

NCHRP 12-49

Comprehensive Specification for the Seismic Design of Bridges

Revised LRFD Design Specifications (Seismic Provisions)

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THIRD DRAFT OF SPECIFICATIONS AND COMMENTARY

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2.1 SCOPE

Minimum requirements are provided for clearances, environmental protection, aesthetics, geological studies, economy, rideability, durability, constructibility, inspectability, and maintainability. Minimum requirements for traffic safety are referenced.

Minimum requirements for drainage facilities and self-protecting measures against water, ice, and water-borne salts are included.

In recognition that many bridge failures have been caused by scour, hydrology and hydraulics are covered in detail.

C2.1

This section is intended to provide the Designer with sufficient information to determine the configuration and overall dimensions of a bridge.

2.2 DEFINITIONS

Control and Repairability Design – A design approach that is similar to conventional ductile design except that construction details provide a replaceable/renewable sacrificial plastic hinge element as described in Article C2.5.6.

Conventional Ductile Design – The design approach most commonly used in current design practice that allows the formation of plastic hinges to dissipate energy as described in Article C2.5.6.

Earthquake Resisting Element (ERE)- A structural element that participates in the Earthquake Resisting System.

Earthquake Resisting System (ERS)- An identifiable structural system designed to resist the effects of the design earthquakes as described in Article 2.5.6.1.

Energy Dissipation – A design approach that relies on specially designed devices usually located between the superstructure and substructure or in a ductile diaphragm to dissipate the energy of an earthquake as described in Article C2.5.6.

Maximum Considered Earthquake (MCE) – The upper level design earthquake used in this specification to represent a rare earthquake that has a 3% probability of being exceeded in 75 years as described in Article 3.10.2.

Seismic Design and Analysis Procedure (SDAP) – One of five design and analysis procedures that are mandated for use by this specification based on the seismic hazard level and the desired performance level as described in Article 3.10.3.

Seismic Detailing Requirements (SDR) – One of six detailing requirements that are mandated by this specification based on the seismic hazard level and the desired performance level as described in Article 3.10.3.

Seismic Isolation – A design approach that reduces the elastic forces a bridge must resist during an earthquake by introducing an isolation bearing and energy dissipating element at the bearing location as described in Article C2.5.6.

Site Class – One of six standard site classifications based on subsurface soil conditions as described in Article 3.10.2.2.1

2.3 LOCATION FEATURES

2.3.5 Geology, Topography, and Land-Use

The geology of the bridge site shall be established as part of the type, size, and location (TS&L) determination

C2.3.5

The geology and topography at a bridge site can play an important role in the bridge type, size, and location

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for the bridge. This evaluation shall consider the potential occurrence of groundwater, soft ground conditions, slope instability, seismicity, faulting, and related geologic hazards on the design and long-term performance of the bridge and its approach fills.

Current topography of the bridge site shall be established via contour maps and photographs. Such studies shall include the history of the site in terms of movement of earth masses, soil and rock erosion, and meandering of waterways.

The history of land-use for the site, such as municipal or hazardous waste disposal and underground mining, shall be established. The potential for flooding or inundation of a site following a major earthquake shall also be identified.

2.4 FOUNDATION INVESTIGATION

2.4.1 General

A subsurface investigation, including borings and laboratory soil tests, shall be conducted in accordance with the provisions of Appendix 2A to provide pertinent and sufficient information for the design of substructure units, including the Site Class of Article 3.10.2.2.1. The type and cost of foundations should be considered in the economic, environmental, and aesthetic studies for location and bridge alternate selection.

2.4.2 Subsurface Investigations

Subsurface explorations shall be made at pier and abutment locations, sufficient in number and depth, to establish a reliable longitudinal and transverse substrata profile. Samples of material encountered shall be taken and preserved for future reference and/or testing. Boring logs shall be prepared in detail sufficient to locate material strata, results of penetration tests, groundwater, any artesian action, and where samples were taken. Special attention shall be paid to the detection of narrow, soft seams that may be located at stratum boundaries.

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(TS&L) determination. Preliminary information about topography and likely geologic conditions should be reviewed. This preliminary information can be obtained from visual reconnaissance by engineering geologists and geotechnical engineers, and from review of geologic maps. With this preliminary information decisions can be made on the possible foundation costs.

Geologic hazards resulting from landslides and earthquakes can lead to extremely high foundation design and construction costs if these hazards are not properly identified during the TS&L phase of the project. As such, it is critical that a representative from the geotechnical area be included in the TS&L process.

In areas of higher seismic activity (Seismic Detailing Requirement (SDR) 3 and above as discussed in Article 3.10.3) special consideration should be given to the identification of potentially active faults that could occur beneath or close to the abutments of the bridge or between the abutments. Appendix 3B provides additional discussion of issues associated with active faults.

C2.4.1

The conduct of the subsurface exploration program is part of the process of obtaining information relevant for the design and construction of substructure elements. Information from the subsurface exploration is particularly critical in areas of higher seismicity (SDR 3, 4, 5, and 6) as information from the exploration will determine the Site Classification for seismic design and the potential for geologic hazards, such as liquefaction and slope stability.

The elements of the process that should precede the actual exploration program include search and review of published and unpublished information at and near the site, a visual site inspection, and design of the subsurface exploration program. Refer to *AASHTO Manual on Subsurface Investigations* (1988) for general guidance regarding the planning and conduct of subsurface exploration programs.

C2.4.2

The exploration phase of the project should be conducted early enough that geologic conditions that could have a significant effect on project costs are identified. If subsurface information is not available from previous work in the area, it may be desirable to conduct a limited exploration program before TS&L to identify conditions that may change either the location or type of bridge.

A variety of subsurface exploration methods are available. The most common methods involve drilling methods or cone penetrometer soundings. In some cases geophysical methods can be used to provide information

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2.4.3 Laboratory Testing

Laboratory tests shall be performed to determine the strength, deformation, and flow characteristics of soils and/or rocks and their suitability for the foundation selected. In areas of higher seismicity (e.g., SDR 3, 4, 5, and 6), it may be appropriate to conduct special dynamic or cyclic tests to establish the liquefaction potential or stiffness and material damping properties of the soil at some sites, if unusual soils exist or if the foundation is supporting a critical bridge.

2.5 DESIGN OBJECTIVES

2.5.6 Seismic Design Approaches

All bridges and their foundations shall have a clearly identifiable earthquake resisting system (ERS) selected to achieve the performance objectives defined in Table 3.10.1-1. The ERS shall provide a reliable and uninterrupted load path for transmitting seismically induced forces into the ground and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements. All structural and foundation elements of the bridge shall be capable of achieving anticipated displacements consistent with the requirements of the chosen mechanism of seismic resistance and other structural requirements.

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relevant to the design of the substructure system. Appendix 2A to this Section provides a discussion of these methods. As noted in this Appendix, each of these methods has limitations. A geotechnical engineer or engineering geologists should be involved in the selection of the most appropriate exploration method.

C2.4.3

The equipment and methods used during laboratory testing will depend on the type of soil or rock, as well as the state of disturbance of the sample to be tested. Therefore, the need for certain types of samples should be considered when planning the field exploration phase of the project.

The number and type of laboratory test should be determined after reviewing boring logs developed from the field exploration plan relative to the range in substructures that will be possibly used for the bridge. Additional details regarding laboratory testing are presented in Appendix 2A.

C2.5.6 Design Approaches

These provisions provide the designer with a range of performance objectives as shown in Table 3.10.1-1. Bridges are seismically designed so that inelastic deformation (damage) intentionally occurs in columns in order that the damage can be readily inspected and repaired after an earthquake. Capacity design procedures are used to prevent damage from occurring in the connections of columns to the foundation and the superstructure as well as in foundations and beams of bents. There are two exceptions to this design philosophy. For pile bents and drilled shafts, some limited inelastic deformation is permitted below the ground level with the owner's approval. The amount of permissible deformation is limited to ensure that no long-term serviceability problems occur due to the amount of cracking that is permitted in the concrete pile or shaft. The second exception is with lateral spreading associated with liquefaction. For the life-safety performance level, significant inelastic deformation is permitted in the piles, primarily because this can be a costly and difficult problem to prevent. There are a number of design approaches that can be used to achieve the performance objectives. These are given in Figure C2.5.6-1 and discussed briefly below.

Conventional Ductile Design - Caltrans first introduced this design approach in 1973 following the 1971 San Fernando earthquake. It was further refined and applied nationally in the 1981 AASHTO Guide Specification for Seismic Design of Highway Bridges (ATC, 1981). These provisions were adopted by AASHTO in 1991 as their Standard Seismic Provisions. The design forces are obtained from an elastic analysis of the bridge using response spectra for the appropriate design event. Component design forces such as column moments

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(M_b) are obtained by dividing the elastic column moment (M_e) by a specified R-Factor as shown in Figure C2.5.6-2. The component's actual strength will be greater than the design strength by an over-strength ratio which will range from 1.3 to 1.6. If the R-Factor for a column is low (i.e., <1.5) then the column should remain essentially elastic for the design event and inelastic deformation (damage) should be avoided. If the R-Factor is high (i.e. $R>3$) then significant plastic hinging may occur and the column may not be repairable. If the R-Factor is between 1.5 and 3.0 then the column should be repairable. The other key premise of the provisions is that displacements caused by the inelastic response of a bridge are approximately equal to the displacements obtained from an analysis using the unreduced elastic response spectrum. As diagrammatically shown in Figure C2.5.6-2 this assumes that D_{max} (or $D_{inelastic}$) is equal to D_e (or $D_{elastic}$). Recent work by Miranda and Bertero (1994) and by Chang and Mander (1994) indicates that this is a reasonable assumption except for short period structures for which it is non-conservative. A correction factor on displacements to address this issue is given in Article 3.10.3.9.4. A plot of the results from Miranda and Bertero's work is given in Figure C2.5.6-3. A more detailed discussion on the basis of the conventional design provisions can be found in ATC 18 (1997).

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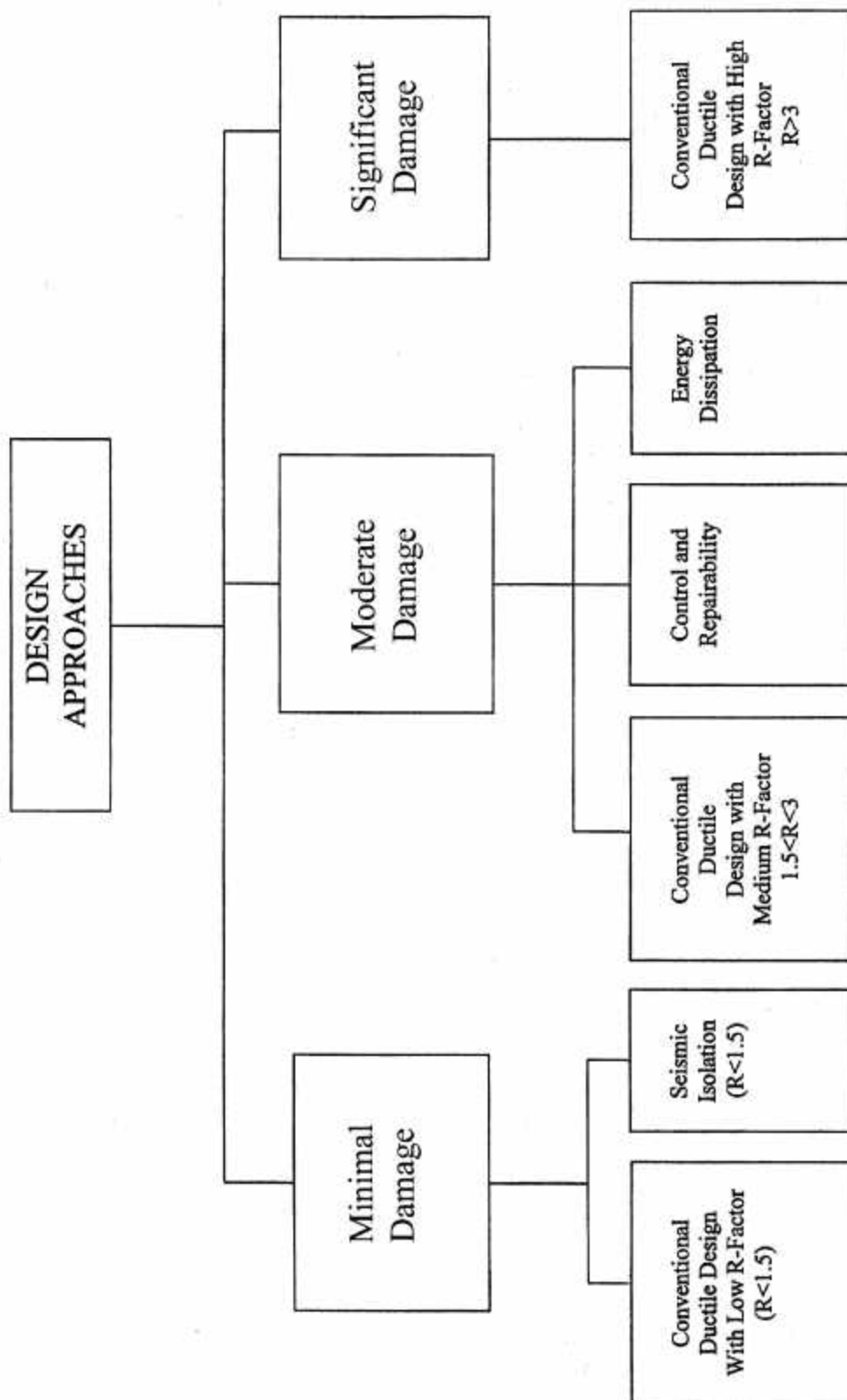


Figure C2.5.6-1 Design Approaches

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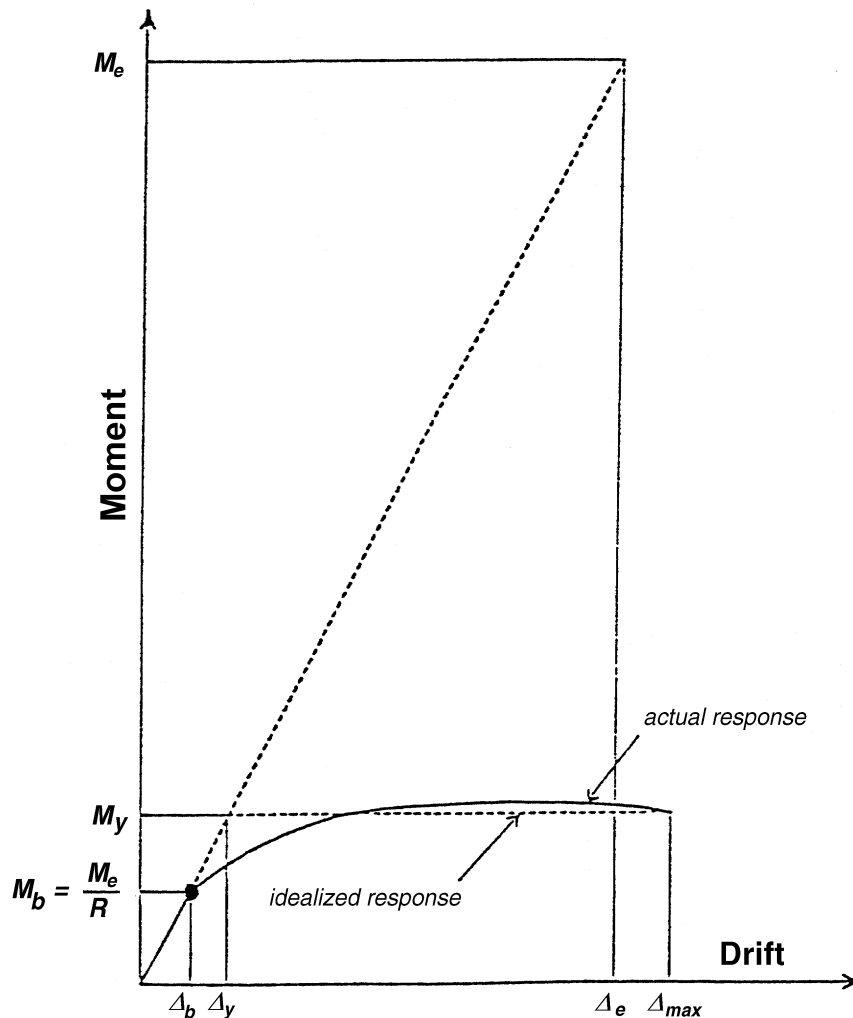


Figure C2.5.6-2 Basis for Conventional Ductile Design

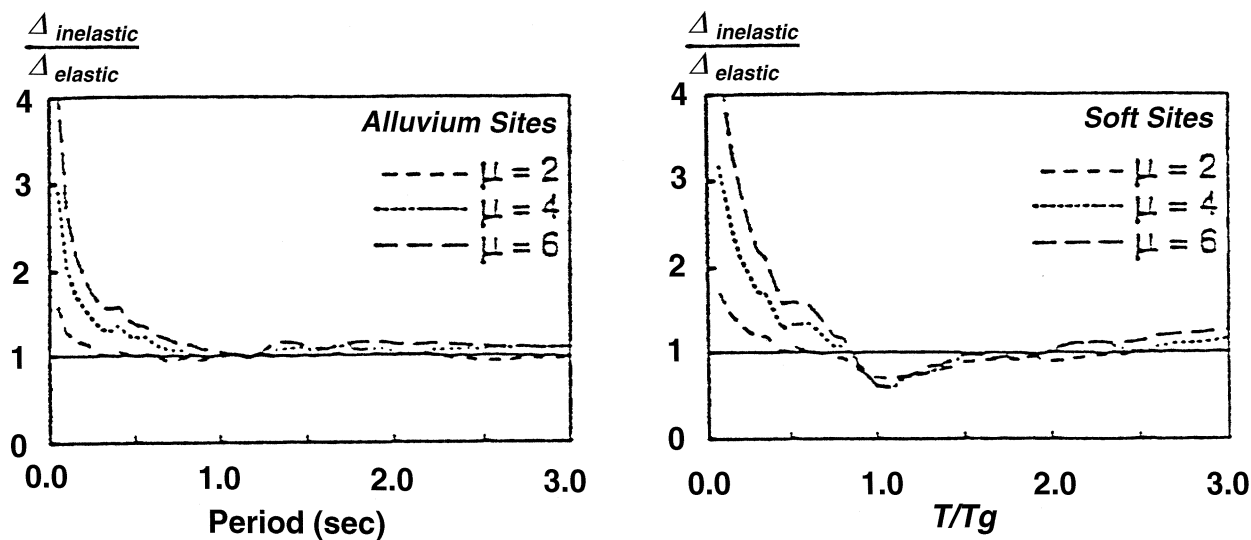


Figure C2.5.6-3 Comparison of Elastic and Inelastic Displacements
(From Miranda and Bertero)

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Seismic Isolation - This design approach reduces the elastic forces a bridge must resist by introducing an isolation bearing and energy dissipation element at the bearing location. The isolation bearing intentionally lengthens the period of a relatively stiff bridge and this results in lower design forces. This design alternate was first applied in the US in 1984 and has been extensively reported on in technical literature. (e.g. ATC, 1986 and 1993; ASCE, 1989, 1991 and 1993; EERI, 1990). As of January 1, 1999 there were over 120 bridges constructed in the U.S. and over 300 worldwide using this concept. AASHTO adopted Guide Specifications for Seismic Isolation Design of Highway Bridges in 1991 and these were substantially revised in 1997. The 1997 and 2000 revisions are now incorporated in these provisions. Elastic response of the substructure elements is possible with seismic isolation, since the elastic forces resulting from seismic isolation are generally less than the reduced design forces required by conventional ductile design using an R factor of 3 to 6.

Energy Dissipation - this design approach adds energy dissipation elements between the deck and the column and/or abutment or in the end diaphragm of a steel girder bridge with the intent of dissipating energy in elements designed specifically for that purpose. This minimizes the energy that is dissipated in the plastic hinge zone of columns. This design approach differs from seismic isolation in that an element of flexibility is generally not part of the system and thus the fundamental period of vibration is not changed. If the equivalent viscous damping of the bridge is increased from 5% to 30% then the displacement of the deck will be reduced by a factor of approximately 2. In general the energy dissipation design concept does not result in reduced design forces but it will reduce the ductility demand on columns due to the reduction in deck displacement (ATC, 1993 and EERI, 1998). As of January 1, 1999 there are approximately 10 applications of this design approach in the U.S. If the energy dissipation is in the end diaphragm of a steel girder bridge then the diaphragm acts as a force-limiting fuse in the transverse direction.

Control and Repairability Design - this design approach is based on the conventional ductile design concept that permits significant inelastic deformation in the plastic hinge zone of a column. The difference with conventional ductile design is that construction details in the hinge zone of reinforced concrete columns provide a replaceable/renewable sacrificial plastic hinge element. Hinge zones are deliberately weakened with respect to their adjoining elements and all regions outside the hinge zone are detailed to remain elastic and undamaged during seismic loading. The concept has been extensively tested but as of January 1, 1999 has not yet been used in practice. Chang and Mander (1997) provides the details for the implementation of this design concept.

The design objectives and performance expectations of the above design approaches are as follows:

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Columns as Primary Energy Dissipation Mechanism

1. The bridge is analyzed to get the elastic design moments in the columns. The elastic moments are reduced by the R-Factor to determine the design moment for the determination of longitudinal column steel. This design value or the minimum longitudinal steel requirement (0.8%) or the P- Δ requirement may govern the amount of longitudinal steel. The design objective is to minimize the amount of longitudinal steel as this will minimize the foundation and connection costs. For the no analysis procedure specified in Sec.3.10.3.3 the amount of longitudinal steel required for non-seismic loads is used as the starting point for the capacity design procedure.
2. In order to force inelastic deformation in the columns the connections of the column to the footing and superstructure are designed for the maximum moments and shears that can be developed by the columns as described in the capacity design procedures of Sec. 3.10.3.8. The design objective is to force inelastic deformation to occur where it can be readily inspected and repaired.
3. The performance expectation is that inelastic deformation will occur primarily in the columns. If large ductility demands occur then the columns may need to be replaced. Replacement of columns can be avoided with the use of the control and repairability design approach or with the use of a low R-Factor (< 3) or with the use of the seismic isolation design alternate to reduce the elastic force demand on the columns.

Abutments as an Additional Energy Dissipation Mechanism

1. In the early phases of the development of the provisions, there was serious debate as to whether or not the abutments would be included and relied upon in the ERS. Some states design a bridge so that the substructures are capable of resisting all the lateral load without any contribution from the abutments. In this design option the abutments are a mechanism to provide an unquantifiable higher level of safety. Rather than mandate this design philosophy it was decided to permit two design alternates. The first is where the Earthquake Resisting System (ERS) does not include the abutments and the substructures are capable of resisting all the lateral loads. The second alternate is where the abutments are an important part of the ERS and in this case, a higher level of analysis is required — SDAP E. The abutments can be designed as part of the ERS and become an additional source for dissipating the earthquake

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energy. In the longitudinal direction the abutment maybe designed to resist the forces elastically utilizing the passive pressure of the backfill. In some cases the displacement of the deck will exceed the passive pressure and cause larger soil movements in the abutment backfill. This requires a more refined analysis to determine the amount of expected movement. In the transverse direction the abutment is generally designed to resist the loads elastically. In some cases (spread footings) limited movement is permitted and the elastic forces are reduced by 1.5. The design objective when abutments are relied upon to resist either longitudinal or transverse loads is to either minimize column sizes and/or reduce the ductility demand on the columns accepting that damage may occur in the abutment.

2. The performance expectation is that inelastic deformation will occur in the columns as well as the abutments. If large ductility demands occur in the columns then the columns may need to be replaced. If large movements of the superstructure occur the abutment back-wall may be damaged as well as some settlement of the abutment backfill. Large movements of the superstructure can be reduced with use of energy dissipators and/or isolation bearings at the abutments and/or column locations. Replacement of columns can be avoided with the use of the control and repairability design approach or with the use of a low R-Factor (< 3) or with the use of the seismic isolation design alternate to reduce the demand on the columns.

There are several design alternates available to a designer and these are summarized separately for concrete and steel superstructures.

Concrete Superstructures

- Columns monolithic with the superstructure with energy dissipation occurring in the columns and at times in the abutment soil backfill. The control and repairability concept can be used in conjunction with this design alternate if the need to avoid replacing a column after a large earthquake is desired.
- Superstructure supported on conventional bearings. Energy dissipation will occur in the

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columns and at times in the abutment soil backfill and to a more limited extent in some types of bearings. Bearings are a critical element in the load path of this design alternate and must be demonstrated by test to be able to resist the MCE forces and displacements in both the longitudinal and transverse directions (Article 3.10.3.14). Alternately restraint systems may be used to resist the MCE forces. If failure of a bearing is part of this design concept the superstructure must have a level surface on which to slide and this configuration must be analyzed since load redistribution will occur (Article 3.10.3.14).

- Superstructure supported on isolation bearings. Energy dissipation will occur in the isolation bearings although some may also occur in the abutment soil backfill. This permits the columns to be designed elastically thus avoiding damage in the columns.

Steel Superstructures

- Steel superstructure supported on either conventional or isolation bearings as discussed above for concrete superstructures. The control and repairability alternate is applicable for concrete substructures but not for steel substructures.
- Steel superstructure designed with the ductile end diaphragm concept. This concept when applicable has the ability to eliminate the ductility demand on columns in the transverse direction only. The columns are capacity protected in the transverse direction by being designed for the maximum forces generated by the ductile end diaphragm.

2.5.6.1 EARTHQUAKE RESISTING SYSTEMS (ERS) FOR SEISMIC DESIGN

For the purposes of encouraging the use of appropriate systems and of ensuring due consideration of performance by the owner, the ERS and earthquake resisting elements (ERE) are categorized as follows:

-
- Permissible
 - Permissible with Owner Approval
 - Not Recommended for New Bridges
-

C2.5.6.1

Selection of an appropriate ERS is fundamental to achieving adequate seismic performance. To this end, the identification of the lateral-force-resisting concept and the selection of the necessary elements to facilitate the concept should be accomplished in the conceptual design or type, size and location or design alternative phase of a project.

Seismic performance is typically better in systems with regular configurations and evenly distributed stiffness and strength. Thus, typical geometric configuration constraints, such as skew, unequal pier heights, sharp curves, etc, conflict, to some degree, with the seismic design goals. For this reason, it is advisable to resolve potential conflicts between configuration and seismic performance early in the design effort. For example,

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These terms apply to both systems and elements. For a system to be in the permissible categories, its primary ERE must all be in the permissible categories. If any ERE are not permissible, then the entire system is not permissible.

Permissible systems and elements have the following characteristics:

1. All significant inelastic action shall be ductile and occur in locations with adequate access for inspection and repair. If all structural elements of a bridge are designed elastically ($R=1.0$) then no inelastic deformation is anticipated and the elastic elements are permissible.
2. Inelastic action does not jeopardize the gravity load support capability of the structure (e.g. cap beam and superstructure hinging)

Permissible systems that require owner approval are those that do not meet either item (1) or (2), above. Such systems may be used; however, the owner shall approve their use. Additionally, these systems will require the use of the highest level of analysis requirement (Seismic Design and Analysis Procedures E – Article 3.10.3.6), as outlined in the flow chart shown in Figure 2.5.6-1. The minimum Seismic Design and Analysis Procedures (SDAP) are defined in Article 3.10.3.1.

Systems that do not fall in either of the two permissible categories are not recommended. In general, they are not allowed. However, if adequate consideration is given to all potential modes of behavior and potential undesirable failure mechanisms are suppressed, then such systems may be used with the owner's approval.

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resolution may lead to decreased skew angles at the expense of longer end spans. The resulting trade-off between performance and cost should be evaluated in the type, size, and location or design alternative phase of a project when design alternatives are viable from a practical viewpoint.

The classification of ERS and ERE into permissible and not recommended categories is done to trigger due consideration of seismic performance that leads to the most desirable outcome, that is seismic performance that ensures wherever possible post-earthquake serviceability. To achieve such an objective, special care in detailing the primary energy dissipating elements is necessary. Conventional reinforced concrete construction with ductile plastic hinge zones can continue to be used, but designers should be aware that such detailing, although providing desirable seismic performance, will leave the structure in a damaged state following a large earthquake. It may be difficult or impractical to repair such damage. Therefore, in order to ensure post-earthquake serviceability of the highway system as a whole, especially on essential routes with high traffic volumes, designers are encouraged to consider the use of replaceable/repairable elements that may consist of plastic hinge zones with purpose-built fuse bars; seismic isolation devices and systems; and systems with supplemental / sacrificial energy dissipation devices, such as dampers or other yielding devices

It should be recognized that under certain conditions the use of ERE that require owners' approval will be necessary. In the earlier AASHTO seismic specifications (1991-2000) some of the ERE in the owners' approval category were simply not permitted for use (i.e., in ground hinging of piles and shafts, foundations permitted to rock beyond $\frac{1}{2}$ uplift, etc.) These elements are now permitted provided their deformation performance is assessed as part of a pushover analysis (Article 3.10.3.6). This approach of allowing their use with additional analytical effort was believed to be preferable to an outright ban on their use. Thus, it is not the objective of this specification to discourage the use of systems that require owner approval. Instead, such systems may be used, but additional design effort and consensus between the designer and owner are required to implement such systems.

Common examples from each of the three categories of systems are shown in Figures C2.5.6-1 through C2.5.6-4.

In general, the soil behind abutments is capable of resisting substantial seismic forces that may be delivered through a continuous superstructure to the abutments. Furthermore, such soil may also substantially limit the overall movements that a bridge may experience. This is particularly so in the longitudinal direction of a straight bridge with little or no skew and a continuous deck. The

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controversy with this design concept is the scenario of what may happen if there is significant abutment damage early in the earthquake ground-motion duration and the columns are reliant on the abutment resisting a proportional amount of load. This would be a problem in a long duration and high magnitude (greater than 7) earthquake. Unless lock up devices are used, a bridge comprised of multiple simply supported spans cannot effectively mobilize the abutments for resistance of longitudinal force. It is recommended that simply supported spans do not rely on abutments for any seismic resistance.

Because structural redundancy is desirable, good design practice dictates the use of the design alternate where the intermediate substructures are designed to resist all seismic loads, if possible. This assures that in the event abutment resistance becomes ineffective, the bridge will still be able to resist the earthquake. In such a situation, the abutments provide an increased margin against collapse.

The same arguments can be made for allowing damage in locations that are very difficult to inspect. For instance, the first approach to a design using drilled shafts is to keep plastic hinging above the ground and some states mandate this design concept. However, situations arise where this is impractical. In such situations, the ERS would require owner approval.

The flow chart in Figure 2.5.6-2 helps facilitate the decision-making process for assessing and accommodating restricted behavior.

The interrelationship between the performance level, the earthquake resisting system and the SDAP is given in Table 2.5.6-1. Abutment design issues are further amplified in Table 2.5.6-2.

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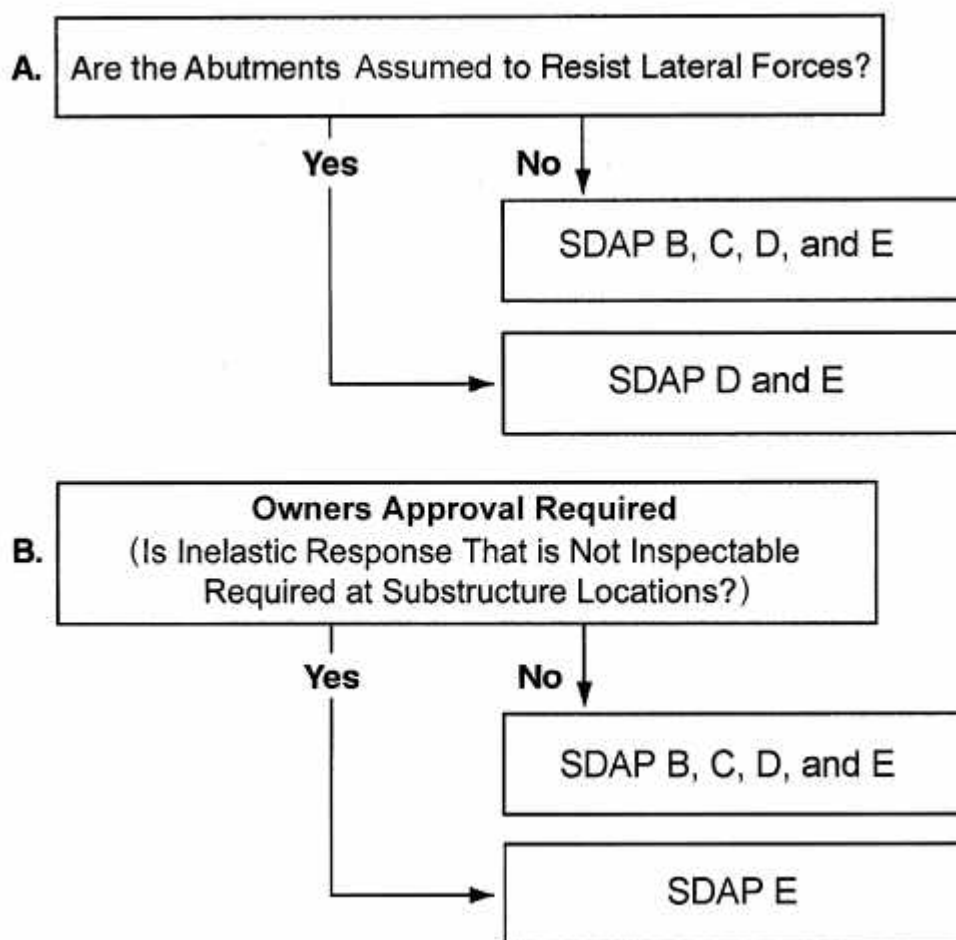


Figure 2.5.6-2 Classification of ERS

Table 2.5.6-1 Performance Levels and Earthquake Resisting Systems

Performance Level	Expected Element Behavior	Earthquake Resisting System	Abutment Performance	
			50% in 75 Years	3% in 75 Years
Operational	Linear Elastic Nonlinear Elastic	Permissible elements designed to resist all seismic loads within displacement constraints. Elements requiring owner approval should not be used.	No damage. Soil passive mobilization is OK if $\Delta \leq 0.01H_E$	No damage. Soil passive mobilization is O.K. if $\Delta \leq 0.02H_E$
Life Safety	Linear Elastic Nonlinear Elastic Nonlinear Inelastic	Permissible elements designed to resist all seismic loads within displacement constraints. Elements requiring owner approval are OK.	Limited damage and soil passive mobilization O.K.	Significant damage. Soil passive mobilization is O.K.

SECTION 2 – GENERAL DESIGN AND LOCATION FEATURES (SI)

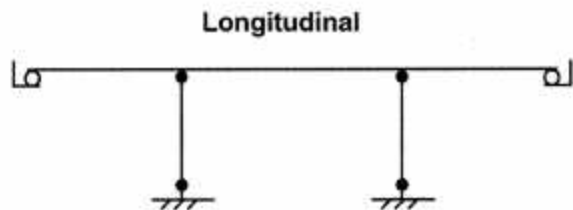
SPECIFICATIONS

COMMENTARY

Table 2.5.6-2 Abutment Design Issues

No Damage		Significant Damage Accepted	
Longitudinal	Transverse	ERS does not Include Abutment Contribution	ERS Includes Abutment Contribution
Alternate 1 – Abutment resists forces by mobilizing passive soil for 3% in 75-year event and displacement constraints of Table 3.10.1-2 are acceptable ($\Delta \leq 0.02H_E$). Needs sufficient backwall clearance for 50% in 75-year event.	Alternate 1 – Design abutments to resist full 3% in 75-year transverse loads within acceptable displacement limits of Table 3.10.1-2 ($\Delta \leq 0.02H_E$)	The ERS is designed to resist all seismic loads without any contribution from abutments (SDAP B and C). Abutments then limit displacement and provide additional capacity and better performance. The bridge is safe even if serious problems occur at the abutments. For SDAP D and E and the 50% in 75-year event, the bridge should be analyzed with the abutments and the abutments are designed for the 50% in 75-year forces and displacements. If sacrificial concrete shear keys are used to protect the piles, the bridge shall be analysed with all combinations of shear key failure considered (i.e. at each abutment separately and both abutments simultaneously).	The ERS is designed with the abutments as a key element of the ERS. Abutment are designed and analyzed for the 3% in 75-year forces and displacements.
Alternate 2 – Abutment does not mobilize passive soil in 3% in 75-year event. Need sufficient clearance to backwall or use top of backwall knockoff detail.	Alternate 2 – Provide capacity protection (force-limiting devices) for abutment, plus sufficient clearance. Transverse force capacity governed by 50% in 75-year forces. Capacity protection by shear key or bearings that provide sufficient nonseismic lateral capacity and then have sufficient displacement capacity for 3% in 75-year event. If sacrificial concrete shear keys are used to protect the piles, the bridge shall be analysed with all combinations of shear key failure considered (i.e. at each abutment separately and both abutments simultaneously).		
Alternate 3 – With either of above alternatives, use displacement-limiting devices (isolation bearing or energy dissipation devices) to limit overall deck displacements. Displacements can be reduced by up to a factor of 2 with 30% damping.	Alternate 3 – Provide sufficient clearance in the transverse direction to permit the deck to move. The movement can be limited with isolation bearings or energy dissipation devices		

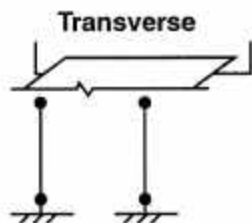
SECTION 2 – GENERAL DESIGN AND LOCATION FEATURES (SI)



Abutment resistance not required as part of ERS

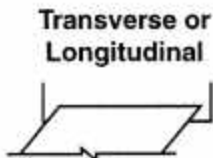
Plastic hinges in inspectable locations or elastic design of columns.

Knock-off backwalls permissible

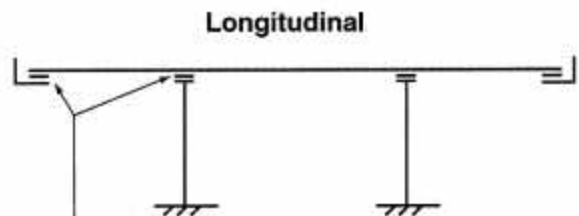


Abutment not required in ERS, breakaway shear keys permissible

Plastic hinges in inspectable locations or elastic design of columns.

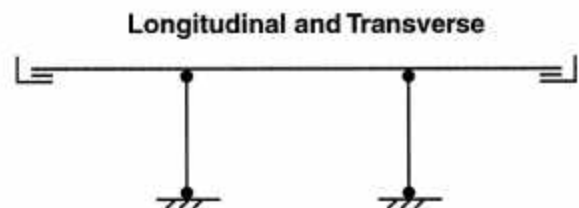


Abutment resistance required, but abutment able to resist 3% in 75-year earthquake elastically and passive soil pressure in longitudinal direction is less than $0.70 \times$ presumptive value given in 11.6.5.1



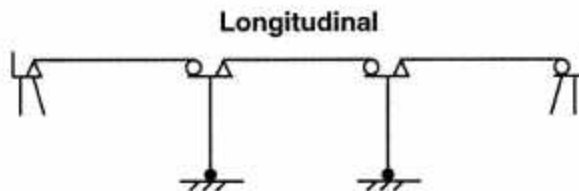
Abutment not required as part of ERS

Isolation bearings accommodate full displacement



Isolation bearings with significant energy dissipation capacity or energy dissipators are used at the abutment to limit overall displacements.

Plastic hinges in inspectable locations or elastic design of columns.



Multiple simply-supported spans with adequate seat widths. Plastic hinges in inspectable locations or elastic design of columns.

Figure C2.5.6-1a

Permissible Earthquake Resisting Systems

SECTION 2 – GENERAL DESIGN AND LOCATION FEATURES (SI)

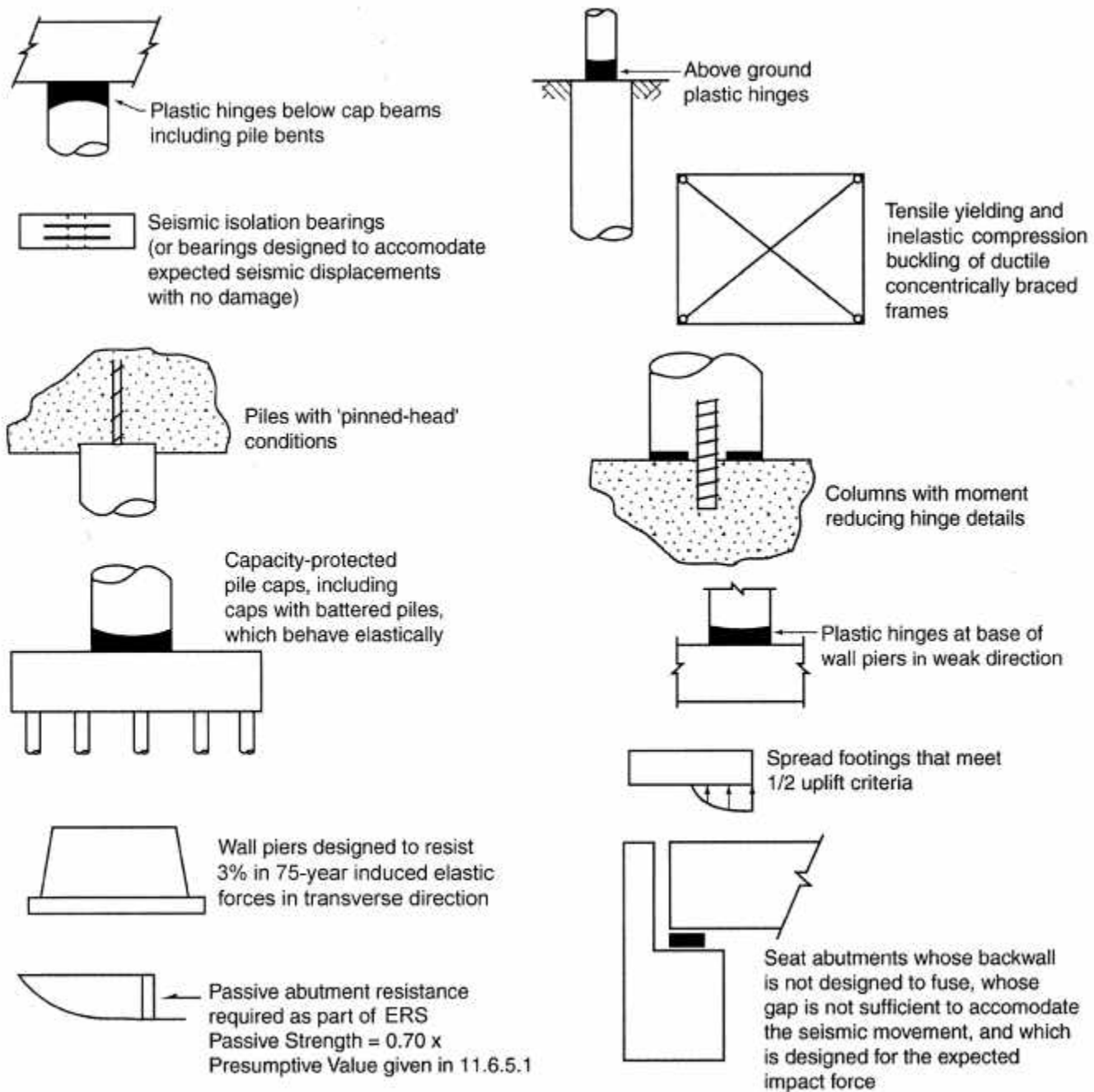
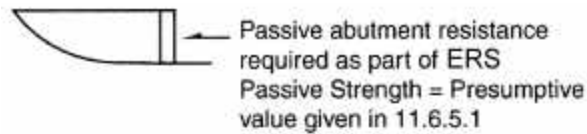


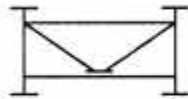
Figure C2.5.6-1b

Permissible Earthquake Resisting Elements

SECTION 2 – GENERAL DESIGN AND LOCATION FEATURES (SI)

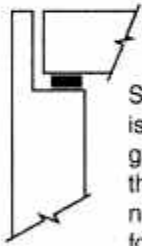


OANR: Use 70% of presumptive strength



Ductile diaphragms in superstructure

OANR: Yielding restricted to substructure



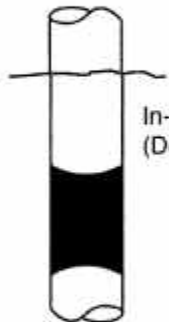
Seat abutments whose backwall is not designed to fuse, whose gap is not sufficient to accommodate the seismic movement, and which is not designed for the expected impact force

OANR: Design to fuse or design for the appropriate design forces and displacements



Wall piers on pile foundations that are not strong enough to force plastic hinging into the wall, and are not designed for the 3% in 75-year elastic forces

OANR: Force hinging into the wall with multiple pile lines and pile cap



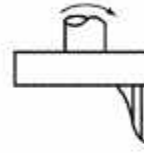
In-ground hinging in shafts or piles
(Deformation limits in Section 5)

OANR: Force hinging to occur above ground with larger in-ground shaft



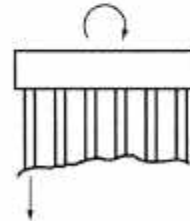
Sliding of spread footing abutment allowed to limit force transferred

OANR: Design for no sliding



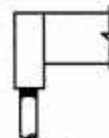
Foundations permitted to rock beyond 1/2 uplift limit or exceed ultimate bearing stress and a linear stress distribution

OANR: Use 1/2 uplift and linear stress distribution



More than the outer line of piles in group systems allowed to plunge or uplift under seismic loadings

OANR: Only outer line is permitted to reach tension capacity



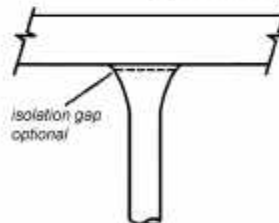
Plumb piles that are not capacity-protected (e.g. integral abutment piles or pile-supported seat abutments that are not fused transversely)

OANR: Use seat abutment or a detail that allows movement



Batter pile systems in which the geotechnical capacities and/or in-ground hinging define the plastic mechanisms

OANR: Plastic hinging forced to occur above ground in column



Columns with Architectural Flares - with or without an isolation gap

OANR: Remove flare

Note: OANR means a design alternate where owners approval is not required and a higher level of analysis (pushover in SDAP E) can be avoided.

Figure C2.5.6-2 Permissible Earthquake Resisting Elements that Require Owner's Approval

SECTION 2 – GENERAL DESIGN AND LOCATION FEATURES (SI)

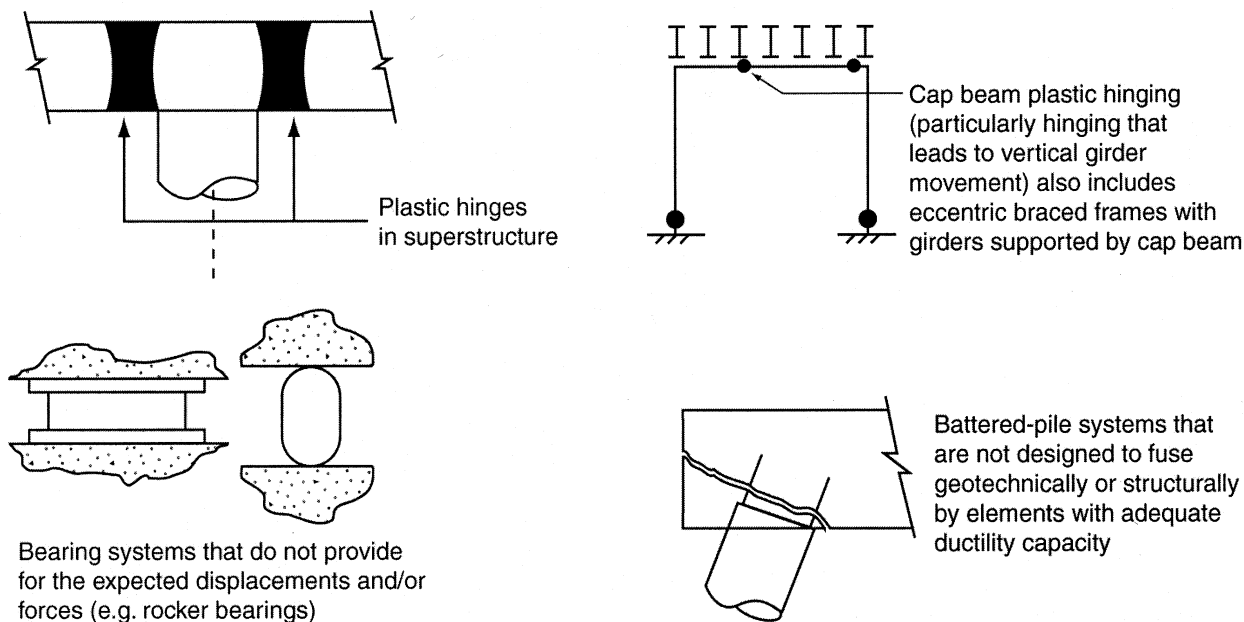


Figure C2.5.6-3 Earthquake Resisting Elements that are not Recommended for New Bridges

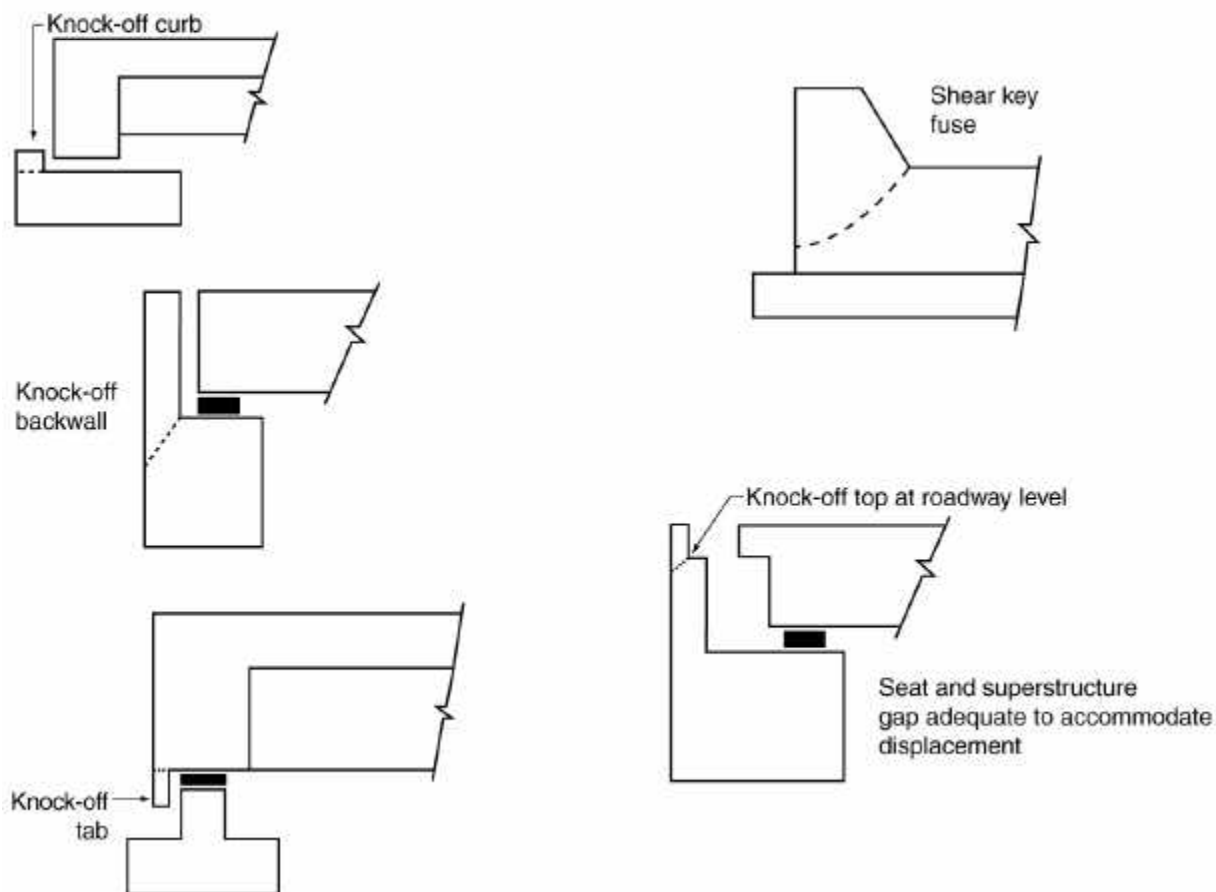


Figure C2.5.6-4 Methods of Minimizing Damage to Abutment Foundation

SECTION 2 – GENERAL DESIGN AND LOCATION FEATURES (SI)

SPECIFICATIONS

COMMENTARY

2.5.6.2 REQUIREMENTS FOR TEMPORARY BRIDGES AND STAGE CONSTRUCTION

C2.5.6.2

Any bridge or partially constructed bridge that is expected to be temporary for more than five years shall be designed using the requirements for permanent structures and shall not use the provisions of this Article.

The requirement that an earthquake shall not cause collapse of all or part of a bridge, as stated in Article 3.10.1, shall apply to temporary bridges expected to carry traffic. It shall also apply to those bridges that are constructed in stages and expected to carry traffic and/or pass over routes that carry traffic. The acceleration coefficient given in Article 3.10.2 may be reduced by a factor of not more than 2 in order to calculate the component elastic forces and displacements. Acceleration coefficients for construction sites that are close to active faults shall be the subject of special study. The response modification factors given in Article 3.10.5 may be increased by a factor of not more than 1.5 in order to calculate the design forces. This factor shall not be applied to connections as defined in Table 3.10.5.1-2.

The minimum seat width provisions of Article 4.7.4.4 shall apply to all temporary bridges and staged construction.

The option to use a reduced acceleration coefficient is provided to reflect the limited exposure period.

SECTION 2 – GENERAL DESIGN AND LOCATION FEATURES (SI)

REFERENCES	SPECIFICATIONS	COMMENTARY
ASCE, 1989, 1991, and 1993, Proceedings ASCE Structures Congress: <u>Seismic Engineering – Research and Practice</u>		
ATC, 1981, <i>Seismic Design Guidelines for Highway Bridges</i> , Report No. ATC-6, Applied Technology Council, Redwood City, California.		
ATC, 1997, <i>Seismic Design Criteria for Bridges and other Highway Structures; Current and Future</i> , Report No. ATC-18, Applied Technology Council, Redwood City, California.		
ATC, 1986, <i>Proceeding of a Seminar on Base Isolation and Energy Dissipation</i> , Report No. ATC-17, Applied Technology Council, Redwood City, California.		
ATC, 1993, <i>Proceeding of a Seminar on Seismic Isolation, Passive Energy Dissipation and Active Control</i> , Report No ATC-17-1, Applied Technology Council, Redwood City, California.		
Andrus, R.D. and Youd, T.L. "Subsurface Investigation of a Liquefaction-Induced Lateral Spread, Thousand Springs Valley, Idaho," <u>U.S. Corps of Engineers Miscellaneous Paper GL-87-8</u> , 1987		
Chang, G.A. and Mander, J.B., 1994 <i>Seismic Energy Based Fatigue Damage Analysis of Bridge Columns – Part I and II</i> , NCEER Technical Report Nos., 94-0006 and 94-0013, National Center for Earthquake Engineering Research, State University of New York, Buffalo, New York.		
EERI, 1990, "Seismic Isolation: From Idea to Reality," Earthquake Engineering Research Institute, Oakland, California.		
Kramer, S.L. <u>Geotechnical Earthquake Engineering</u> , Prentice Hall, Upper Saddle River, NJ, 653 p., 1996		
Miranda, E. and Bertero, V.V., 1994, "Evaluation of Strength Reduction Factors for Earthquake-Resistant Design," <i>Earthquake Spectra</i> , Vol. 10, No. 2, Earthquake Engineering research Institute, Oakland, California.		
Nassar, A.A. and Krawinkler, H., 1991, <i>Seismic Demands for SDOF and MDOF Systems</i> , Report Nol 95, John A. Blume Earthquake Engineering Center, Stanford University, Stanford, California.		
Vallee, R.P. and Skryness, R.S. "Sampling and In Situ Density of a Saturated Gravel Deposit," <u>ASTM Geotechnical Testing Journal</u> , Vol. 2, No. 3, pp. 136-142, 1980.		
Youd, T.L. and Idriss, I.M. (Editors), <u>Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils</u> , NCEER Technical Report NCEER-97-0022, Salt Lake City, UT, January 5-6, 1997.		

2A.1 GENERAL

Site characterization shall be performed for each substructure element, as appropriate, to provide the necessary information for the design and construction of foundations. The type and extent of site characterization shall be based on subsurface conditions, structure type, and project requirements. The site characterization program shall be extensive enough to reveal the nature and types of soil deposits and/or rock formations encountered, the engineering properties of the soils and/or rocks, the potential for liquefaction, and the groundwater conditions.

2A.2 SUBSURFACE EXPLORATIONS

Subsurface explorations shall be made to competent material of suitable bearing capacity or to a depth where added stresses due to estimated footing load is less than 10 percent of the existing effective soil overburden stress, whichever is the greater. If bedrock is encountered at shallow depths, the exploration shall advance a minimum of 3000 mm into the bedrock or to 1000 mm beyond the proposed foundation depth, whichever is greater.

Site characterization normally includes subsurface explorations and laboratory testing of samples of soil/rock recovered during the exploration work. Subsurface exploration can include drilling and sampling of the soil or rock, as well as in situ testing.

C.2A.2

As a minimum, the subsurface exploration and testing program should obtain information to analyze foundation stability and settlement with respect to:

- Geological formation(s);
- Location and thickness of soil and rock units;
- Engineering properties of soil and rock units, including density, shear strength and compressibility;
- Groundwater conditions;
- Ground surface topography
- Local considerations, such as expansive or dispersive soil deposits, collapse potential of soil in arid regions, underground voids from solution weathering or mining activity, or slope instability potential; and
- Behavior under seismic loading, including liquefaction, seismic-induced ground settlement, lateral flow and spreading (e.g., sloping ground underlain by very loose saturated soil and the presence of a free face), and ground motion amplification or attenuation.

Issues related to the constructibility of the foundation system should also be identified during the subsurface investigation process. These issues can include the drivability of piles, the excavability/stability of holes for drilled shafts and similar bored systems (e.g., Cast-in-Drill Hole (CIDH) piles), occurrence of boulders and rocks that could affect pile or retaining wall construction, need for and ability to de-water soils or control groundwater flow.

2A.2.1 In Situ Tests

In situ tests may be performed to obtain deformation and strength parameters of foundation soils or rock for the purposes of design and/or analysis. The tests shall be performed in accordance with the appropriate standards recommended by ASTM or AASHTO and may include the following in-situ soil tests and in-situ rock tests:

In Situ Soil Tests

- Standard Penetration Test - AASHTO T 206 (ASTM D 1586)
- Static Cone Test - ASTM D 3441
- Field Vane Test - AASHTO T 223 (ASTM D 2573)
- Pressuremeter Test - ASTM D 4719
- Plate Bearing Test - AASHTO T 235 (ASTM D 1194)
- Well Test (Permeability) - ASTM D 4750

In Situ Rock Tests

- Deformability and Strength of Weak Rock by an In-Situ Uniaxial Compressive Test - ASTM D 4555
- Determination of Direct Shear Strength of Rock Discontinuities - ASTM D 4554
- Modulus of Deformation of Rock Mass Using the Flexible Plate Loading Method - ASTM D 4395•
Modulus of Deformation of Rock Mass Using a Radial Jacking Test - ASTM D 4506
- Modulus of Deformation of Rock Mass Using the Rigid Plate Loading Method - ASTM D 4394
- Stress and Modulus of Deformation Determination Using the Flatjack Method - ASTM D 4729
- Stress in Rock Using the Hydraulic Fracturing Method - ASTM D 4645

If so requested by the Owner, boring and penetration test holes shall be plugged to prevent water contamination.

C.2A.2.1

The most suitable type of exploration method will depend on the type of soil/rock encountered, the type and size of the foundation, and the requirements of design. Often a combination of one or more methods is required. In nearly every situation at least one boring with soil/rock sampling should be planned. Results of other soil exploration methods, such as the cone penetrometer or field vane, should be compared to information recovered in the soil boring. Table 2A.1-1 provides a summary of the suitability and information that can be obtained from different in situ testing methods.

Parameters derived from field tests, such as standard penetration, cone penetrometer, dynamic penetrometer, and pressuremeter tests, can often be used directly in design calculations based on empirical relationships. These are sometimes found to be more reliable than analytical calculations, especially in familiar ground conditions for which the empirical relationships are well established.

Appendix 2A – Provisions for Site Characterization
(Subsurface Explorations, In Situ Testing, Laboratory Testing)

Table 2A.1-1 - In-Situ Tests

TYPE OF TEST	BEST SUITED TO	NOT APPLICABLE TO	PROPERTIES THAT CAN BE DETERMINED
Standard Penetration Test (SPT)	Sand	Coarse Gravel	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification.
Dynamic Cone Test	Sand and Gravel	Clay	Qualitative evaluation of compactness. Qualitative comparison of subsoil stratification.
Static Cone Test	Sand, Silt, and Clay	Coarse Gravel, Cemented Soil, Rock	Continuous evaluation of density and strength of sands. Continuous evaluation of undrained shear strength in clays.
Field Vane Test	Clay	All Other Soils	Undrained shear strength.
Pressuremeter Test	Soft Rock, Sand, Gravel, and Till	Soft Sensitive Clays	Bearing capacity and compressibility.
Plate Bearing Test and Screw Plate Test	Sand and Clay	-	Deformation modulus. Modulus of subgrade reaction. Bearing capacity.
Flat Plate Dilatometer Test	Sand and Clay	Gravel	Empirical correlation for soil type, K_e , overconsolidation ratio, undrained shear strength, and modulus.
Permeability Test	Sand and Gravel	-	Evaluation of coefficient of permeability.

2A.2.2 Explorations for Seismic Studies

In areas of high seismic activity (e.g., Seismic Detailing Requirement (SDR) 3 and above), special consideration shall be given to the seismic response of the site during the planning of field explorations. The planning process shall consider the potential for liquefaction and the requirement to determine the Site Class Definition, as required for establishing the Seismic Hazard Level and SDR. Articles 3.10.2.2 and 3.10.3 provides definitions for the Site Class Definition, Seismic Hazards Level, and SDR, respectively.

C.2A.2.2

Subsurface exploration methods in areas of high seismicity are generally the same as those used for standard subsurface explorations. However, the empirical correlations used to estimate the potential for liquefaction or the shear wave velocity of the soil normally require use of equipment that have been calibrated according to certain standards. The geotechnical engineer or engineering geologist responsible for having the subsurface explorations carried out should become familiar with these methods and confirm during the exploration program that correct methods and calibrated equipment are being used. If incorrect methods or uncalibrated equipment are used, it is possible to predict overly conservative or unconservative ground response for

a design seismic event.

2A.2.2.1 LIQUEFACTION POTENTIAL

Field explorations shall be performed to evaluate the potential for liquefaction in SDR 3, 4, 5, and 6 at those sites potentially susceptible to liquefaction. For sites that are potentially liquefiable, it is important to obtain an accurate determination of soil stratigraphy, the groundwater location, and the density of cohesionless soil. Of particular importance is the identification of thin layers that, if liquefied, could result in lateral flows or spreading of the soil above the liquefied layers.

C.2A.2.2.1

A potential for liquefaction exists if the following conditions are present: (1) the peak horizontal acceleration at the ground surface is predicted to be greater than 0.15g (g = acceleration of gravity); (2) the soil consists of loose to medium dense non-plastic silts, sands, and in some cases gravels; and (3) the permanent groundwater location is near the ground surface. Appendix B in Section 3 provides specific guidance on the determination and evaluation of liquefaction.

Depth of Exploration

The potential depth of liquefaction is an important decision. Normally, liquefaction is assumed to be limited to the upper 15 to 20 m of soil profile. However, it appears that this limiting depth is based on the observed depth of liquefaction rather than the maximum depth of liquefaction that is physically possible. For this reason an exploration program should extend at least to 25 m or until a competent bearing layer (with no underlying loose layers) is encountered, whichever occurs first.

Methods of Exploration

Several different exploration methods can be used to identify soils that could be susceptible to liquefaction. These include the Standard Penetration Test (SPT), the cone penetration test (CPT), and certain types of shear wave velocity measurements (e.g., crosshole, downhole, and SASW methods). ASTM standards exist for conducting SPTs, CPTs, and certain types of shear wave velocity (see Article 2A.2.1). These methods should be followed. If standards are not available, then it is essential to have testing completed by experienced individuals, who understand the limitations of the test methods and who understand the level of accuracy needed by the engineer for Site Class Definition or liquefaction determination.

Standard Penetration Test (SPT) Method: The SPT is currently the most common field exploration method for liquefaction studies. It is critical that if SPTs are conducted to obtain information for liquefaction assessments, procedures follow those recommended by Youd and Idriss (1997). These procedures have strict requirements for hammer energy, sampler size, and drilling method. If these methods are not followed, the value of the blow count determined from the SPT can vary by 100 percent, resulting in great uncertainty in any liquefaction assessment based on the SPT results. Recommended SPT procedures are summarized in Table 2A.2.2-1.

An automatic trip hammer should be used wherever possible; hammer energy calibrations should be obtained for the hammer, whether it is a donut hammer or an

automatic hammer. Records should also be available that indicate whether the SPT sampler used liners or not, and the type of drilling method that was used. It will usually be necessary to conduct the SPTs at close depth intervals, rather than the conventional 1.5-m interval, because thin liquefiable layers could be important to design.

Sites with gravel deposits require special consideration when performing SPTs. Because of the coarse size of gravel particles, relative to the size of the sampler, these deposits can result in misleadingly high blow counts. Three procedures can be considered for these sites:

- If a site has only a few gravel layers or if the gravel is not particularly abundant or large, it may be possible to obtain an equivalent SPT blow count if “incremental” blow counts are measured. To perform “incremental” blow count measurements, the number of blows for each 25 mm of penetration is recorded, rather than the blows for 150 mm. By plotting the blow counts per 25 mm versus depth, it is sometimes possible to distinguish between the blow count obtained in the matrix material and blow counts affected by large gravel particles. The equivalent blow count for 150 mm can then be estimated by summing and extrapolating the number of blows for the representative 25 mm penetrations that appear to be uninfluenced by coarse gravel particles. This procedure is described in Vallee and Skryness (1980).
- Andrus and Youd (1987) describe an alternate procedure for determining blow counts in gravel deposits. They suggest that the penetration per blow be determined and the cumulative penetration versus blow count be plotted. With this procedure, changes in slope can be identified when gravel particles interfere with penetration. From the slope of the cumulative penetration, estimates of the penetration resistance can be made where the gravel particles did or did not influence the penetration resistance.
- An alternative in gravel deposits is to obtain Becker Hammer blow counts, which have been correlated to the standard penetration test blow count (Youd and Idriss, 1997).

Cone Penetrometer Test (CPT) Method: For many locations the CPT is the preferred method of determining liquefaction potential. This method is preferred because it is able to provide an essentially continuous indication of soil consistency and type with depth. It is also less susceptible to operator-related differences in measurements. The CPT method may not be applicable at sites where cobbles and gravels overlie looser sandy soils. At these sites it may be impossible to push the CPT rod and sensor through the gravel. For these sites it is sometimes possible to auger through the gravel materials to provide access for the cone penetrometer rod and

Appendix 2A – Provisions for Site Characterization
(Subsurface Explorations, In Situ Testing, Laboratory Testing)

sensor.

Most CPT equipment are not capable of obtaining soil samples. Empirical correlations can, however, be used to estimate soil type and grain size. Although these correlations often provide very good indirect estimations of soil type and grain size, it is generally desirable to perform a limited number of SPTs at the site to obtain soil samples for laboratory determination of grain size, to confirm soil descriptions, and to provide a comparison to SPT blow counts.

Procedures for interpreting liquefaction resistance from the CPT measurement are given in Youd and Idriss (1997).

Shear Wave Velocity Methods: Shear wave velocity can also be used for both liquefaction evaluations and the determination of soil shear modulus, which is required when establishing spring constants for spread footing foundations. The shear wave velocity of the soil is also fundamental to the determination of Site Class Definition, as discussed in Article 3.10.2.2.1.

A variety of methods are available for making shear wave velocity measurements. They include downhole and crosshole methods which are performed in boreholes, seismic-cone methods which are conducted in conjunction with a CPT, and Spectral Analysis of Surface Wave (SASW) methods which are conducted from the ground surface without a borehole. Experienced individuals should perform these methods, as the collection and interpretation of results requires considered skill. In the absence of this experience, it is possible to obtain misleading results. Surface wave refraction procedures should not be used, as they are generally not able to obtain information in low-velocity layers. Additional information about the shear wave velocity can be found in Kramer (1996).

Procedures for interpreting liquefaction resistance from shear wave velocity data are discussed in Youd and Idriss (1997).

Table 2A.2.2-1 - Recommended SPT Procedure

Borehole size	66 mm < Diameter < 115 mm
Borehole support	Casing for full length and/or drilling mud
Drilling	Wash boring; side discharge bit Rotary boring; side or upward discharge bit Clean bottom of borehole*
Drill rods	A or AW for depths of less than 15 m N or NW for greater depths
Sampler	Standard 51 mm O.D. +/- 1 mm 35 mm I.D. +/- 1 mm >457 mm length

Appendix 2A – Provisions for Site Characterization
(Subsurface Explorations, In Situ Testing, Laboratory Testing)

Penetration Resistance	Record number of blows for each 150 mm; N = number of blows from 150 to 450 mm penetration
Blow count Rate	30 to 40 blows per minute

* Maximum soil heave within casing <70 mm

2A.2.2.2 SITE RESPONSE DETERMINATION

The field exploration shall provide sufficient information to determine the Site Class Definition (see Article 3.10.2.2.1), which is used to determine the Seismic Hazards Level.

C.2A.2.2.2

The Site Class Definition is used to determine whether amplification or de-amplification of ground motions occurs as earthquake-induced motions propagate from depth to the ground surface. Five general site classes have been defined (Article 3.10.2.2.1) for seismic studies. These categories generally require determination of soil properties in the upper 30 m of soil profile. Procedures for establishing the soil properties include the SPT, the shear wave velocity, and the strength of the material. It is important when planning the field explorations to recognize that this information could be important to a site and make explorations plans accordingly.

2A.3 LABORATORY TESTING

Laboratory tests shall be performed to determine the strength, deformation, and flow characteristics of soils and/or rocks and their suitability for the foundation selected. In areas of higher seismicity (e.g., SDR 3, 4, 5, and 6), it may be appropriate to conduct special dynamic or cyclic tests to establish the liquefaction potential or stiffness and material damping properties of the soil at some sites if unusual soils exist or if the foundation is supporting a critical bridge.

C.2A.3

An understanding of the engineering properties of soils is essential to the use of current methods for the design of foundations and earth structures. The purpose of laboratory testing is to provide the basic data with which to classify soils and to measure their engineering properties. The design values selected from the laboratory tests should be appropriate to the particular limit state and its correspondent calculation model under consideration.

For the value of each parameter, relevant published data together with local and general experience should be considered. Published correlations between parameters should also be considered when relevant.

2A.3.1 Standard Laboratory Tests

Laboratory soil tests may include:

- Water Content - ASTM D 4643
- Specific Gravity - AASHTO T 100 (ASTM D 854)
- Grain Size Distribution - AASHTO T 88 (ASTM D 422)
- Soil Compaction Testing – ASTM D 698 or D 1557
- Liquid Limit and Plastic Limit - AASHTO T 90 (ASTM D 4318)

C2A.3.1

Standard laboratory tests of soils may be grouped broadly into two general classes:

- Classification tests: These can be performed on either disturbed or undisturbed samples.
- Quantitative tests for permeability, compressibility, and shear strength. These tests are generally performed on undisturbed samples, except for materials to be placed as controlled fill or materials that do not have an unstable soil-structure. In these cases, tests should be performed on specimens prepared in the laboratory.

Appendix 2A – Provisions for Site Characterization
(Subsurface Explorations, In Situ Testing, Laboratory Testing)

- Direct Shear Test - AASHTO T 236 (ASTM D 3080)
- Unconfined Compression Test - AASHTO T 208 (ASTM D 2166)
- Unconsolidated-Undrained Triaxial Test - ASTM D 2850
- Consolidated-Undrained Triaxial Test - AASHTO T 297 (ASTM D 4767)
- Consolidation Test - AASHTO T 216 (ASTM D 2435 or D 4186)
- Permeability Test - AASHTO T 215 (ASTM D 2434)

A certain number of classification tests should be conducted at every bridge site; the number of quantitative tests will depend on the types of soils encountered. In many cases disturbance associated with the soil sampling process can limit the usefulness of quantitative test results. This is particularly the case for cohesionless soil. It can also occur for cohesive soil if high quality Shelby tube samples are not obtained. High quality sampling also requires careful sampling and careful soil setup once the sample is retrieved from the ground.

2A.3.2 Special Testing for Seismic Studies

For some important projects it may be necessary or desirable to conduct special soil laboratory tests to establish the liquefaction strength or stiffness and material damping properties of the soil. These tests can include resonant column, cyclic triaxial, and cyclic simple shear tests. Only a limited number of academic and consulting organizations are currently conducting these types of tests; therefore, special care is required when selecting a testing laboratory for these tests. Kramer (1996) provides a summary of the laboratory testing for determination of dynamic properties of soil.

2A.3.3 Rock Testing

Laboratory rock tests may include:

- Determination of Elastic Moduli - ASTM D 3148
- Triaxial Compression Test - AASHTO T 266 (ASTM D 2664)
- Unconfined Compression Test - ASTM D 2938
- Splitting Tensile Strength Test - ASTM D 3967

C.2A.3.2

For liquefaction assessments it is generally preferable to rely on in situ methods for determining the liquefaction strength of the soil, because of difficulties associated with sample disturbance. The exception to this general rule is for non-plastic silty soil, where the database for in situ-based correlations is not as well established. For these soils cyclic laboratory test may be necessary to estimate liquefaction strengths.

Empirical correlations have also been developed to define the effects of shearing strain amplitude and confining pressure on shear modulus and material damping of cohesionless and cohesive soils. Laboratory determination of these properties may be warranted where special soil conditions exist or where the stress state on the soil could change. Kramer (1996) provides a summary of the available methods for estimating shear modulus and material damping as a function of shearing strain amplitude and confining pressure.

C.2A.3.3

Laboratory testing of rock has very limited applicability for measuring significant rock properties, such as:

- Compressive strength,
- Shear strength,
- Hardness,
- Compressibility, and
- Permeability.

Rock samples small enough to be tested in the laboratory are usually not representative of the entire rock mass. Laboratory testing of rock is used primarily for classification of intact rock samples, and, if performed properly, serves a useful function in this regard.

Laboratory tests on intact samples provide upper bounds on strength and lower bounds on compressibility. Frequently, laboratory tests can be used in conjunction with field tests to give reasonable estimates of rock mass behavioral characteristics.

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3.2 DEFINITIONS

Capacity Design – A method of component design that allows the designer to prevent damage in certain components by making them strong enough to resist loads that are generated when adjacent components reach their overstrength capacity.

Capacity Spectrum Design – SDAP C – A design and analysis procedure that combines a demand and capacity analysis (See Article 3.10.3.4.1)

Collateral Seismic Hazard – Seismic hazards other than direct ground shaking such as liquefaction, fault rupture, etc.

Complete Quadratic Combination (CQC) – A statistical rule for combining modal responses from an earthquake load applied in a single direction to obtain the maximum response due to this earthquake load.

Damage Level – A measure of seismic performance based on the amount of damage expected after one of the design earthquakes.

Displacement Capacity Verification – SDAP E – A design and analysis procedure that requires the designer to verify that his or her structure has sufficient displacement capacity. It generally involves a non-linear static (i.e. “pushover”) analysis.

Earthquake Resisting System – A system that provides a reliable and uninterrupted load path for transmitting seismically induced forces into the ground and sufficient means of energy dissipation and/or restraint to reliably control seismically induced displacements.

Expected Earthquake – The largest earthquake that is likely to occur during the life of a bridge. It has a 50 percent chance of being exceeded during a 75 year period.

Lateral Ground Movement – Seismically induced permanent horizontal ground movement

Life Safety Performance Level – The minimum acceptable level of seismic performance allowed by this specification. It is intended to protect human life during and following a rare earthquake.

Liquefaction – Seismically induced loss of shear strength in loose, cohesionless soil that results from a build up of pore pressure as the soil tries to consolidate when exposed to seismic vibrations.

Maximum Considered Earthquake – The upper level, or rare, design earthquake that has a 3 percent chance of being exceeded in 75 years.

Minimum Seat Width – The minimum prescribed width of a bearing seat that must be provided in a new bridge designed according to these specifications.

Operational Performance Level – A higher level of seismic performance that may be selected by a bridge owner who wishes to have immediate service and minimal damage following a rare earthquake.

Overstrength Capacity – The maximum expected force or moment that can be developed in a yielding structural element assuming overstrength material properties and large strains and associated stresses.

Performance Criteria – The levels of performance in terms of post earthquake service and damage that are expected to result from specified earthquake loadings if bridges are designed according to this specification.

Plastic Hinge – The region of a structural component, usually a column or a pier in bridge structures, that undergoes flexural yielding and plastic rotation while still retaining sufficient flexural strength.

Plastic Hinge Zone – Those regions of structural components that are subject to potential plastification and thus must

SECTION 3 – LOADS AND LOAD FACTORS

be detailed accordingly.

Rare Earthquake – The upper level design event, or maximum considered earthquake. It has a 3 percent probability of being exceeded during a 75 year period.

Response Modification Factor – Factors used to modify the element moment demands from an elastic analysis to account for ductile behavior and obtain design moment demands.

Seismic Design and Analysis Procedure (SDAP) – One of five defined procedures for conducting seismic design and analysis. Minimum requirements are based on seismic hazard level, performance objective, structural configuration, and the type of ERS and/or ERE's.

Seismic Detailing Requirements (SDR) – One of six categories of minimum detailing requirements based on the seismic hazard level and the performance objective.

Seismic Hazard Level – One of four levels of seismic ground shaking exposure measured in terms of the rare earthquake design spectral accelerations for 0.2 and 1.0 seconds.

Service Level – A measure of seismic performance based on the expected level of service that the bridge is capable of providing after one of the design earthquakes.

Site Class – One of six classifications used to characterize the effect of the soil conditions at a site on ground motion.

Square Root of the Sum of the Squares (SRSS) Combination – In this specification, this classical statistical combination rule is used in two ways. The first is for combining forces resulting from two or three orthogonal ground motion components. The second use is for establishing orthogonal moments for biaxial design.

Tributary Weight – The portion of the weight of the superstructure that would act on a pier participating in the ERS if the superstructure between participating piers consisted of simply supported spans. A portion of the weight of the pier itself may also be included in the tributary weight.

3.3 NOTATION

3.3.1 General

A_g	=	gross cross-sectional area of column
C_s	=	seismic coefficient
C_{sm}	=	elastic seismic response coefficient for the m^{th} mode of vibration
C_v	=	dead load multiplier coefficient for vertical earthquake effects
D	=	transverse dimension of a column or pile
D_p	=	pile dimension about the weak axis at ground line
d_b	=	longitudinal reinforcing bar diameter
d_c	=	total thickness of cohesive soil at a site
d_i	=	thickness of soil layer "i"
d_s	=	total thickness of cohesionless soil at a site
F_a	=	site coefficient for short-period portion of design response spectrum curve
F_v	=	site coefficient for long-period portion of design response spectrum curve
M_n	=	nominal moment capacity of a column
M_{po}	=	plastic overstrength capacity of a column

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M_X	=	maximum moment about the “x” axis due to earthquake load applied in all directions
M_X^L	=	maximum moment about the “x” axis due to earthquake load applied in the longitudinal direction
M_X^{LC1}	=	maximum moment about the “x” axis due to earthquake load case 1
M_X^{LC2}	=	maximum moment about the “x” axis due to earthquake load case 2
M_X^T	=	maximum moment about the “x” axis due to earthquake load applied in the transverse direction
M_Y	=	maximum moment about the “y” axis due to earthquake load applied in all directions
M_Y^L	=	maximum moment about the “y” axis due to earthquake load applied in the longitudinal direction
M_Y^{LC1}	=	maximum moment about the “y” axis due to earthquake load case 1
M_Y^{LC2}	=	maximum moment about the “y” axis due to earthquake load case 2
M_Y^T	=	maximum moment about the “y” axis due to earthquake load applied in the transverse direction
$\bar{N}$	=	average standard penetration test blow count for the top 100 ft (30 m) of a site
$\bar{N}_{ch}$	=	average standard penetration test blow count for cohesionless layers of top 100 ft (30 m) of a site
N	=	minimum seat width
N_i	=	standard penetration test blow count of soil layer “i”
PI	=	plasticity index of soil
P_C	=	axial compression capacity of timber pile
P_e	=	column axial load
P_y	=	axial yield force of steel pile
Q	=	total factored force effect
Q_i	=	force effect from specified load
R	=	response modification factor
R_B	=	base response modification factor
R_d	=	ratio of estimated actual displacement to displacement determined from elastic analysis
S_a	=	design response spectral acceleration
S_{DS}	=	design earthquake response spectral acceleration at short periods
S_{DI}	=	design earthquake response spectral acceleration at 1 second period
S_s	=	0.2-second period spectral acceleration on Class B rock from national ground motion maps
S_1	=	1-second period spectral acceleration on Class B rock from national ground motion maps
$\bar{S}_u$	=	average undrained shear strength of cohesive layers in the top 100 ft (30 m) of a site
T	=	period of vibration
T_s	=	period at the end of constant design spectral acceleration
T_0	=	period at beginning of constant design spectral acceleration
T^*	=	period used to calculate R and R_d
t	=	thickness of pier wall
$\bar{V}_s$	=	average shear wave velocity for the top 100 ft (30 m) of a site
V_{si}	=	shear wave velocity of soil layer “i”
w	=	moisture content in percent
q	=	principal crack angle in reinforced concrete column
q_p	=	plastic rotation at a plastic hinge

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a_{skew}	=	skew angle of the bridge, (0 degrees being the angle for a right bridge)
e_y	=	yield strain of longitudinal reinforcement
r_ℓ	=	longitudinal reinforcement ratio of a column or pier
Δ	=	displacement from an elastic seismic analysis
Δ_m	=	estimated actual displacement at the center of mass

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3.3.2 Load and Load Designation

The following permanent and transient loads and forces shall be considered:

- Permanent Loads

DD = downdrag
DC = dead load of structural components and nonstructural attachments
DW = dead load of wearing surfaces and utilities
EL = accumulated locked-in force effects resulting from the construction process
EH = horizontal earth pressure load
ES = earth surcharge load
EV = vertical pressure from dead load of earth fill

- Transient Loads

BR = vehicular braking force
CE = vehicular centrifugal force
CR = creep
CT = vehicular collision force
CV = vessel collision force
EQ = earthquake
FR = friction
IC = ice load
IM = vehicular dynamic load allowance
LL = vehicular live load
LS = live load surcharge
PL = pedestrian live load
SE = settlement
SH = shrinkage
TG = temperature gradient
TU = uniform temperature
WA = water load and stream pressure
WL = wind on live load
WS = wind load on structure

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3.4 LOAD FACTORS AND COMBINATIONS

3.4.1 Load Factors and Load Combinations

The total factored force effect shall be taken as:

$$Q = \sum h_i g_i Q_i \quad (3.4.1-1)$$

where:

h_i = load modifier specified in Article 1.3.2

Q_i = force effects from loads specified herein

g_i = load factors specified in Tables 1 and 2

Components and connections of a bridge shall satisfy Equation 1.3.2.1-1 for the applicable combinations of factored extreme force effects as specified at each of the following limit states:

-
- EXTREME EVENT I - Load combination including rare and expected earthquakes.
 - EXTREME EVENT II - Load combination relating to ice load, collision by vessels and vehicles, and certain hydraulic events with a reduced live load other than that which is part of the vehicular collision load, CT.
-

C3.4.1

The background for the load factors specified herein, and the resistance factors specified in other sections of these specifications is developed in Nowak (1992).

This limit state includes water loads, WA. The probability of a major flood and an earthquake occurring at the same time is very small. Therefore, consideration of basing water loads and scour depths on mean discharges may be warranted. Live load coincident with an earthquake is discussed elsewhere in this article.

The recurrence interval of extreme events is thought to exceed the design life.

The joint probability of these events is extremely low, and, therefore, the events are specified to be applied separately. Under these extreme conditions, the structure is expected to undergo considerable inelastic deformation by which locked-in force effects due to TU, TG, CR, SH, and SE are expected to be relieved.

The 0.50 live load factor signifies a low probability of the concurrence of the maximum vehicular live load (other than CT) and the extreme events.

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Table 3.4.1-1 - Load Combinations and Load Factors

Load Combination Limit State	DC DD DW EH EV ES	LL IM CE BR PL LS EL	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These at a Time			
										EQ	IC	CT	CV
STRENGTH-I (unless noted)	g_p	1.75	1.00	-	-	1.00	0.50/1.20	g_{TG}	g_{SE}	-	-	-	-
STRENGTH-II	g_p	1.35	1.00	-	-	1.00	0.50/1.20	g_{TG}	g_{SE}	-	-	-	-
STRENGTH-III	g_p	-	1.00	1.40	-	1.00	0.50/1.20	g_{TG}	g_{SE}	-	-	-	-
STRENGTH-IV EH, EV, ES, DW DC ONLY	g_p 1.5	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
STRENGTH-V	g_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	g_{TG}	g_{SE}	-	-	-	-
EXTREME EVENT-I	1.00	g_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
EXTREME EVENT-II	g_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
SERVICE-I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	g_{TG}	g_{SE}	-	-	-	-
SERVICE-II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
SERVICE-III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	g_{TG}	g_{SE}	-	-	-	-
FATIGUE-LL, IM & CE ONLY	-	0.75	-	-	-	-	-	-	-	-	-	-	-

The load factor for live load in Extreme Event Load Combination I, γ_{EQ} , shall be determined on a project specific basis. The inertia effects of live load do not need to be considered when performing a dynamic analysis.

It is generally not necessary to consider the gravity effects of live load for Extreme Event I except for bridges with heavy truck traffic (i.e. high ADTT) and/or elements particularly sensitive to gravity loading such as C-bents, outrigger bents or superstructures with nonsymmetrical geometry. Because of the difficulty in predicting the partial live load to be applied with earthquake, and the probability that this live load will be significantly below the AASHTO design live load, it is acceptable to use γ_{EQ} values with live load effect envelopes or the AASHTO lane loading. Universally acceptable methods for determining values for γ_{EQ} have not been established, but values between 0.25 and 0.40 have been suggested for use in design.

A load factor for passive earth pressure is not given in Table 2 because, strictly speaking, passive earth pressure is a resistance and not a load. For discussion of the selection of a passive earth pressure resistance factor see Article C10.5.4.

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3.10 EARTHQUAKE EFFECTS: EQ

3.10.1 General

C3.10.1

3.10.1.1 APPLICABILITY

C3.10.1.1

The provisions herein shall apply to bridges of conventional slab, beam girder, box girder, and truss superstructure construction. For other types of construction (i.e. cable stayed and suspension), the Owner shall specify and/or approve appropriate provisions. Unless otherwise specified by the Owner, these provisions need not be applied to completely buried structures.

Seismic effects for box culverts and buried structures need not be considered, except where they cross active faults.

The potential for soil liquefaction and slope movements shall be considered.

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3.10.1.2 DESIGN EARTHQUAKES AND SEISMIC PERFORMANCE OBJECTIVES

Bridges shall be designed to satisfy the performance criteria given in Table 3.10.1-1. As a minimum, bridges shall be designed for the life safety level of performance. Higher levels of performance may be required at the discretion of the bridge owner. Development of design earthquake ground motions for the probabilities of exceedance in Table 3.10.1-1 are given in Article 3.10.2.

When required by the provisions of this specification, seismic performance shall be assured by verifying that displacements are limited to satisfy geometric, structural and foundation constraints on performance.

COMMENTARY

C3.10.1.2

The design earthquake ground motions and forces specified herein are based on the probabilities of exceedance stated in Table 3.10.1-1 for a nominal life expectancy of a bridge of 75 years. As a minimum these specifications are intended to achieve minimal damage to the bridge during expected ground motions during the life of the bridge; and to prevent collapse during rare ground motions. Bridge owners may choose to mandate higher levels of bridge performance.

For sites close to highly active faults, the upper-level earthquake ground motions (Maximum Considered Earthquake or MCE) defined probabilistically can reach values that exceed ground motions estimated deterministically for the maximum magnitude earthquake considered capable of occurring on the fault. For such sites, it is considered reasonable to limit or bound the design ground motions to conservative deterministic estimates of the ground motion for the maximum magnitude earthquake. As indicated in the footnote to Table 3.10.1-1, deterministic bounds on ground motions have been incorporated on MCE maps where applicable (Hamburger and Hunt, 1997; BSSC, 1998; Leyendecker et al., 2000). These bounds were defined as 1.5 times the median ground motions calculated using appropriate attenuation relationships assuming the occurrence of the maximum magnitude earthquake on the fault, but not less than 1.5g for the short-period acceleration plateau (S_s) and 0.6g for 1.0-second spectral acceleration (S_1). The magnitude of a maximum earthquake is the best estimate of the largest magnitude considered capable of occurring on the fault. On the current MCE maps, deterministic bounds are applied only in portions of California, in local areas along the California-Nevada border, along coastal Oregon and Washington, and in portions of Alaska and Hawaii.

Probabilistic ground motions developed for MCE ground motion maps by the USGS were actually calculated for a probability of exceedance of 2% in 50 years. These ground motion values are nearly identical to ground motions for 3% probability of exceedance in 75 years because the corresponding ground motion return periods are nearly the same (2475 year return period for 2% probability of exceedance in 50 years and 2462 years return period for 3% probability in 75 years). Therefore, the map values may be taken as the ground motions for 3% probability of exceedance in 75 years.

Allowable displacements are constrained by geometric, structural and geotechnical considerations. The most restrictive of these constraints will govern displacement capacity. These displacement constraints may apply to either transient displacements as would

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occur during ground shaking, or permanent displacements as may occur due to seismically induced ground failure or permanent structural deformations or dislocations, or both. The magnitude of allowable displacements depends on the desired performance level of the bridge design. The following paragraphs discuss the geometric constraints that should be considered in establishing displacement capacities. It should be noted that these recommendations are order of magnitude values and are not meant to be precise. Structural and geotechnical constraints are discussed in Sections 5, 6, 10 and 11.

Allowable displacements shown in Table C3.10.1.2 were developed at a Geotechnical Performance Criteria Workshop conducted on September 10 & 11, 1999 as an extension of the NCHRP 12-49 project. The original intent of the workshop was to develop detailed foundation displacement criteria based on geotechnical constraints. The final recommendation of the workshop was that, except in special circumstances, foundations are able to accommodate large displacements without strength degradation and that displacement capacities are usually constrained by either structural or geometric considerations. The values in the table reflect geometric constraints and are based largely on judgment that represents the consensus opinion of the workshop participants.

Geometric constraints generally relate to the usability of the bridge by traffic passing on or under it. Therefore, this constraint will usually apply to permanent displacements that occur as a result of the earthquake. The ability to repair, or the desire not to be required to repair, such displacements should be considered when establishing displacement capacities. When uninterrupted or immediate service is desired, the permanent displacements should be small or non-existent, and should be at levels that are within an accepted tolerance for normally operational highways of the type being considered. A guideline for determining these displacements should be the AASHTO publication "A Policy on Geometric Design of Highways and Streets". When limited service is acceptable, the geometric constraints may be relaxed. These may be governed by the geometry of the types of vehicles that will be using the bridge after an earthquake and by the ability of these vehicles to pass through the geometric obstruction. Alternately, a jurisdiction may simply wish to limit displacements to a multiple of those allowed for uninterrupted service. In the case of a no collapse performance objective, when liquefaction occurs, post earthquake use of the bridge is not guaranteed and therefore no geometric constraints would be required to achieve these goals. However, because life safety is at the heart of the no collapse requirement, jurisdictions may consider establishing some geometric

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displacement limits for this performance level for important bridges or those with high ADT. This can be done by considering the risk to highway users in the moments during or immediately following an earthquake. For example, an abrupt vertical dislocation of the highway of sufficient height could present an insurmountable barrier and thus result in a head-on type collision that could kill or severely injure occupants of the vehicle. Usually these types of geometric displacement constraints will be less restrictive than those resulting from structural considerations and for bridges on liquefied sites it may not be economic to prevent significant displacements from occurring. Table C3.10.1-2 shows the order of magnitude of suggested displacement limits based on geometric constraints.

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Table 3.10.1-1 Design Earthquakes and Seismic Performance Objectives

Probability of Exceedance For Design Earthquake Ground Motions ⁽⁴⁾		Performance Level ⁽¹⁾	
		Life Safety	Operational
Rare Earthquake (MCE) 3% in 75 years	Service ⁽²⁾	Significant Disruption	Immediate
	Damage ⁽³⁾	Significant	Minimal
Expected Earthquake 50% in 75 years	Service	Immediate	Immediate
	Damage	Minimal	Minimal to None

Notes:

(1) Performance Levels

These are defined in terms of their anticipated performance objectives in the upper level earthquake. Life safety in the MCE event means that the bridge should not collapse but partial or complete replacement may be required. Since a dual level design is required the Life Safety performance level will have immediate service and minimal damage for the expected design earthquake. For the operational performance level the intent is that there will be immediate service and minimal damage for both the rare and expected earthquakes.

(2) Service Levels*:

- *Immediate* – Full access to normal traffic shall be available following an inspection of the bridge.
- *Significant Disruption* – Limited access (Reduced lanes, light emergency traffic) may be possible after shoring, however the bridge may need to be replaced.

(3) Damage Levels*:

- None – Evidence of movement may be present but no notable damage.
- Minimal – Some visible signs of damage. Minor inelastic response may occur, but post-earthquake damage is limited to narrow flexural cracking in concrete and the onset of yielding in steel. Permanent deformations are not apparent, and any repairs could be made under non-emergency conditions with the exception of superstructure joints.
- Significant – Although there is no collapse, permanent offsets may occur and damage consisting of cracking, reinforcement yield, and major spalling of concrete and extensive yielding and local buckling of steel columns, global and local buckling of steel braces, and cracking in the bridge deck slab at shear studs on the seismic load path is possible. These conditions may require closure to repair the damage. Partial or complete replacement of columns may be required in some cases. For sites with lateral flow due to liquefaction, significant inelastic deformation is permitted in the piles, whereas for all other sites the foundations are capacity-protected and no damage is anticipated. Partial or complete replacement of the columns and piles may be necessary if significant lateral flow occurs. If replacement of columns or other components is to be avoided, the design approaches producing minimal or moderate damage (Figure 2.5.6-1) such as seismic isolation or the control and reparability design concept should be assessed.

- * See commentary and design sections for geometric and structural constraints on displacements and deformations.

(4) The upper-level earthquake considered in these provisions is designated the Maximum Considered Earthquake, or MCE. In general the ground motions on national MCE ground motion maps have a probability of exceedance of

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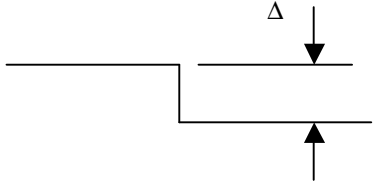
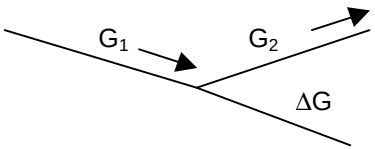
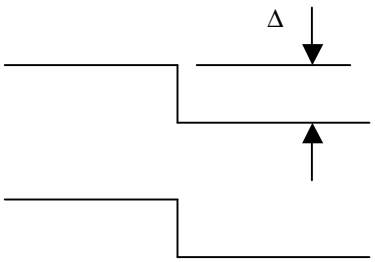
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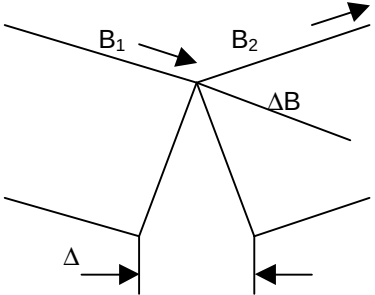
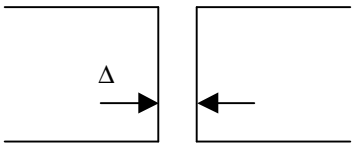
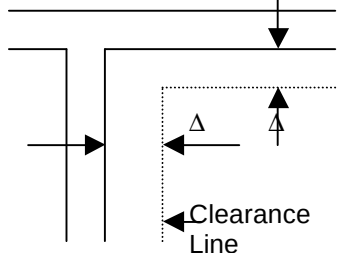
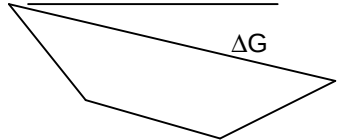
approximately 3% in 75 years. However, adjacent to highly active faults, ground motions on MCE maps are bounded deterministically as described in the commentary for Article 3.10.1.2. When bounded deterministically, MCE ground motions have a probability of exceedance higher than 3% in 75 years. The performance objective for the expected earthquake is either explicitly included as an elastic design for the 50% in 75 year force level or results implicitly from design for the 3% in 75 year force level.

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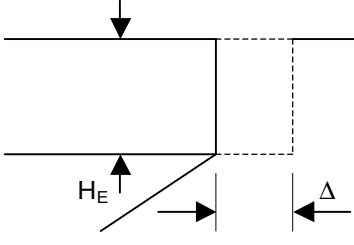
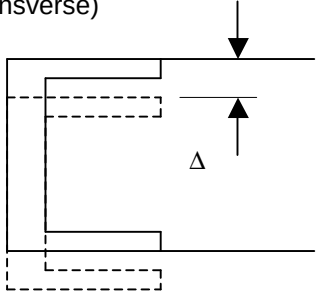
Table C3.10.1-2 Geometric Constraints on Service Level

Permanent Displacement Type	Possible Causes	Mitigation Measures	Immediate	Significant Disruption
Vertical Offset 	- Approach fill settlement - Bearing failure	- Approach slabs - Approach fill stabilization - Bearing type selection	0.083 feet (0.03 meters)	0.83 feet (0.2 meters) To avoid vehicle impact
Vertical Grade Break (2) 	- Interior support settlement - Bearing failure - Approach slab settlement	- Strengthen foundation - Bearing type selection - Longer approach slab	Use AASHTO "Green Book" requirements to estimate allowable grade break	None
Horizontal Alignment Offset 	- Bearing failure - Shear key failure - Abutment foundation failure	- Bearing type selection - Strengthen shear key - Strengthen foundation	0.33 feet (0.1 meters) Joint seal may fail	Shoulder Width (To avoid vehicle impact)

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Horizontal Alignment Break (3) 	- Interior support failure - Bearing failure - Lateral foundation movement	- Strengthen interior support - Bearing type selection - Strengthen foundation	Use AASHTO "Green Book" requirements to estimate allowable alignment break	None $\Delta = 3.28$ feet (1.0 meters)
Longitudinal Joint Opening 	- Interior support failure - Bearing failure - Lateral foundation movement	- Strengthen interior support - Bearing type selection - Strengthen foundation	0.33 feet (0.1 meters)	3.28 feet (1.0 meters) To avoid vehicle impact
Encroachment on Clearance 	- Foundation settlement - Lateral foundation movement - Bearing failure	- Strengthen foundation - Bearing type selection	Δ (Actual Clearance)	Depends on facility being encroached upon
Tilting of Cross-Section 	- Interior support settlement - Bearing failure - Approach slab settlement	- Strengthen foundation - Bearing type selection - Longer approach slab	$\Delta G = .001$ radians	None

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Movement into Abutment Fill (Longitudinal) 	Engagement of abutment backfill due to horizontal movement of superstructure	- Increase gap between superstructure and abutment backwall - Stiffen interior supports - Increase amount of fill that is engaged	$\Delta = .02H_E$	No Constraint Controlled by Adjacent Seat Width
Movement through Abutment Fill (Transverse) 	Transverse movement of strengthened or supplemental interior wingwalls through approach fill	- Isolate transverse movement with sacrificial shear keys and/or isolation bearings - Increase transverse strength and stiffness of abutment	$\Delta = .02H_E$	No Constraint

Notes:

- Geometric constraints, with the exception of longitudinal and transverse movement through abutment fill, usually apply to permanent displacements which may be difficult to predict accurately. Therefore, the constraints in this table shall be taken as order of magnitude values.
- The AASHTO publication "A Policy on Geometric Design of Highways and Streets" (otherwise known as the "Green Book") specifies criteria for determining vertical curve length based on site distance. This criteria, which is based on design speed and whether the curve is a "crest" or a "sag" can be used to determine the allowable change in grade resulting from support settlement. A curve length equal to the sum of adjacent spans may be used in the case of a continuous superstructure or a zero curve length may be used in the case of adjacent simply supported span lengths. Bridge owners may also wish to consider the AASHTO recommendations on appearance and driver comfort in establishing allowable grade changes.
- In the case of horizontal curves, minimum curve radius is usually controlled by superelevation and side friction. These radii are specified in the AASHTO "Green Book". When lateral displacement of an interior support results in an abrupt angle break in horizontal alignment a vehicle shall be able to safely achieve the desired turning radius at design speed within the provided lane width minus a margin of safety at each edge of the lane. Consideration shall also be given to the opening of the expansion joint at the edge of the bridge.
- Joint seals may be damaged at the immediate service level. If no damage at the seal is desired the designer should check the actual longitudinal and transverse capacity or reduce some of the permissible movements.

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3.10.2 Design Ground Motion

Design response spectra acceleration parameters shall be obtained using either a general procedure (Article 3.10.2.1) or a site-specific procedure (Article 3.10.2.3). A site-specific procedure shall be used if any of the following apply:

Soils at the site require site-specific evaluation (i.e. Site Class F soils, Article 3.10.2.2.1), unless a determination is made that the presence of such soils would not result in a significantly higher response of the bridge.

The bridge is considered to be a major or very important structure for which a higher degree of confidence of meeting the seismic performance objectives of Article 3.10.1.2 is desired.

- The site is located within 10 km (6.25 miles) of a known active fault and its response could be significantly and adversely influenced by near-fault ground motion characteristics.

3.10.2.1 DESIGN RESPONSE SPECTRA BASED ON GENERAL PROCEDURE

Design response spectra for the rare earthquake (MCE) and expected earthquake shall be constructed using the accelerations from national ground motion maps described

C3.10.2

Using either the general procedure or the site-specific procedure, a decision as to whether the design motion is defined at the ground surface or some other depth needs to be made as an initial step in the design process. Article 3.10.2.2.2 provides a commentary on this issue.

Examples of conditions that could lead to a determination that Site Class F soils would not result in a significantly higher bridge response are (1) localized extent of Site Class F soils and (2) limited depth of soft soils. (1) As discussed in Commentary to Article 3.10.2.3.2, for short bridges with a limited number of spans and having earth approach fills, ground motions at the abutments will generally principally determine the response of the bridge. If Site Class F soils are localized to the interior piers and are not present at the abutments, the bridge engineer and geotechnical engineer might conclude that the response of interior piers would not significantly affect bridge response. (2) Commentary to Article 3.10.2.3.2 also describes cases where the effective depth of input ground motion is determined to be in stiffer soils at depth, below a soft surficial layer. If the surficial layer results in a classification of Site Class F and the underlying soil profile classifies as Site Class E or stiffer, a determination might be made that the surficial soils would not significantly increase bridge response.

For purposes of these specifications, an active fault is defined as a fault having a location that is known or can reasonably be inferred and has exhibited evidence of displacement in Holocene time (past approximately 11,000 years). Active fault locations can be determined from maps showing active faults prepared by state geological agencies or the U.S. Geological Survey. Article C.3.10.2.2 describes near-fault ground motion effects that are not included in national ground motion mapping and could potentially increase the response of some bridges. Normally, site specific evaluation of these effects would be considered only for major or very important bridges.

C3.10.2.1

National ground motion maps described in this specification are based on probabilistic national ground motion mapping conducted by the U.S. Geological Survey

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in this section and site factors described in Section 3.10.2.2. The construction of the response spectra shall follow the procedures described below and illustrated in Figure 3.10.2.1-3.

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(USGS) and, in California, as a joint effort between the USGS and the California Division of Mines and Geology (CDMG) (Frankel et al., 1996; 1997a; 1997b; 1997c; 2000; Klein et al., 1999; Peterson et al. 1996; Wessen et al., 1999a; 1999b). As described in Commentary to Article 3.10.1.2, maps for the rare earthquake (MCE) are for a probability of exceedance of 3% in 75 years but are bounded deterministically near highly active faults. These maps were originally published in the 1997 edition of the NEHRP Provisions (BSSC, 1998), and subsequently in the 2000 edition of the International Building Code (ICC, 2000). The development of the MCE maps is described in BSSC (1998), Hamburger and Hunt (1997), and Leyendecker et al. (2000b). Ground motions for the expected earthquake are for a probability of exceedance of 50% in 75 years. Paper maps for the expected earthquake have not been prepared as of February, 2001; however map values at any location may be obtained by interpolation from the seismic hazard curves on the CD-ROM published by the USGS (Frankel and Leyendecker, 2000)).

In lieu of using national ground motion maps referenced in this Specification, ground motion response spectra may be constructed based on approved state ground motion maps. To be accepted, the development of state maps should conform to the following:

1. The definition of design ground motions should be the same as described in Article 3.10.1.2 and Table 3.10.1-1.
2. Ground motion maps should be based on a detailed analysis demonstrated to lead a quantification of ground motion at a regional scale that is as or more accurate than achieved at the scale of the national maps. The analysis should include: characterization of seismic sources and ground motion that incorporates current scientific knowledge; incorporation of uncertainty in seismic source and ground motion models and parameter values used in the analysis; detailed documentation of map development; detailed peer review. The peer review process should preferably include one or more individuals from the U.S. Geological Survey who participated in the development of the national maps.

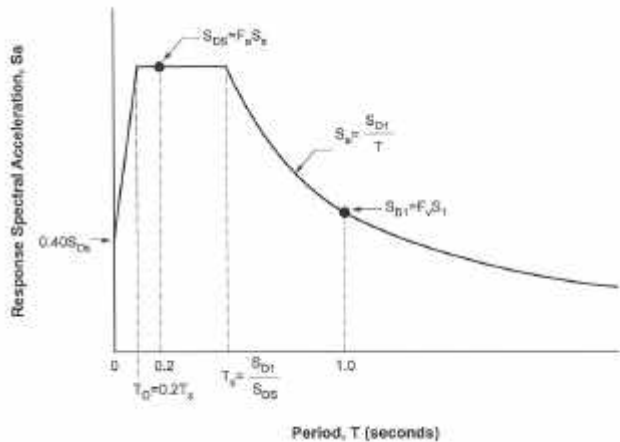


Figure 3.10.2.3-1 Design Response Spectrum, Construction Using Two-Point Method

Design earthquake response spectral acceleration at short periods, S_{DS} , and at 1 second period, S_{D1} , shall be determined from Eq. 3.10.2.1-1 and 3.10.2.1-2, respectively:

$$S_{DS} = F_a S_s \quad (3.10.2.1-1)$$

and

$$S_{D1} = F_v S_1 \quad (3.10.2.1-2)$$

where S_s and S_1 are the 0.2-second period spectral acceleration and 1-second period spectral acceleration, respectively, on Class B rock from ground motion maps described below and F_a and F_v are site coefficients described in Article 3.10.2.2.3. Values of S_s and S_1 may be obtained by the following methods:

1. For the MCE
 - (a) S_s and S_1 may be obtained from national ground motion maps (Figures 3.10.2.1-1(a) through 3.10.2.1-1(l) located at the end of Section 3). Large scale MCE maps may be obtained from the United States Geological Survey, Golden, Colorado
 - (b) S_s and S_1 may be obtained from the CD-ROM published by the U.S. Geological Survey (Leyendecker et al., 2000a) for site coordinates specified by latitude and longitude, or alternatively, by zip code.
2. For the expected earthquake, S_s and S_1 may be obtained by linear interpolation from hazard curves on the CD-ROM published by the U.S. Geological

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Survey (Frankel and Leyendecker, 2000) for site coordinates specified by latitude and longitude or alternatively by zip code.

The design response spectrum curve shall be developed as indicated in Figure 3.10.2.1-3 and as follows:

1. For periods less than or equal to T_0 , the design response spectral acceleration, S_a , shall be defined by Equation 3.10.2.1-3:

$$S_a = 0.60 \frac{S_{DS}}{T_0} T + 0.40 S_{DS} \quad (3.10.2.1-3)$$

T and T_0 are defined in 2. below.

Note that for $T = 0$ seconds, the resulting value of S_a is equal to peak ground acceleration, PGA.

2. For periods greater than or equal to T_0 and less than or equal to T_s , the design response spectral acceleration, S_a , shall be defined by Equation 3.10.2.1-4:

$$S_a = S_{DS} \quad (3.10.2.1-4)$$

where $T_0 = 0.2T_s$, and $T_s = S_{D1}/S_{DS}$, and
 T = period of vibration (sec).

3. For periods greater than T_s , the design response spectral acceleration, S_a , shall be defined by Equation 3.10.2.1-5:

$$S_a = \frac{S_{D1}}{T} \quad (3.10.2.1-5)$$

Response spectra constructed using maps and procedures described in Article 3.10.2.1 are for a damping ratio of 5%.

For single mode method of analysis, Equations 3.10.2.1-1, -2, -4, and -5 may be used to calculate C_s

(i.e. $C_{sm} = \frac{S_{D1}}{T} \leq S_{DS}$) where $S_{DS} = S_a = C_s$ for

$T \leq T_s$ (i.e no reduction in S_a for $T < T_0$ as permitted by Equation 3.10.2.1-3)

For periods exceeding approximately 3 seconds, depending on the seismic environment, Equation 3.10.2.1-5 may be conservative because the ground motions may be approaching the constant spectral displacement range for which S_a decays with period as $1/T^2$. Equation 3.10.2.1-5 should be used unless a more appropriate long-period spectrum decay is determined based on a site specific study.

3.10.2.2 SITE EFFECTS ON GROUND MOTIONS

The generalized site classes and site factors described in this section shall be used with the general procedure for constructing response spectra described in Article 3.10.2.1. Site-specific analysis of soil response effects shall be conducted where required by Article 3.10.2 and in accordance with the requirements in Article 3.10.2.2.

C3.10.2.2

The site classes and site factors described in this article were originally recommended at a site response workshop in 1992 (Martin, ed., 1994). Subsequently they were adopted in the 1994 and 1997 NEHRP Provisions (BSSC, 1995, 1998), the 1997 Uniform Building Code (UBC) (ICBO, 1997), the Seismic Design Criteria of Caltrans (1999), and the 2000 International Building Code (IBC) (ICC, 2000). The basis for the adopted site classes and site factors are described by Martin and Dobry (1994), Rinne (1994), and Dobry et al. (2000).

Procedures described in this Article were originally developed for computing ground motions at the ground surface for relatively uniform site conditions. Depending on the site classification and the level of the ground motion, the motion at the surface could be different than the motion at depth. This creates some question as to the location of the motion to use in the bridge design. It is also possible that the soil conditions differ between abutments or between abutments and central piers. An example would be where one abutment is on firm ground or rock and the other is on a loose fill. These variations are not always easily handled by simplified procedures described in this commentary. For critical bridges it may be necessary to use more rigorous numerical modeling to represent these conditions. The decision to use more rigorous numerical modeling should be made after detailed discussion of the benefits and limitations of more rigorous modeling by the bridge and geotechnical engineer.

Geologic Differences

If geotechnical conditions at abutments and intermediate piers result in different soil classifications, then response spectra should be determined for each abutment and pier having a different site classification. The design response spectra may be taken as the envelope of the individual spectra. However, if it is assessed that the bridge response is dominated by the abutment ground motions, only the abutment spectra need be enveloped (Section C3.10.2.2.2).

3.10.2.2.1 Site Class Definitions

The site shall be classified as one of the following classes according to the average shear wave velocity, SPT blow count (N-value), or undrained shear strength in the upper 30 m (100 ft) of site profile. Procedures given in Article 3.10.2.2.2 shall be used to determine the average condition.

C3.10.2.2.1

Steps for Classifying a Site (also see Table 3.10.2.2.1-1 below):

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- A Hard rock with measured shear wave velocity, $\bar{V}_S > 1500$ m/s (5000 ft/sec)
- B Rock with $760 \text{ m/s} < \bar{V}_S \leq 1500 \text{ m/s}$ ($2500 \text{ ft/sec} < \bar{V}_S \leq 5000 \text{ ft/sec}$)
- C Very dense soil and soft rock with $360 \text{ m/s} < \bar{V}_S \leq 760 \text{ m/s}$ ($1200 \text{ ft/sec} < \bar{V}_S \leq 2500 \text{ ft/sec}$) or with either $\bar{N} > 50$ blows/0.30 m (blows/ft) or $\bar{S}_u > 100$ kPa (2000 psf)
- D Stiff soil with $180 \text{ m/s} \leq \bar{V}_S \leq 360 \text{ m/s}$ ($600 \text{ ft/sec} \leq \bar{V}_S \leq 1200 \text{ ft/sec}$) or with either $15 \leq \bar{N} \leq 50$ blows/0.30 m (blows/ft) or $50 \text{ kPa} \leq \bar{S}_u \leq 100 \text{ kPa}$ ($1000 \text{ psf} \leq \bar{S}_u \leq 2000 \text{ psf}$)
- E A soil profile with $\bar{V}_S < 180 \text{ m/s}$ (600 ft/sec) or with either $\bar{N} < 15$ blows/0.30 m (blows/ft) or $\bar{S}_u < 50$ kPa (1000 psf), or any profile with more than 3 m (10 ft) of soft clay defined as soil with $PI > 20$, $w = 40$ percent, and $\bar{S}_u < 25$ kPa (500 psf)
- F Soils requiring site-specific evaluations:
1. Peats and/or highly organic clays ($H > 3$ m [10 ft] of peat and/or highly organic clay where H = thickness of soil)
 2. Very high plasticity clays ($H > 8$ m [25 ft] with $PI > 75$)
 3. Very thick soft/medium stiff clays ($H > 36$ m [120 ft])

Exception: When the soil properties are not known in sufficient detail to determine the Site Class, Site Class D may be used. Site Classes E or F need not be assumed unless the authority having jurisdiction determines that Site Classes E or F could be present at the site or in the event that Site Classes E or F are established by geotechnical data.

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Step 1: Check for the three categories of Site Class F requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific evaluation.

Step 2: Categorize the site using one of the following three methods with $\bar{V}_S$, $\bar{N}$, and $\bar{S}_u$ computed in all cases as specified by the definitions in Art. 3.10.2.2.2:

- a. $\bar{V}_S$ for the top 30 m (100 ft) ($\bar{V}_S$ method)
- b. $\bar{N}$ for the top 30 m (100 ft) ($\bar{N}$ method)
- c. $\bar{N}_{ch}$ for cohesionless soil layers ($PI < 20$) in the top 30 m (100 ft) and average $\bar{S}_u$ for cohesive soil layers ($PI > 20$) in the top 30 m (100 ft) ($\bar{S}_u$ method)

$\bar{N}_{ch}$ and $\bar{S}_u$ are averaged over the respective thickness of cohesionless and cohesive soil layers within the upper 30 m (100 ft). Refer to Article 3.10.2.2.2 for equations for calculating average parameter values for the methods a, b, and c. If method c is used, the site class is determined as the softer site class resulting from the averaging to obtain $\bar{N}_{ch}$ and $\bar{S}_u$ (for example, if $\bar{N}_{ch}$ were equal to 20 blows/0.30 m (blows/ft) and $\bar{S}_u$ were equal to 40 kPa (800 psf), the site would classify as E in accordance with Table 3.10.2.2.1-1). Note that when using method b, $\bar{N}$ values are for both cohesionless and cohesive soil layers within the upper 30 m (100 feet).

As described in Commentary to Article 3.10.2.2.2, it may be appropriate in some cases to define the ground motion at depth, below a soft surficial layer, where the surficial layer would not significantly influence bridge response. In this case, the Site Class may be determined on the basis of the soil profile characteristics below the surficial layer.

Within Site Class F, soils requiring site-specific evaluation, one category has been deleted in these specifications from the four categories contained in the aforementioned documents. This category consists of soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils. It was judged that special analyses for the purpose of refining site ground motion amplifications for these soils was too severe a requirement for ordinary bridge design because such analyses would require utilization of effective stress and/or strength degrading nonlinear analyses techniques that are difficult to apply even by experts. Also, limited case history data and analysis results indicate that liquefaction reduces spectral response rather than increases it, except at long periods in some cases.

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Because of the general reduction in response spectral amplitudes due to liquefaction, the designer may wish to consider special analysis of site response for liquefiable soil sites to avoid excessive conservatism in assessing bridge inertia loads when liquefaction occurs. Site-specific analyses are required for major or very important structures in some cases (Article 3.10.2), so that appropriate analysis techniques would be used for such structures. The deletion of liquefiable soils from Site Class F only affects the requirement to conduct site-specific analyses for the purpose of determining ground motion amplification through these soils. It is still required to evaluate liquefaction occurrence and its effect on a bridge as a potential site ground failure hazard as specified in Article 3.10.4.

Table 3.10.2.2.1-1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
E	< 180 m/sec (<600 ft/sec)	<15 blows/0.30 m (blows/ft)	< 50 kPa (<1000 psf)
D	180 to 360 m/sec (600 to 1200 ft/sec)	15 to 50	50 to 100 kPa (1000 to 2000 psf)
C	360 to 760 m/sec (1200 to 2500 ft/sec)	> 50	> 100 kPa (> 2000 psf)
B	760 to 1500 m/sec (2500 to 5000 ft/sec)	—	—
A	> 1500 m/sec (> 5000 ft/sec)	—	—

NOTE: If the $\bar{s}_u$ method is used and the $\bar{N}_{ch}$ and $\bar{s}_u$ criteria differ, select the category with the softer soils (for example, use Site Class E instead of D).

The shear wave velocity for rock, Site Class B, shall be either measured on site or estimated on the basis of shear wave velocities in similar competent rock with moderate fracturing and weathering. Softer and more highly fractured and weathered rock shall either be measured on site for shear wave velocity or classified as Site Class C.

The hard rock, Site Class A, category shall be supported by shear wave velocity measurements either on site or on profiles of the same rock type in the same formation with an equal or greater degree of weathering and fracturing. Where hard rock conditions are known to be continuous to a depth of 30 m (100 ft) surficial shear wave velocity measurements may be extrapolated to assess $\bar{v}_s$.

The rock categories, *Site Classes* A and B, shall not be used if there is more than 3 m (10 ft) of soil between the rock surface and the bottom of the spread footing or mat foundation.

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3.10.2.2.2 Definitions Of Site Class Parameters

The definitions presented below apply to the upper 30 m (100 ft) of the site profile. Profiles containing distinctly different soil layers shall be subdivided into those layers designated by a number that ranges from 1 to n at the bottom where there are a total of n distinct layers in the upper 30 m (100 ft). The symbol l_i then refers to any one of the layers between 1 and n .

The average $\bar{v}_s$ for the layer is as follows:

$$\bar{v}_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{si}}} \quad (3.10.2.2.2-1)$$

where $\sum_{i=1}^n d_i$ is equal to 30 m (100 ft), v_{si} is the shear wave velocity in m/s (ft/sec) of the layer, and d_i is the thickness of any layer between 0 and 30 m (100 ft).

N_i is the Standard Penetration Resistance (ASTM D1586-84) not to exceed 100 blows/0.30 m (blows/ft) as directly measured in the field without corrections.

$\bar{N}$ is:

$$\bar{N} = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{N_i}} \quad (3.10.2.2.2-2)$$

$\bar{N}_{ch}$ is

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^m \frac{d_i}{N_i}} \quad (3.10.2.2.2-3)$$

where $\sum_{i=1}^m d_i = d_s$

In Equation 3.10.2.2.2-3, d_i and N_i are for cohesionless soils only and d_s is the total thickness of cohesionless soil layers in the top 30 m (100 ft).

s_{ul} is the undrained shear strength in kPa (psf), not to exceed 250 kPa (5,000 psf), ASTM D2166-91 or D2850-

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C3.10.2.2.2

An alternative to applying Equations 3.10.2.2.2-2, -3, and -4 to obtain values for $\bar{N}$, $\bar{N}_{ch}$ and $\bar{s}_u$ is to convert the N -values or s_u values into estimated shear wave velocities and then apply Equation 3.10.2.2.2-1. Procedures given in Kramer (1996) can be used for these conversions.

If the site profile is particularly erratic or if the average velocity computed in this manner does not appear reasonable or if the project involves special design issues, it may be desirable to conduct shear wave velocity measurements, using one of the procedures identified in Article 2.4.3.1b. In all evaluations of site classification, the shear wave velocity should be viewed as the fundamental soil property, as it was the soil property that was used when conducting the original studies which defined the site categories.

Depth of Motion Determination

For short bridges that involve a limited number of spans, the motion at the abutment will generally be the primary mechanism by which energy is transferred from the ground to the bridge superstructure. If the abutment involves an earth approach fill, the site classification should be determined at the base of the approach fill. The potential effects of the approach fill overburden pressure on the shear wave velocity of the soil should be accounted for in the determination of site classification.

It may be necessary for some long bridges to determine the site classification at a central pier. If this central pier is supported on spread footings, then the motion computed at the ground surface is appropriate. However, if deep foundations (i.e., driven piles or drilled shafts) are used to support the central pier, then the location of the motion will depend on the horizontal stiffness of the soil-cap system relative to the horizontal stiffness of the soil-pile system. If the pile cap is the stiffer of the two, then the motion should be defined at the pile cap. If the pile cap provides little horizontal stiffness or if there is no pile cap (i.e., pile extension), then the controlling motion will likely be at some depth below the ground surface. Typically this will be approximately 4 to 7 pile diameters below the pile cap or where a very large increase in soil stiffness occurs. The determination of this elevation requires considerable judgment and should be discussed by the geotechnical and bridge engineer.

For cases where the controlling motion is more appropriately specified at depth, site-specific ground response analyses can be conducted following guidelines given in Appendix 3A of this section to establish ground motions at the point of fixity. This approach or alternatives to this approach should be used only with the owner's approval.

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 $\bar{s}_u$ is:

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^k \frac{d_i}{s_{ui}}} \quad (3.10.2.2.2-4)$$

where $\sum_{i=1}^k d_i = d_c$

d_c is the total thickness (30- d_s m (100- d_s ft)) of cohesive soil layers in the top 30 m (100 ft).

PI is the plasticity index, ASTM D4318-93.

w is the moisture content in percent, ASTM D2216-92.

3.10.2.2.3 Site Coefficients

Site coefficients for the short-period range (F_a) and for the long-period range (F_v) are given in Tables 3.10.2.2.3-1 and 3.10.2.2.3-2, respectively. Application of these coefficients to determine elastic seismic response coefficients of ground motions is described in Article 3.10.2.1

Table 3.10.2.2.3-1 Values of F_a as a Function of Site Class and Mapped Short-Period Spectral Acceleration

Site Class	Mapped Spectral Response Acceleration at Short Periods				
	$S_s \leq 0.25 \text{ g}$	$S_s = 0.50 \text{ g}$	$S_s = 0.75 \text{ g}$	$S_s = 1.00 \text{ g}$	$S_s \geq 1.25 \text{ g}$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	<i>a</i>	<i>a</i>	<i>a</i>	<i>a</i>	<i>a</i>
NOTE: Use straight line interpolation for intermediate values of S_s , where S_s is the spectral acceleration at 0.2 seconds obtained from the ground motion maps.					
^a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.10.2). For the purpose of defining Seismic Hazard Levels in Article 3.10.3.1 Type E values may be used for Type F soils.					

Table 3.10.2.2.3-2 Values of F_v as a Function of Site Class and Mapped 1 Second Period Spectral Acceleration

Site Class	Mapped Spectral Response Acceleration at 1 Second Periods				
	$S_1 \leq 0.1 \text{ g}$	$S_1 = 0.2 \text{ g}$	$S_1 = 0.3 \text{ g}$	$S_1 = 0.4 \text{ g}$	$S_1 \geq 0.5 \text{ g}$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	<i>a</i>	<i>a</i>	<i>a</i>	<i>a</i>	<i>a</i>
NOTE: Use straight line interpolation for intermediate values of S_1 , where S_1 is the spectral acceleration at 1.0 second obtained from the ground motion maps.					
^a Site-specific geotechnical investigation and dynamic site response analyses shall be performed (Article 3.10.2). For the purpose of defining Seismic Hazard Levels in Article 3.10.3.1 Type E values may be used for Type F soils.					

3.10.2.3 RESPONSE SPECTRA BASED ON SITE-SPECIFIC PROCEDURE

A site-specific procedure to develop design response spectra of earthquake ground motions shall be performed when required by Article 3.10.2 and may be performed for any site. A site-specific probabilistic ground motion analysis shall be comprehensive and shall include the following: characterization of seismic sources and ground motion attenuation that incorporates current scientific interpretations, including uncertainties in seismic source and ground motion models and parameter values; detailed documentation; and detailed peer review.

Where analyses to determine site soil response effects are required by Articles 3.10.2.2 and 3.10.2 for Site Class F soils, the influence of the local soil conditions shall be determined based on site-specific geotechnical investigations and dynamic site response analyses.

For sites located within 10km of an active fault (as defined in Article 3.10.2), studies shall be considered to quantify near-fault effects on ground motions if these could significantly influence the bridge response.

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The intent in conducting a site-specific probabilistic ground motion study is to develop ground motions that are more accurate for the local seismic and site conditions than can be determined from National ground motion maps and the general procedure of Article 3.10.2.1. Accordingly, such studies must be comprehensive and incorporate current scientific interpretations at a regional scale. Because there are typically scientifically credible alternatives for models and parameter values used to characterize seismic sources and ground motion attenuation, it is important to formally incorporate these uncertainties in a site-specific probabilistic analysis. Examples of these uncertainties include seismic source location, extent and geometry; maximum earthquake magnitude; earthquake recurrence rate; and ground motion attenuation relationship.

Guidelines are presented in Appendix 3A for site-specific geotechnical investigations and dynamic site response analyses for Site Class F soils. These guidelines are applicable for site-specific determination of site response for any site class when the site response is determined on the basis of a dynamic site response analysis.

Near-fault effects on horizontal response spectra include: (1) higher ground motions due to the proximity of the active fault; (2) directivity effects that increase ground motions for periods greater than 0.5 second if the fault rupture propagates toward the site; and (3) directionality effects that increase ground motions for periods greater than 0.5 second in the direction normal (perpendicular) to the strike of the fault. If the active fault is included and appropriately modeled in the development of national ground motion maps, then effect (1) is already included in the national ground motion map. Effects (2) and (3) are not included in the national map. These effects are significant only for periods longer than 0.5 second and normally would be evaluated only for major or very important bridges having

In cases where the 0.2-second or 1.0-second response spectral accelerations of the site-specific probabilistic response spectrum for the MCE exceeds the response spectrum shown in Figure 3.10.2.3-1, a deterministic spectrum may be utilized in regions having known active faults if the deterministic spectrum is lower than the probabilistic spectrum. The deterministic spectrum shall be the envelope of median-plus-standard-deviation spectra calculated for characteristic maximum magnitude earthquakes on known active faults, but shall not be lower than the spectrum shown in Figure 3.10.2.3-1. If there is more than one active fault in the site region, the deterministic spectrum shall be calculated as the envelope of spectra for the different faults. Alternatively, deterministic spectra may be defined for each fault, and each spectrum, or the spectrum that governs bridge response, may be used for the analysis of the bridge.

natural periods of vibration longer than 0.5 second. Further discussion of effects (2) and (3) are contained in Sommerville (1997) and Somerville et al. (1997). The ratio of vertical-to-horizontal ground motions increases for short-period motions in the near-fault environment. Site-specific vertical response spectra should be developed where required based on Article 3.10.2.6.

The application of site-specific deterministic limits on response spectra in areas of active faults follows criteria that are similar to the criteria used in constructing deterministic bounds for national ground motion maps for the MCE. However, site-specific deterministic spectra are calculated as median-plus-standard-deviation values rather than the nominal 1.5-times-median values used for national ground motion maps (refer to commentary to Article 3.10.1.2).

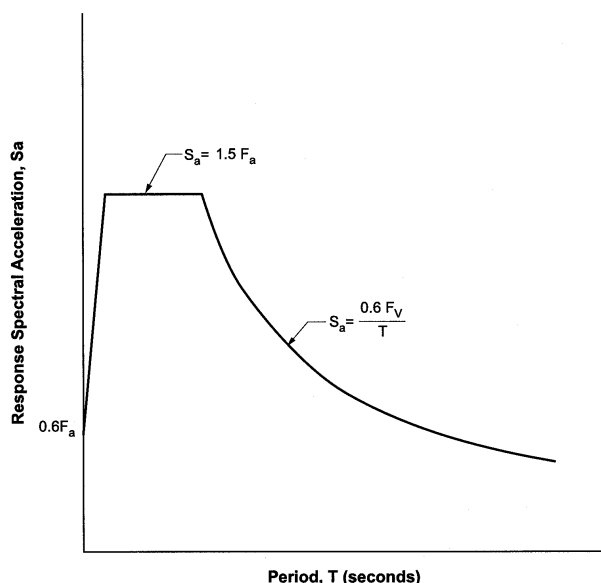


Figure 3.10.2.3-1 Minimum Deterministic Response Spectrum

When response spectra are determined from a site-specific study, the spectra shall not be lower than two-thirds of the response spectra determined using the general procedure in Article 3.10.2.1.

3.10.2.4 COMBINATION OF SEISMIC FORCE EFFECTS

C3.10.2.4

The maximum seismic force due to seismic load in any one direction shall be based on the CQC combination of modal responses due to ground motion in that direction. The maximum force due to two or three orthogonal ground motion components shall be obtained either by the SRSS combination or the 100% - 40% combination forces due to the individual seismic loads.

SRSS Combination Rule – the maximum response quantity of interest is the SRSS combination of the response quantity from each of the orthogonal directions.

(i.e., $M_x = \sqrt{(M_x^T)^2 + (M_x^L)^2}$ where M_x^T and M_x^L are the x-component moments from a transverse and longitudinal analysis)

If biaxial design of an element is important (e.g. circular columns) and the bridge has a maximum skew angle less than 10 degrees and/or a subtended angle less than 10 degrees then the maximum response quantities in the two orthogonal directions (M_x , M_y) shall use the 100% - 40% rule prior to obtaining the vector sum. The maximum vector moment is the maximum of:

$$\sqrt{M_x^2 + (0.4M_y)^2} \text{ or } \sqrt{(0.4M_x)^2 + M_y^2}$$

If the maximum skew angle or the subtended angle in a horizontally curved bridge exceeds 10 degrees then the maximum response quantities in the two horizontal directions shall be combined as the vector sum:

$$\sqrt{M_x^2 + M_y^2}$$

100% - 40% Combination Rule – the maximum response quantity of interest shall be obtained from the maximum of two load cases.

Load Case 1 (LC1) – 100% of the absolute value of the response quantity resulting from the analysis in one orthogonal direction (transverse) added to 40% of the response quantity resulting from the analyses in the other orthogonal direction(s) (longitudinal).

$$M_x^{LC1} = 1.0M_x^T + 0.4M_x^L$$

Load Case 2 (LC2) – 100% of the absolute value of the response quantity resulting from an analysis in the other orthogonal direction (longitudinal) added to 40% of the response quantity resulting from an analysis in the original direction (transverse).

$$M_x^{LC2} = 0.4M_x^T + 1.0M_x^L$$

The combination of seismic forces computed from a response spectrum analysis has three aspects. The first is the combination of the vibration modes due to ground motion in one direction (longitudinal, transverse, or vertical). The CQC method ("complete quadratic combination") provides a good estimate of the maximum force, including the correlation of modal responses closely-spaced in frequency.

The second issue is the contribution of two or three orthogonal ground motion components to a single force effect. The SRSS rule ("square root sum of the squares") is the most appropriate rule for combining the contribution of orthogonal, and uncorrelated, ground motion components to a single seismic force. The SRSS method is recommended particularly for seismic analysis including vertical ground motion (Button et. al. 1999). Since the prior AASHTO seismic provisions were based on a 100% - 30% combination it was decided to modify this and permit the 100% - 40% combination rule as an alternate to the SRSS combination rule. The 100%-40% combination of forces provides results similar to the SRSS combination when the same response spectrum is used in two orthogonal directions (Clough and Penzien, 1993).

For three components of ground motions the combination rules of a bending moment are as follows.

$$\text{SRSS Combination: } M_x = \sqrt{(M_x^T)^2 + (M_x^L)^2 + (M_x^V)^2}$$

100% – 40% Combination:

$$M_x^{LC1} = 1.0M_x^T + 0.4M_x^L + 0.4M_x^V$$

$$M_x^{LC2} = 0.4M_x^T + 1.0M_x^L + 0.4M_x^V$$

$$M_x^{LC3} = 0.4M_x^T + 0.4M_x^L + 1.0M_x^V$$

The third issue is the combination of two force quantities when biaxial design of a member is important (e.g. circular column). This is the most difficult of the three issues since the maxima of the three components (axial force P , and bending moments about two local axes M_x and M_y) are not likely to occur at the same time. A sophisticated approach to determining the critical combination is difficult to justify for design. Instead a simpler approach is adopted.

For the SRSS combination and a very regular bridge the two components to be combined M_x and M_y utilize the 100% - 40% rule prior to obtaining the vector sum which is then used with +/- of the maximum axial force in the design of the column. If the bridge has any significant skew or curvature, the vector sum is applied to the maximum moment quantities. This is because the 100% - 40% rule as applied in biaxial design can be non-conservative when significant skew and curvature exist.

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If biaxial design of an element is important then the maximum response quantities in the two orthogonal directions from each load case shall be combined to obtain a vectorial sum and the maximum vector from the two load cases shall be used for design, i.e., the maximum of:

$$\sqrt{(M_x^{LC1})^2 + (M_y^{LC1})^2} \text{ or } \sqrt{(M_x^{LC2})^2 + (M_y^{LC2})^2}$$

3.10.2.5 ACCELERATION TIME HISTORIES

When time history dynamic analysis of structures is performed, the development of time histories shall meet the requirements of this section. The developed time histories shall have characteristics that are representative of the seismic environment of the site and the local site conditions.

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conservative when significant skew and curvature exist.

For the 100% - 40% combination rule the M_x and M_y components from each load case are combined to obtain the vectorial sum and the maximum moment of the two load cases is used with the maximum axial load in the design of the column. The combination rules are as follows:

SRSS Combination for Biaxial Design:

- For bridges with skew or curvature less than 10 degrees - Maximum of $\sqrt{M_x^2 + (0.4M_y)^2}$ and $\sqrt{(0.4M_x)^2 + M_y^2}$ with the maximum axial load $\pm P$
- For bridges with skew or curvature greater than 10 degrees- $\sqrt{M_x^2 + M_y^2}$ with the maximum axial load $\pm P$

100%- 40% Combination for Biaxial Design:

- Maximum of $\sqrt{(M_x^{LC1})^2 + (M_y^{LC1})^2}$ and $\sqrt{(M_x^{LC2})^2 + (M_y^{LC2})^2}$ and $\sqrt{(M_x^{LC3})^2 + (M_y^{LC3})^2}$ with the maximum axial load $\pm P$

C3.10.2.5 ACCELERATION TIME HISTORIES

Characteristics of the seismic environment of the site to be considered in selecting time histories include: tectonic environment (e.g. subduction zone; shallow crustal faults in western United States or similar crustal environment; eastern United States or similar crustal environment); earthquake magnitude; type of faulting (e.g. strike-slip; reverse; normal); seismic source-to-site distance; local site conditions; and design or expected ground motion characteristics (e.g. design response spectrum; duration of strong shaking; special ground motion characteristics such as near-fault characteristics. Dominant earthquake magnitudes and distances that principally contribute to the probabilistic design response spectra at a site as determined from national ground motion maps can be obtained from deaggregation information from the U.S. Geological Survey website:

<http://geohazards.cr.usgs.gov/eq/>.

It is desirable to select time histories that have been recorded under conditions similar to the seismic conditions at the site listed above, but compromises are usually required because of the multiple attributes of the seismic environment and the limited data bank of recorded time histories. Selection of time histories having similar earthquake magnitudes and distances, within reasonable ranges, are especially important parameters because they have a strong influence on response spectral content,

Time histories may be either recorded time histories or spectrum-matched time histories. If sufficient recorded motions are not available, simulated-recorded time histories may be developed using theoretical ground motion modeling methods that simulate the earthquake rupture and the source-to-site seismic wave propagation.

If spectrum-matched time histories are developed, the initial time histories to be spectrum matched shall be representative recorded or simulated-recorded motions. Analytical techniques used for spectrum matching shall be demonstrated to be capable of achieving seismologically realistic time series that are similar to the time series of the initial time histories selected for spectrum matching.

When using recorded or simulated-recorded time histories, they shall be scaled to the approximate level of the design response spectrum in the period range of significance. For each component of motion, an aggregate match of the design response spectrum shall be achieved for the set of acceleration time histories used. A mean spectrum of the individual spectra of the time histories shall be calculated period-by-period. Over the defined period range of significance, the mean spectrum shall not be more than 15% lower than the design spectrum at any period, and the average of the ratios of the mean spectrum to the design spectrum shall be equal to or greater than unity. When developing spectrum-matched time histories, before the matching process, they shall be scaled to the approximate level of the design response spectrum in the period range of significance. Thereafter, the set of time histories for each component shall be spectrum-matched to achieve the aggregate fit requirement stated above.

response spectral shape, duration of strong shaking, and near-source ground motion characteristics. It is desirable that selected recorded motions be somewhat similar in overall ground motion level and spectral shape to the design spectrum to avoid using very large scaling factors with recorded motions and very large changes in spectral content in the spectrum-matching approach. If the site is located within 10 km (6.25 miles) of an active fault, then intermediate-to-long period ground motion pulses that are characteristic of near-source time histories should be included if these types of ground motion characteristics could significantly influence structural response. Similarly, the high short-period spectral content of near-source vertical ground motions should be considered.

Ground motion modeling methods of strong motion seismology are being increasingly used to supplement the recorded ground motion database. These methods are especially useful for seismic settings for which relatively few actual strong-motion recordings are available, such as in the central and Eastern United States. Through analytical simulation of the earthquake rupture and wave propagation process, these methods can produce seismologically reasonable time series.

Response spectrum-matching approaches include methods in which time series adjustments are made in the time domain (Lilhanard and Tseng, 1988; Abrahamson, 1992) and those in which the adjustments are made in the frequency domain (Gasparini and Vanmarcke, 1976; Silva and Lee, 1987; Bolt and Grigor, 1993). Both of these approaches are capable of modifying existing time histories to achieve a close match to the design response spectrum while maintaining fairly well the basic time domain character of the recorded or simulated-recorded time histories. To minimize changes to the time domain characteristics, it is desirable that the overall shape of the spectrum of the recorded or simulated-recorded time history not be greatly different from the shape of the design response spectrum and that the time history initially be scaled so that its spectrum is at the approximate level of the design spectrum before spectrum matching.

When developing three-component sets of time histories by simple scaling rather than spectrum matching, it is difficult to achieve a comparable aggregate match to the design spectra for each component of motion when using a single scaling factor for each time history set. It is desirable, however, to use a single scaling factor to preserve the relationship between the components. Approaches of dealing with this scaling issue include: (1) use of a higher scaling factor to meet the minimum aggregate match requirement for one component while exceeding it for the other two; (2) use of a scaling factor to meet the aggregate match for the most critical component with the match somewhat deficient for other components; (3) compromising on the scaling by using different factors as required for different components of a time history set.

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For use in nonlinear inelastic time history analysis using either recorded, simulated-recorded, or spectrum-matched motions for either the 3% in 75 yr or 50% in 75 yr event, at least three time histories shall be used for each component of motion. The design actions shall be taken as the maximum response calculated for the three ground motions in each principal direction. If a minimum of seven recorded, simulated-recorded, or spectrum-matched time histories are used for each component of motion, the design actions may be taken as the mean response calculated for each principal direction.

3.10.2.6 VERTICAL ACCELERATION EFFECTS

The impact of vertical ground motion may be ignored if the bridge site is greater than 50km from an active fault as defined in Article 3.10.2 and can be ignored for all bridges in the central and Eastern U.S. and those areas impacted by subduction earthquakes in the Northwest. If the bridge site is located within 10km of an active fault then a site specific study is required if it is determined that the response of the bridge could be significantly and adversely affected by vertical ground motion characteristics. In such cases response spectra and acceleration time histories as appropriate shall be developed for use and shall include appropriate vertical ground motions for inclusion in the design and analysis of the bridge. For vertical design forces the linear analysis shall use the CQC modal combination method and the SRSS directional combination method.

If the bridge site is located between 10km and 50km of an active fault a site specific study may be performed including the effects of appropriate vertical ground motion.

In lieu of a dynamic analysis that incorporates the effect of vertical ground motions the following variations in column axial loads and superstructure moments and shears shall

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While the second approach is acceptable, it requires careful examination and interpretation of the results and possibly dual analyses for application of the horizontal higher horizontal component in each principal horizontal direction.

The requirements for the number of time histories to be used in nonlinear inelastic dynamic analysis and for the interpretation of the results take into account the dependence of response on the time domain character of the time histories (duration, pulse shape, pulse sequencing) in addition to their response spectral content.

Additional guidance on developing acceleration time histories for dynamic analysis may be found in publications by the Caltrans Seismic Advisory Board Adhoc Committee on Soil-Foundation-Structure Interaction (CSABAC) (1999) and the U.S. Army Corps of Engineers (2000). CSABAC (1999) also provides detailed guidance on modeling the spatial variation of ground motion between bridge piers and the conduct of seismic soil-foundation-structure interaction (SFSI) analyses. Both spatial variations of ground motion and SFSI may significantly effect bridge response. Spatial variations include differences in seismic wave arrival times between bridge piers (wave passage effect), ground motion incoherence due to seismic wave scattering, and differential site response due to different soil profiles at different bridge piers. For long bridges, all forms of spatial variations may be important. For short bridges, limited information appears to indicate that wave passage effects and incoherence are, in general, relatively unimportant in comparison to effects of differential site response (Shinozuka et al., 1999; Martin, 1998). Somerville et al. (1999) provide guidance on the characteristics of pulses of ground motion that occur in time histories in the near-fault region.

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The most comprehensive study (Button et al., 1999) performed to date on the impact of vertical acceleration effects indicates that for some design parameters (superstructure moment and shear, column axial forces) and for some bridge types the impact can be significant. The study was based on vertical response spectra developed by Silva (1997) from recorded Western U.S. ground motions. Until more information is known about the characteristics of vertical ground motions in the Eastern U.S. and those areas impacted by subductions zones in the Northwest the specification cannot impose mandatory requirements. However, it is advisable for designers to be aware that vertical acceleration effects may be important (Button et al., 1999) and for more important bridges the impact be assessed.

Recent studies (e.g. Abrahamson and Silva, 1997; Silva, 1997; Campbell and Bozorgnia, 2000) have shown that the ratio of the vertical response spectrum to the horizontal response spectrum of ground motions can differ substantially from the nominal two-thirds ratio commonly

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be included in the design of the columns and the superstructure to account for the effects of vertical ground motion.

Column Axial Loads (AL) = DL Axial Force $\pm C_v$ (DL Axial Force)

Superstructure Bending Moments = DL Moment $\pm C_v$ (DL Moment)

Superstructure Shears = DL Shear $\pm C_v$ (DL Shear)

C_v is the coefficient given in Table 3.10.2.6-1 if the maximum magnitude of the design earthquake is 6.5, or Table 3.10.2.6-2 if the maximum magnitude of the design earthquake is 7.5. Note that the coefficient C_v for the superstructure has a value specified at the mid-span location and at the column/pier support. Linear interpolation is used to determine C_v for points on the superstructure between these locations.

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assumed in engineering practice. These studies show that the ratios of vertical to horizontal response spectral values are functions of the tectonic environment, subsurface soil or rock conditions, earthquake magnitude, earthquake source-to-site distance, and period of vibration. Whereas the two-thirds ratio may be conservative for longer periods of vibration (say greater than 0.3 second) in many cases, at shorter periods the ratio of vertical to horizontal response spectra may exceed two-thirds and even substantially exceed unity for close earthquake source-to-site distances and periods less than 0.2 second. At present, detailed procedures have not been developed for constructing vertical spectra having an appropriate relationship to the horizontal spectra constructed using the general procedure of Article 3.10.2.1. When developed, these procedures could be used in conjunction with deaggregation information on dominant earthquake source-to-site distance and earthquake magnitude from the USGS national map Internet website [<http://geohazards.cr.usgs.gov/eq/>] to construct vertical spectra at any location.

At present, this specification requires explicit consideration of vertical acceleration effects in design only as a function of the distance of a bridge site from an active fault. As such, these requirements would generally not be applied to sites in the central and eastern United States because few active faults meeting the definition in Article 3.10.2 have been accurately located in that part of the country. Also, because the characteristics of vertical ground motions in subduction zones has been the subject of only limited studies, the specification does not at present impose requirements for vertical acceleration effects as a function of distance from subduction zone faults.

For use in Tables 3.10.2.6-1 and 3.10.2.6-2, earthquake magnitude is taken as the largest (maximum) magnitude, based on the moment magnitude scale, of an earthquake considered capable of occurring on the active fault. Usually, maximum magnitude is estimated on the basis of the longest rupture length or the largest rupture area assessed to be capable of occurring on the fault (e.g., Wells and Coppersmith, 1994). Maximum magnitude should be estimated by a knowledgeable geologist or seismologist.

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Table 3.10.2.6-1 Fault distance zones and corresponding dead load multiplier for all bridges observed for rock and soil site conditions and a magnitude 6.5 event..

Response Quantity	Fault Distance Zones (km)				
	0-10	10-20	20-30	30-40	40-50
Pier Axial Force DL Multiplier	0.7	0.3	0.20	0.1	0.1
Superstructure Shear Force at Pier DL Multiplier	0.7	0.4	0.2	0.1	0.1
Superstructure Bending Moment at Pier DL Multiplier	0.6	0.3	0.2	0.1	0.1
Superstructure Shear Force at Mid-Span DL Multiplier	0.1	0.1	0.1	0.1	0.1
Superstructure Bending Moment at Mid-Span* DL Multiplier	1.4	0.7	0.4	0.3	0.2
<u>Footnotes</u> (1) The DL Multiplier values given above are in addition to the dead load; thus, an actual "load factor" would be 1.0 plus/minus the above numbers. (2) The Live Load (LL) typically used in the design of bridge types shown in this study is in the range of 20-30% of the Dead Load (DL).					

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Table 3.10.2.6-2 Fault distance zones and corresponding dead load multiplier for all bridges observed for rock and soil site conditions and a magnitude 7.5 event.

Response Quantity	Fault Distance Zones (km)				
	0-10	10-20	20-30	30-40	40-50
Pier Axial Force DL Multiplier	0.9	0.4	0.2	0.2	0.1
Superstructure Shear Force at Pier DL Multiplier	1.0	0.5	0.3	0.2	0.2
Superstructure Bending Moment at Pier DL Multiplier	1.0	0.5	0.3	0.2	0.2
Superstructure Shear Force at Mid-Span DL Multiplier	0.2	0.1	0.1	0.1	0.1
Superstructure Bending Moment at Mid-Span* DL Multiplier	1.9	1.0	0.6	0.5	0.3
<u>Footnotes</u> (1) The DL Multiplier values given above are in addition to the dead load; thus, an actual “load factor” would be 1.0 plus/minus the above numbers. (2) The Live Load (LL) typically used in the design of bridge types shown in this study is in the range of 20-30% of the Dead Load (DL).					

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3.10.3 Seismic Design and Analysis Procedures

3.10.3.1 GENERAL

For single-span bridges, regardless of seismic zone and in lieu of a rigorous analysis, the minimum design force at the connections in the restrained direction between the superstructure and the substructure shall not be less than the product of $F_a S_s / 2.5$, and the tributary permanent load. The minimum seat widths shall comply with Article 3.10.3.10.

Each bridge shall be assigned a Seismic Hazard Level that shall be the highest level determined by the value of $F_v S_1$ or $F_a S_s$ from Tables 3.10.3-1.

Table 3.10.3-1 – Seismic Hazard Levels

Seismic Hazard Level	Value of $F_v S_1$	Value of $F_a S_s$
I	$F_v S_1 \leq 0.15$	$F_a S_s \leq 0.15$
II	$0.15 < F_v S_1 \leq 0.25$	$0.15 < F_a S_s \leq 0.35$
III	$0.25 < F_v S_1 \leq 0.40$	$0.25 < F_a S_s \leq 0.60$
IV	$0.40 < F_v S_1$	$0.60 < F_a S_s$

Notes:

1. For the purposes of determining the Seismic Hazard Level for Site Class E Soils (Article 3.10.2.2.1) the value of F_v and F_a need not be taken larger than 2.4 and 1.6 respectively when S_1 is less than or equal to 0.10 and S_s is less than 0.25.
2. For the purposes of determining the Seismic Hazard Level for Site Class F Soils (Article 3.10.2.2.1) F_v and F_a values for Site Class E soils may be used with the adjustment described in Note 1 above.

C3.10.3.1

Requirements for single span bridges are not as rigorous as for multi-span bridges because of their favorable response to seismic loads in past earthquakes. As a result, single span bridges need not be analyzed for seismic loads regardless of the SDR and design requirements are limited to minimum seat widths and connection forces. Adequate seat widths must be provided in both the transverse and longitudinal directions. Connection forces based on the premise that the bridge is very stiff and that the fundamental period of response will be short. This assumption acknowledges the fact that the period of vibration is difficult to calculate because of significant interaction with the abutments.

These reduced requirements are also based on the assumption that there are no vulnerable substructures (i.e., no columns) and that a rigid (or near rigid) superstructure is in place to distribute the in-plane loads to the abutments. If, however, the superstructure is not able to act as a stiff diaphragm and sustains significant in-plane deformation during horizontal loading, it should be analyzed for these loads and designed accordingly. Single span trusses may be sensitive to in-plane loads and the designer may need to take additional precautions to ensure the safety of truss superstructures.

The Seismic Hazard Level is defined as a function of the magnitude of the ground surface shaking as expressed by $F_v S_1$ and $F_a S_s$. Bridges with a period greater than 1 second would be more appropriately governed by the $F_v S_1$ definition whereas bridges with a period less than 0.7 second would be more appropriately governed by the $F_a S_s$ definition. Since the period of the bridge is not known at an early stage in the design process both criteria are therefore used to define the Seismic Hazard Level. The two footnotes to the Tables 3.10.3-1(a) and 3.10.3-1(b) effectively limit boundaries for Soil Types E and F in Hazard Levels I and II to those of Soil Type D. This decision was made in part because of the greater uncertainty in the values of F_v and F_a for Type E and F soils when ground shaking is relatively low ($S_1 < 0.10$ and $S_s < 0.25$) and in part to not extend the boundaries beyond those of Soil Type D until the impact of this major revision of the specification is better understood. Further discussion on the Hazard Level boundaries is given in Appendix 3C.

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Each bridge shall be designed, analyzed and detailed for seismic effects in accordance with Table 3.10.3-2. Seismic Design and Analysis Procedures (SDAP) are described in Sections 3.10.3.2 through 3.10.3.5, and Section 4. Minimum seismic detailing requirements (SDR) are given in Table 3.10.3-3, and are discussed further in Sections 5 and 6.

COMMENTARY

Seismic design and analysis procedures reflect the variation in seismic risk across the country and are used to permit different requirements for methods of analysis, minimum support lengths, column design details, and foundation and abutment design procedures.

Table 3.10.3-2 - Seismic Design and Analysis Procedures (SDAP)
and Seismic Detailing Requirements (SDR)

Seismic Hazard Level	Life Safety		Operational	
	SDAP	SDR	SDAP	SDR
I	A1	1	A2	2
II	A2	2	C/D/E	3
III	B/C/D/E	3	C/D/E	5
IV	C/D/E	4	C/D/E	6

Notes:

1. SDAP B/C – The use of these two design/analysis procedures is governed by regularity requirements as defined in Sections 3.10.3.3.2 and 3.10.3.4.2 respectively.
2. SDAP D – The use of the uniform load method is only permitted for the life safety performance level and limits on its use are given in Art. 4.8.4.3.2
3. If abutments are required to deform inelastically and act as part of the ERS then only SDAP D or E can be used and the ULM is not permitted.
4. If owners approval of an ERE is required (Article 2.5.6.1 – i.e. inelastic behavior that is not inspectable occurs in a substructure) then SDAP E must be used.

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Table 3.10.3-3 – Component Detailing Provisions for SDR's

Component		SDR 1	SDR 2	SDR 3	SDR 4	SDR 5	SDR 6
Seat Width		Art. 3.10.3.10	Art. 3.10.3.10	Art. 3.10.3.10	Art. 3.10.3.10	Art. 3.10.3.10	Art. 3.10.3.10
Bearing	Conventional	0.1DL – Art. 3.10.3.2	0.25DL- Art. 3.10.3.2	Capacity Design Procedures – Art. 3.10.3.8 or Elastic Forces With $R=0.8$	Same as SDR 3	Same as SDR 3	Same as SDR 3
	Isolation	Detailed and tested for 1.1 times 3% in 75 year forces and displacements.	Same as SDR 1	Same as SDR 1	Same as SDR 1	Same as SDR 1	Same as SDR 1
Column (Reinforced Concrete)	Flexure	Non-seismic Requirements. (0.8% minimum longitudinal steel)	Non-seismic Requirements (0.8% minimum longitudinal steel)	SDAP B and C – non-seismic or min. steel or $P-\Delta$ or 50% in 75 year forces for SDAP C SDAP D/E – moment demand divided by R or min. steel or $P-\Delta$	Same as SDR 3	Same as SDR 3	Same as SDR 3
	Shear	Non-seismic Requirements	Minimum Shear Reinforcement per Art. 5.10.11.4.1c – Method 1	From Capacity Design Procedures – Art. 3.10.3.8 or Elastic Forces with $R=0.67$	Same as SDR 3	Same as SDR 3	Same as SDR 3
	Confinement, Longitudinal Bar Restraint	None	None	Maximum of Art. 5.10.11.4.1d to f within plastic hinge zone defined in Art. 3.10.3.9.	Same as SDR 3	Same as SDR 3	Same as SDR 3
Column (Steel)		None	$P_e \leq 0.4A_g F_y$	b/t ratios comply with Table 6.15.1. Full penetration welds for column-to-beam connections $P_e \leq 0.2A_g F_y$ Laterally support plastic hinge zones			
Connection of Column to Superstructure, Bent Beam, Footing/Pile Cap		N/A except for Bearings above	N/A except for Bearings above	Design Forces from Capacity Design Procedures – Art. 3.10.3.8 or if Elastic Forces are used in column moment design see Note 1	Same as SDR 3	Same as SDR 3	Same as SDR 3
Soil and Pile Aspects of Foundation Design	Concrete	N/A	Top 3D of piles – shear reinforcement per Art. 5.10.11.4.1c – Method 1 plus Note 2. N/A for spread foundations	Design Forces from Capacity Design Procedures using an over-strength ratio of 1.0. – Art. 3.10.3.8 plus Notes 2, 3 and 4. Maximum of shear, confinement and bar restraint reinforcement in top 3D – Art. 5.10.11.4.1c to e. Maximum of shear and confinement reinforcement for piles 3D to 10D from pile cap – Art. 5.10.11.4.1c and d.	Same as SDR 3 except higher over-strength ratios are used for concrete and steel respectively. – Art. 3.10.3.8. Notes 2, 3 and 4. Shear, confinement and bar restraint reinforcement per SDR 3 is required in top 10D	Same as SDR 4	Same as SDR 4
	Steel	N/A	N/A				

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Pile Bents	Concrete	N/A	Shear reinforcement per Art. 5.10.11.4.1c – Method 1 provided from top of bent to 10D below ground level. plus Note 2	Design Forces from Capacity Design Procedures using higher over-strength ratios for concrete and steel – Art. 3.10.3.8 plus Notes 2, 3 and 4 Reinforcement for piles in plastic hinge zone of Art. 3.10.3.9 shall be maximum of shear, confinement and bar restraint reinforcement – Art. 5.10.11.4.1c to e.	Same as SDR 3	Same as SDR 3	Same as SDR 3
	Steel	N/A	N/A				
Abutments		N/A	N/A	Non-seismic requirements for SDAP B and C, Seismic design. for SDAP D/E – Table 2.5.6-1 and 2.5.6-3 and Art. 11.6.5.1	See Table 2.5.6-1 and -3 and Art.11.6.5.1	Same as SDR 4	Same as SDR 4
Liquefaction				If predominant moment magnitude is less than 6 – no requirements. If greater than 6 see Art. 3.10.4.1	See Art. 3.10.4.1	Same as SDR 4	Same as SDR 4
ERS/ERE		N/A	N/A	See Figures C2.5.6-1 through 3 for permitted systems	Same as SDR 3	Systems and Elements requiring Owner's Approval in Figure C2.5.6-2 are not permitted	Same as SDR 5
Approach/Settlement Slab		N/A	N/A	N/A	N/A	Encouraged but not mandated	Required

NOTES:

1. See Article 3.10.3.11
2. If scour occurs then this amount of transverse reinforcement shall be provided to 3D below the lowest scour depth where D is the diameter of the pile.
3. Connection of all potential tension piles to the pile cap shall be designed for the greater of the nominal geotechnical pullout capacity of the pile or the maximum pile pullout demand calculated assuming elastic axial stiffness of the piles.
4. If liquefaction occurs and the dominant moment magnitude is greater than 6 then the transverse reinforcement shall be provided to a depth of 3D below the liquefiable layer. Guidance on determining the dominant moment magnitude is contained in Article 3.B.2.4 of Appendix 3B.

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3.10.3.2 SDAP A1 AND A2 - CONNECTION FORCES

For bridges in SDAP A1 the horizontal design connection force in the restrained directions shall not be taken to be less than 0.1 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake.

For SDAP A2, the horizontal design connection force in the restrained directions shall not be taken to be less than 0.25 times the vertical reaction due to the tributary permanent load and the tributary live loads assumed to exist during an earthquake.

For SDR 2 reinforced concrete columns, pile bents and the top 3D of concrete piles shall meet the shear reinforcement requirements of Article 5.10.11.4.1c.

For each uninterrupted segment of a superstructure, the tributary permanent load at the line of fixed bearings, used to determine the longitudinal connection design force, shall be the total permanent load of the segment.

If each bearing supporting an uninterrupted segment or simply supported span is restrained in the transverse direction, the tributary permanent load used to determine the connection design force shall be the permanent load reaction at that bearing.

Each elastomeric bearing and its connection to the masonry and sole plates shall be designed to resist the horizontal seismic design forces transmitted through the bearing. For all bridges in SDAP A1 and A2 and all single-span bridges, these seismic shear forces shall not be less than the connection force specified herein.

COMMENTARY

C3.10.3.2

In areas of low seismicity only minimum seat widths (Article 3.10.3.10) and connection design forces for bearings and minimum shear reinforcement in concrete columns and piles in SDR 2 are deemed necessary for the life safety performance objective. These default values are used as minimum design forces in lieu of rigorous analysis.

The division of SDAP A1 and A2 at a short period spectral response acceleration of 0.10 is an arbitrary expedience intended to provide some relief to parts of the country with very low seismicity.

This article describes the minimum connection force that must be transferred from the superstructure to its supporting substructures through the bearings. It does not apply if the connection is a monolithic structural joint. Similarly, it does not apply to unrestrained bearings (such as elastomeric bearings) or in the unrestrained directions of bearings that are free to move (slide) in one direction but fixed (restrained) in an orthogonal direction. The minimum force is simply 0.1 or 0.25 times the weight that is effective in the restrained direction. The calculation of the effective weight requires care and may be thought of as a tributary weight. It is calculated from the length of superstructure that is tributary to the bearing in the direction under consideration. For example, in the longitudinal direction at a fixed bearing, this length will be the length of the segment and may include more than one span if it is a continuous girder (i.e. it is the length from one expansion joint to the next). But in the transverse direction at the same bearing, this length may be as little as one-half of the span, particularly if it is supporting an expansion joint. This is because the expansion bearings at the adjacent piers will generally be transversely restrained and able to transfer lateral loads to the substructure.

It is important that not only the bearing but also the details that fasten the bearing to the sole and masonry plates (including the anchor bolts which engage the supporting members), have sufficient capacity to resist the above forces. At a fixed bearing, it is necessary to consider the simultaneous application of the longitudinal and transverse connection forces when checking these capacities.

Note that the primary purpose of this requirement is to ensure that the connections between the superstructure and its supporting substructures remain intact during the design earthquake and thus protect the girders from being unseated. The failure of these connections has been observed in many earthquakes and imposing minimum strength requirements is considered to be a simple but effective strategy to minimize the risk of collapse. However, in low seismic zones it is not necessary to design the substructures or their foundations for these forces since it is expected that if a column does yield it will have sufficient

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3.10.3.3 SDAP B - NO SEISMIC DEMAND ANALYSIS

Bridges qualifying for SDAP B do not require a seismic demand analysis but capacity design principles and minimum design details are required. The capacity design forces are covered in more detail in Section 3.10.3.8.

3.10.3.3.1 No Analysis Approach

SDAP B consists of the following steps:

- *Step 1* - Check Article 3.10.3.3.2 for restrictions on structural and site characteristics to determine if SDAP B is applicable. The bridge site must not exceed $F_v S_1$ limitations and the structure must meet certain regularity requirements as defined in Section 3.10.3.3.2.
- *Step 2* - Reinforced concrete columns shall be designed using non-seismic loading cases and checked for minimum longitudinal reinforcement (0.8%).
Step 3 - Reinforced concrete columns shall be detailed to meet the shear, confinement and bar restraint reinforcement requirements of Article 5.10.11.4.1c through e in the plastic hinge zones defined in Article 3.10.3.9.
- *Step 4* - Steel columns shall be designed using non-seismic loading cases and checked for minimum width to thickness ratios as described in Chapter 6. Plastic hinge zone forces shall be those from capacity design procedures of Article 3.10.3.8.
- *Step 5* - Members connecting to columns shall be designed to resist column plastic moments and shears using the principles of capacity design described in Article 3.10.3.8 using an overstrength ratio of 1.5 and 1.2 for concrete and steel respectively.
- *Step 6* - Foundations (soils and piles) shall be designed to resist column moment and shears

inherent ductility to survive without collapse. Even though bridge columns in SDR 2 are not required to be designed for seismic loads, shear reinforcement requirements will provide a minimum level of capacity for ductile deformations which is considered to be adequate for the magnitude and duration of the ground motion expected in SDR 2.

The magnitude of live load assumed to exist at the time of the earthquake should be consistent with the value of g_{eq} used in conjunction with Table 3.4.1-1.

C3.10.3.3

The no analysis procedures are an important new addition to the provisions because they apply in the expanded areas now requiring more detailed seismic design. The purpose of these provisions is to provide the designers of regular bridges, that comply with certain restrictions, the ability to design their structure without the need to undertake a dynamic analysis. The bridge is designed for all non-seismic requirements and capacity design procedures are then used to determine shear reinforcement and confining reinforcement requirements. Capacity design principles are also used for the connection forces of the columns to the pile cap or spread footing and the superstructure or bent cap. There are no seismic design requirements for abutments except that integral abutments need to be designed for passive pressure. The superstructure displacements anticipated in these lower zones are expected to be relatively modest and significant abutment contribution to the response of the bridge is not anticipated but if it occurs it will reduce substructure displacements. The design forces for the soil and pile aspects of foundation design are the overstrength forces from the columns but using an overstrength ratio of 1.0. The use of the lower overstrength ratio for SDR 3 implies that there will be some limited ductility demand on the piles in the event of the 3% in 75-year earthquake. Since shear, confining and bar restraint reinforcement is also required in the top 3D of the piles this reduction in foundation design forces was believed to be prudent in the lower seismic risk areas. Current AASHTO Division 1A requirements (SPC B) do not require capacity design of the foundation, rather the foundations are designed for twice the column design forces. Converting to a capacity design approach with an overstrength ratio of 1.0 will lead to a more uniform level of seismic resistance in these lower seismic areas.

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using the principles of capacity design described in Article 3.10.3.8 using an overstrength ratio of 1.0.

3.10.3.3.2 Restrictions

SDAP B shall be used only at sites where:

$$F_V S_1 < 0.4 \cos a_{skew} \quad (3.10.4.3-1)$$

where a_{skew} = the skew angle of the bridge, (0 degrees being the angle for a right bridge).

Additionally, SDAP B shall be used only on structures that comply with the following restrictions:

- For concrete column and pile bents

- $P_e < 0.15 f'_c A_g$
- $r_\ell > 0.008$
- $D > 300\text{mm (12 inches)}$
- $\frac{M}{VD} < 6$

where P_e = column axial load
 f'_c = nominal 28 day concrete strength
 A_g = gross cross-sectional area of column
 r_ℓ = longitudinal reinforcement ratio
 D = column transverse dimension
 M = maximum column moment
 V = maximum column shear

- For concrete wall piers with low volumes of longitudinal steel:

- $P_e < 0.1 f'_c A_g$
- $r_\ell > 0.0025$
- $\frac{M}{VT} < 10$
- $t > 300\text{mm (12 inches)}$

where t = wall thickness, or smallest cross-sectional dimension.

- For steel pile bents framing into reinforced concrete caps:

- $P_e < 0.15 P_y$

Structures with lower axial loads or stronger columns (i.e., more steel and large column/pile sizes) have a greater intrinsic strength and are able to resist the design ground motions with less damage. However, ductile detailing still needs to be provided in accordance with Section 5.

The no analysis provisions are not applicable to steel braced frame substructures. In the case of a cantilever column, in a pile bent configuration, the length L in the $L/b < 10$ criteria would be equal the length above ground to the top of the bent plus 3 pile bent diameters.

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- $D_p \geq 250\text{mm (10 inches)}$
- $L/b < 10$
Column fixity at base or embedded in soil for pile bent where b is the flange width and L is the length from the point of maximum moment to the inflexion point of the column when subjected to a pure transverse load.

where D_p = pile dimension about the weak axis bending at ground line.

P_y = axial yield force of steel pile

- For timber piles framing into reinforced concrete caps or steel moment-frame columns:
 - $P_e < 0.1P_c$
 - $D_p \geq 250\text{mm (10 inches)}$
 - $\frac{M}{VD_p} < 10$

where P_c = axial compression capacity of the pile.

SDAP B shall NOT be used for bridges where:

- Individual interior bent stiffnesses vary by more than a factor of 2 with respect to the average bent stiffness of the bridge.
- The maximum span exceeds 80 m.
- The maximum span length is more than 50 percent longer than the average span length.
- The maximum skew angle exceeds 30 degrees
- For horizontally curved bridges the subtended angle exceeds 30 degrees.
- For frames in which the superstructure is continuous over the bents and for which some bents do not participate in the ERS, $F_v S_1$ factored by the ratio of the total number of bents in the frame divided by the number of bents in the frame that participate in the ERS in the longitudinal direction exceeds $0.4 \cos \alpha_{skew}$
- If the bridge site has a potential for liquefaction and the piers are seated on spread footings.
- The bridge site has a potential for liquefaction and the piers are seated on piled foundations unless the piles shall be detailed for ductility, in accordance

COMMENTARY

These provisions do not apply for bridges with variable height piers. Designers are encouraged to design the portion of piers participating in a seismic mechanism to have similar column lengths.

Variable span lengths can create uneven loading conditions on the piers resulting for unusual modal behavior.

For highly skewed bridges, biaxial loading of the piers can be problematic from a design point-of-view. Moreover, extra care needs to be taken in assessing the displacement demands at joints and bearings.

Designers are actively discouraged from using one pier to resist all longitudinal inertia loads when using this analysis method. Its use is most appropriate when all supporting bents participate in the ERS.

Careful and site specific analysis of the soil-structure interaction is needed at sites with liquefaction or lateral spreading potential.

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with these provisions over the length passing through the liquifiable soil layer plus an additional length of three-pile diameters or 3 m (10 ft) whichever is larger, above and below the liquefiable soil layer.

3.10.3.3.3 Capacity Design and Strength Requirements of Members Framing into Columns

Except for the geotechnical design of foundations, SDAP B requires the use of capacity design for all components connected to the columns (Article 3.10.3.8). For the geotechnical design of foundations, the moment overstrength capacity of columns that frame into the foundations need not be taken as greater than:

$$M_{po} = 1.0 M_n$$

Where

M_{po} = plastic overstrength capacity of a column

M_n = nominal moment capacity of a column

3.10.3.4 SDAP C – CAPACITY SPECTRUM DESIGN METHOD

3.10.3.4.1 Capacity Spectrum Design Approach

SDAP C combines a demand and capacity analysis, including the effect of inelastic behavior of ductile earthquake resisting elements. The procedure applies only to bridges that behave essentially as a single degree-of-freedom system. SDAP C is restricted to bridges with a very regular configuration as described in Article 3.10.3.4.2 and with the recommended earthquake resisting systems (ERS) as described in Section 2.

The major steps in applying the capacity spectrum method for the two levels of earthquake are as follows:

- *Step 1* - Design the bridge for the non-seismic load combinations. Determine the applicability of SDAP C.
- *Step 2* - Check if the design for non-seismic loads satisfies the requirements for the 50% in 75-year

COMMENTARY

C3.10.3.3

The principles of capacity design require that the strength of those members that are not part of the primary energy dissipating system be stronger than the overstrength capacity of the primary energy dissipating members—that is, the columns with hinges at their member ends.

The geotechnical features of foundations (i.e. soil bearing, and side friction and end bearing on piles) possess inherent ductility. At low to moderate levels of seismic input this manifests itself as minor rocking of the foundation and/or nominal permanent settlements which do not significantly affect the service level of the bridge.

Full capacity protection of the geotechnical features of the foundation in SDAP B is not required. Should the rare earthquake occur, some limited ductility demand may occur in the piles and some minor rocking and permanent settlement may occur. This trade-off, compared to current practice for SPC-B in the existing AASHTO provisions, was believed to be prudent.

C3.10.3.4

The capacity spectrum design method is conceptually the same as the Caltrans displacement based design method.

The primary difference is that the capacity spectrum approach begins with the existing nonseismic capacity of the columns and then assesses the adequacy of the resulting displacements. The Caltrans procedure uses methods to estimate the maximum displacement that can be tolerated and then assesses the minimum strength requirements for the column.

The key equation used in the capacity spectrum method is the relationship between the seismic coefficient, C_s , and displacement, Δ :

$$C_s \Delta = \left(\frac{F_v S_1}{2p B_L} \right)^2 g$$

in which S_1 is the spectral acceleration coefficient at 1

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earthquake event.

- *Step 3* - Design for the 50% in 75-year earthquake event if necessary from Step 2.
- *Step 4* - With a design that satisfies the non-seismic load combinations and the 50% in 75-year earthquake event, check that the requirements for the 3% in 75 year earthquake event are satisfied.
- *Step 5* - If necessary from Step 4, modify the design to satisfy the requirements for the 3% in 75-year event.
- *Step 6* - Design and detail the columns, the connections of the columns to the foundation, and superstructure or column bent using the capacity design procedures of Article 3.10.3.8. For bridges in SDR 3, the requirements of Article 3.10.3.3.3 are applicable.

Details for each of these steps are discussed in the Commentary.

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second period, F_v is the site factor for the earthquake event, and g is the acceleration due to gravity (32.2 ft/sec^2 or 9.8 m/sec^2). The factor B_L reduces the demand to account for inelastic deformation capacity of the earthquake resisting elements; Table 4.8.5.1-1 gives B_L for the two earthquake events and two performance levels. This equation is valid in the velocity-sensitive region of the response spectrum and is applicable to most bridges. The complete design procedure includes steps for shorter period bridges, such as those with pier walls, but such cases are not discussed in this commentary.

The following detailed summary of this method expands on the procedure outlined in the Specification. It focuses on conservative estimates of strength and displacement. More refined techniques may be used which still satisfy the capacity spectrum method, but for most cases the simple approach described herein provides efficient designs that will satisfy the performance requirements defined in the Specifications.

Step 1

With the design for all non-seismic requirements determine if the configuration and component requirements for a very regular bridge are satisfied. If so, the capacity spectrum procedure may be used.

Step 2

Determine F_v and S_1 for the 50% in 75-year earthquake event. In the longitudinal and transverse direction, perform the following sub steps:

2-1. Compute the yield displacement, Δ_y , for each participating bent or pier; set Δ_y to 1.3 times the smallest value. Note that a participating pier or bent is one whose fixity conditions permits it to resist horizontal lateral loads. It is possible a pier may participate transversely but not longitudinally due to a bearing that has transverse fixity and longitudinal movement.

2-2. Compute the lateral strength of each participating pier or bent, and sum the strengths to give the lateral strength of the bridge, V_n . The seismic coefficient for the bridge is $C_s = V_n/W$, in which W is the weight of the bridge responding to earthquake ground motion (generally the superstructure and a portion of the substructure).

2-3. If the following equation is satisfied for the 50% in 75-year values of F_v and S_1 ,

$$C_s \Delta_y \leq \left(\frac{F_v S_1}{2p} \right)^2 g$$

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the bridge is expected to meet the performance requirement for the 50% in 75-year earthquake event.

Step 3

If the equation in step 2-3 is not satisfied, increase the strength of the participating piers or reconfigure the bridge so more piers participate such that V_n and C_s satisfy step 2-3.

Step 4

Determine F_v and S_1 for the 3% in 75-year earthquake event. For the strength of the bridge in step 3, determine if the bridge has sufficient deformation capacity according to the following sub steps in the longitudinal and transverse directions:

4-1. Using the strength from step 3, determine the maximum displacement from:

$$\Delta = \frac{1}{C_s} \left(\frac{F_v S_1}{2p B_L} \right)^2 g$$

where B_L is obtained from Table 4.8.5.1-1.

4-2 Check that the maximum displacement is less than the deformation capacity for the shortest pier, with height H :

$$\Delta \leq q_p H$$

for reinforced concrete columns satisfying the requirements of Section 5, the plastic rotation capacity, q_p , may be taken as 0.035 or as given in Article 5.16.2. A similar value is applicable for steel columns that satisfy the requirements of Section 6 or as given in Article 6.15.6.

4-3. Check that the P-delta requirement is met using the height of the shortest participating pier:

$$\Delta \leq 0.25 C_s H$$

If the displacement limits in steps 4-2 and 4-3 are met, the design is satisfactory for the 3% in 75-year earthquake event.

Step 5

If the displacement limits in step 4 are not satisfied, the strength of the participating piers must be increased or additional piers must participate. For reinforced concrete columns it is necessary to increase the longitudinal

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reinforcement. If the reinforcement ratio exceeds 2.5%, the column size may need to be increased. The new strength can be determined as follows, in the longitudinal and transverse directions:

5-1. If step 4.2 is not satisfied, set the maximum displacement to $\Delta = q_p H$, where H is the height of the shortest participating column. Determine the required seismic coefficient from,

$$C_s = \frac{1}{\Delta} \left(\frac{F_v S_1}{2pB_L} \right)^2 g$$

5-2. If step 4.3 is not satisfied, determine the required seismic coefficient from,

$$C_s = 4 \frac{\Delta}{H}$$

where H is the height of the shortest column.

5-3. The required lateral strength is $V_n = C_s W$, where W is the total weight of the bridge. Apportion V_n to the individual piers participating in resisting lateral loads in proportion to the tributary mass for the pier. Redesign the piers to provide the required strength.

Bridges that satisfy step 4 and 5 are expected to have satisfactory performance in the 3% in 75-year earthquake event for each performance level.

Step 6

Capacity design procedures of Article 3.10.3.8 are used to determine the shear and confinement reinforcement requirements, the column connection forces and the foundation design forces. The bridge is designed so it can resist the 3% in 75-year event without any contribution from the abutment and hence there are no seismic design requirements for the abutments.

3.10.3.4.2 Restrictions

SDAP C shall only be used on bridges that satisfy the following requirements:

- The number of spans per frame or unit shall not exceed six.
- The number of spans per frame or unit shall be at least three, unless seismic isolation bearings are utilized at the abutments.
- Abutments shall not be assumed to resist significant seismic forces in the transverse or

The configuration requirements for Capacity spectrum analysis restrict application to individual frames or units that can be reasonably assumed to respond as a single degree-of-freedom in the transverse and longitudinal directions. When abutments do not resist significant seismic forces, the superstructure will respond as a rigid-body mass. The lateral load-resisting piers or bents must be uniform in strength and stiffness to justify the assumption of independent translational response in the

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significant seismic forces in the transverse or longitudinal directions.

- Span length shall not exceed 60 m (200 feet).
- The ratio of span lengths in a frame or unit shall not exceed 1.5.
- Pier wall substructures must have bearings that permit transverse movement.
- The maximum skew angle shall not exceed 30 degrees, and skew of piers or bents shall not differ by more than 5 degrees in the same direction.
- For horizontally curved bridges, the subtended angle of the frame shall not exceed 20 degrees.
- The ratio of bent or pier stiffness shall not vary by more than 2 with respect to the average bent stiffness, including the effect of foundation stiffness.
- The ratio of lateral strength (or seismic coefficient) shall not exceed 1.5 of the average bent strength.
- For concrete columns and pile bents:
 - $P \leq 0.20f_c A_g$
 - $r_{\ell} > 0.008$
 - $D \geq 300\text{mm (12 inches)}$
- When liquefaction potential is determined to exist according to the requirements in Article 3.10.4.1, the piers or bents must have pile foundations.

COMMENTARY

longitudinal and transverse directions.

SDAP C may be appropriate for pier wall substructures in the longitudinal direction but will not work in the transverse direction if bearings are fixed. If bearings permit movement transversely, then the capacity spectrum method for isolation bearings (Article 15.4) shall be used.

These requirements are similar to the ones for no-analysis in Article 3.10.9.3.2.

3.10.3.5 SDAP D - ELASTIC RESPONSE SPECTRUM METHOD

C3.10.3.5

SDAP D is a one step design procedure using an elastic (cracked section properties) analysis. Either the Uniform Load or Multimode method of analysis may be used. The analysis shall be performed for the governing design spectra (either the 50% in 75-year or the 3% in 75-year) and the R-Factors given in Tables 3.10.3.7.1-1 and 3.10.3.7.1-2 shall be used to modify elastic response values. The analysis shall determine the elastic moment demand at all plastic hinge locations in the columns. Capacity design principles shall be used for column shear design and the design of all column connections and foundation design. If sacrificial elements are part of the design (i.e. shear keys)

This is essentially a two level design procedure, however in many parts of the US, and in the Eastern US in particular, the 50% in 75 year event will rarely govern. In most cases designers will be able to quickly assess which of the two events will produce the maximum column moments by dividing the ground response spectra by the respective R factors and comparing the relative values. Only when the two spectra are relatively close will two analyses be required.

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they shall be sized to resist the 50% in 75-year forces and the bridge shall be capable of resisting the 3% in 75-year forces without the sacrificial elements (i.e. two analyses are required if sacrificial elements exist in a bridge).

This design procedure consists of the following steps:

- *Step 1* - Design the bridge for non-seismic loading conditions.
- *Step 2* - Perform an elastic dynamic analysis as described in Article 4.7 for the 3% in 75-year earthquake loading to determine displacement demands. Analysis shall reflect the anticipated condition of the structure and the foundation during this earthquake.
- *Step 3* - Determine controlling seismic design forces for the moment design of all columns from an elastic dynamic analysis using either the 50% in 75- or 3% in 75-year earthquake. Analyses shall reflect the anticipated condition of the structure and the foundation during each of these earthquakes. Elastic forces from the analyses shall be modified using the appropriate R factors from Tables 3.10.3.7.1-1 and 3.10.3.7.1-2.
- *Step 4* – Determine the minimum design base shear for each column using the *P-D* requirements from Article 3.10.3.10.4 using the elastic displacements obtained in Step 2. Modify column design as necessary.
- *Step 5* - Determine the design forces for other structural actions using Capacity Design as described in Article 3.10.3.8.
- *Step 6* - Design sacrificial elements to resist forces generated by the 50% in 75-year earthquake.

3.10.3.6 SDAP E – ELASTIC RESPONSE SPECTRUM METHOD WITH DISPLACEMENT CAPACITY VERIFICATION

SDAP E requires an elastic (cracked section properties) response spectrum analysis for the governing design spectra (50% in 75-year or 3% in 75-year) and *P-?* design. The results of these analyses shall be used to perform preliminary flexural design of hinging members and to determine the displacement of the structure. To take advantage of the higher R Factors in Table 3.10.3.7.1-1, displacement capacities shall be verified using two-dimensional nonlinear static (pushover) analyses in the principal structural directions. Design forces on substructure elements may be reduced below those

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obtained for the 3% in 75-year event divided the the R-Factor, but not lower than 70% of these forces nor the 50% in 75-year forces and only if the displacement capacity of the element is satisfied as part of the pushover analysis. If column sizes are reduced as part of a force redistribution process in the pushover analysis then the elastic analysis used as the basis of the design process shall reflect the final sizing of the substructure members. Capacity design principles of Article 3.10.3.8 shall be used to design the foundations and for column shear design. SDAP E is required when owner approved ERE are used that have inelastic action that cannot be inspected.

This design procedure shall consist of the following steps:

- *Step 1* - Perform Steps 1 through 4 for SDAP D except that the appropriate R factors from Tables 3.10.3.7.1-1 and 3.10.3.7.1-2 shall be used.
- *Step 2* - Perform a Displacement Capacity Verification analysis using the procedures described in Article 4.8.5.4. If sufficient displacement capacity exists the substructure design forces may be further reduced from those at Stem 1, but not less than 70% of the Stem 1 forces nor less than design forces from the 50% in 75-year event. If column sizes are reduced, repeat Step 2 of SDAP D and these displacements shall be used in repeat of this step in SDAP E.
- *Step 3* - Perform Steps 5 and 6 for SDAP D.

3.10.3.7 RESPONSE MODIFICATION FACTORS

Structures that are designed using SDAP D or E shall use the response modification factors defined in this article.

3.10.3.7.1 General

To apply the response modification factors specified herein, the structural details shall satisfy the provisions of Articles 5.10.2.2, 5.10.11, and 5.13.4.6 and Section 6.

Except as noted herein, seismic design force effects for flexural design of the primary plastic hinges in substructures shall be determined by dividing the force effects resulting from elastic analysis by the appropriate response modification factor, R , as given by

$$R = 1 + (R_B - 1) \frac{T}{T^*} \leq R_B$$

where R_B is given in Table 3.10.3.7.1-1., T is the period of vibration and $T^* = 1.25 T_s$, where T_s is defined in Figure 3.10.2.1-3

C3.10.3.6.1

These Specifications recognize that it is uneconomical to design a bridge to resist large earthquakes elastically. Columns are assumed to deform inelastically where seismic forces exceed their design level, which is established by dividing the elastically computed force effects by the appropriate R-factor. Most other elements of the ERS are designed by capacity design procedures for the maximum forces that can be developed by plastic hinges in the columns or the elastic forces from the analysis.

The most important R-Factor is that of the supporting substructure. Since a bridge closely approximates a single-degree-of-freedom (SDOF) system, the design process is schematically shown Figure C2.5.6-2 and discussed in C2.5.6. There has been a considerable amount of research over the past ten years on the relationship

between the ductility demand of a SDOF system and its design strength. For example, if we assume an element has a displacement ductility capacity m at a given value, we would like to know the design force necessary to ensure that this ductility is not exceeded. A good overview of this issue can be found in ATC-18 (1997), which summarizes the work of Mirander and Bertero (1996), Nasser and Krawinkler (1991) and Chang and Mander (1994). Figure C3.10.3.6.1-1 shows a smoothed relationship (Mirander and Bertero, 1996) between the ductility factor m and R for two sites. Note that R is less than m for periods less than one second and hence the need for the short period modifier on R given by Equation 3.10.3.6.1-1

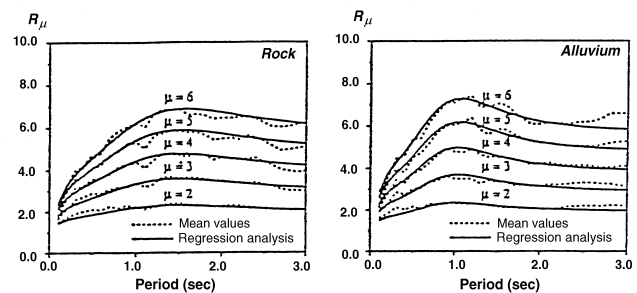


Figure C3.10.3.6.1-1 Comparison of Mean Strength-Reduction Factors of Rock and Alluvium Sites with Regression Analysis

The R -Factors of Table 3.10.3.7.1-1 were based on an evaluation of existing test data of structural components, parameter studies that were performed in conjunction with the development of these provisions and engineering judgment. The Project Team first reviewed the test data on reinforced concrete columns (Taylor and Stone, 1993; Hose, Silvan and Sieble, 1999) to establish the range of ductility capacity that could be relied upon. This was in the range of 6-10 for well-detailed columns, depending on the range of design parameters (e.g., axial load, longitudinal and confinement reinforcement, etc.). The parameter study associated with the development of this criteria showed that there were only a limited number of instances where use of an R -Factor greater than 6 would not be limited either by the minimum longitudinal steel requirement of 0.8% in concrete columns or the P - D requirements of Article 3.10.3.10.4. As a consequence the R -Factor for concrete and steel columns was set at 6 for SDAP E with a provision that the design forces could be further reduced (not lower than 70%) provided the displacement capacity of the element was satisfied in the pushover analysis.

TABLE 3.10.3.7.1-1 – BASE RESPONSE MODIFICATION FACTORS, R_B , FOR SUBSTRUCTURE

Substructure Element	Performance Objective			
	Life Safety		Operational	
	SDAP D	SDAP E	SDAP D	SDAP E
Wall Piers – larger dimension	2	3	1	1.5
Columns – Single and Multiple	4	6	1.5	2.5
Pile Bents and Drilled Shafts – Vertical Piles – above ground	4	6	1.5	2.5
Pile Bents and Drilled Shafts – Vertical Piles – 2 diameters below ground level-No owners approval required.	1	1.5	1	1
Pile Bents and Drilled Shafts – Vertical Piles – in ground - Owners approval required.	N/A	2.5	N/A	1.5
Pile Bents with Batter Piles	N/A	2	N/A	1.5
Seismically Isolated Structures	1.5	1.5	1	1.5
Steel Braced Frame – Ductile Components	3	4.5	1	1.5
Steel Braced frame – Nominally Ductile Components	1.5	2	1	1
All Elements for expected Earthquake	1.3	1.3	0.9	0.9

Notes:

1. The substructure design forces resulting from the elastic analysis divided by the appropriate R-Factor for SDAP E cannot be reduced below 70% at these R-Factor reduced forces as part of the pushover analysis.
2. There maybe design situations (e.g architecturally oversized columns) where a designer opts to design the column for an $R=1.0$ (i.e. elastic design). In concrete columns the associated elastic design shear force may be obtained from the elastic analysis forces using an R-Factor of 0.67 or by calculating the design shear by capacity design procedures using a flexural overstrength factor of 1.0. In steel braced frames if an $R=1.0$ is used the connection design forces shall be obtained using an $R=0.67$. If an $R=1.0$ is used in any design the foundations shall be designed for the elastic forces plus the SDR 2 detailing requirements are required for concrete piles. (i.e. minimum shear requirements). – Article 3.10.3.11.
3. Unless specifically stated, the R factors apply to both steel and concrete.
4. N/A in this case means that owners approval is required and thus SDAP E is required to use this design option.

TABLE 3.10.3.7.1-2 - RESPONSE MODIFICATION FACTORS - CONNECTIONS

Connection	All Performance Objectives
Superstructure to abutment	.8
Expansion joints within a span of the superstructure	.8
Columns, piers, or pile bents to cap beam or superstructure	.8
Columns or piers to foundations	.8

Note: These factors are not intended for those cases where capacity design principles are used to design the connections.

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3.10.3.7.2 Application

A wall-type concrete pier may be analyzed as a single column in the weak direction if all the provisions for columns, as specified in Section 5, are satisfied.

C3.10.3.7.2

Wall-type piers may be treated as wide columns in the strong direction, provided the appropriate R-factor in this direction is used.

3.10.3.8 CAPACITY DESIGN

C3.10.4.8

3.10.3.8.1 General

C3.10.3.8.1

Capacity design principles require that those elements not participating as part of the primary energy dissipating system (flexural hinging in columns), such as column shear, joints and cap beams, spread footings, pile caps and foundations be “capacity protected”. This is achieved by ensuring the maximum moment and shear from plastic hinges in the columns (overstrength) can be dependably resisted by adjoining elements.

Exception: Elastic design of all substructure elements (Article 3.10.3.11), seismic isolation design (Article 3.10.3.13) and in the transverse direction of a column when a ductile diaphragm is used.

The objective of these provisions for conventional design is that inelastic deformation (plastic hinging) occurs at the location in the columns (top and/or bottom) where they can be readily inspected and/or repaired. To achieve this objective all members connected to the columns, the shear capacity of the column and all members in the load path from the superstructure to the foundation, shall be capable of transmitting the maximum (overstrength) force effects developed by plastic hinges in the columns. The exceptions to the need for capacity design of connecting elements is when all substructure elements are designed elastically (Article 3.10.3.11), seismic isolation design (Article 3.10.3.13) and in the transverse direction of columns when a ductile diaphragm is used.

3.10.3.8.2 Inelastic Hinging Forces

C3.10.3.8.2

Inelastic hinges shall form before any other failure due to overstress or instability in the structure and/or in the foundation. Except for pile bents and drilled shafts, and with owners' approval, inelastic hinges shall only be permitted at locations in columns where they can be readily inspected and/or repaired.

Superstructure and substructure components and their connections to columns that are designed not to yield shall be designed to resist overstrength moments and shears of yielding members. Except for the geotechnical aspects of design of foundations in SDR 3, the moment overstrength capacity (M_{po}) of column/pier/pile members that form part of the primary mechanism resisting seismic loads shall be assessed using one of the following approaches:

- $M_{po} = 1.5 M_n$, for concrete columns
= $1.2 M_n$ for steel columns
= $1.3 M_n$ for concrete filled steel tubes
= $1.5 M_n$ for steel piles in weak axis bending and for steel members in shear (e.g. eccentrically braced frames)

where M_n is the nominal moment strength in which expected yield strengths are used for steel members (Article 6.15.2)

- For reinforced concrete columns the plastic analysis approach given by Article 5.10.11.4.1h.

The principles of capacity design require that the strength of those members that are not part of the primary energy dissipating system be stronger than the overstrength capacity of the primary energy dissipating members—that is, the columns with hinges at their member ends.

This clause permits three approaches of increasing sophistication (but also of increasing effort to conduct) for assessing the overstrength capacity of reinforced concrete columns. See Article 3.10.3.3.3 for foundation design in SDR 3.

Overstrength factors applied to nominal moment capacities are a simplified method for determining flexural overstrength. For reinforced concrete columns, detailed calculations of overstrength factors for a variety of column properties (Mander, Dutta and Goel (1997)) ranged from 1.25 to 1.50. A conservative default value of 1.5 is specified for the first approach but a designer can calculate a more precise project specific value using one of the remaining two approaches.

For the second approach, the flexural moment overstrength capacity (M_{po}) of reinforced concrete column/pier/pile members that form part of the primary mechanism resisting seismic loads may be assessed using the simplified plastic moment-axial load interaction formula method developed in Mander, Dutta and Goel (1997) – See Article 5.10.11.4h. It is recommended that for this approach f'_{co} for concrete be assumed to be $1.7f'_c$ and

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- For reinforced concrete columns a compatibility section analysis, taking into account the expected strengths of the materials and the confined concrete properties and the strain hardening effects of the longitudinal reinforcement.

These overstrength moments and associated shear forces, calculated on the basis of inelastic hinging at overstrength, shall be taken as the extreme seismic forces that the bridge is capable of developing. Typical methods of applying capacity design at a bent in the longitudinal and transverse directions are shown in Figure 3.10.3.8.2-1.

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f_{yo} of steel be $1.3f_y$

When assessing overstrength capacity of flexural members using the third approach, compatibility section analysis (i.e the moment-curvature method), it is important to differentiate between overstrength resulting from the response of the section to high curvature demands, and overstrength resulting from upper bound material properties.

For example, in the case of reinforced concrete columns, confined concrete will have enhanced capacity and reinforcing steel will strain harden at high plastic curvatures. This will result in increased flexural capacity of the column that will be captured by a moment curvature analysis that considers these factors. In addition, reinforcing steel can have a higher than nominal yield point, and concrete is likely to be stronger than specified and will gain strength with age beyond the 28 day specified strength. It has been recommended that for the purpose of a rigorous calculation that f'_{co} for concrete be assumed to be $1.7f'_c$ and f_{yo} of steel be $1.3f_y$. In this case the overstrength moment is taken at the design curvature from the moment curvature analysis (ATC, 1996).

For structural steel, f_{yo} may be taken as $1.2F_{ye}$ where F_{ye} is the expected yield strength considering the likelihood that higher than nominal strength steel will be used. The plastic section modulus should be used in overstrength moment calculations for steel members.

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3.10.3.8.2(a) Single Columns and Piers

Column shear forces and design moments in the superstructure, bent caps, and the foundation structure shall be calculated for the two principal axes of a column and in the weak direction of a pier or bent as follows:

- **Step 1.** Determine the column overstrength moment capacities. For reinforced concrete columns, use an overstrength factor given in Article 3.10.3.8 times the nominal moment. The nominal moment for steel members is calculated using the expected yield strengths of Article 6.15.2. For both materials use the maximum elastic column axial load from Section 3.10.2.4 added to the column dead load. Column overstrength moments should be distributed to the connecting structural elements. (Exception: when calculating the design forces for the geotechnical aspects of foundations in SDR 3, use an overstrength factor of 1.0 on the nominal moment.)
- **Step 2.** Using the column overstrength moments, calculate the corresponding column shear force assuming a quasi-static condition. For flared columns designed to be monolithic with the superstructure or with isolation gaps less than required by Article 5.10.11.4.1, the shear shall be calculated as the greatest shear obtained from using:
 - a) The overstrength moment at both the top of the flare and the top of the foundation with the appropriate column height.
 - b) The overstrength moment at both the bottom of the flare and the top of the foundation with the reduced column height.

If the foundation of a column is significantly below ground level, the column height for the capacity shear force shall be based on the mud or ground line, not the top of the foundation.

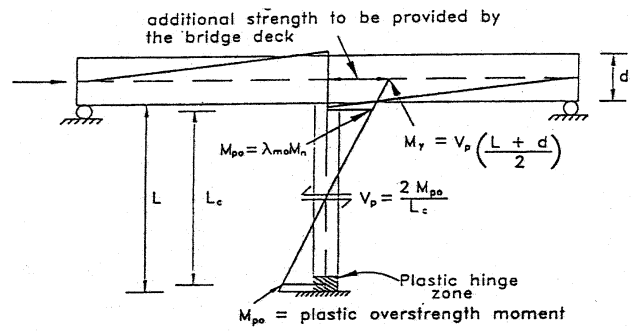
For pile bents or drilled shafts, the length of the pile or drilled shaft shall be not lower than the ground line for the purpose of calculating the shear force.

The forces corresponding to a single column hinging are:

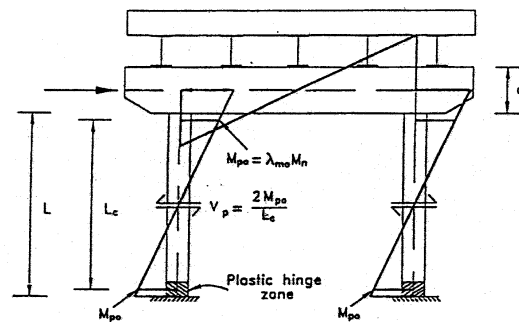
- **Axial Forces** —unreduced maximum and minimum seismic axial load of Article 3.10.2.6 plus the dead load.

COMMENTARY

C3.10.3.8.2(a)



(a) Longitudinal Response



(b) Transverse Response

Figure 3.10.3.8.2-1 Capacity Design of Bridges using Overstrength Concepts.

This conservative requirement to calculate the capacity design shear force will be adequate if fixity of the column occurs any time in the future. If a concrete traffic barrier could reduce the fixity at the column then the height down to the barrier should be considered in the shear force calculation.

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- Moments—those calculated in Step 1.
- Shear Force—that calculated in Step 2.

3.10.3.8.2(b) *Bents with Two or More Columns*

The forces for bents with two or more columns shall be calculated both in the plane of the bent and perpendicular to the plane of the bent. Perpendicular to the plane of the bent the forces shall be calculated as for single columns in Article 3.10.3.8.2(a). In the plane of the bent the forces shall be calculated as follows:

- *Step 1.* Determine the column overstrength moment capacities. Use an overstrength factor of 1.5 on the nominal strength for reinforced concrete and 1.2 on the nominal strength calculated using the expected yield strength for structural steel. For both materials use the axial load corresponding to the dead load. (Exception: When calculating the design forces for the geotechnical aspects of foundations in SDR 3 use an overstrength factor of 1.0 on the nominal moment.
- *Step 2.* Using the column overstrength moments calculate the corresponding column shear forces. Sum the column shears of the bent to determine the maximum shear force for the bent. Note that, if a partial-height wall exists between the columns, the effective column height is taken from the top of the wall. For flared columns and foundations below ground level see Article 3.10.3.8.2(a) - Step 2. For pile bents the length of pile from the pile cap to the mud or ground line shall be used to calculate the shear force.
- *Step 3.* Apply the bent shear force to the top of the bent (center of mass of the superstructure above the bent) and determine the axial forces in the columns due to overturning when the column overstrength moments are developed.
- *Step 4.* Using these column axial forces combined with the dead load axial forces, determine revised column overstrength moments. With the revised overstrength moments calculate the column shear forces and the maximum shear force for the bent. If the maximum shear force for the bent is not within 10% of the value previously determined, use this maximum bent shear force and return to Step 3.

The forces in the individual columns in the plane of a

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bent corresponding to column hinging, are:

- Axial Forces—the maximum and minimum axial load is the dead load plus, or minus, the axial load determined from the final iteration of Step 3.
- Moments—the column overstrength plastic moments corresponding to the maximum compressive axial load specified in (1) with an overstrength factor specified in Article 3.10.3.8.2 (1.5 on the nominal moment for reinforced concrete and 1.2 on the nominal moment using expected yield strengths for structural steel). Exception: An overstrength factor of 1.0 is required for geotechnical design forces in SDR 3.
- Shear Force—the shear force corresponding to the final column overstrength moments in Step 4 above.

3.10.3.8.2(c) Capacity Design Forces

Design forces for columns and pile bents shall be determined using the provisions of Article 3.10.3.8.2(a) and/or (b). Design forces for pier walls in the weak direction shall be determined using the provisions of Article 3.10.3.8.2(a). The capacity design forces for the shear design of individual columns, pile bents or drilled shafts shall be those determined using Article 3.10.3.8.2(a) and/or (b). The capacity design forces for the connection of the column to the foundation, cap beam or superstructure shall be the axial forces, moments and shears determined using the provisions of Article 3.10.3.8.2(a) and/or (b). The bearing supporting a superstructure shall be capable of transferring the shear forces determined using the provisions of Article 3.10.3.8.2(a) and/or (b) in both the longitudinal and transverse directions. The capacity design forces for superstructure design (Article 3.10.3.12) shall be the shear forces and where appropriate the moments of Article 3.10.3.8.2(a) and/or (b). The abutment forces associated with the superstructure design shall be the elastic forces from the analysis.

3.10.3.9 PLASTIC HINGE ZONES

C3.10.3.9

Columns, pile bents/caissons and piles that participate in the ERS will have plastic hinges occurring and special detailing in these zones is specified in Sections 5 and 6. The plastic hinge zones defined below cover the potential range of locations where a plastic hinge may occur.

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3.10.3.9.1 Top Zone of Columns, Pile Bents and Drilled Shafts

For concrete and steel columns, pile bents and drilled shafts the plastic hinge zone at the top of the member is defined as the length of the member below the soffit of the superstructure for monolithic construction and below the soffit of girders or cap beams for bents. The plastic hinge zone length shall be the maximum of the following.

- The maximum cross-sectional dimension of a reinforced concrete column
- One sixth of the clear height of a reinforced concrete column
- One eighth of the clear height of a steel column
- 450mm
- For reinforced concrete columns the following additional criteria are applicable

$$\begin{aligned} & D(\cot q + \frac{1}{2} \tan q) \\ & 1.5(0.08 M/V + 4400 e_y d_b) \\ & M/V(1 - M_y/M_{po}) \end{aligned}$$

where

- D = transverse column dimension in direction of bending
- T = principal crack angle from Eqn. 5.10.11.4.1-6
- e_y = yield strain of longitudinal reinforcement
- d_b = longitudinal bar diameter
- M = maximum column moment
- V = maximum column shear
- M_y = column yield moment
- M_{po} = column plastic overstrength moment

- For flared columns the plastic hinge zone shall extend from the top of the column to a distance equal to the maximum of the above criteria below the bottom of the flare.

3.10.3.9.2 Bottom Zone of a Column Above a Footing or Above an Oversized In-ground Drilled Shaft

The plastic hinge zone above the top of the footing of a column or a drilled shaft designed so that the maximum moment is above ground shall be the maximum of the items given in 3.10.3.9.1 unless the footing or the transition between in- ground and above ground drilled shafts is below the ground level in which case it shall extend from the top of the footing or the transition between the two shafts to a distance above the mud or ground line equal to the maximum of the items given in 3.10.3.9.1.

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3.10.3.9.3 Bottom Zone of Pile Bents and Drilled Shafts/Caissons

The plastic hinge zone at the bottom of a pile bent or a uniform diameter drilled shaft/caisson shall extend a distance above the mud or ground line equal to the maximum of the items specified in 3.10.3.9.1 to a distance 10D below the mud or ground line or 15 ft. whichever is greater. It need not exceed 3D below the point of maximum moment. If scour or liquefaction may occur it shall extend a distance of 3D below the mean scour depth or 3D below the lowest liquefiable layer. If a drilled shaft has an oversized in-ground shaft the top 10D of the oversized shaft shall be treated like the Zone of a pile below the pile cap.

3.10.3.9.4 Zone of a Pile Below the Pile Cap

It shall extend a depth equal to 10D below the pile cap or 15ft whichever is greater. It need not exceed 3D below the point of maximum moment. If scour or liquefaction may occur the zone shall extend to 3D below the mean scour depth or 3D below the lowest liquefiable layer.

3.10.3.10 MINIMUM DISPLACEMENT REQUIREMENTS

C3.10.3.10

3.10.3.10.1 General

C3.10.3.10.1

For this section, displacement is the displacement at the center of mass for a pier or bent in the transverse or longitudinal direction determined from the seismic analysis.

3.10.3.10.2 Minimum Seat Width Requirement

C3.10.3.10.2

The seat width shall not be less than (1) 1.5 times the displacement of the superstructure at the seat according to Equation (3.10.3.10.4-2); or (2):

Unseating of girders at abutments and piers must be avoided in all circumstances. The current Division I-A requirement for minimum seat width is:

$$N = 0.20 + 0.0017L + 0.0067H$$

$$N = \left[0.10 + 0.0017L + 0.007H + 0.05\sqrt{H} \cdot \sqrt{1 + \left(\frac{B}{L} \right)^2} \right] \frac{(1 + 1.25F_v S_1)}{\cos \alpha} \quad (3.10.3.10.1-1)$$

where,

L is the distance between joints in meters

H is the tallest pier between the joints in meters

B is the width of the superstructure in meters

α is the skew angle

The ratio B/L need not be taken greater than 3/8.

for seismic performance categories A and B. The seat width is multiplied by 1.5 for SPC C and D. The seat width is further multiplied by $1/\cos \alpha$ to account for skew effects. The current expression gives reasonable minimum seat widths, but it is modified herein for larger seismic zones.

The requirement for minimum seat width accounts for (1) relative displacement due to out-of-phase ground motion of the piers, (2) rotation of pier footings, and (3) longitudinal and transverse deformation of the pier. The current expression provides reasonable estimates of the first two effects, but underestimates the third. The maximum deformation demand is given by the P- Δ limitation because P- Δ generally controls the displacement of the piers. The capacity spectrum gives:

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$$C_s \Delta = \frac{F_v S_1}{2pB} \frac{\Delta}{H} g$$

and the P-Δ limitation is:

$$C_s > 4 \frac{\Delta}{H}$$

Combining the two expressions gives the maximum displacement when P-Δ controls:

$$\Delta = \frac{\sqrt{g}}{4pB} \sqrt{H} \cdot F_v S_1$$

Assuming $B=1.4$, with moderate ductility capacity, the longitudinal displacement limit in meter units is $\Delta_s = 0.18 \sqrt{H} \cdot F_v S_1$.

Transverse displacement of a pier supporting a span with fixed bearing and a span with a longitudinal release will result in additional seat displacement. The seat displacement at the edge of the span with the longitudinal release is $2\Delta_s B / L$. Combining the seat displacement due to longitudinal and transverse displacement of the pier using the SRSS combination rule gives the pier displacement contribution to seat width as:

$$N = 0.18 \sqrt{H} \sqrt{1 + \left(2 \frac{B}{L}\right)^2} \cdot F_v S_1$$

For $F_v S_1 = 0.40$ the coefficient is 0.072. Because transverse displacement of a pier is limited by "arching" of the superstructure, the maximum of $B/L=3/8$ is reasonable for determining the seat displacement.

Using this approach, the minimum seat width in (3.10.3.10.1-1) is a linear function of the seismic hazard, $F_v S_1$. The factor on seat width varies from unity for $F_v S_1 = 0$ to 1.5 for $F_v S_1 = 0.40$. The factor for $F_v S_1 = 0.80$ is 2.0. The coefficient for the pier deformation term provides a contribution to the seat width for $F_v S_1 = 0.40$ of:

$$N = 0.075 \sqrt{H} \sqrt{1 + \left(2 \frac{B}{L}\right)^2}$$

which is close to the value from the P-Δ analysis. The constant term is reduced from 0.20 to 0.10 because the pier deformation is included directly.

Equation (3.10.3.10.1-1) provides seat width that are slightly larger than the Division I-A requirement for low seismic zones and larger seat widths for $F_v S_1 = 0.80$ are larger by a factor of 1.5 to 1.8.

3.10.3.10.3 Displacement Compatibility

All components that are not designed to resist seismic loads must have deformation capacity sufficient to transfer

C3.10.3.10.3

Certain components may be designed to carry only dead and live loads (e.g. bearings, non-participating bents,

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non-seismic loads.

3.10.3.10.4 P-? Requirements

The displacement of a pier or bent in the longitudinal and transverse direction must satisfy

$$\Delta \leq 0.25C_s H \quad (3.10.3.10.4-1)$$

where,

$$\Delta = R_d \Delta_e \quad (3.10.3.10.4-2)$$

$$R_d = \left(1 - \frac{1}{R}\right) \frac{T^*}{T} + \frac{1}{R} \text{ for } T < T^* \quad (3.10.3.10.4-3)$$

where $T^* = 1.25 T_s$ where T_s is defined in Figure 3.10.2.3-1, otherwise $R_d = 1$,

Δ_e is the displacement demand from the seismic analysis, R is the ratio between elastic lateral force and the lateral strength of the pier or bent, C_s is the seismic coefficient based on the lateral strength, and H is the height of the pier from the point of fixity for the foundation.

If a nonlinear time history seismic analysis is performed, the displacement demand, Δ , may be obtained directly from the analysis in lieu of Equation 3.10.3.9.4-2. However, the displacement Δ shall not be taken less than 0.67 of the displacement determined from an elastic response spectrum analysis.

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etc.). Other components are non-structural, but their failure would be unacceptable or could result in structural problems (e.g. large diameter water pipes that could erode away soils if they failed). Under seismic loads these components must deform to remain compatible with their connections. The purpose of this section is to require a check that the non-seismic load resisting components have sufficient deformation capacity under seismically induced displacements of the bridge.

C3.10.3.10.4

Structures subject to earthquake ground motion may be susceptible to instability from P-?. Inadequate strength can result in "ratcheting" of structural displacement, with large residual deformation, and eventually instability. The intent of this section is to provide a minimum strength, or alternatively, a maximum displacement, for which P-? effects will not significantly affect seismic behavior of a bridge.

P-? produces a negative slope in a structures' force-displacement relationship equal to P/H .

The basis for the requirement in Equation 3.10.3.10.4-1 is that the maximum displacement is such that the reduction in resisting force is limited to a 25 percent reduction from the later strength assuming no post yield stiffness:

$$\Delta \frac{P}{H} < 0.25V \quad (C3.10.3.10.4-1)$$

where P is the gravity load on the substructure. Stating a limitation on displacement in terms of lateral strength is justified from dynamic analysis of SDF systems with various hysteretic relationships. requirement has been shown to limit P- Δ effects from dynamic analysis of single degree-of-freedom systems (Mahin and Boroschek, 1991, MacRae 1994). The requirement of Equation (C3.10.3.10.4-1) will avoid "ratching" in structures with typical post-yield stiffness.

The lateral strength can be expressed in terms of the seismic coefficient, $C_s = V / W$, which upon substitution into (C3.10.3.10.4-1) gives:

$$\Delta \leq 0.25C_s \left(\frac{W}{P} \right) H \quad (C3.10.3.10.4-2)$$

where W is the weight of the bridge responding to horizontal earthquake ground motion. For bridges in which the weight responding to horizontal ground motion is equal to gravity load on the substructure, Equation C3.10.3.10.4-2 gives Equation 3.10.3.10.4-1.

However, bridges with abutments may have a W/P ratio greater than unity if the abutments do not deform

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significantly, thus reducing P-Δ effects because a portion of the gravity load is resisted by the abutments. The Engineer may consider using Equation C3.10.3.10.4-2 with $W/P \leq 2$ when such an assumption is documented.

Equation 3.10.3.10.4-1 can also be stated as a minimum seismic coefficient to avoid P-Δ effects.

$$C_s > 4 \frac{\Delta}{H} \quad (\text{C3.10.3.10.4-3})$$

In the short period range, the equal displacement rule does not apply. Inelastic displacement will be greater than the elastic displacement according to:

$$\Delta_{inelastic} = \frac{R_B}{R} \Delta \quad (\text{C3.10.3.10.4-4})$$

in which R_B is the target reduction factor and R is the ratio of the lateral strength to the elastic force according to Article 3.10.3.6.1. Substitution of Equation 3.10.3.6.1-1 into C3.10.3.10.4-3 gives Equation 3.10.3.10.4-4.

3.10.3.10.5 Minimum Displacement Requirements for Lateral Load Resisting Piers and Bents

For SDAP E the displacement capacity from the Displacement Capacity Verification must be greater than the displacement demand according to the following requirement:

$$1.5\Delta \leq \Delta_{capacity}$$

where the Δ is defined in Article 3.10.3.10.4 and $\Delta_{capacity}$ is the maximum displacement capacity.

When a nonlinear dynamic analysis is performed the displacement demand may not be taken less than 0.67 times the demand from a elastic response spectrum analysis, nor may the displacement capacity be taken greater than the capacity from the Displacement Capacity Verification.

C3.10.3.10.5

The requirement in this section is based on the “equal displacement rule”, that is the maximum displacement from dynamic analysis with a linear model using cracked section properties is approximately equal to the maximum displacement for the yielding structure – Figure C2.5.6-2.

The factor of 1.5 on the displacement demand recognizes the approximations in the modeling for the seismic analysis. Furthermore, the demand analysis is performed for a model of the entire bridge including three-dimensional effects. However, the displacement capacity verification is done using a two-dimensional pushover analysis on individual bents. Since the relationship between the two methods of analysis is not well-established, the factor of 1.5 represents a degree of conservatism to account the lack of a rigorous basis for comparing displacement demand and capacity.

For very regular bridges satisfying the requirements for SDAP C in Article 3.10.3.4.2, the displacement requirement implied in the capacity spectrum approach does not include the 1.5 factor.

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3.10.3.11 ELASTIC DESIGN OF SUBSTRUCTURES

There may be instances where a designer chooses to design all of the substructure supports elastically (i.e., $R=1.0$ for all substructures) or in some cases a limited number of substructure elements are designed elastically

3.10.3.11.1 All Substructure Supports are Designed Elastically

The elastic design forces for all elements are obtained from SDAP D using either an $R=1.0$ or 0.8 as specified in Table 3.10.3.7.1-2. The design force for any elements that could result in a brittle mode at failure (e.g., shear in concrete columns and pile bents, connections in braced frames) shall use an R -Factor of 0.67 with the elastic force. As an alternate to the use of the elastic forces, all elements connected to the column can be designed using the capacity design procedures of Article 3.10.3.8 using an overstrength ratio of 1.0 times the nominal moment capacities.

3.10.3.11.2 Selected Substructure Supports are Designed Elastically

If selected substructure supports are designed elastically then the moment demand can be established using an $R=1.0$ from the SDAP D analysis. The column or pile bent shear force and all connecting elements shall be designed using the capacity design procedures of Article 3.10.3.8 or the requirements of Article 3.10.3.11.1. Exception: The component design procedures of Article 3.10.3.11.1 may be used, provided the SDAP D analytical model uses the secant modulus of columns that are not designed elastically. The secant stiffness of the columns shall be based on the elastic displacements from an iterated analysis.

3.10.3.12 SUPERSTRUCTURE SEISMIC DESIGN

The provisions of this section apply in SDAP C, D and E for SDR 4, 5, and 6. Unless noted otherwise these provisions apply to both levels of earthquake.

3.10.3.12.1 General

The superstructure shall either be capacity-protected, such that inelastic response is confined to the substructure or designed for the elastic seismic forces of the 3% in 75-year event. If capacity protection is used, the overstrength forces developed in the piers and the elastic forces at the abutments shall be used to define the forces that the superstructure must resist. In addition to the earthquake forces, the other applicable forces for the Extreme Event combination shall be used. The combined action of the vertical loads and the seismic loads shall be considered. The superstructure shall remain essentially elastic using nominal properties of the members under the overstrength

C3.10.3.11.1

If all the supporting substructures elements (columns, piers, pile bents) are designed elastically, there will be no redistribution of lateral loads due to plastic hinges developing in one or more columns. As a consequence the elastic analysis results are appropriate for design. The recommended provisions attempt to prevent any brittle modes of failure from occurring.

C3.10.3.11.2

If only one or a selected number of supporting substructure elements are designed elastically, there will be a significant redistribution of lateral loads when one or more of the columns develop plastic hinges. Generally, the elastically designed elements will attract more lateral load. Hence the need to either use capacity design principles for all elements connected to the elastically designed column. If this is not practical, the complete bridge needs to be reanalyzed using the secant stiffness of any columns in which plastic hinges will form in order to capture the redistribution of lateral loads that will occur.

C3.10.3.12

C3.10.3.12.1 General

Capacity-protection or elastic design of the superstructure is required to reduce the possibility of earthquake induced damage in the superstructure. It is generally felt that such damage is not easily repairable and may jeopardize the vertical load-carrying capability of the superstructure.

The elastic forces from the 3% in 75-year event may be used in lieu of capacity-protecting the superstructure, because their use will typically satisfy the performance objective for the design level ground motion.

When the superstructure can effectively span transversely

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forces or elastic forces corresponding to the 3% in 75-year earthquake, whichever are selected by the designer.

3.10.3.12.2 Load Paths

Load paths for resistance of inertial forces, from the point of origin to the points of resistance, shall be engineered. Positive connections between elements that are part of the earthquake resisting system (ERS) shall be provided. Article 4.8.3.2 contains additional requirements. Bridges with a series of multi – simple spans cannot use the abutments to resist longitudinal forces from spans other than the two end spans. Longitudinal forces from interior spans may only be transferred to the abutments when the superstructure is continuous.

3.10.3.12.3 Effective Superstructure Width

The width of superstructure that is effective in resisting longitudinal seismic forces is dependent on the ability of the piers and abutments to effectively resist such forces. In the case of longitudinal moment transfer from the superstructure to the substructure, the pier cap beam shall be designed to resist forces transferred at the connection locations with the substructure. If such resistance is not provided along the cap beam, then a reduced effective superstructure width shall be used. This width shall be the sum of the column width along the transverse axis and the superstructure depth for open-soffit superstructures (e.g. I-girder bridges) or the column width plus twice the superstructure depth for box girders and solid superstructures. The effective width is to be taken transverse to the column at the pier and may be assumed to increase at a 45-degree angle as one moves along the

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between abutments as a diaphragm, then the resistance of the intermediate piers may not contribute significantly to the lateral resistance. In such cases, the elastic forces for the design earthquake should be used for the design of the superstructure lateral capacity. However, when designed in this manner, the superstructure could be vulnerable in earthquakes that produce shaking at the site that is larger than the design ground motion. If the maximum resistances of the abutments are defined, then they may be used to define the maximum forces in the superstructure, as an alternate to the use of the elastic seismic forces.

C3.10.3.12.2 Load Paths

The path of resistance for the seismic loads should be clearly defined, and the mechanisms for resistance engineered to accommodate the expected forces. In general, the seismic forces in the superstructure should be those corresponding to a plastic mechanism (yielding elements at their respective overstrength conditions) or the elastic demand analysis forces. The load path in the superstructure should be designed to accommodate these forces elastically.

Where non-seismic constraints preclude the use of certain connection elements, alternate positive connections should be made. For instance, non-composite action is often used in the negative moment regions of continuous steel plate girders. Consequently, studs are not present to transfer inertial loads from the deck to the diaphragm. In such cases, the girder pad portion of the deck slab could be extended beside the girder flange to provide a bearing surface.

Longitudinal forces may only be transferred to the abutment by a continuous superstructure. If a series of simple spans are used the seismic loads must be resisted at each substructure location.

C3.10.3.12.3 Effective Superstructure Width

In the case of longitudinal seismic force resistance, the piers will receive loads at the connection points between the superstructure and substructure. For longitudinal loading the primary load path from the superstructure to the pier is along the girder or web lines. To effectively transfer these forces to the substructure, connections to the piers should be made close to the girder or web lines. This requires that the cap beam of the pier in a single- or multi-column bent should be capable of resisting the effects of these forces, including shears, moments, and torsion.

In the case of longitudinal moment (moment about the superstructure transverse axis) transferred between super- and substructure, significant torsion may develop in the cap beam of the pier. The designer may choose to resist the longitudinal moment directly at the column locations and

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superstructure until the full section becomes effective.

For superstructures with integral cap beams at the piers, the effective width of the cap beam may be as defined in Section 4.6.2.6.

3.10.3.12.4 Superstructure-To-Substructure Connections

The provisions of this section apply in SDAP B, D, and E. These provisions apply to both levels of earthquake.

3.10.3.12.4.a Connection Design Forces

The forces used for the design of connection elements shall be the lesser of the 3% in 75-year elastic forces or the overstrength forces developed in the substructure below the connection as per Article 3.10.3.8.

3.10.3.12.4.b Fuse Elements and Adjacent Connections

Where connections or adjacent structure is designed to fuse (e.g. shear keys at abutments that might be intended to breakaway in the 3% in 75-year earthquake), the design forces shall correspond to an upper-bound estimate of the force required to fuse the element. The materials and details used to create fuse elements shall be chosen such that reasonable predictability of the fuse strength is assured.

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avoid these torsions. However, in a zone adjacent to the column, the longitudinal moment in the superstructure must then be transferred over an effective superstructure width, which accounts for the concentration of forces at the column location. The provisions used to specify the effective width are based on Caltrans' *Seismic Design Criteria* (1999). On the other hand, if the cap beam is designed for the longitudinal moments applied at the girder lines, no effective width reduction of the superstructure is required.

C3.10.3.12.4 Superstructure-To-Substructure Connections

C3.10.3.12.4.a Connection Design Forces

In general the connections between the superstructure and substructure should be designed for the maximum forces that could be developed. In the spirit of capacity design, this implies that the forces corresponding to the full plastic mechanism (with yielding elements at their overstrength condition) should be used to design the connections. In cases where the full mechanism might not develop during the 3% in 75-year earthquake, it is still good practice to design the connections to resist the higher forces corresponding to the full plastic mechanism. It is also good practice to design for the best estimate of forces that might develop in cases such as pile bents with battered piles. In such bents the connections should be stronger than the expected forces, and these forces may be quite large and may have large axial components. In such cases, the plastic mechanism may be governed by the pile geotechnical strengths, rather than the piles' structural strengths.

C3.10.3.12.4.b Fuse Elements and Adjacent Connections

Elements that fuse to capacity protect attached elements should be treated similarly to elements that form a plastic hinge. The overstrength force from the fusing element may be used to design the adjacent elements and connections. Just as with plastic hinging, the designer should attempt to control the failure mechanism, as much as is possible. This implies that some modes of failure may be suppressed by adding strength, and others promoted by reducing strength. In general, the upper bound strength of the fuse should be about 75 percent of capacity of the elements being protected. For instance, strength of a fusible shear key at a pile-supported abutment might be sized to be 75 percent of the lateral strength of the pile group. The connections of adjacent elements to the abutment would then be designed to provide at least this capacity.

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3.10.3.13 SEISMIC ISOLATION DESIGN

The design and testing requirements for the isolators are given in Articles 15.12 through 15.15

The analysis requirements for a seismically isolated bridge are given in Article 4.8.4.6 and Article 4.8.5.2 for the capacity spectrum method and Article 4.8.5.3 for a multi-mode analysis and Article 4.8.5.5 for a nonlinear time-history analysis. Other analysis and modeling issues are given in Article 15.4 and design properties of the isolators are given in Article 15.5. If an upper and lower bound analysis is performed as per Article 15.4, then the design forces and displacement shall be the maximum of those obtained from the upper and lower bound analyses respectively.

The supporting substructures may be all designed elastically using the provisions of Article 3.10.3.11.1. If an R of 1.5 as per Table 3.10.3.7.1-1 is used to design the substructure, all other elements connected to the column shall be designed using the Capacity Design procedures of Article 3.10.3.8. The design and testing of the isolator units is given in Article 15.10 and other design issues related to the isolators are given in Section 15.

3.10.3.14 SEISMIC DESIGN AND TESTING OF BEARINGS

The provisions of this section apply to the design and/or testing of all bearings in SDR 3 through 6. There are three design or testing alternates for bearings that are not designed and tested as seismic isolation bearings as per article 3.10.3.13. Alternate 1 requires both prototype and quality control testing of bearings as per Article 3.10.3.14.1. If testing of bearings is not performed for the required forces and displacements, then Alternate 2 provides a design option to provide a positive restraint system for the bearing. The restraint shall be capable of resisting the forces generated in the 3% in 75 year event utilizing an analytical model that assumes that all bearings so designed are restrained. Alternate 3 provides a design option that permits a bearing to fail, provided there is a flat surface on which the girders can slide. The bearing or masonry plinth cannot impede the movement. The bridge must be analyzed in this condition and allowance for 150% of the calculated movement shall be provided.

If Alternate 3 is selected then a non-linear time history analysis is required using an appropriate coefficient of friction for the sliding surface to determine the amount of displacement that will result. The bearings shall be assumed to have failed early in the time history so a conservative value of the displacement is obtained.

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C3.10.3.13 SEISMIC ISOLATION DESIGN

The design and testing requirements for the isolators are given in Articles 15.12 through 15.15

The analysis requirements for a seismically isolated bridge are given in Article 4.8.4.6 and Article 4.8.5.2 for the capacity spectrum method and Article 4.8.5.3 for a multi-mode analysis and Article 4.8.5.5 for a nonlinear time-history analysis. Other analysis and modeling issues are given in Article 15.4 and design properties of the isolators are given in Article 15.5. If an upper and lower bound analysis is performed as per Article 15.4, then the design forces and displacement shall be the maximum of those obtained from the upper and lower bound analyses respectively.

The supporting substructures may be all designed elastically using the provisions of Article 3.10.3.11.1. If an R of 1.5 as per Table 3.10.3.7.1-1 is used to design the substructure, all other elements connected to the column shall be designed using the Capacity Design procedures of Article 3.10.3.8. The design and testing of the isolator units is given in Article 15.10 and other design issues related to the isolators are given in Section 15.

C3.10.3.14 SEISMIC DESIGN AND TESTING OF BEARINGS

One of the significant issues that arose during the development of these provisions was the critical importance of bearings as part of the overall bridge load path. The 1995 Kobe earthquake, and others that preceded it and have occurred since, clearly showed poor performance of some very recent bearing types and the disastrous consequences that a bearing failure can have on the overall performance of a bridge. A consensus was developed that some testing of bearings would be desirable provided a designer had the option of providing restraints or permitting the bearing to fail if an adequate surface for movement is provided. A classic example occurred in Kobe where a bearing failed and it destroyed the steel diaphragm and steel girder because the girder became jammed on the failed bearing and could not move.

There has been a number of studies performed when girders slide either on specially designed bearings or concrete surfaces. A good summary of the range of the results that can be anticipated from these types of analyses can be found in Dicleli, M., Bruneau, M. (1995).

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3.10.3.14.1 Prototype and Quality Control Tests

Prototype Tests – each manufacturer shall perform a set of prototype tests on two full size bearings to qualify that particular bearing type and size for the rated forces or displacements of it's application. The sequence of tests shall be those given in Article 15.10.2 for the displacement or force for which it is to be qualified. For fixed bearings, the sequence of tests shall be performed for 110% of the lateral force capacity of the bearing where 110% of the force capacity replaces the total design displacement in Article 15.10.2. For bearings that permit movement, the total design displacement shall be 110% of the displacement for which they are to be qualified.

Quality Control Tests – a set of quality control tests shall be performed on 1 out of every 10 bearings of a given type and size. The tests shall be similar to those required for isolation bearings as specified in Articles 15.12.2, 15.14.2 and 15.15.6. For fixed bearings, the total design displacement shall be replaced by the lateral force capacity for which they are qualified.

3.10.4 Collateral Seismic Hazards

Collateral hazards resulting from seismic ground shaking shall be evaluated. These collateral hazards include liquefaction, as well as other hazards caused by or associated with earthquake-induced ground movement, such as faulting, landsliding, differential compaction, and flooding or inundation from failure of dams or reservoirs during earthquake loading.

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C3.10.3.14.1

The types of tests that are required are similar but significantly less extensive than those required for seismically isolated bridges. Each manufacturer is required to conduct a prototype qualification test to qualify a particular bearing type and size for it's design forces or displacements. This series of tests only needs to be performed once to qualify the bearing type and size, whereas on an isolated project, prototype tests are required on every project. The quality control tests required on 1 out of every 10 bearings is the same as that required for every isolator on seismic isolation bridge projects. The cost of the much more extensive prototype and quality control testing of isolation bearings is approximately 10 to 15% of the total bearing cost, which is of the order of 2% of the total bridge cost. The testing proposed herein is much less stringent than that required for isolation bearings and is expected to be less than 0.1% of the total bridge cost. However, the benefits of testing are considered to be significant since owners would have a much higher degree of confidence that each new bearing will perform as designed during an earthquake. The testing capability exists to do these tests on full size bearings. Caltrans has invested in a full size test machine located at the University of California, San Diego, and similar capabilities exist at other universities, government laboratories, and commercial facilities.

C3.10.4

These hazards result from movement of the earth during a seismic event. Generally, there are two types of ground movement during an earthquake: (1) vibration of the ground, and (2) permanent displacement of the ground.

Vibration occurs as energy propagates from below to the ground surface. These motions are dynamic; they result in straining of the soil and sometimes buildup in porewater pressures, which can lead to loss in soil stiffness and strength. It is generally assumed that with the cessation of dynamic shaking, dynamic strains and porewater pressures return to their pre-earthquake condition.

The second type of movement involves permanent displacement of the soil. These displacements can be in the form of lateral movement, as occurs during liquefaction-related flows and soil spreading, or they can be vertical settlement, as occurs during dynamic compaction. Permanent ground movement can also result from faulting and landsliding. The magnitude of these movements can range from less than a few centimeters to meters.

Both vibrational movement and permanent movement of the earth can result in significant loads on a bridge foundation system, particularly in SDR 3, 4, 5, and 6, and therefore warrant careful consideration during design.

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3.10.4.1 LIQUEFACTION

C3.10.4.1

An evaluation of the potential for and consequences of liquefaction within near-surface soil shall be made in accordance with the following requirements:

SDR 1 and 2

Not required unless directed otherwise by the Owner.

SDR 3, 4, 5, and 6

Required unless one of the following conditions is met or as directed otherwise by the Owner.

- Mean magnitude for the 3% in 75-year event is less than 6.0 (Figures 3.10.4-1 to 3.10.4-4);
- Mean magnitude of the 3% in 75-year event is less than 6.4 and equal to or greater than 6.0, and the normalized Standard Penetration Test (SPT) blow count $[(N_1)_{60}]$ is greater than 20;
- Mean magnitude for the 3% in 75-year event is less than 6.4 and equal to or greater than 6.0, $(N_1)_{60}$ is greater than 15, and $F_a S_s$ is between 0.25 and 0.375; or
- A liquefaction evaluation is required for the 50% in 75 year event if $F_a S_s$ is greater than 0.375.

If the mean magnitude shown in Figures 3.10.4-1 to 3.10.4-4 is greater than or equal to 6.4, or if the above requirements are not met for magnitudes between 6.0 and 6.4, evaluations of liquefaction and associated phenomena such as lateral flow, lateral spreading, and dynamic settlement shall be evaluated in accordance with these Specifications.

Liquefaction has been perhaps the single most significant cause of damage to bridge structures during past earthquakes. Most of the damage has been related to lateral movement of soil at the bridge abutments. However, cases involving the loss in lateral and vertical bearing support of foundations for central piers of a bridge have also occurred.

In SDR 1 and 2 the potential for liquefaction is generally low. In some cases (Type E and F soils in SDR 2) the peak ground acceleration in these SDR's may exceed 0.15g ($F_a S_s$ in excess of 0.375). While this level of peak ground acceleration is sufficient to cause liquefaction, the magnitude of the earthquake causing liquefaction for these categories will generally be less than 6 and hence the duration of strong shaking will be relatively short. For magnitudes less than 6.0, liquefaction develops slowly at most sites, and results in minimal effects to the structure during dynamic shaking, and therefore the effects of liquefaction on dynamic response can be neglected. In addition little potential exists for permanent movement of the ground, again because of the small size and limited duration of seismic events in these areas.

The potential for liquefaction in SDR 3, 4, 5, and 6 is higher, and therefore careful attention to the determination of the potential for and consequences of liquefaction is needed for sites with this classification. If the mean magnitude of the 3% in 75 year event is less than 6.0, then the discussion above with regard to duration is applicable in these SDR's. For the magnitude interval of 6.0 to 6.4, a liquefaction analysis is not required when the combination of ground shaking and blow count are below values that would cause liquefaction. This transition interval is based on an assessment of available data from past earthquakes and engineering judgment.

The mean magnitudes shown in Figures 3.10.4-1 to 3.10.4-4 are based on deaggregation information, which can be found in the USGS website (<http://geohazards.cr.usgs.gov/eq/>). A site-specific determination of the mean magnitude can be obtained from this website using the coordinates of the project site.

If liquefaction occurs in the 50% in 75 year event then the performance criteria for piles will need to be operational for the life safety performance level.

3.10.4.1.1 Evaluation of Liquefaction Potential

C3.10.4.1.1

Procedures given in Appendix 3B shall be used to evaluate the potential for liquefaction.

A site is considered potentially susceptible to liquefaction if one or more of the following conditions exists (SCEC, 1999):

- Liquefaction has occurred at the site during historical earthquakes.
- The site consists of uncompacted or poorly compacted fills containing liquefaction-susceptible

materials that are saturated, nearly saturated, or may be expected to become saturated.

- The site has sufficient existing geotechnical data, and analyses indicate that the soils are potentially susceptible to liquefaction.

For sites where geotechnical data are lacking or insufficient, the potential for liquefaction can be delineated using one or more of the following criteria:

- The site consists of soil of late Holocene age (less than 1,000 years old, current river channels and their historical flood plains, marshes, and estuaries) where the groundwater is less than 12 m deep and the anticipated earthquake ground shaking $F_a S_s$ is greater than 0.375 (peak ground acceleration (PGA) greater than 0.15g.)
- The site consists of soils of Holocene age (less than 11,000 years old) where the ground water is less than 10 m below the surface and $F_a S_s$ is greater than 0.50 (PGA is greater than 0.2g.)
- The site consists of soils of latest Pleistocene age (11,000 to 15,000 years before present) where the ground water is less than 5 m below the surface and $F_a S_s$ is greater than 0.75 (PGA is greater than 0.3g).

3.10.4.1.2 Evaluation of the Effects of Liquefaction and Lateral Ground Movement

Procedures given in Appendix 3B shall be used to evaluate the potential for and effects of liquefaction and liquefaction-related permanent ground movement (i.e., lateral spreading, lateral flow, and dynamic settlement). If both liquefaction and ground movement occur, they shall be treated as separate and independent load cases, unless agreed to or directed otherwise by the Owner.

C3.10.4.1.2

The design of bridge structures for liquefaction effects generally has two components.

- Vibration Effects: The first is that the bridge must perform adequately with just the liquefaction-induced soil changes alone. This means that the mechanical properties of the soil that liquefy are changed to reflect their liquefied conditions (i.e., “p-y” curves or modulus of subgrade reaction for lateral stiffness are reduced). Design for these cases is in reality a design for structural vibration effects, and these are the effects that the code-based procedures typically cover for design.
- Permanent Displacement Effects: The second component of the design is the consideration of liquefaction-induced ground movements. These can take several forms: lateral spreading, lateral flow, and dynamic settlement. Lateral spreading is a lateral movement that is induced by the ground shaking and develops in an incremental fashion as shaking occurs. Flow, on the other hand, is movement that occurs due to the combined effects of sustained pore pressure and gravity without the inertial loading from the earthquake. Flows can occur several minutes following an earthquake when

porewater pressures redistribute to form a critical combination with gravity loading. Dynamic settlement occurs following an earthquake as porewater pressures dissipate.

Vibration and permanent movement occur simultaneously during a seismic event. Their simultaneous occurrence is a complicated process that is difficult to represent without the use of very complex computer modeling. For most bridges the complexity of the modeling doesn't warrant performing a combined analysis. In these cases the recommended methodology is to consider the two effects independently, i.e., de-coupled. The reasoning behind this is that it is not likely that the peak vibrational response and the peak spreading or flow effect will occur simultaneously. For many earthquakes the peak vibration response occurs somewhat in advance of maximum ground movement loading. For very large earthquakes where liquefaction may occur before peak ground accelerations occur, the peak vibration response is like to be significantly attenuated and, hence, inertial loading reduced from peak design values. In addition peak displacements demands arising from lateral ground spreading are likely to generate maximum pile moments at depths well below peak moments arising from inertial loading. Finally, the de-coupling of response allows the flexibility to use separate and different performance criteria for design to accommodate the two phenomena. Two detailed case studies on the application of the recommended design methods for both liquefaction and lateral flow design are given in an NCHRP Report (ATC/MCEER, 2000)

While the de-coupled method is recommended for most bridges, more rigorous approaches are sometimes necessary, such as when a critical bridge might be involved. Coupled approaches are available to represent the large-strain, pore-water pressure buildup mechanisms that occurs during liquefaction. However, these methods are difficult to use, and should only be considered after detailed discussions between the Owner and the Engineer regarding the capabilities and limitations of these methods.

If lateral flow occurs, significant movement of the abutment and foundation systems can result. Inelastic deformation of the piles is permitted for this condition (e.g., plastic rotation of 0.05 radians). The geometric constraints of Table C3.10.1.2-2 provide guidance for meeting the desired performance objective. The range of design options include designing the piles for the flow forces to an acceptance of the predicted lateral flow movements realizing the bridge may need to be replaced. Structural and/or soil mitigation measures may be used to minimize the amount of movement to meet higher performance objectives.

3.10.4.1.3 Design Requirements if Liquefaction and Group Movement Occurs

C3.10.4.1.3

If it is determined from Appendix 3B that liquefaction can occur at a bridge site, then one or more of the following approaches shall be implemented in the design.

SDR 3

If liquefaction and no lateral flow occurs, then the bridge shall be designed by conventional procedures including the following requirements:

1. Piled Foundations, Drilled Shafts and Pile Bents: The pile or shaft shall penetrate beyond the bottom of the liquefied layer by at least 3 pile diameters or to a depth that is not affected by liquefaction of the overlying layer or by partial build-up in pore-water pressure, whichever is deeper. In addition the shear reinforcement in a concrete or pre-stressed concrete pile shall meet the requirements of Sec 5.10.11.4.1c from the pile or bent cap to a depth of 3 diameters below the lowest liquefiable layer.
2. Spread Footings: The bottom of the spread footing shall either be below the liquefiable layer or it shall be at least twice the minimum width of the footing above the liquefiable layer. If liquefaction occurs beneath the base of the footing, the magnitude of settlement caused by liquefaction shall be estimated, and its effects on bridge performance assessed.

If lateral flow or lateral spreading is predicted to occur, the following options shall be considered as detailed in Appendix 3B.

1. Design the piles or spread footings to resist the forces generated by the lateral spreading.
2. If the structure cannot be designed to resist the forces, assess whether the structure is able to tolerate the anticipated movements and meet the geometric and structural constraints of Table 3.10.1-2. The maximum plastic rotation of the piles shall be as defined in Article 5.16.3.
3. If the structure cannot meet the performance requirements of Table 3.10.1-1, assess the costs and benefits of various mitigation measures to minimize the movements to a tolerable level to meet the desired performance objective. If a higher performance is desired so that the spread footings or piles will not have to be replaced, the allowable plastic rotations of Article 5.16.3 shall be met.

SDR 4, 5, and 6

Bridges located in SDR 4, 5, and 6 shall be supported

If liquefaction and no lateral flow occur for SDR 3 bridges, then the only additional design requirements are those reinforcement requirements specified for the piles and spread foundation. Additional analyses are not required, although for major or important bridges the additional analyses specified in Article 3.10.6.1.1b may be considered to assess the impact on the substructures above the foundation.

If liquefaction and lateral flow are predicted to occur for SDR 3, a detailed evaluation of the effects of lateral flow on the foundation should be performed. Lateral flow is one of the more difficult issues to address because of the uncertainty in the movements that may occur. The design steps to address lateral flow are given in Appendix 3B. Note that a liberal plastic rotation of the piles is permitted. This plastic rotation does imply that the piles and possibly other parts of the bridge will need to be replaced if these levels of deformation do occur. Design options range from an acceptance of the movements with significant damage to the piles and columns if the movements are large to designing the piles to resist the forces generated by lateral spreading. Between these options are a range of mitigation measures to limit the amount of movement to tolerable levels for the desired performance objective.

Spread footings are not normally used in SDR 4, 5,

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on deep foundations unless (1) the footing is located below the liquefiable layer, (2) special design studies are conducted to demonstrate that the footing will tolerate liquefaction, or (3) the ground is improved so that liquefaction does not occur. If spread footings are being considered for use at a liquefiable site in SDR 4, 5, and 6, Owner approval shall be obtained before beginning the design process.

If liquefaction occurs, then the bridge shall be designed and analyzed in two configurations as follows:

1. Nonliquefied Configuration: The structure shall be analyzed and designed, assuming no liquefaction occurs using the ground response spectrum appropriate for the site soil conditions.
2. Liquefied Configuration: The structure as designed in Nonliquefied Configuration above shall be reanalyzed and redesigned, if necessary, assuming that the layer has liquefied and the liquefied soil provides whatever residual resistance is appropriate (i.e., “p-y curves” or modulus of subgrade reaction values for lateral pile response analyses consistent with liquefied soil conditions). The design spectra shall be the same as that used in Nonliquefied Configuration unless a site-specific response spectra has been developed using nonlinear, effective stress methods (e.g., computer program DESRA or equivalent) that properly account for the buildup in pore-water pressure and stiffness degradation in liquefiable layers. The reduced response spectra resulting from the site-specific nonlinear, effective stress analyses shall not be less than 2/3's of that used in Nonliquefied Configuration. The Designer shall provide a drawing of the load path and energy dissipation mechanisms in this condition as required by Article 2.5.6 since it is likely that plastic hinges will occur in different locations than for the non-liquefied case. Shear reinforcement given in Article 5.10.11.4.1c shall be used in all concrete and prestressed concrete piles to a depth of 3 pile diameters below the liquefied layer.

If lateral flow or lateral spreading occurs, the following options shall be considered.

1. Design the piles to resist the forces generated by the lateral spreading.
2. If the structure cannot be designed to resist the forces, assess whether the structure is able to tolerate the anticipated movements and meet the geometric and structural constraints of Table 3.10.1-2. The maximum plastic rotation of the piles is 0.05 radians.
3. If the structure cannot meet the performance requirements of Table 3.10.1-1, assess the costs and benefits of various mitigation measures to minimize

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and 6 if liquefiable soils are present. Spread footings can be considered if the spread footing is located below the bottom of the liquefiable layer, the ground will be improved to eliminate the potential for liquefaction, or special studies are conducted to demonstrate that the spread footing will perform adequately during and following liquefaction. In most situations these requirements will result in the use of either driven pile or drilled shaft foundations.

The approach used to design the foundation first involves designing to accommodate the non-seismic load conditions and the vibration case of seismic loading without liquefaction. This structure and foundation system should then be assessed for its capability to resist the inertial loads when the soil layers have liquefied. In general this second case will only impact the design of the structure above the foundation system when the upper layers of soil have liquefied.

As noted above for SDR 3, lateral flow is one of the more difficult issues to address because of the uncertainty in the movements that may occur. The design steps to address lateral flow are given in Appendix 3B. A liberal plastic rotation of the piles is permitted, but this does imply that the piles and possibly other parts of the bridge will need to be replaced if these levels of deformation do occur. Design options range from an acceptance of the movements with significant damage to the piles and columns if the movements are large to designing the piles to resist the forces generated by lateral spreading. Between these options are a range of mitigation measures to limit the amount of movement to tolerable levels for the desired performance objective.

Because the foundation will typically possess some lateral resistance capable of reducing the magnitude of spreading, this capacity should be utilized. If the lateral displacements are too great for the structure to adequately accommodate, then geotechnical improvements will be necessary, unless the performance objective under spreading loads is to accept a severely damaged bridge that likely will need to be replaced. Therefore the most cost-effective approach is to account for the beneficial restraint action of the existing (as-designed for non-spreading effects) foundation.

Additionally, if the foundation can provide significant restraint, but not fully adequate restraint, then additional piles may be considered. Depending on the soil profile and the manner in which spreading develops, simple “pinch” piles provided in addition to the foundation may prove effective. The cost trade-off between pinch piles and geotechnical remediation should be assessed to determine the most effective means of achieving appropriate soil restraint.

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the movements to a tolerable level to meet the desired performance objective. If a higher performance is desired so that the piles will not have to be replaced the allowable plastic rotations of Article 5.16.3 shall be met.

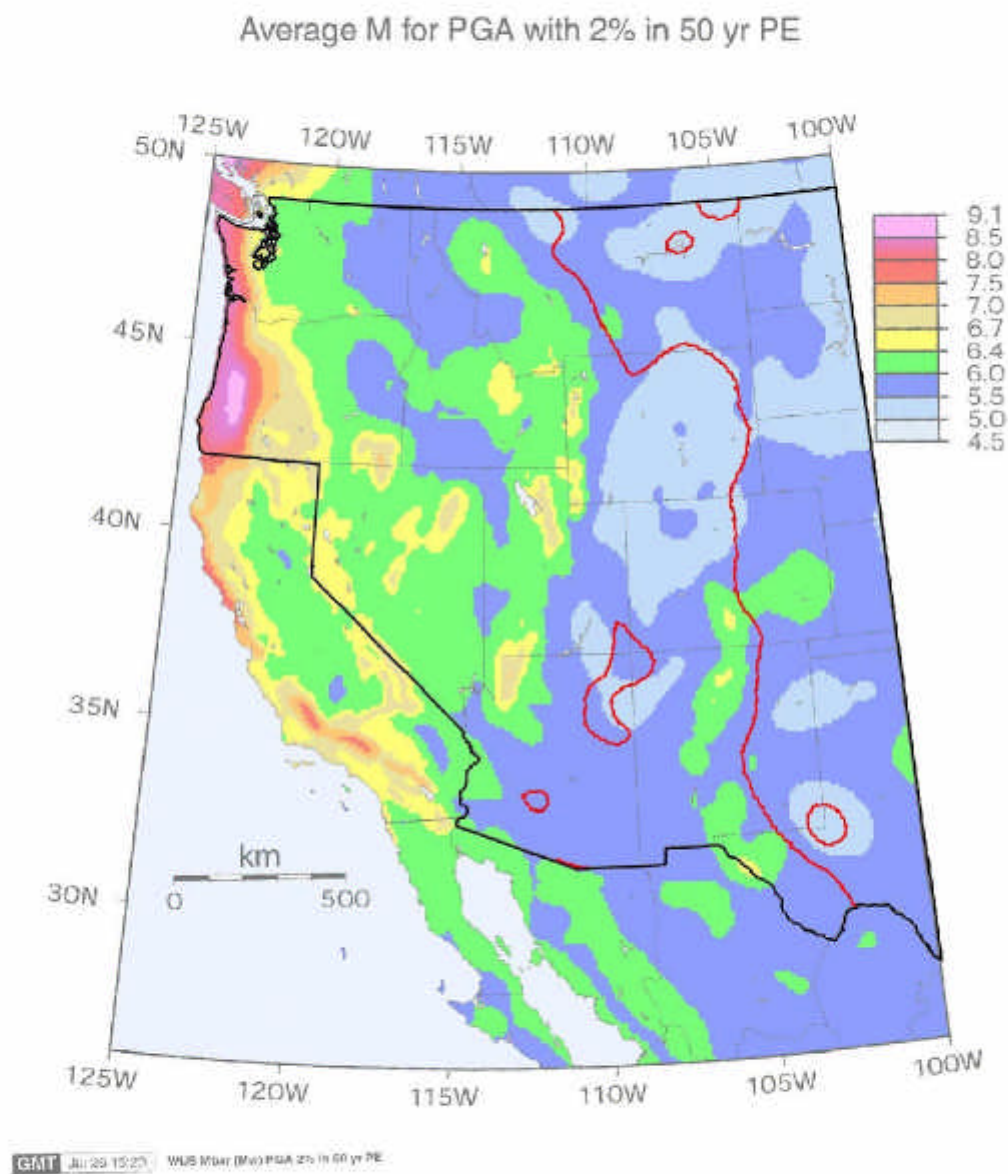
3.10.4.2 OTHER HAZARDS

The potential occurrence of collateral hazards resulting from fault rupture, landsliding, differential ground compaction, and flooding and inundation shall be evaluated for SDR 3, 4, 5, and 6. Procedures for making these evaluations are summarized in Appendix 3B.

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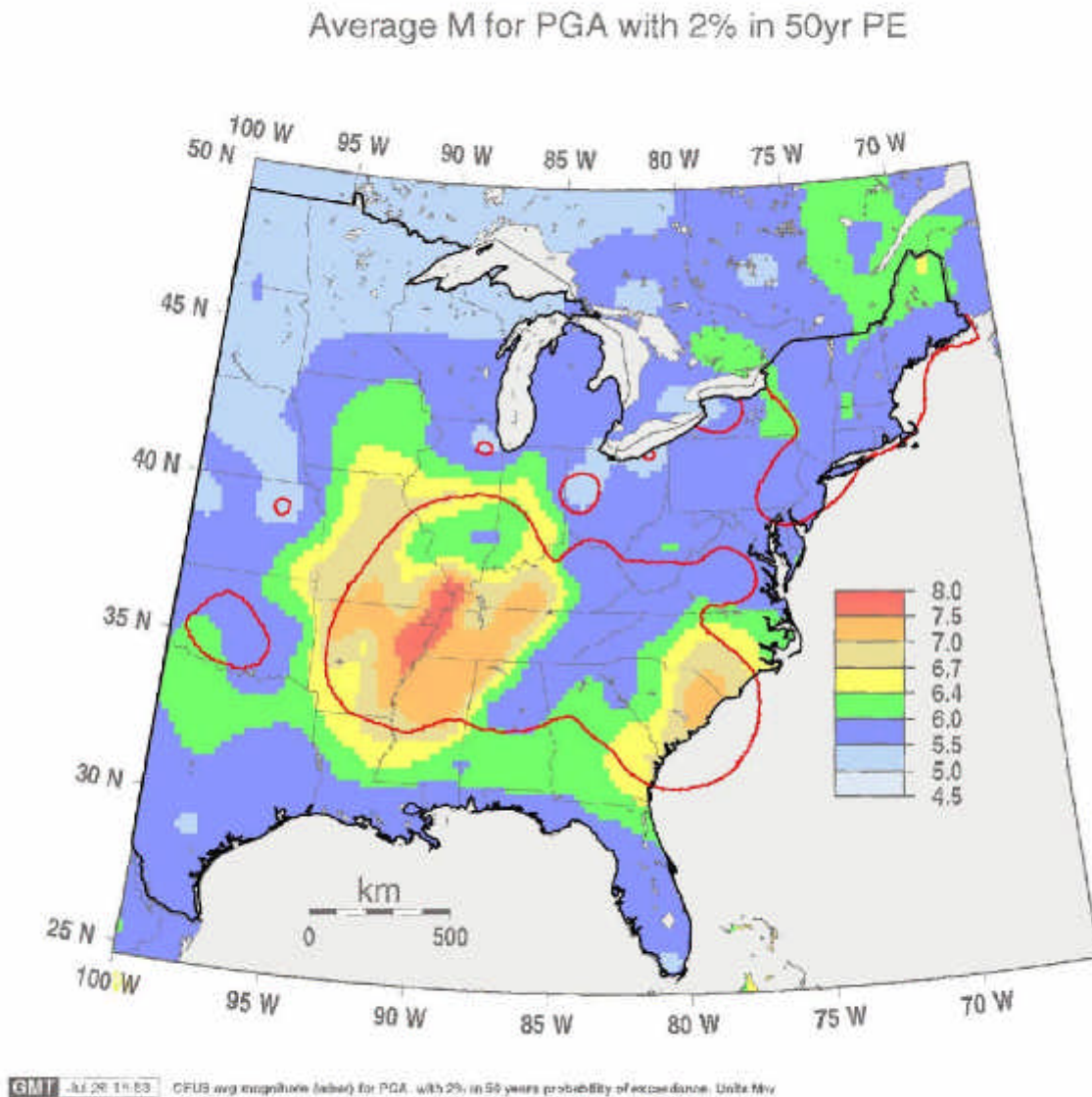
C3.10.4.2

The assessment of these collateral hazards will normally be limited to bridges located in SDR 3, 4, 5, and 6 as the potential for any of these hazards in SDR 1 and 2 will generally be small.



Improved Figure being Developed

Figure 3.10.4-1 Mean Earthquake Magnitude Map for Western United States



Improved Figure being Developed

Figure 3.10.4-2 Mean Earthquake Magnitude Map for Central and Eastern United States

Figure being Developed

Figure 3.10.4-3 Mean Earthquake Magnitude Map for Alaska (Map 1)

Figure being Developed

Figure 3.10.4-4 Mean Earthquake Magnitude Map for Alaska (Map 2)

3.11 EARTH PRESSURE: EH, ES, LS, and DD

3.11.4 Effect of Earthquake

The effects of probable amplification of active earth pressure and/or mobilization of passive earth masses by earthquake shall be considered.

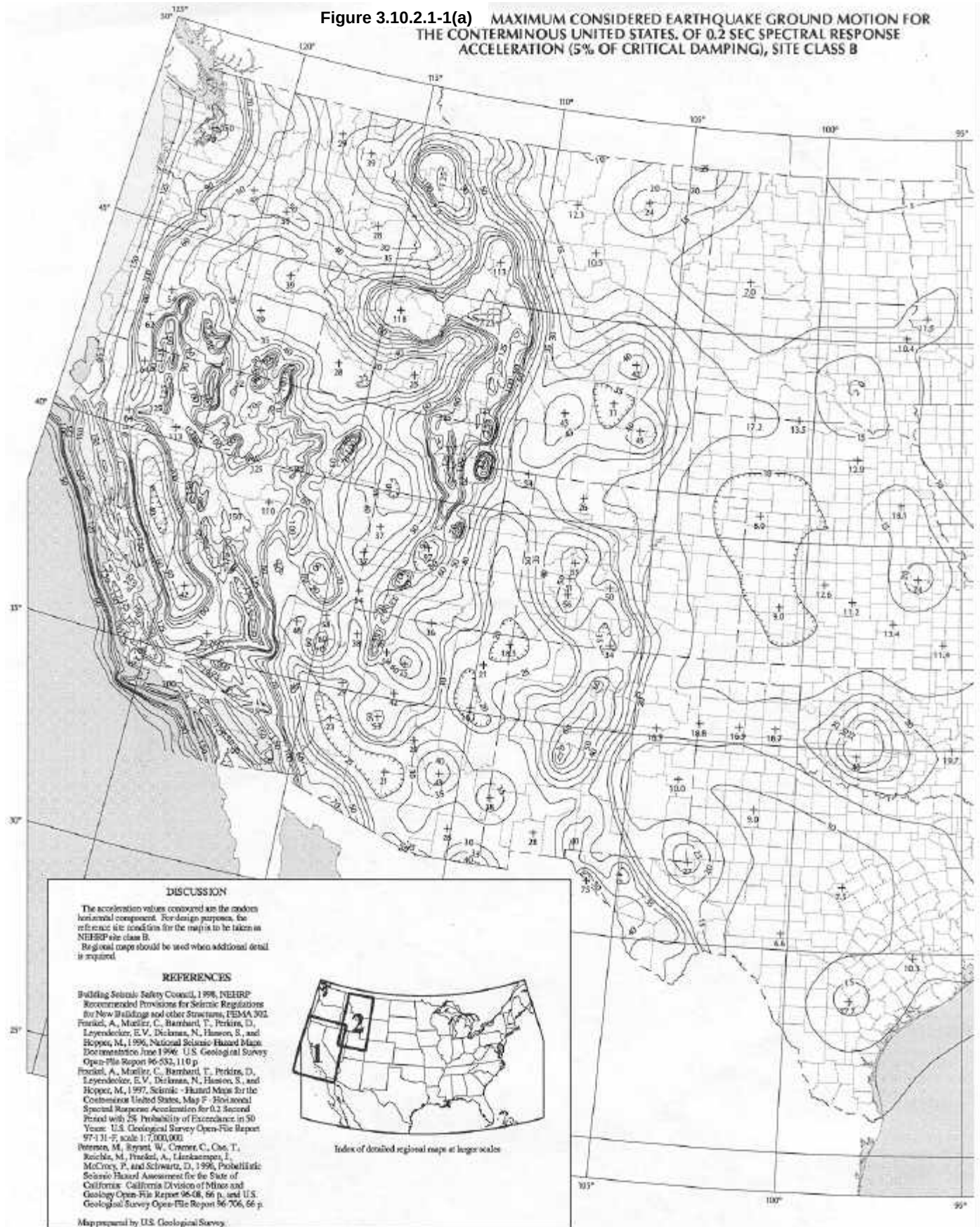
C3.11.4

The Mononobe-Okabe method for determining equivalent static fluid pressures for seismic loads on gravity and semigravity retaining walls is presented in the appendix to Section 11.

The Mononobe-Okabe analysis is based, in part, on the assumption that the backfill soils are unsaturated and thus not susceptible to liquefaction.

Where soils are subject to both saturation and seismic or other cyclic/instantaneous loads, special consideration should be given to addressing the possibility of soil liquefaction.

Figure 3.10.2.1-1(a) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES, OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



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Figure 3.10.2.1-1(a) continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES, OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

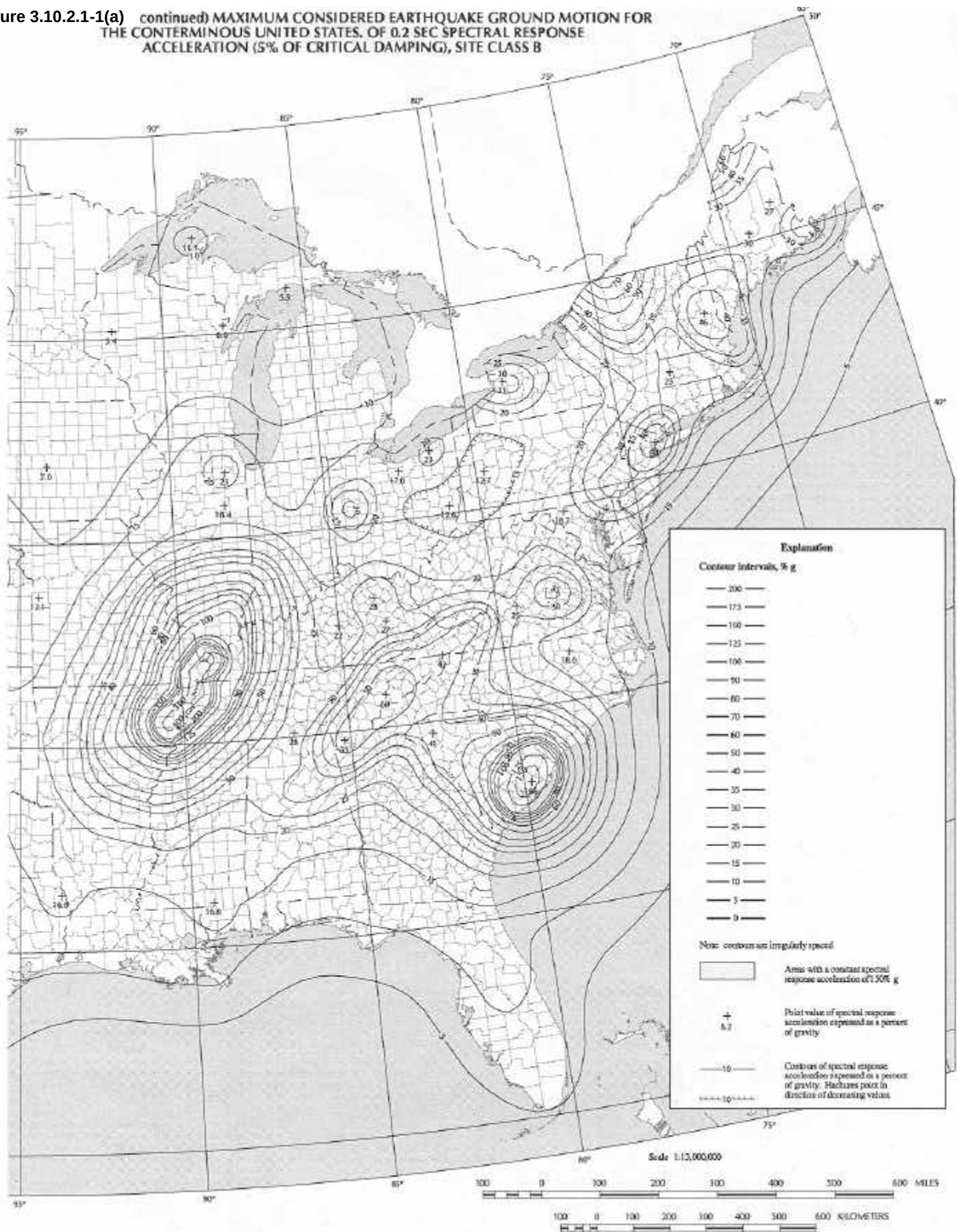
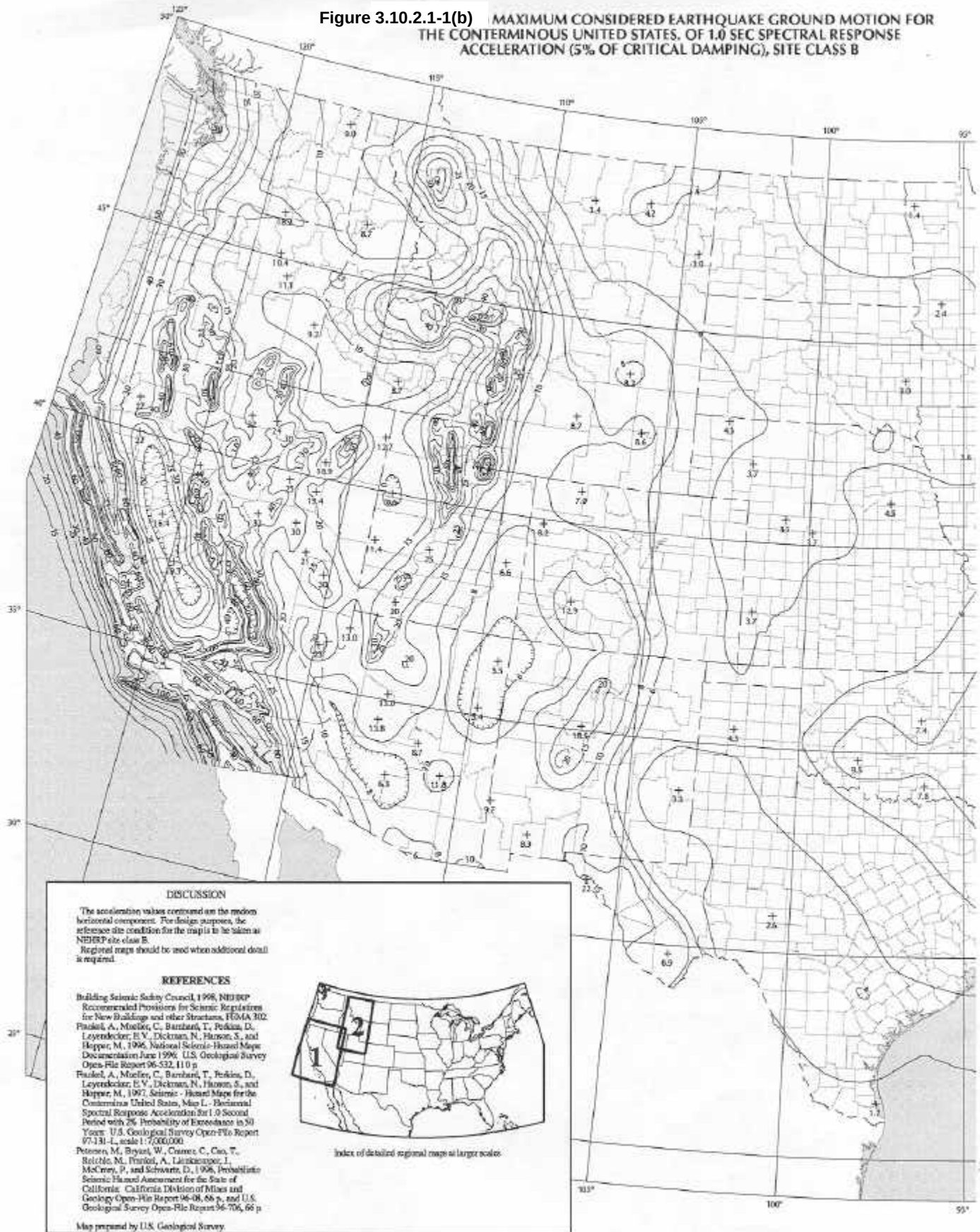
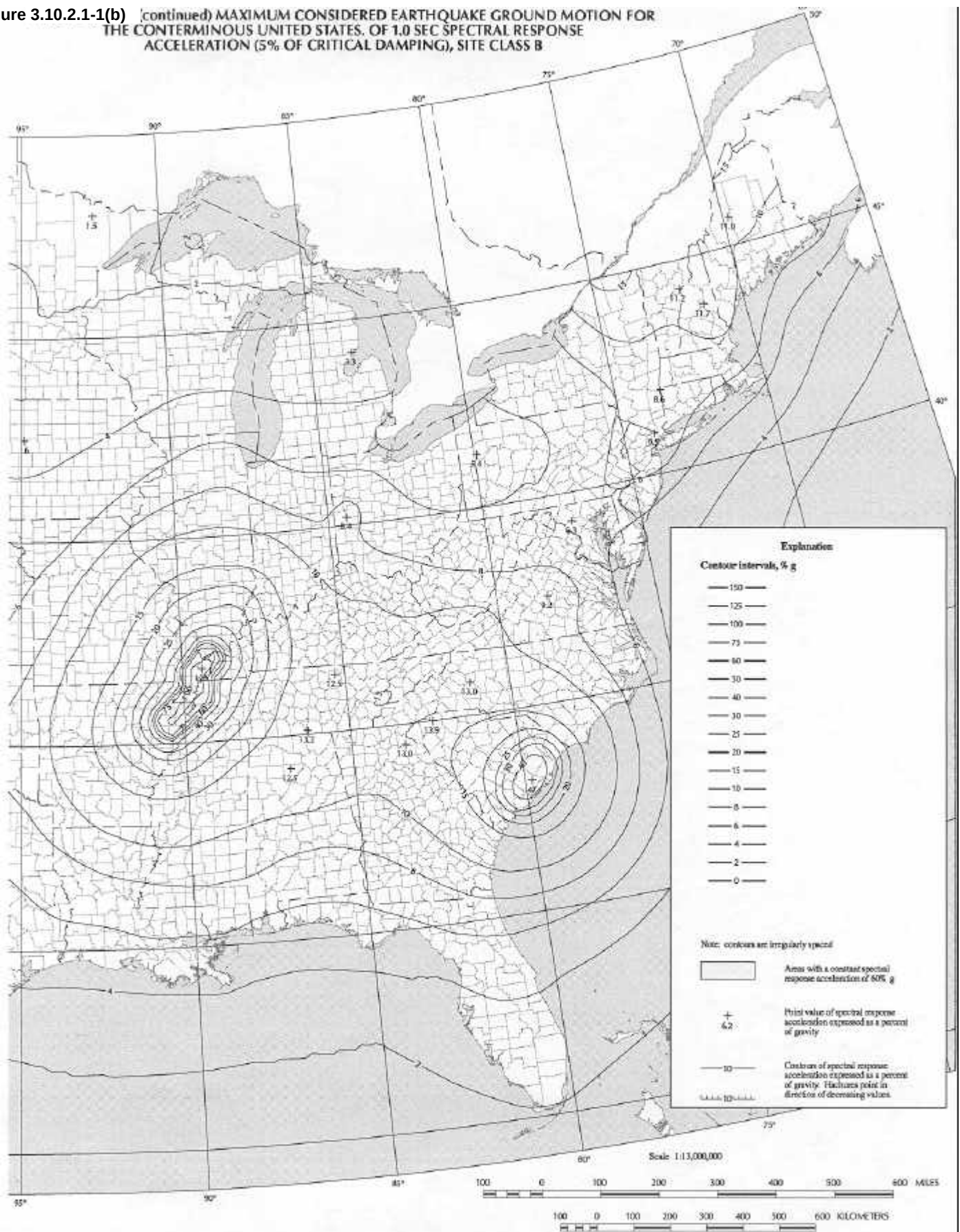


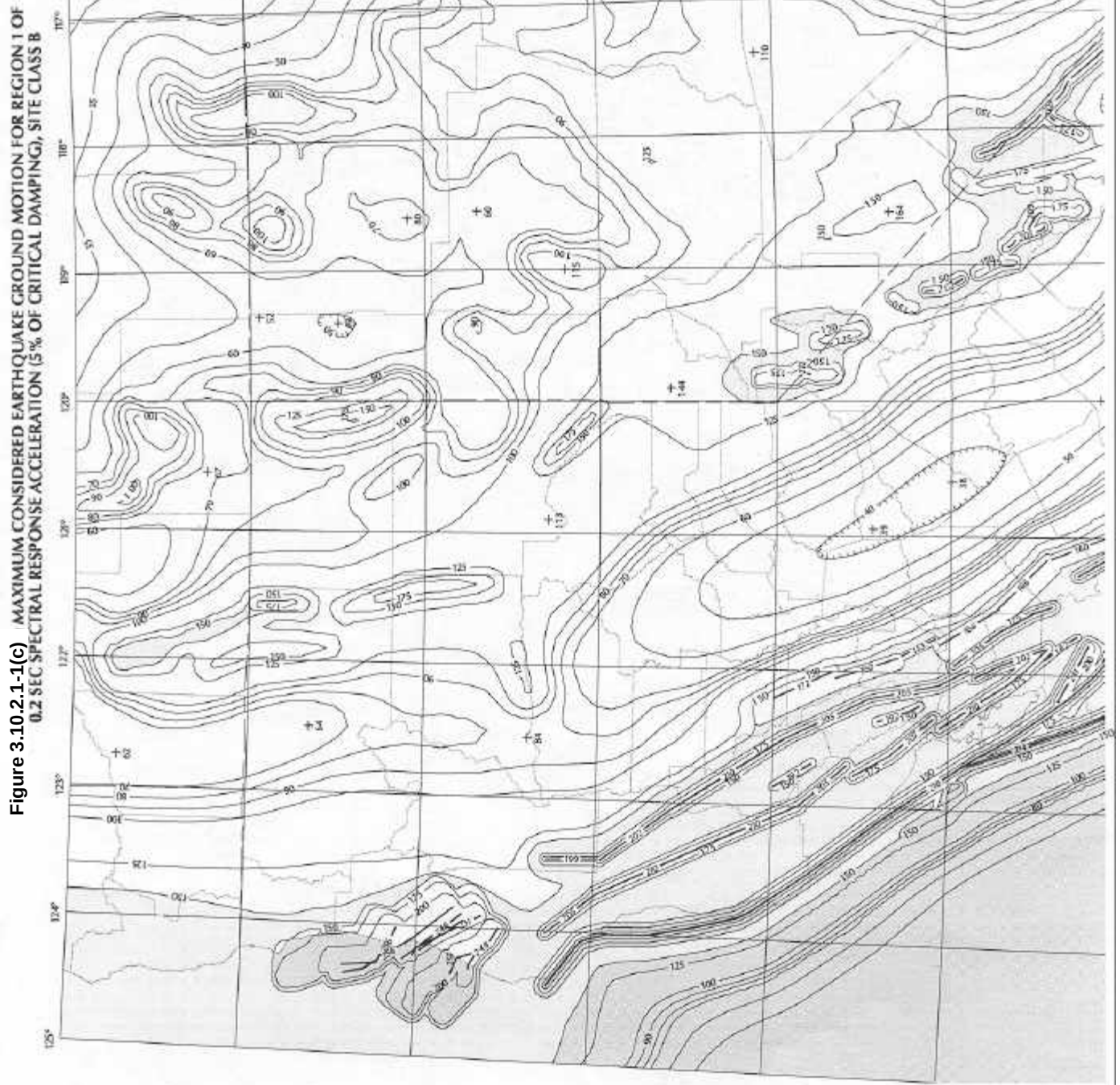
Figure 3.10.2.1-1(b) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES, OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



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Figure 3.10.2.1-1(b) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR THE CONTERMINOUS UNITED STATES. OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B





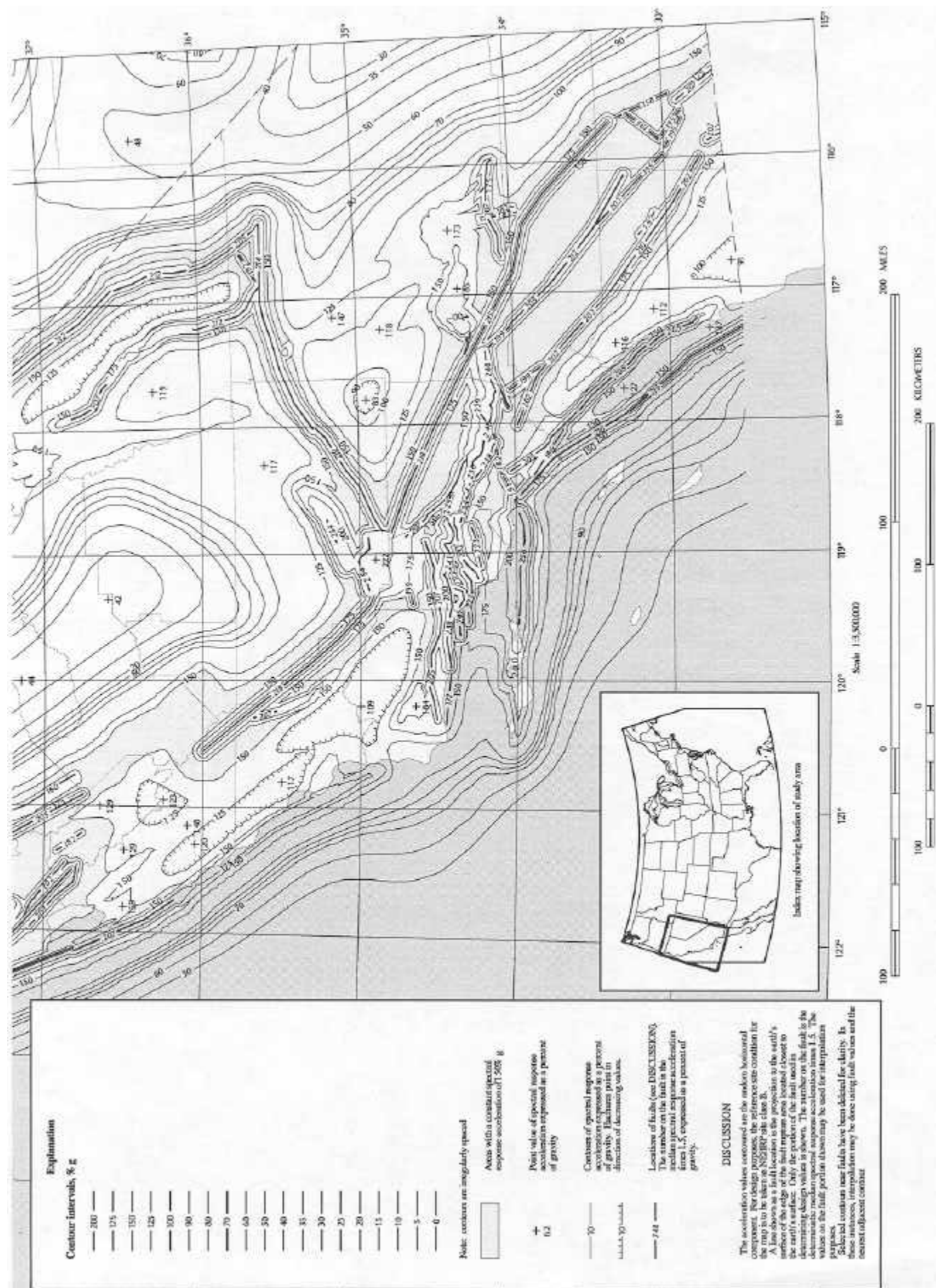
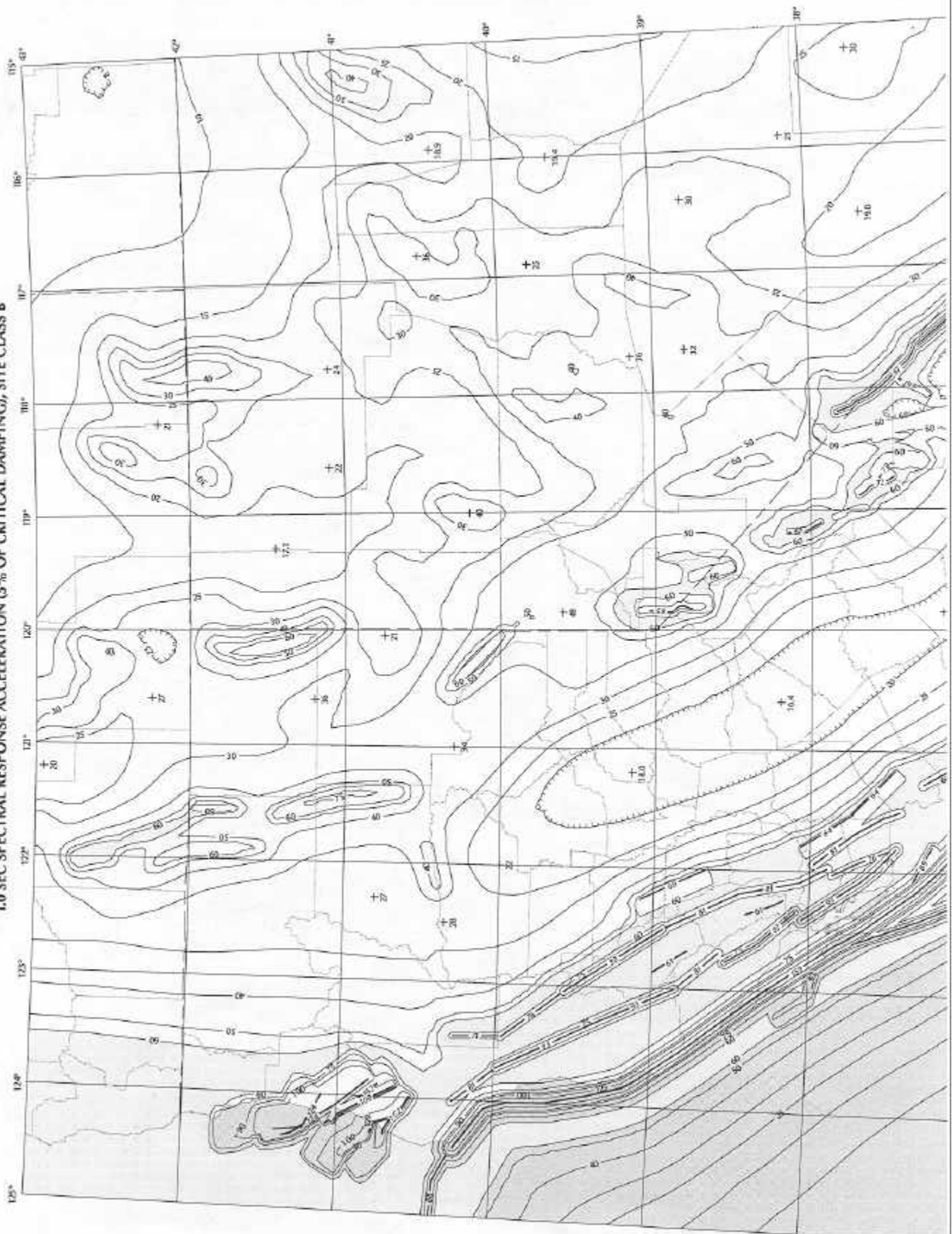


Figure 3.10.2.1-1(c) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION I OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B.

Figure 3.10.2.1-1(d) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 1 OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



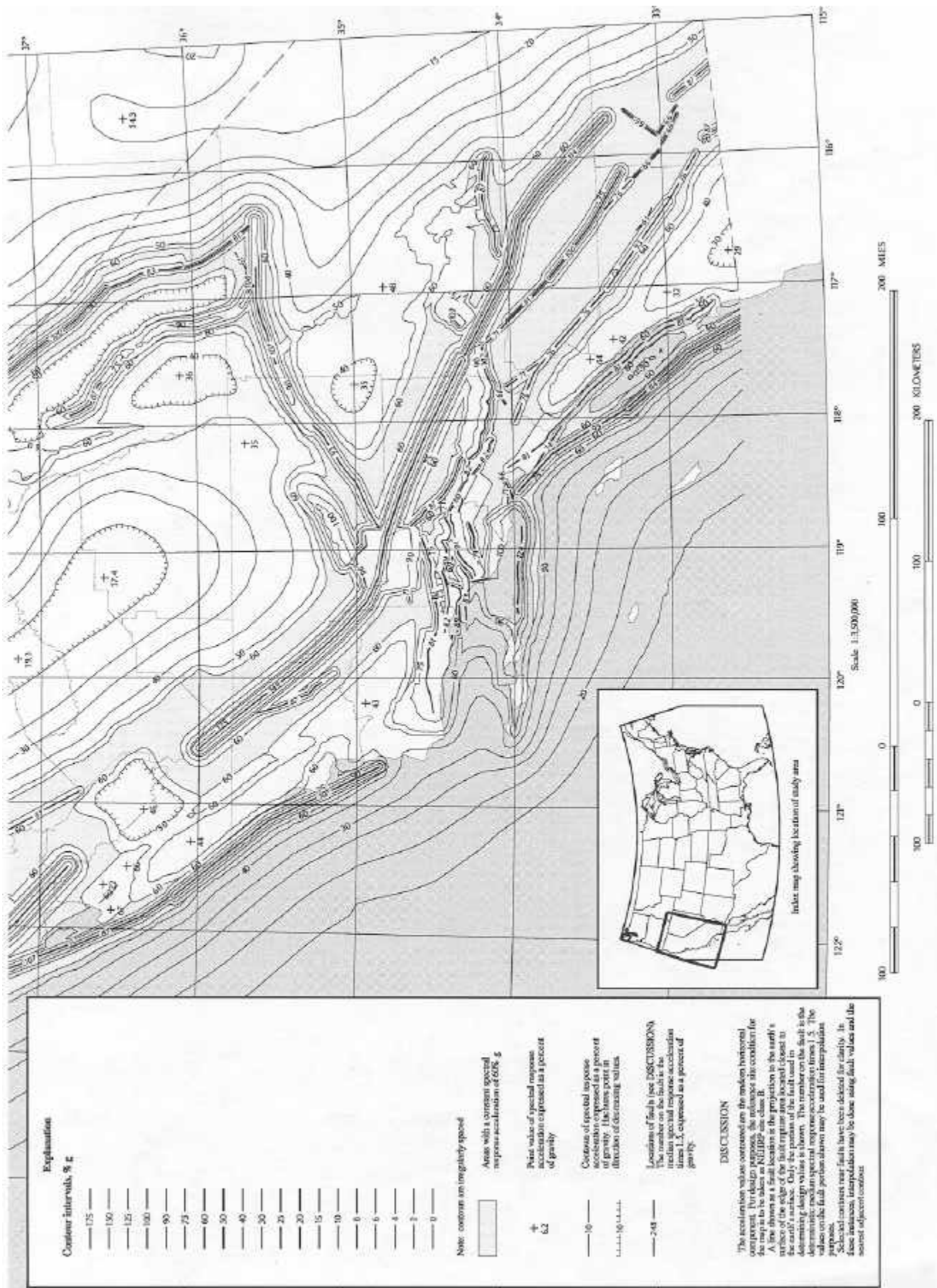


Figure 3.10.2.1-1(d) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 1 OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

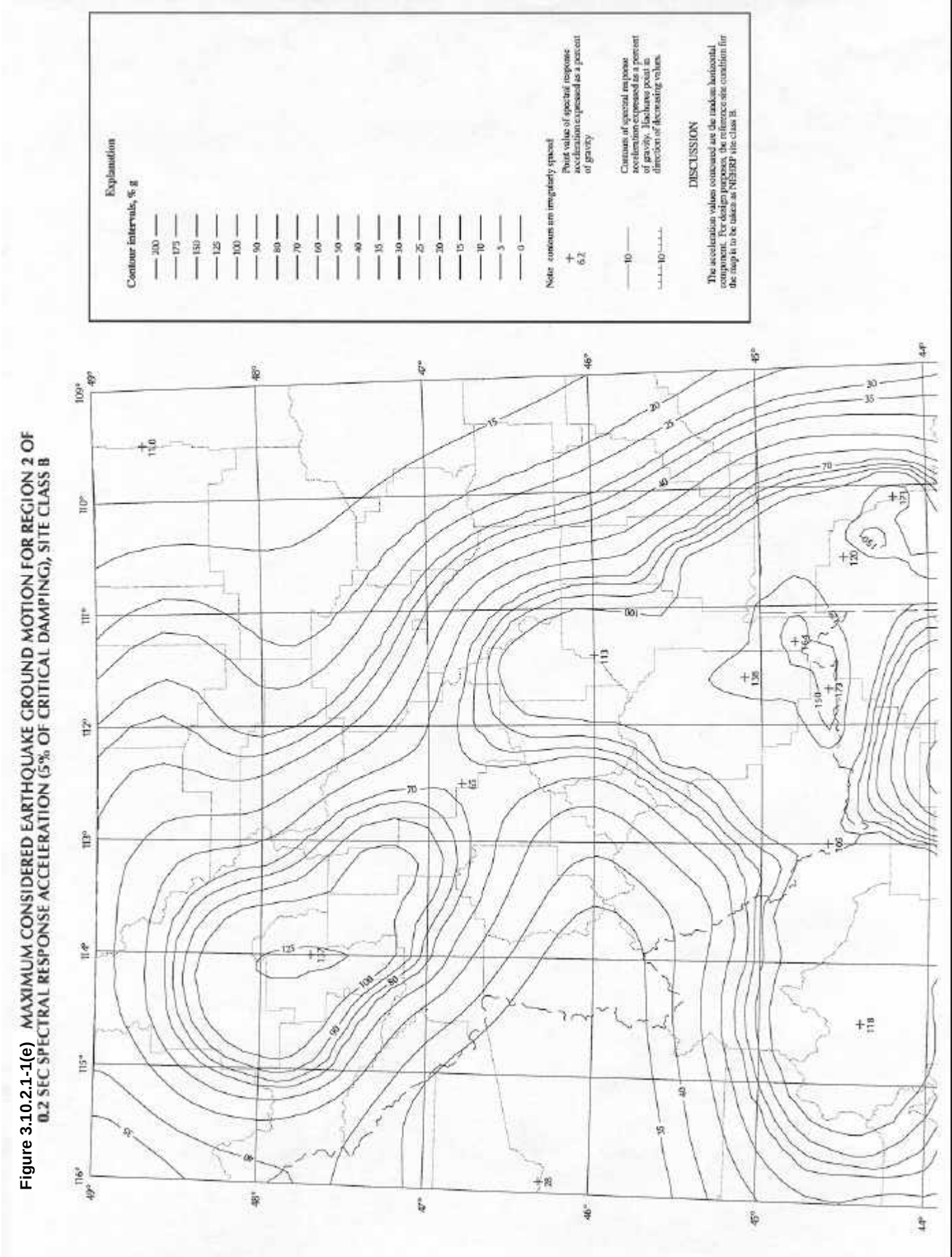
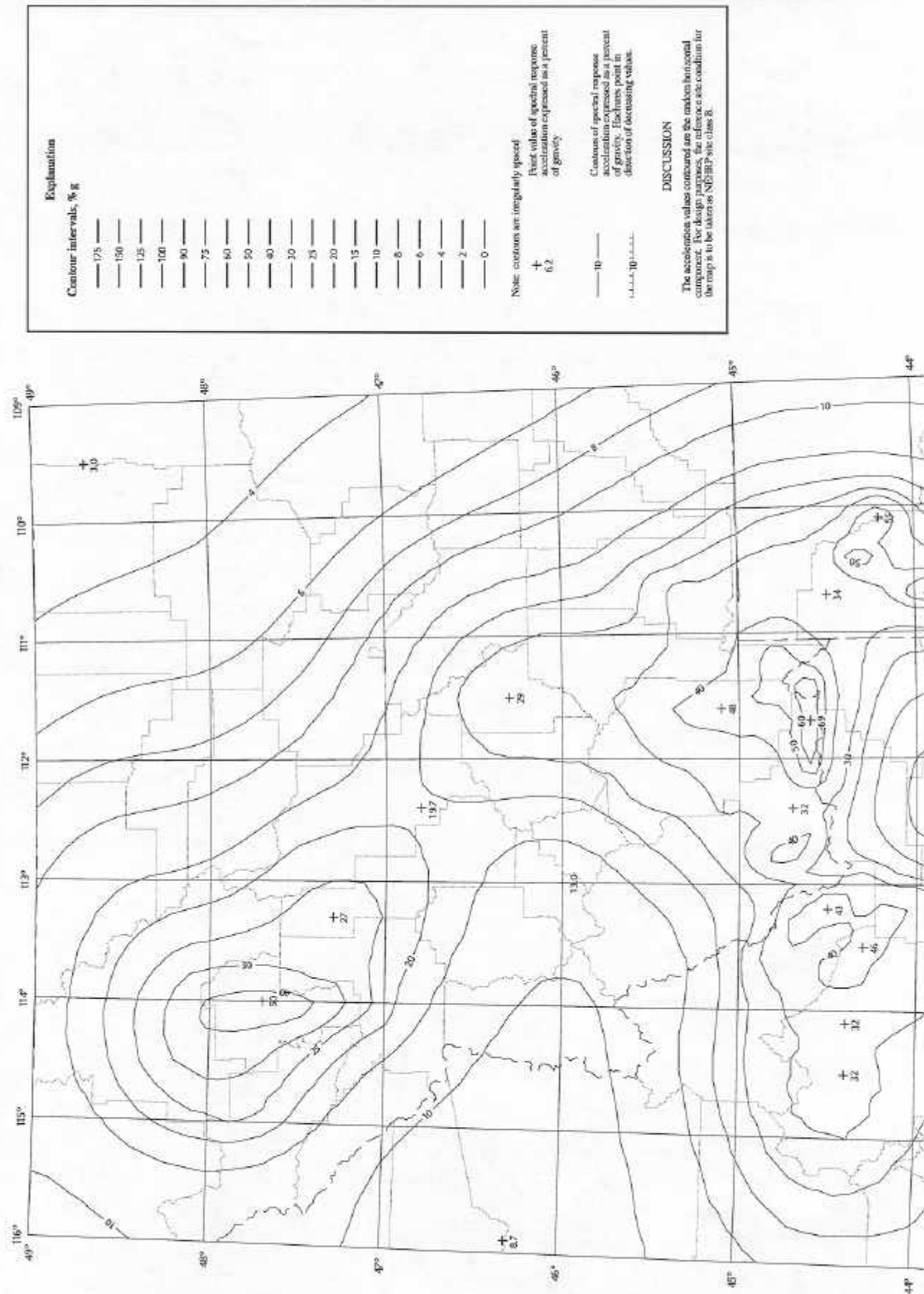




Figure 3.10.2.1-1(f) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 2 OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



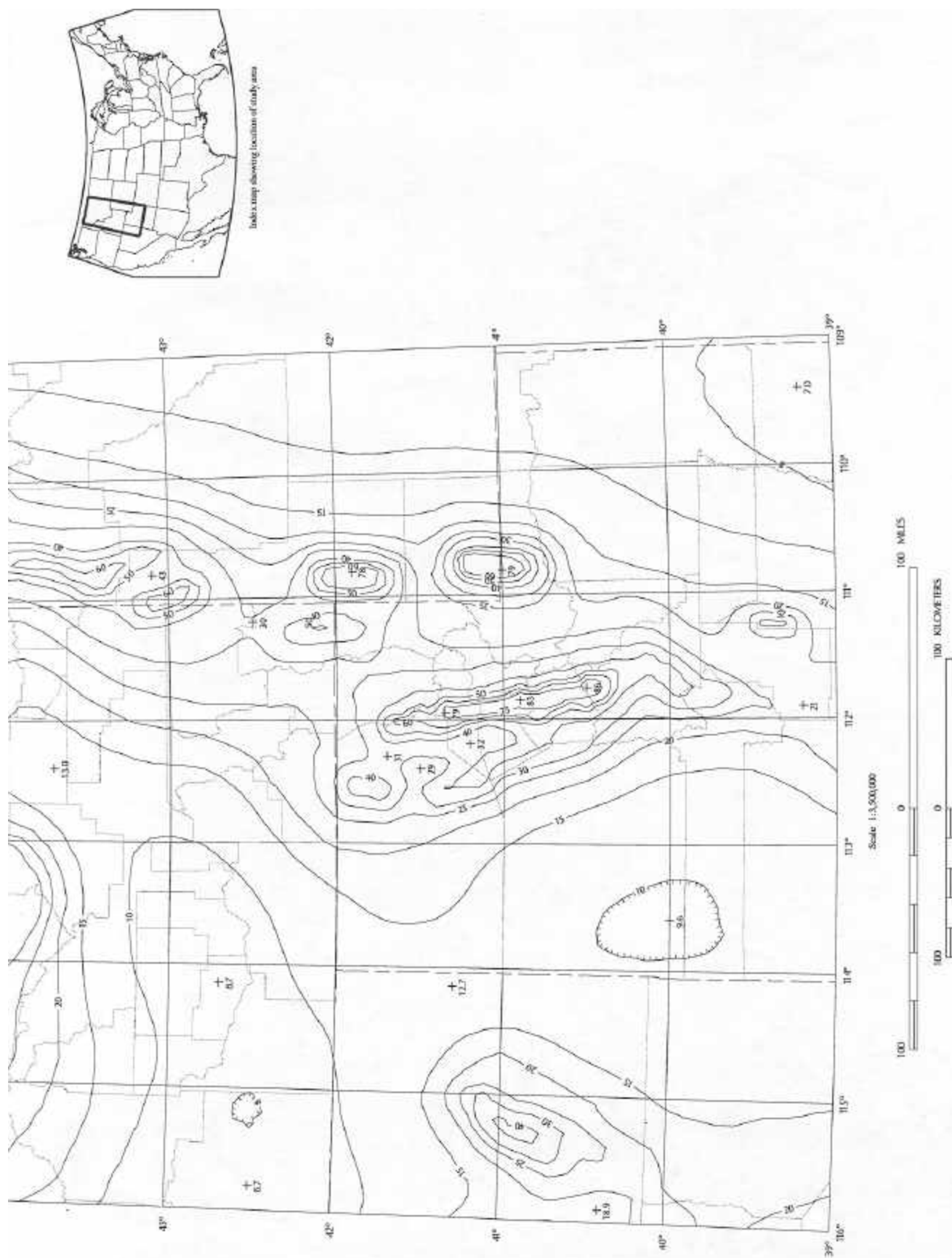


Figure 3.10.2.1-1(f) (continued) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR REGION 2 OF 1.0 SEC. SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

Figure 3.10.2.1-1(g) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR ALASKA OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

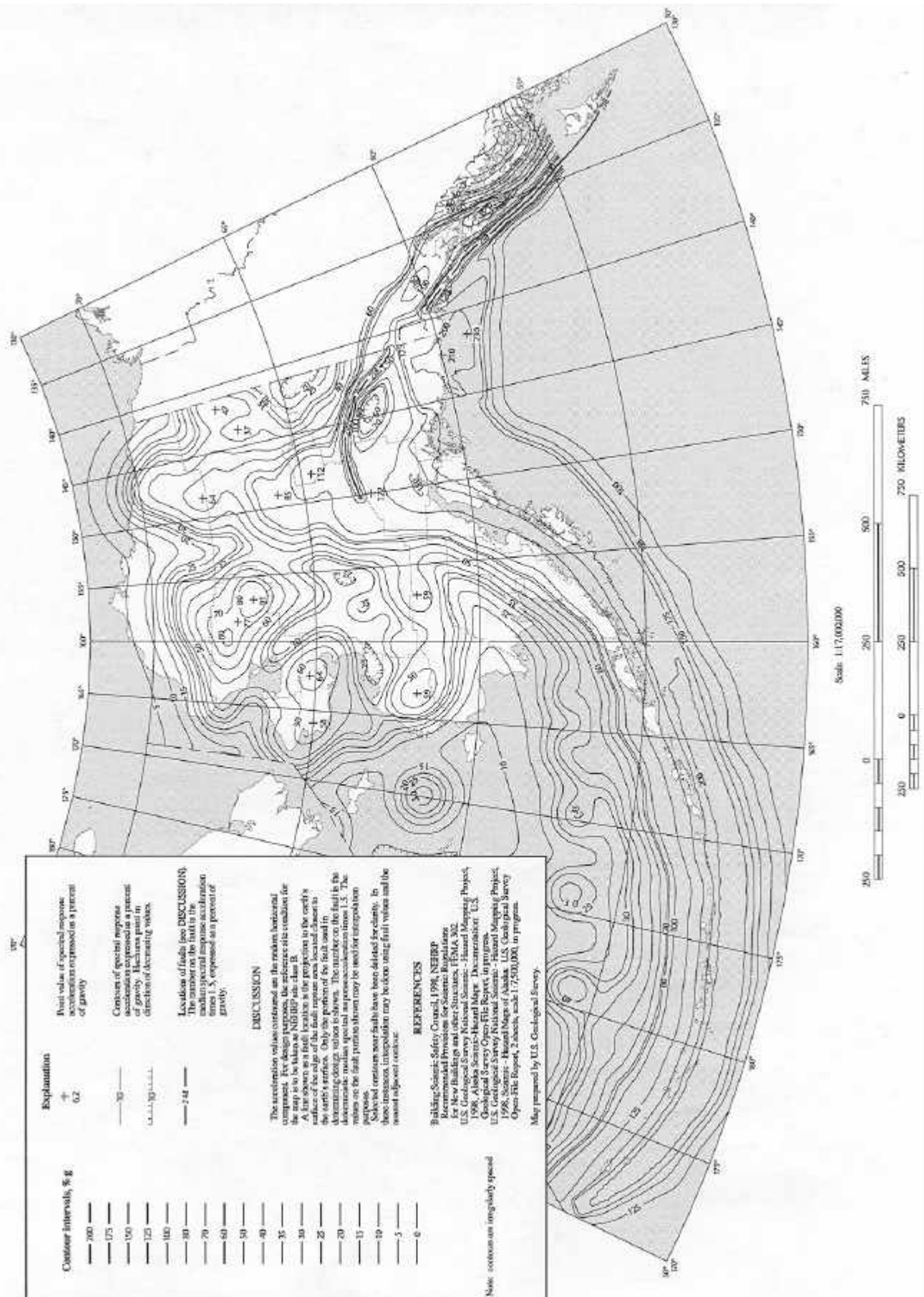


Figure 3.10.2.1-1(h) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR ALASKA OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

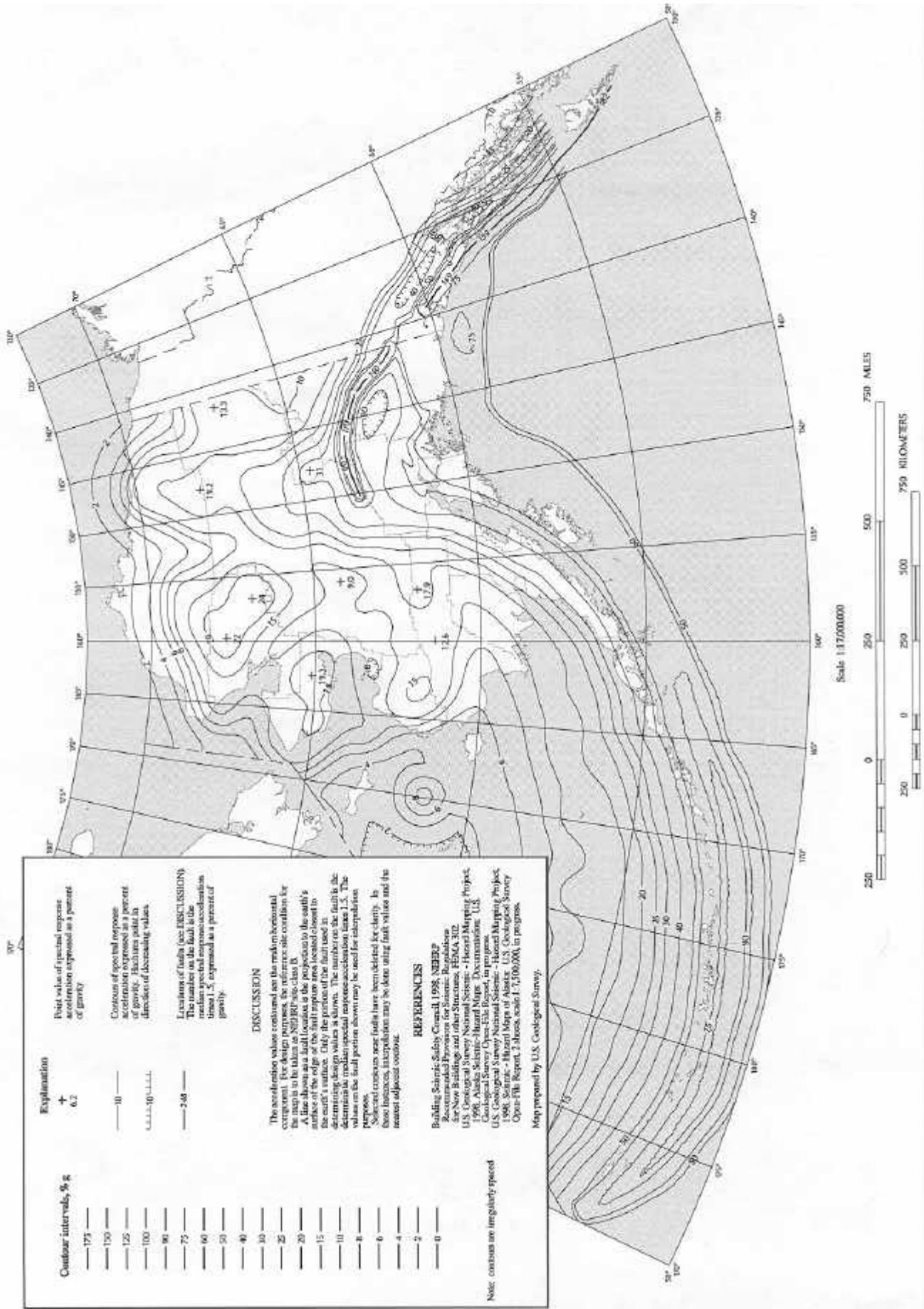


Figure 3.10.2.1-1(i) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR HAWAII OF 0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

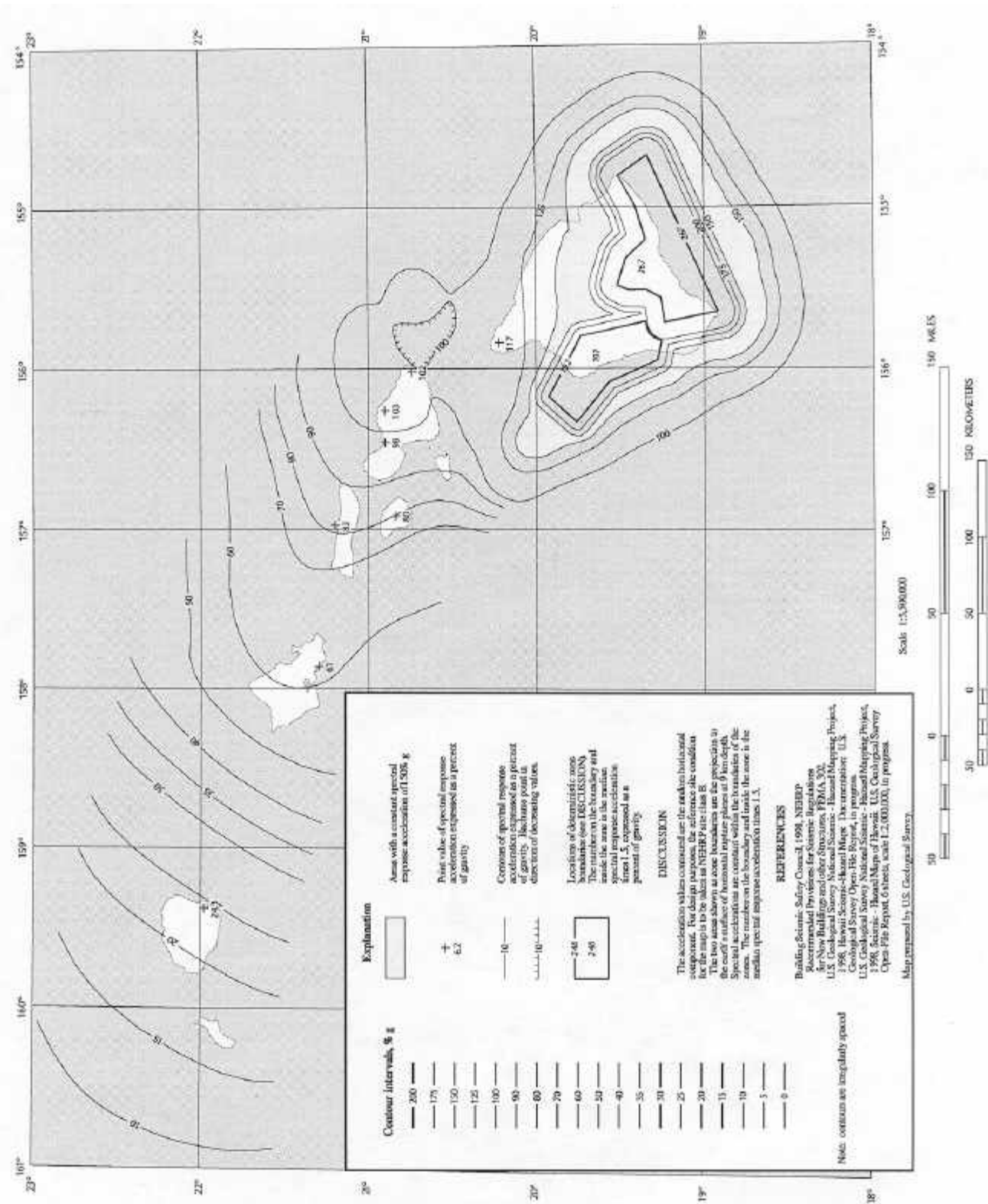
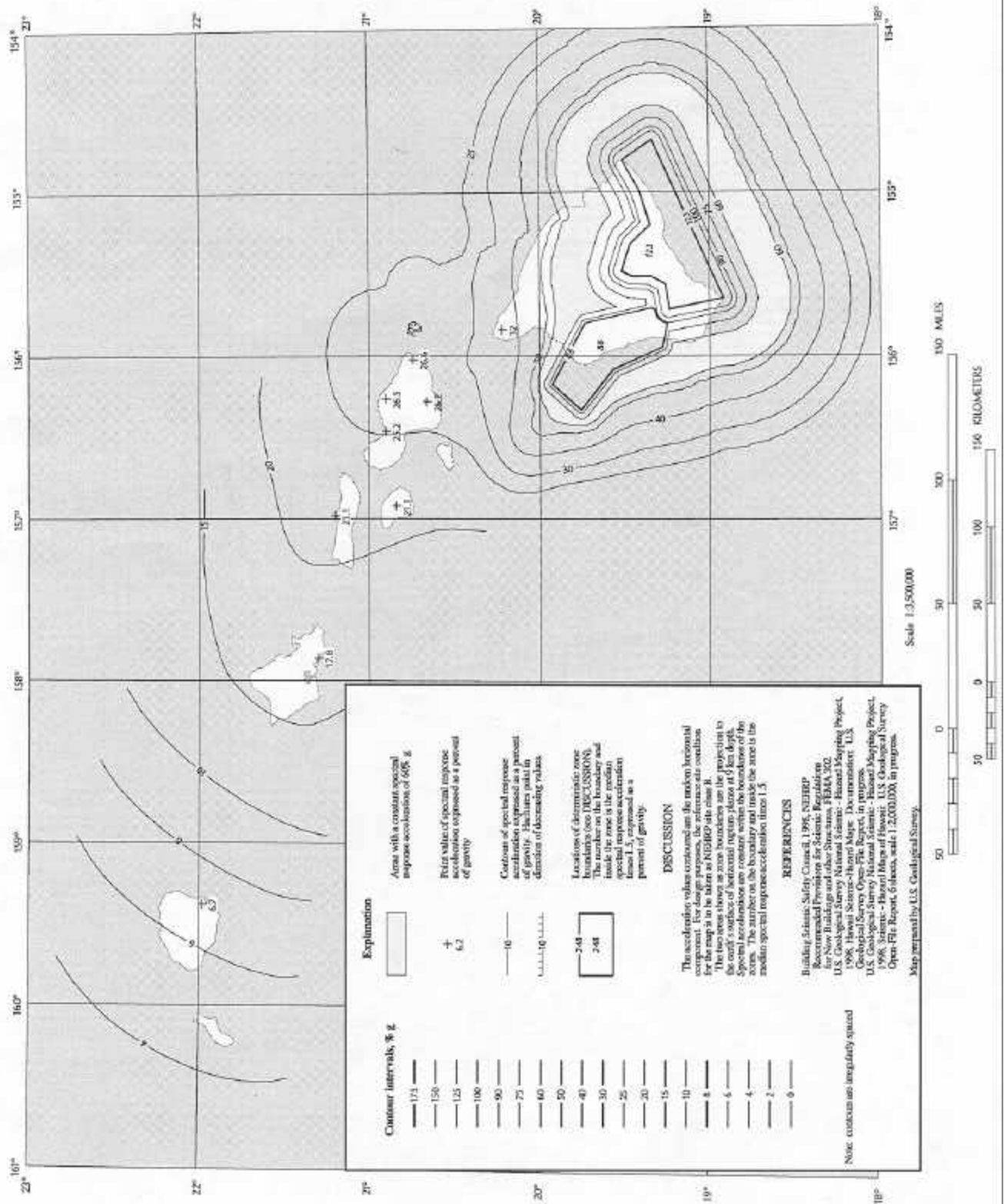
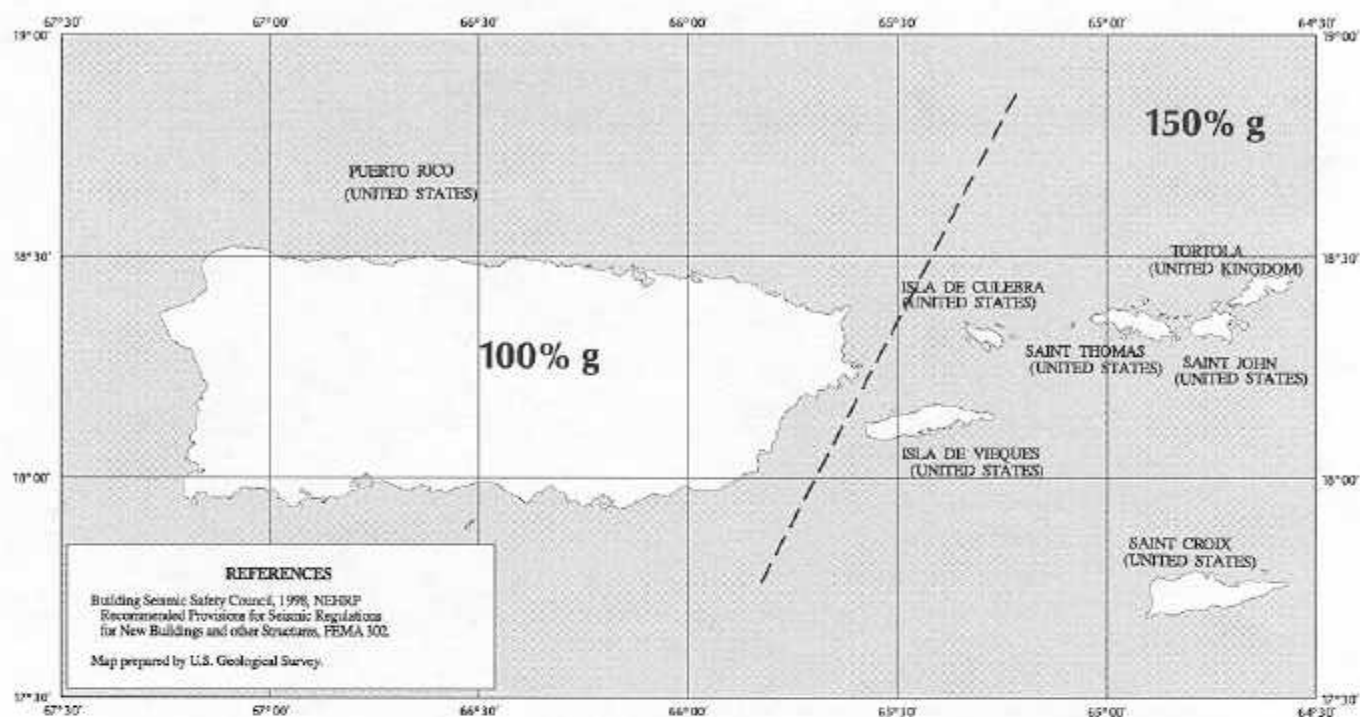


Figure 3.10.2.1-1(i) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR HAWAII OF 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B

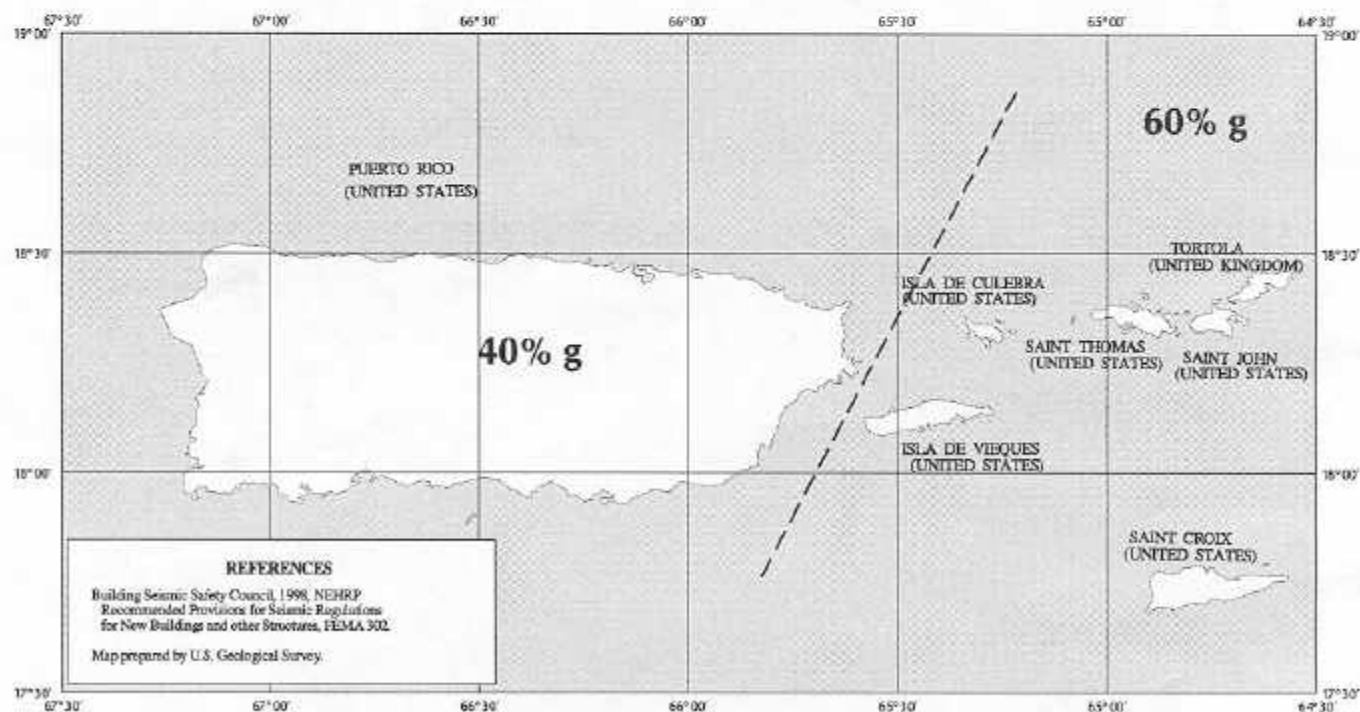


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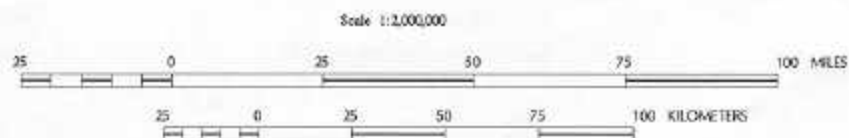
Figure 3.10.2.1-1(k) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR PUERTO RICO, CULEBRA, VIEQUES, ST. THOMAS, ST. JOHN, AND ST. CROIX OF 0.2 AND 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



0.2 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)

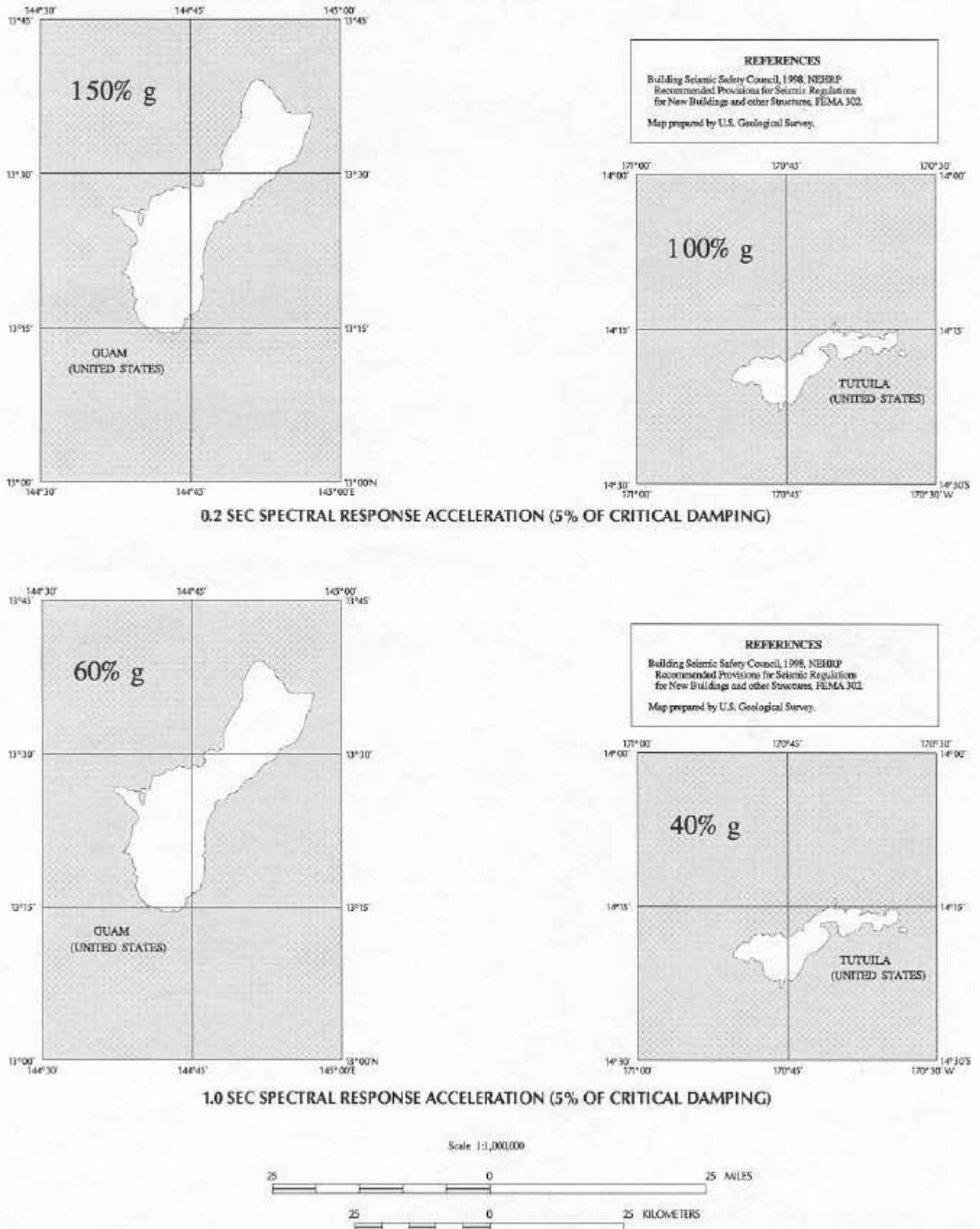


1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING)



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Figure 3.10.2.1-1(l) MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION FOR GUAM AND TUTUILA OF 0.2 AND 1.0 SEC SPECTRAL RESPONSE ACCELERATION (5% OF CRITICAL DAMPING), SITE CLASS B



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As indicated in Article 3.10.2.3.3 and Tables 3.10.2.3.3-1 and -2, site coefficients F_a and F_v are not provided for Site Class F soils and site-specific geotechnical investigations and dynamic site response analyses are required for these soils. Guidelines are provided below for conducting site-specific investigations and site response analyses for Site Class F soils. These guidelines are also applicable if it is desired to conduct dynamic site response analyses for other soil types. Additional guidance on the topics addressed below is presented in a report by the Caltrans Seismic Advisory Board Ad Hoc Committee on Soil-Foundation-Structure-Interaction (CSABAC, 1999).

Site-Specific Geotechnical Investigation.

For purposes of obtaining data to conduct a site response analysis, site-specific geotechnical investigations should include borings with sampling, standard penetration tests (SPTs) cone penetrometer tests (CPTs), and/or other subsurface investigative techniques and laboratory soil testing to establish the soil types, properties, and layering and the depth to rock or rock-like material. It is desirable to measure shear wave velocities in all soil layers. Alternatively, shear wave velocities may be estimated based on shear wave velocity data available for similar soils in the local area or through correlations with soil types and properties. A number of such correlations are summarized by Kramer (1996).

Dynamic Site Response Analysis:

Components of a dynamic site response analysis include: (1) modeling the soil profile; (2) selecting rock motions to input into the soil profile; and (3) conducting a site response analysis and interpreting the results.

1. Modeling the soil profile: Typically, a one-dimensional soil column extending from the ground surface to bedrock is adequate to capture first-order site response characteristics. However, two- to three-dimensional models may be considered for critical projects when two or three-dimensional wave propagation effects may be significant (e.g., in basins). The soil layers in a one-dimensional model are characterized by their total unit weights, shear wave velocities from which low-strain (maximum) shear moduli may be obtained and by relationships defining the nonlinear shear stress-strain relationships of the soils. The required relationships for analysis are often in the form of curves that describe the variation of shear modulus with shear strain (modulus reduction curves) and by curves that describe the variation of damping with shear strain (clamping curves). In a two- or three-dimensional model, compression wave velocities or moduli or Poissons ratios are also required. In an analysis to estimate the effects of liquefaction on soil site response, the nonlinear soil model must also incorporate the buildup of soil pore water pressures and the consequent effects on reducing soil stiffness and strength. Typically, modulus reduction curves and damping curves are selected on the basis of published relationships for similar soils (e.g., Seed and Idriss, 1970; Seed et al., 1986; Sun et al., 1988; Vucetic and Dobry, 1991; Electric Power Research Institute, 1993; Kramer, 1996). Site-specific laboratory dynamic tests on soil samples to establish nonlinear soil characteristics can be considered where published relationships are judged to be inadequate for the types of soils present at the site. The uncertainty in soil properties should be estimated, especially the uncertainty in the selected maximum shear moduli and modulus reduction and damping curves.
2. Selecting input rock motions: Acceleration time histories that are representative of horizontal rock motions at the site are required as input to the soil model. Unless a site-specific analysis is carried out

to develop the rock response spectrum at the site, the Maximum Credible Earthquake (MCE) rock spectrum for Site Class B rock can be defined using the general procedure described in Section 2.5. For hard rock (Site Class A), the spectrum may be adjusted using the site factors in Tables 3.10.4.3-1 and –2. For profiles having great depths of soil above site class A or B rock, consideration can be given to defining the base of the soil profile and the input rock motions at a depth at which soft rock or very stiff soil of Site Class C is encountered. In such cases, the design rock response spectrum may be taken as the spectrum for Site Class C defined using the site factors in Tables 3.10.4.3-1 and –2. Several acceleration time histories, typically at least four, recorded during earthquakes having magnitudes and distances that significantly contribute to the site seismic hazard should be selected for analysis. The U.S. Geological Survey results for deaggregation of seismic hazard (website address: <http://geohazards.cr.usgs.gov/eq/>) can be used to evaluate the dominant magnitudes and distances contributing to the hazard. Prior to analysis, each time history should be scaled so that its spectrum is at the approximate level of the design rock response spectrum in the period range of interest. It is desirable that the average of the response spectra of the suite of scaled input time histories be approximately at the level of the design rock response spectrum in the period range of interest. Because rock response spectra are defined at the ground surface rather than at depth below a soil deposit, the rock time histories should be input in the analysis as outcropping rock motions rather than at the soil-rock interface.

3. Site response analysis and results interpretation. Analytical methods may be equivalent linear or nonlinear. Frequently used computer programs for one-dimensional analysis include the equivalent linear program SHAKE (Schnabel et al., 1972; Idriss and Sun, 1992) and nonlinear programs DESRA-2 (Lee and Finn, 1978), MARDES (Chang et al., 1991), SUMDES (Li et al., 1992), D-MOD (Matasovic, 1993), TESS (Pyke, 1992), and MUSC (Qiu, 1998). If the soil response is highly nonlinear (e.g. high acceleration levels and soft clay soils), nonlinear programs are generally preferable to equivalent linear programs. For analysis of liquefaction effects on site response, computer programs incorporating pore water pressure development (effective stress analyses) must be used (e.g., DESRA-2, SUMDES, D-MOD and TESS). Response spectra of output motions at the ground surface should be calculated and the ratios of response spectra of ground surface motions to input outcropping rock motions should be calculated. Typically, an average of the response spectral ratio curves is obtained and multiplied by the design rock response spectrum to obtain a soil response spectrum. This response spectrum is then typically adjusted to a smooth design soil response spectrum by slightly decreasing spectral peaks and slightly increasing spectral valleys. Sensitivity analyses to evaluate effects of soil property uncertainties should be conducted and considered in developing the design response spectrum.

Appendix 3A – Guidelines for Conduction Site-Specific Geotechnical Investigations and Dynamic Site Response Analyses

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Appendix 3A – Guidelines for Conduction Site-Specific Geotechnical Investigations and Dynamic Site Response Analyses

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3B Collateral Seismic Hazards

The term collateral seismic hazards refers to earthquake-caused movement of the earth that either results in loads being imposed on a bridge foundation system or causes changes in the resistance of the earth that affects the response of a bridge-foundation system. These effects can be either dynamic or static in form. Liquefaction is one of the most well-known examples of a collateral hazard. This Appendix provides an overview of methods used to evaluate and design for these collateral hazards. This overview includes

- a general discussion of the term collateral hazards and the implication of these hazards on design of bridge foundations (Article 3B.1)
- a summary of methods used to screen for and evaluate liquefaction and associated hazards, such as lateral flows, lateral spreading, settlement, and differential settlement (Article 3B.2)
- an overview of other collateral hazards such as faulting, landsliding, differential compaction, and flooding and inundation (Article 3B.3), and
- a review of methods for designing spread footings and deep foundations for the most common collateral hazards, liquefaction (Article 3B.4)

The design of a bridge structure should consider the potential for these collateral hazards during the initial type, size, and location (TS&L) phase of the project, as significant cost can be incurred to design for, mitigate, or avoid these hazards.

3B.1 General

The most common of the collateral hazards is liquefaction. During liquefaction, saturated granular soil loses stiffness and strength, which can affect the vertical or lateral bearing support of a foundation. Under normal circumstances, these losses in support can be handled during design. The more serious consequences of liquefaction are permanent lateral ground movements and settlement of the soil, both of which can damage a bridge foundation system.

Several other types of hazards associated with seismic-related ground behavior also can lead to damage of a bridge. These hazards include ground faulting, landsliding, differential compaction, and inundation and flooding resulting from earthquake-induced failures of dams or reservoirs.

3B.1.1 Evaluation of Collateral Hazards

Various procedures have been developed over the past 20 years for quantifying the potential for and the consequences of these geologic hazards. The discussions in this Appendix summarize procedures and approaches commonly employed within the profession. The applicability of these procedures will

C3B.1

The term collateral hazards has been selected to differentiate loads that are imposed on a structure by displacement of soil from loads developed within a structure due to the inertial response of the bridge deck and abutments. These hazards are also called geologic or geotechnical hazards by those practicing in the areas of geology and geotechnical engineering. In this Appendix the terms geologic hazards and collateral hazards are used interchangeably.

Displacement associated with these collateral hazards can be very large, often being on the order of a meter and sometimes being as large as several meters. In some cases such as liquefaction-induced flow failures or landsliding, it will be difficult to prevent or limit displacement without significant expenditure of project funds. In the case of faulting the displacement cannot be prevented; all that can be done is to design the structure to withstand or avoid the movement.

C.3B.1.1

As time passes and more is learned about seismic response of soil, methods for identifying and dealing with collateral seismic hazards will likely change. For this reason this Appendix is intended to provide guidance and not be prescriptive.

depend on the soil conditions at the site, the complexity of the structure, and the risk that the Owner is prepared to assume.

Much of the following discussion will focus on the evaluation of liquefaction and its related hazards. Procedures given in this Appendix for the assessment of liquefaction are based on a consensus document prepared after a workshop sponsored by the National Earthquake Engineering Research (NCEER) in 1996 (Youd and Idriss, 1997). The workshop was attended by a group of leading professionals working or conducting research in the area of liquefaction. The NCEER Workshop participants were not always in complete agreement in all areas dealing with liquefaction or design for liquefaction; however, the participants did agree that the NCEER Workshop report would form a minimum basis for conducting liquefaction evaluations. It was expected that the profession would build on these methods as more information became available.

The dilemma that an Owner will face is deciding when methods advocated by an individual or group of individuals should be used to upgrade the procedures developed during the consensus NCEER Workshop. There is no simple process of making these decisions, a situation that is common to any evolving technology.

3B.1.2 Designing for Collateral Hazards

The design of bridge structures for collateral hazards must consider the movement of the earth and the changes in soil properties resulting from this movement. In the case of liquefaction both effects must be considered in design. The first is that the bridge must perform adequately with just the liquefaction-induced soil changes alone. This means that the mechanical properties of the soil that liquefy are changed to reflect their post-liquefaction values (e.g., properties such as “p-y curves” and modulus of subgrade reaction values used to evaluate the lateral stiffness of a pile foundation are reduced). The second component of the design is the consideration of liquefaction-related ground movements. These can take several forms: lateral spreading, lateral flow, and ground settlement.

- Lateral spreading is a lateral movement that is induced by the ground shaking and develops in an incremental fashion as shaking occurs.
- Lateral flow is movement that occurs due to the combined effects of sustained porewater pressure and gravity loads without the inertial loading from the earthquake. Flows can occur several minutes following an earthquake, when porewater pressures redistribute to form a

C3B.1.2

The focus of this Appendix is the design for liquefaction and liquefaction-related hazards, as liquefaction has been perhaps the single most significant cause of damage to bridge structures during past earthquakes. Most of the damage has been related to lateral movement of soil at the bridge abutments. However, cases involving the loss in lateral and vertical bearing support of foundations for central piers of a bridge have also occurred.

Loss in lateral support and permanent ground movement can occur simultaneously during a seismic event. Their simultaneous occurrence is a complicated process that is difficult to represent without the use of very complex computer modeling. For most bridges the complexity of the modeling does not warrant performing a combined analysis. In these cases the recommended methodology is to consider these effects independently, i.e., de-coupled. The reasoning behind this is that it is not likely that the peak vibrational response and the peak spreading or flow effect will occur simultaneously. For many earthquakes the peak vibration response occurs somewhat in advance of maximum ground movement loading. For very large earthquakes where liquefaction may occur before peak ground accelerations occur, the peak vibration response is

Appendix 3B – Provisions for Collateral Seismic Hazards

Liquefaction and other geologic Hazards

critical combination with gravity loading.

- Dynamic settlement occurs following an earthquake as porewater pressures dissipate.

These liquefaction-related effects are normally considered separately as uncoupled events.

like to be significantly attenuated and, hence, inertial loading reduced from peak design values. In addition peak displacements demands arising from lateral ground spreading are likely to generate maximum pile moments at depths well below peak moments arising from inertial loading. Finally, the de-coupling of response allows the flexibility to use separate and different performance criteria for design to accommodate these phenomena.

Two detailed case studies on the application of the recommended design methods for both liquefaction and lateral flow design are given in an NCHRP Report (ATC/MCEER, 2000).

3B.2 Liquefaction¹

The need for an evaluation of liquefaction and liquefaction-related hazards depends on the level of ground shaking and the magnitude of the earthquake that could occur at a site. In areas of very low seismicity (SDR 1 and SDR 2), no specific seismic design requirements occur. On the other hand, the potential for liquefaction at sites should be determined for sites located in SDR 3, 4, 5, and 6.

The evaluation of liquefaction potential should follow procedures given in Youd and Idriss (1997) and SCEC (1999). These procedures are summarized in Article 3B.2.

C3B.2

In SDR's 1 and 2 the potential for liquefaction is generally low. In some cases the peak ground acceleration in these SDR's may exceed 0.15g. While this level of peak ground acceleration is sufficient to cause liquefaction, the magnitude of the earthquake causing liquefaction in these categories will generally be less than 6. For this earthquake magnitude liquefaction develops slowly for most soils, and results in minimal effects other than ground settlement.

The potential for liquefaction in SDR's 3, 4, 5, and 6 is much higher, and therefore careful attention to the determination of the potential for and consequences of liquefaction is needed for sites with these classifications. At some locations it may be necessary to use ground improvement methods to mitigate the potential effects of liquefaction. As these methods are often expensive, detailed consideration of the potential for liquefaction is warranted.

3B.2.1 Preliminary Screening for Liquefaction

An evaluation of liquefaction hazard potential may not be required if the following conditions occur at a site:

C3B.2.1

Liquefaction will generally occur in loose, saturated granular materials. These granular materials can include silts, sands, and in some cases loose gravels. Liquefaction of loose gravels

¹ Much of the contents of this discussion of liquefaction was taken from a report titled "Recommended Procedures for Implementation of DMG Special Publication 117, Guideline for Analyzing and Mitigating Liquefaction in California" and referenced as SCEC (1999). The SCEC report was prepared by a group of consultants and government agency staff led by Dr. G.R. Martin of the University of Southern California and Dr. M. Lew of Law/Crandall. Funding for the report was provided by the City of Los Angeles, the County of Los Angeles, the California Division of Mines and Geology, the Federal Emergency Management Agency, as well as the Counties of Riverside, San Bernadino, San Diego, Orange, and Ventura. The intent of the SCEC report was to provide practical guidance to design engineers in the implementation of liquefaction prediction and hazards evaluation methods. The SCEC report represented the current state-of-the-practice at the time that these LFRD specifications were being prepared. Where appropriate, the SCEC report recommendations have been updated or augmented in this Appendix to be more consistent with requirements for bridge design or new developments in liquefaction assessment methodologies.

Appendix 3B – Provisions for Collateral Seismic Hazards

Liquefaction and other geologic Hazards

- The estimated maximum-past-, current-, and maximum-future-groundwater-levels (i.e., the highest groundwater level applicable for liquefaction analyses) are determined to be deeper than 15 m below the existing ground surface or proposed finished grade, whichever is deeper.
- “Bedrock” or similar lithified formational material underlies the site. In many areas glacially overridden (till) deposits fall in this classification.
- The corrected standard penetration blow count, $(N_1)_{60}$, is greater than or equal to 30 in all samples with a sufficient number of tests. If cone penetration test soundings are made, the corrected cone penetration test tip resistance, q_{c1N} , should be greater than or equal to 160 in all soundings in sand materials.
- The soil is clayey. For purposes of this screening, clayey soils are those that have a clay content (i.e., particle size <0.005 mm) greater than 15 percent. However, based on the so-called “Chinese Criteria,” (Seed and Idriss, 1982) clayey soils having all of the following characteristics may be susceptible to severe strength loss:
 - Percent finer than 0.005 mm less than 15 percent
 - Liquid Limit less than 35
 - Water Content greater than $0.9 * \text{Liquid Limit}$

If the screening investigation clearly demonstrates the absence of liquefaction hazards at a project site and the Owner concurs, the screening investigation will satisfy the site investigation report requirement for liquefaction hazards. If not, a quantitative evaluation will be required to assess the liquefaction hazards.

3B.2.2 Field Explorations for Liquefaction Hazards Assessment

Two field exploration methods are normally used during the evaluation of liquefaction potential, Standard Penetration Test (SPT) methods and Cone Penetrometer Test (CPT) methods. Appendix 2A gives a brief discussion of these methods. These methods should be regarded as the minimum requirement for evaluating site liquefaction potential. A geologic reconnaissance and review of the available geotechnical information for the site should supplement any field investigation.

has been observed during several earthquakes when cohesive soils overlying the gravel prevented drainage of porewater pressures.

Geologically young cohesionless materials are more susceptible than geologically old cohesionless soils, as a result of cementation and other similar aging effects that tend to occur in geologically old materials. Common geologic settings for liquefaction-susceptible soils include unlithified sediments in coastal regions, bays, estuaries, river floodplains and basins, areas surrounding lakes and reservoirs, and wind-deposited dunes and loess. In many coastal regions, liquefiable sediments occupy back-filled river channels that were excavated during Pleistocene low stands of sea level, particularly during the most recent glacial stage. Among the most easily liquefiable deposits are beach sand, dune sand, and clean alluvium that were deposited following the rise in sea level at the start of the Holocene age, about 11,000 years ago.

Preliminary screening can often be used to eliminate a site from further liquefaction consideration. The screening investigation should include a review of relevant topographic, geologic, and soils engineering maps and reports, aerial photographs, groundwater contour maps, water well logs, agricultural soil survey maps, the history of liquefaction in the area, and other relevant published and unpublished reports. The purpose of the screening investigations for sites within zones of required study is to filter out sites that have no potential or low potential for liquefaction.

No specific limitation is placed on the depths of liquefiable soils in the screening process. As discussed in a following section of this Appendix, liquefaction can occur to depths of 25 m or more.

C3B.2.2

A number of factors must be considered during the planning and conduct of the field exploration phase of the liquefaction investigation.

Location of Liquefiable Soils

During the field investigation, the limits of unconsolidated deposits with liquefaction potential should be mapped within and beyond

SPT Method

Procedures for evaluating liquefaction potential using SPT methods are described in detail by Youd and Idriss (1997) and by SCEC (1999). These procedures include consideration of correction factors for drilling method, hole diameter, drive-rod length, sampler type, energy delivery, and spatial frequency of tests.

Information presented in Youd and Idriss (1997) and in SCEC (1999) indicate that the results of SPT explorations are affected by small changes in measurement method; therefore, it is critical for these tests that standard procedures are followed and that all information regarding the test method and equipment used during the field work be recorded. The energy of the SPT hammer system should also be established for the equipment, as this energy directly affects the determination of liquefaction potential. The variation in hammer energy can be as much as a factor of 2, which can easily cause a liquefiable site to be identified as being nonliquefiable, if a correct hammer calibration factor is not introduced.

CPT Method

The CPT is gaining recognition as the preferred method of evaluating liquefaction potential in many locations. Methods for assessing liquefaction potential from CPT results are given in Youd and Idriss (1997). The primary advantages of the CPT method are:

- The method provides an almost continuous penetration resistance profile that can be used for stratigraphic interpretation, which is particularly important in determining the potential for lateral spreading, lateral flows, and significant differential post-liquefaction settlements.
- The repeatability of the test is very good.
- The test is fast and economical compared to drilling and laboratory testing of soil samples.

The limitations of the method are:

- The method does not routinely provide soil samples for laboratory tests.
- The method provides approximate, interpreted soil behavior types and not the actual soil types according to ASTM Test Methods D 2488 (Visual Classification) or D 2487 (USCS Classification) [ASTM, 1998].

the footprint of the bridge. Typically, this will involve an investigations at each pier location and at enough location away from the approach fill to establish the spatial variability of the material. The investigation should establish the thickness and consistency of liquefiable deposits from the ground surface to the depth at which liquefaction is not expected to occur. The “zone of influence” where liquefaction could affect a bridge approach fill will generally be located within a 2H:1V (horizontal to vertical) projection from the bottom of the approach fill.

Location of Groundwater Level

The permanent groundwater level should be established during the exploration program. Shallow groundwater may exist for a variety of reasons, some of which are of natural or man-made origin. Groundwater may be shallow because the ground surface is only slightly above the elevation of the ocean, a nearby lake or reservoir, or the sill of a basin. Another concern is man-made lakes and reservoirs that may create a shallow groundwater table in young sediments that were previously unsaturated. If uncertainty exists in the location of the groundwater level, piezometers should be installed during the exploration program. The location of the groundwater level should be monitored in the piezometers over a sufficient duration to establish seasonal fluctuations that may be due to rainfall, river runoff, or irrigation.

Usually, soils located below the groundwater level are fully saturated; however, at locations where fluctuations in groundwater occur, soil can be in a less than fully saturated conditions. The liquefaction resistance of the soil is affected by the degree of saturation, with the resistance increasing significantly as the degree of saturation decreases. If the groundwater level fluctuates due to tidal action or season river fluctuations, then the zone of fluctuation will often have a lower degree of saturation, making the soil more resistant to liquefaction. Unless the seasonal fluctuation is in place for an extended period of time, say weeks at a higher level, it is usually acceptable to use an long-term groundwater level as a basis for design.

Depth of Liquefaction

The field exploration should be conducted to the maximum depth of liquefiable soil. A depth of about 15 m has often been used as the depth of analysis for the evaluation of liquefaction.

[ASTM, 1998].

- The test cannot be performed in gravelly soils and sometimes the presence of hard/dense crusts or layers at shallow depths makes penetration to desired depths difficult.

The CPT method should be performed in accordance with ASTM D 3441 (ASTM, 1998). Generally, it is recommended that at least one boring be drilled to confirm soil types and obtain samples for laboratory testing if the CPT method is used for evaluating liquefaction potential.

3B.2.4 Ground Motions for Liquefaction Analysis

To perform an analysis of liquefaction triggering, liquefaction settlement, seismically induced settlement, and lateral spreading, a peak horizontal ground acceleration and a mean earthquake magnitude must be established for the site:

- **Peak Ground Acceleration (PGA):** Either the seismic hazard maps described in Article 3.10.2 or a site-specific probabilistic seismic hazard analysis (PSHA), as discussed in Appendix 3A to this section, can be used to determine the design value of PGA. In both methods, appropriate adjustments must be made to correct the firm-ground motion obtained from the map or from the PSHA for local site effects.
- **Earthquake Magnitude:** The magnitude required in the liquefaction analysis can be determined from magnitude-distance deaggregation information for PGA given in the

However, the Seed and Idriss EERI Monograph on “Ground Motions and Soil Liquefaction During Earthquakes” (1982) does not recommend a minimum depth for evaluation, but notes 12 m as a depth to which some of the numerical quantities in the “simplified procedure” can be estimated reasonably. Liquefaction has been known to occur during earthquakes at deeper depths than 15 m given the proper conditions such as low-density granular soils, presence of groundwater, and sufficient cycles of earthquake ground motion. For example, liquefaction occurred to depths in excess of 25 m during the 1964 Alaska earthquake.

For this reason it is recommended that a minimum depth of 25 m below the existing ground surface or lowest proposed finished grade (whichever is lower) be investigated for liquefaction potential. For deep foundations (e.g., shafts or piles), the depth of investigation should extend to a depth that is a minimum of 6 m below the lowest expected foundation level (e.g., shaft bottom or pile toe) or 25 m below the existing ground surface or lowest proposed finished grade, whichever is deeper.

If, during the investigation, the indices to evaluate liquefaction indicate that the liquefaction potential may extend below that depth, the exploration should be continued until a significant thickness (e.g., at least 3 m, to the extent possible) of nonliquefiable soils is encountered.

C3B.2.4

The peak ground acceleration used in the simplified liquefaction evaluation is defined at the ground surface. Maps and most site-specific hazard evaluations also define the PGA at the ground surface; however, the soil conditions used to develop the PGA maps or the attenuation relationships in the PHSA are relatively stiff (Site Classification B/C) as defined in Article 3.10.2.2.1 of the Specifications. It is necessary to adjust these accelerations for local site effects. This adjustment can be made by either using the factors given in Table 3.10.2.3.3-1 or by conducting site-specific ground response studies with a computer program such as SHAKE (Idriss and Sun, 1992) or DESRA 2 (Lee and Finn, 1978).

When Table 3.10.2.3.3-1 is used to estimate site factors, the amplification or attenuation factor is determined on the basis of the Site Class before liquefaction and the Spectral Acceleration

USGS Website (<http://geohazards.cr.usgs.gov/eq/>) or as part of the site-specific PSHA. The mean magnitude of the deaggregation will be applicable for most locations; however, if a single or few magnitude-distance peaks dominate the distribution (e.g., characteristic earthquake on a seismic source), the peak or the mean of the few peaks should be used to define the magnitude. In locations where bi- or tri-modal magnitude-distance distributions occur, each magnitude and an associated acceleration level should be considered.

at Short Periods (S_s), where S_s is equal to $2.5 * \text{PGA}$.

Although for most analyses, information in the USGS Website will be sufficient for determining the PGA and the earthquake magnitude, a site-specific PSHA may provide better estimation of the ground motions at some locations. The decision to perform a PSHA should be made after detailed discussions with the Owner.

3B.2.5 Evaluation of Liquefaction Hazard

Two basic procedures are used to evaluate the potential for liquefaction at a site. These involve

- a simplified procedure that is based on empirical correlations to observations of liquefaction, or
- more rigorous numerical modeling.

The decision between the two procedures should be made after careful review of conditions at the site and the risks associated with liquefaction, and with the concurrence of the Owner.

3B.2.5.1 Simplified Method

The most basic procedure used in engineering practice for assessment of site liquefaction potential is that of the "Simplified Procedure" originally developed by Seed and Idriss (1971, 1982) with subsequent refinements by Seed et al. (1983), Seed et al. (1985), Seed and De Alba (1986), and Seed and Harder (1990). The procedure essentially compares the cyclic resistance ratio (CRR) [the cyclic stress ratio required to induce liquefaction for a cohesionless soil stratum at a given depth] with the earthquake-induced cyclic stress ratio (CSR) at

C3B.2.5

For most projects the simplified procedure will be acceptable. However, for critical projects, more rigorous modeling using equivalent linear and nonlinear computer codes may be appropriate. Conditions warranting use of more rigorous methods include (1) sites where liquefiable soils extend to depths greater than 25 m, (2) sites that have significant interlayering, particularly where interlayers comprise highly permeable soils or soft clay layers, and (3) sites where the cost of ground remediation methods to mitigate liquefaction is great. Most site-specific ground response analyses result in lower estimations of ground acceleration and shearing stresses within the soil profile because the energy dissipative mechanisms occurring during liquefaction are explicitly considered in this approach.

C3B.2.5.1

Adjustments for changes in water table and overburden condition should be made during the simplified analyses. The following guidance can be used in making these adjustments.

Overburden Corrections for Differing Water Table Conditions

To perform analyses of liquefaction triggering, liquefaction settlement, seismically induced settlement, and lateral spreading, it is

that depth from a specified design earthquake (defined by a peak ground surface acceleration and an associated earthquake magnitude).

CRR

Values of CRR for the Simplified Method were originally established from databases for sites that did or did not liquefy during past earthquakes and where values of the normalized SPT value, $(N_1)_{60}$, could be correlated with liquefied strata. The current version of the baseline chart defining values of CRR as a function of $(N_1)_{60}$ for magnitude 7.5 earthquakes is shown on Figure 3B.2.5.-1. This chart was established by a consensus at a 1996 NCEER Workshop, which convened a group of experts to review current practice and new developments in the area of liquefaction evaluations (Youd and Idriss, 1997). The CRR value can also be obtained using CPT, Becker Hammer Tests (BHT), or shear wave velocity methods, as discussed by Youd and Idriss (1997). The determination of CRR must consider the fines content of the soil, the energy of the hammer for the SPT and BHT methods, the effective overburden pressure, and the magnitude of the earthquake.

CSR

For estimating values of the earthquake-induced cyclic shearing stress ratio, CSR, the NCEER Workshop recommended essentially no change to the original simplified procedure (Seed and Idriss, 1971), where the use of a mean r_d factor defining the reduction in CSR with depth is usually adopted for routine engineering practice, as shown in Figure 3B.2.5-3. As an alternative, a site-specific response analysis of the ground motions can be performed, as mentioned in the next section.

CSR is calculated using the following equation:

$$CSR = (\tau_{av}/\sigma'_{vo}) = 0.65(a_{max}/g)(\sigma_{vo}/\sigma'_{vo})r_d$$

where τ_{av}/σ'_{vo} is the earthquake-induced shearing stress, a_{max}/g is the PGA at the ground surface, σ_{vo}/σ'_{vo} is the ratio of total overburden stress to effective overburden stress, and r_d is a soil flexibility number.

Liquefaction Potential

Once values of CRR and CSR are established for a soil stratum at a given depth, the factor of safety against liquefaction (i.e., $FS = CRR/CSR$)

necessary to develop a profile of SPT blow counts or CPT q_c -values that have been normalized using the effective overburden pressure.

This normalization should be performed using the effective stress profile that existed at the time the SPT or CPT testing was performed. Then, those normalized values are held constant throughout the remainder of the analyses, regardless of whether or not the analyses are performed using higher or lower water-table conditions. Although the possibility exists that softening effects due to soil moistening can influence SPT or CPT results if the water table fluctuates, it is commonly assumed that the only effect that changes in the water table have on the results is due to changes in the effective overburden stress.

Raw, field N -values (or q_c -values) obtained under one set of groundwater conditions should not be input into an analysis where they are then normalized using C_N correction factors based on a new (different) water table depth.

Overburden Corrections for Differing Fill Conditions

Approach fills and other increases in overburden pressure should be handled similar to that described above for changes in groundwater location. It is necessary to develop a profile of SPT blow counts or CPT q_c -values that have been normalized using the effective overburden pressure existing before the fill is placed. Then, these normalized values are held constant throughout the remainder of the analyses, regardless of whether or not the analyses are performed using a higher fill condition.

Although the overburden effects of the fill will modify the effective stress condition and could change the SPT or CPT results, it is commonly assumed that these effects will be minor.

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can be computed. The ratio of CRR to CSR should be greater than 1.0 to preclude the development of liquefaction. As the ratio drops below 1.0, the potential for liquefaction increases. Even when the ratio of CRR to CSR is as high as 1.5, increases in porewater pressure can occur. The potential consequences of these increases should be considered during design.

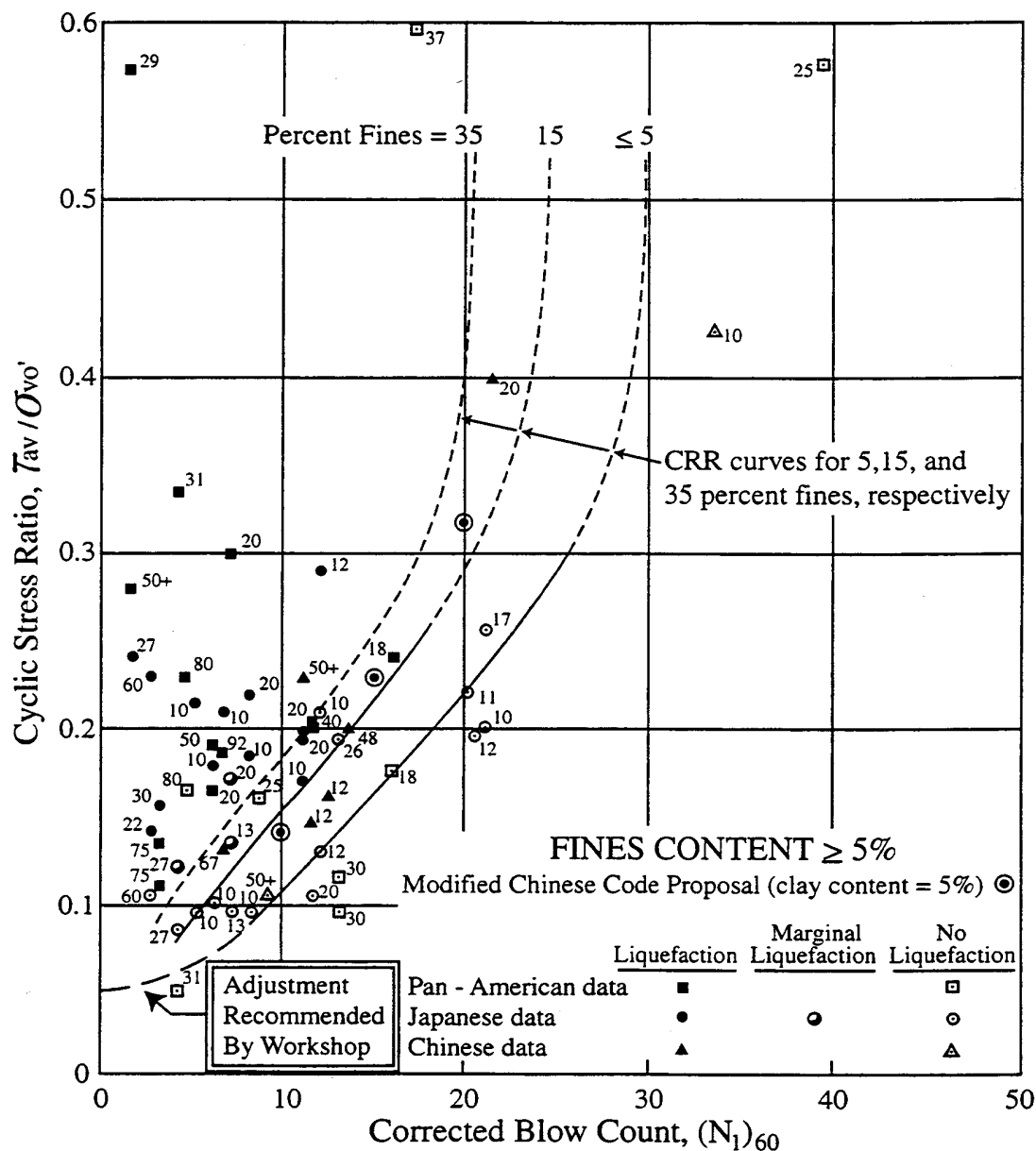


Figure 3B.2.5-1. Simplified Base Curve Recommended for Determination of CRR from SPT Data for Magnitude 7.5 along with Empirical Liquefaction Data (after Youd and Idriss, 1997)

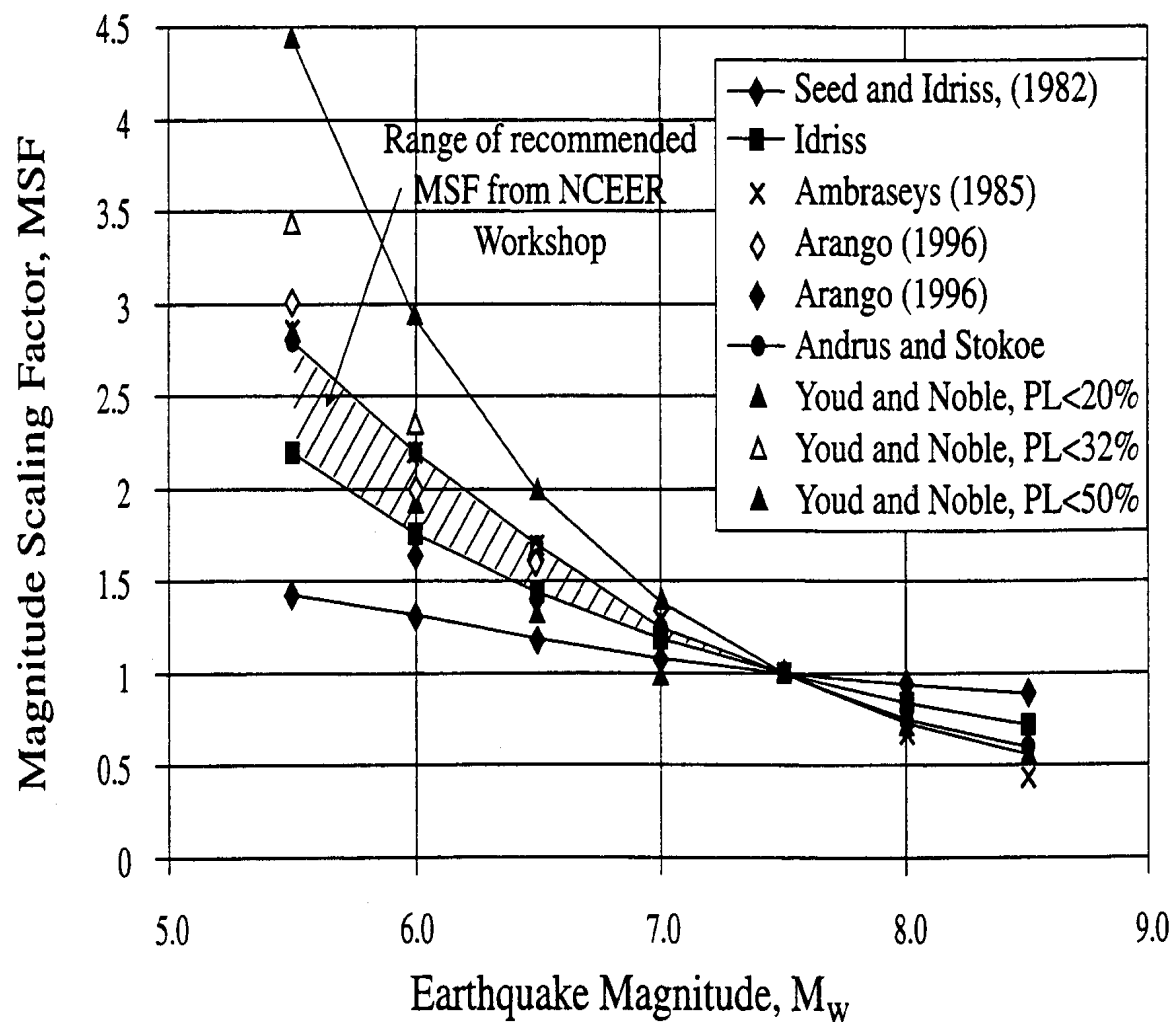


Figure 3B.2.5-2. Magnitude Scaling Factors derived by Various Investigators (after Youd and Idriss, 1997)

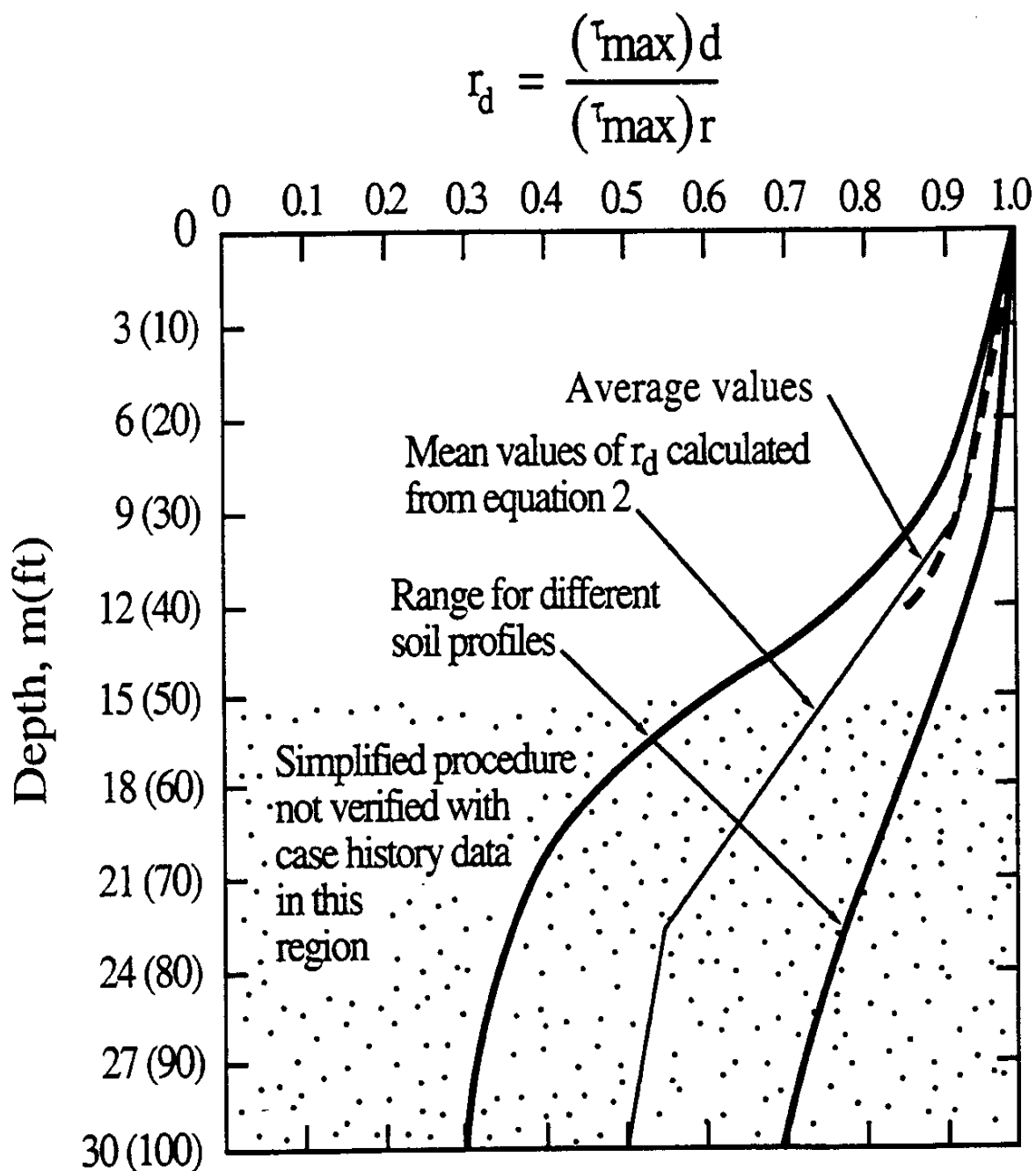


Figure 3B.2.5-3. Soil Flexibility Factor (r_d) versus Depth Curves Developed by Seed and Idriss (1971) with Added Mean Value Lines (after Youd and Idriss, 1997)

3B.2.5.2 Numerical Modeling Methods

For critical projects, the use of equivalent linear or non-linear site specific, one-dimensional ground response analyses may be warranted to assess the liquefaction potential at a site. For these analyses, acceleration time histories representative of the seismic hazard at the site are used to define input ground motions at an appropriate firm-ground interface at depth.

One common approach is to use the equivalent linear total stress computer program SHAKE (Idriss and Sun, 1992) to determine maximum earthquake-induced shearing stresses at depth for use with the simplified procedure described above, in lieu of using the mean values of r_d shown in Figure 3B.2.5-3. Another alternative involves the use of nonlinear, effective stress methods, such as with the computer program DESRA 2 (Lee and Finn, 1978) or DESRA-MUSC (Martin and Qiu, 2000) a modified version of DESRA 2.

3B.2.6 Liquefaction Hazards Assessment

Results of the liquefaction assessment are used to evaluate the potential severity of three liquefaction-related hazards to the bridge:

- Flow failures which involve large translational or rotational slope failures mobilized by existing static stresses (i.e., the site static factor of safety drops below 1.0 due to low strengths of liquefied soil layers).
- Limited lateral spreads which involve a progressive accumulation of deformations during ground shaking with eventual deformations that can range from a fraction of a meter to several meters.
- Ground settlement.

The potential for these hazards can be determined initially on the basis of the factor of safety calculated from the ratio of CRR to CSR. If the ratio is less than 1.0 to 1.3, the hazard should be evaluated following guidelines given below, unless agreed otherwise by the Owner.

C3B.2.5.2

In general, equivalent linear analyses are considered to have reduced reliability as ground shaking levels increase to values greater than about 0.4g in the case of softer soils, or where maximum shearing strain amplitudes exceed 1 to 2 percent. For these cases, true non-linear site response programs should be used, where non-linear shearing stress-shearing strain models (including failure criteria) can replicate the hysteric soil response over the full time history of earthquake loading. The computer program DESRA 2, originally developed by Lee and Finn (1978), was perhaps the first of the widely recognized non-linear, one-dimensional site response program. Since the development of DESRA 2, a number of other non-linear programs have been developed, including MARDES (Chang et al., 1991), D-MOD (Matasovic, 1993) and SUMDES (Li et al., 1992), and DESRA-MUSC (Martin and Qiu, 2000).

C3B.2.6

The factor of safety from the liquefaction analysis can be used to determine if a more detailed evaluation of these hazards is warranted. No single factor of safety value can be cited in a Specification, as considerable judgment is needed in weighing the many factors involved in the decision. A number of those factors are noted below:

- The type of structure and its vulnerability to damage. Structural mitigation solutions may be more economical than ground remediation.
- Levels of risk accepted by the Owner regarding design for life safety, limited structural damage, or essentially no damage.
- Damage potential associated with the particular liquefaction hazards. Flow failures or major lateral spreads pose more damage potential than differential settlement. Hence, factors of safety could be adjusted accordingly.
- Damage potential associated with design earthquake magnitude. A magnitude 7.5 event is potentially far more damaging than a

6.5 event.

- Damage potential associated with SPT values, i.e., low blow counts have a greater cyclic strain potential than higher blow counts.
- Uncertainty in SPT- or CPT- derived liquefaction strengths used for evaluations. Note that a change in silt content from 5 to 15 percent could change a factor of safety from say 1.0 to 1.25.
- For high levels of design ground motion, factors of safety may be indeterminate. For example, if $(N_1)_{60} = 20$, $M = 7.5$ and fines content = 35 percent liquefaction strengths cannot be accurately defined due to the vertical asymptote on the empirical strength curve.

In addition a change in the required factor of safety from 1.0 to 1.25 often only makes minor differences in the extent of liquefiable zones, albeit it would increase the blow count requirements for ground remediation. However, for the example cited, the additional costs of remediation from $(N_1)_{60} = 20$ to $(N_1)_{60} = 25$ say, could be small.

The final choice of an appropriate factor of safety must reflect the particular conditions associated with a specific site and the vulnerability of site-related structures.

3B.2.6.1 Lateral Flows

Flow failures are the most catastrophic form of ground failure that may be triggered when liquefaction occurs. These large translational or rotational flow failures are mobilized by existing static stresses when average shearing stresses on potential failure surfaces exceed the average residual strength developing in the liquefied soil.

To assess the potential for flow failure, the static strength properties of the soil in a liquefied layer is replaced with the residual strength determined from Figure 3B.2.6-1. A conventional slope stability check is then conducted. No seismic coefficient is used during this evaluation, thus representing conditions after the completion of the earthquake. The resulting factor of safety defines the potential for flow failures. If the factor of safety is less than 1.0, lateral flow is predicted.

The estimation of deformation associated with lateral flow cannot be easily made. The deformations

C3B.2.6.1

Valuable commentary on this problem may be found, for example, in publications by NRC (1985), Seed (1987), Seed and Harder, (1990), Dobry (1995), and Kramer (1996). The topic of Post-Liquefaction Shear Strength of Granular Soils was also the subject of an NSF-sponsored NCEER Workshop at the University of Illinois in 1997, a summary of which has been published by Stark et. al. (1998). The complexities of the problem have also been illustrated in centrifuge tests, as described by Arulandan and Zeng (1994) and Fiegel and Kutter (1994).

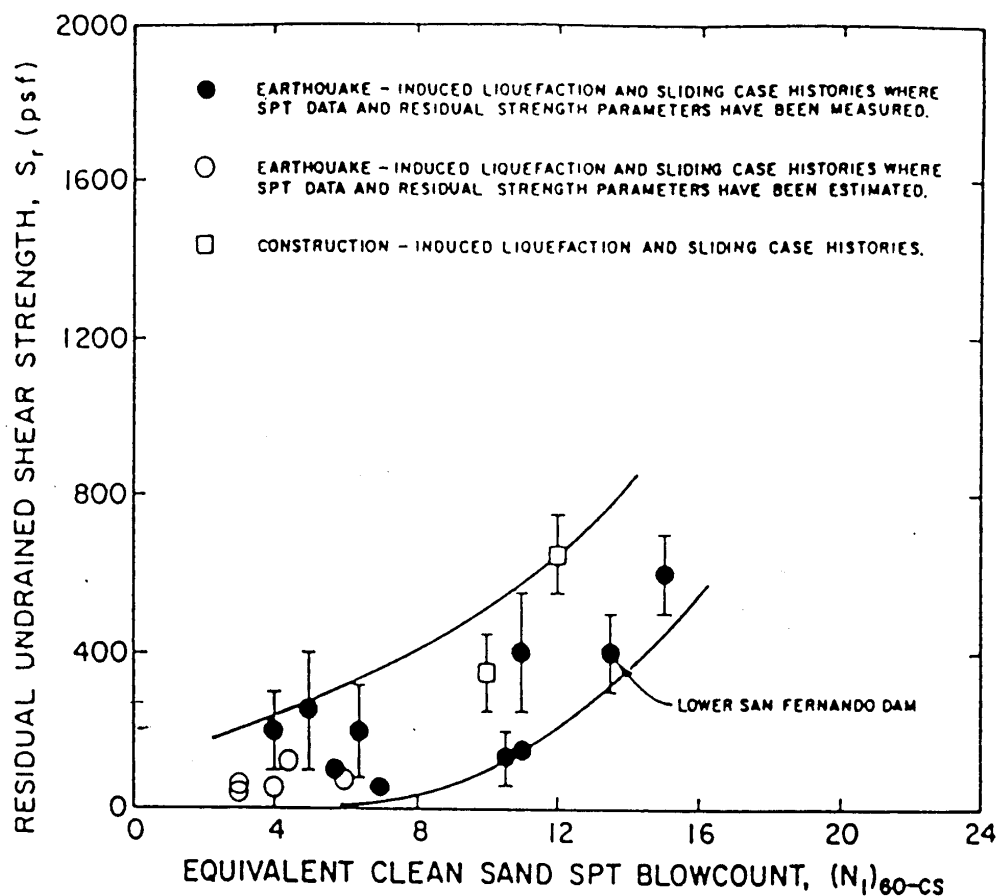
The most difficult step in the flow analysis is the determination of the residual strength of the soil. The most common procedure for evaluating the residual strength involves an empirical correlation between SPT blow counts and apparent residual strength back-calculated from observed flow slides. This relationship is shown in Figure

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can be in excess of several meters, depending on the geometry of the flowing ground and the types and layering of soil. In the absence of reliable methods for predicting deformations, it is usually necessary to assume that the soil will undergo unlimited deformations. If the loads imposed by these movements exceed those that can be tolerated by the structure, some type of ground remediation will likely be required. This situation should be brought to the attention of the Owner and a strategy for dealing with the flow problem agreed upon.

3B.2.6-1. Mean or lower-bound values in the data range shown are often adopted. Some experimental work suggests that residual strength is related to confining pressure (Stark and Mesri, 1992). Steady state undrained shear strength concepts based on laboratory tests have also been used to estimate post liquefaction residual strengths (Poulos et. al., 1985; Kramer, 1996). Due to the difficulties of test interpretation and corrections for sample disturbance, the empirically base correlations are normally used.



RECOMMENDED FINES CORRECTION FOR S_r EVALUATION USING SPT DATA

Percent Fines	N_{corr} (blows/ft)
10%	1
25%	2
50%	4
75%	5

Figure 3B.2.6-1. Relationship between Residual Strength (S_r) and Corrected "Clean Sand" SPT Blowcount $(N_1)_{60}$ from Case Histories (after Seed and Harder, 1990)

3B.2.6.2 Lateral Spreading

The degradation in undrained shearing resistance arising from liquefaction can lead to limited lateral spreads induced by earthquake inertial loading. Such spreads can occur on gently sloping ground or where nearby drainage or stream channels can lead to static shearing stress biases on essentially horizontal ground (Youd, 1995).

Four general approaches can be used to assess the magnitude of the lateral spread hazard:

- **Youd Empirical Approach:** Using regression analyses and a large database of lateral spread case histories from past earthquakes, Bartlett and Youd (1992) developed empirical equations relating lateral-spread displacements to a number of site and source parameters. A refined version of this approach was recently presented by Youd et al. (1999). Generally, this approach should be used only for screening of the potential for lateral spreading, as the uncertainty associated with this method of estimating displacement is generally assumed to be too large for bridge design.
- **Newmark Time History Analyses:** The simplest of the numerical methods is the so called Newmark sliding block analysis, (Newmark, 1965; Kramer, 1996), where deformation is assumed to occur on a well-defined failure plane and the sliding mass is assumed to be a rigid block. This approach requires (1) an initial pseudo-static stability analysis to determine the critical failure surface and associated yield acceleration coefficient (k_y) corresponding to a factor of safety of 1.0, and (2) a design earthquake acceleration record at the base of the sliding mass. Cumulative displacements of the sliding mass generated when accelerations exceed the yield acceleration are computed using computer programs such as described by Houston et al. (1987). These methods are most appropriate when local site effects modify the ground motion as it propagates through the soil profile and when the database for the chart method is not adequate. This latter consideration generally involves sites where the source mechanism will be from a magnitude 8 or higher event.
- **Simplified Newmark Charts:** Charts have been developed by a number of individuals (Franklin and Chang, 1977; Hynes and Franklin, 1984; Wong and Whitman, 1982; and Martin and Qiu, 1994) using large databases of earthquake records and the Newmark Time History Analysis

C3B.2.6.2

The lateral spreading mechanism is a complex process involving the post-liquefaction strength of the soil, coupled with the additional complexities of potential porewater pressure redistribution and the nature of earthquake loading on the sliding mass. At larger cyclic shearing strains, the effects of dilation can also significantly increase post-liquefaction undrained shearing resistance of the liquefied soil. Incremental permanent deformations will still accumulate during portions of the earthquake load cycles when low residual resistance is available. Such low resistance will continue even while large permanent shearing deformations accumulate through a ratcheting effect. These effects have recently been demonstrated in centrifuge tests to study liquefaction-induced lateral spreads, as described by Balakrishnan et al. (1998). Once earthquake loading has ceased, the effects of dilation under static loading can mitigate the potential for a flow slide

The four methods available for estimating deformations from lateral spreading account for this complex process in varying degrees.

The Youd Empirical Approach

The Youd empirical approach uses a variety of earthquake parameters, including magnitude, geometry, and soil grain size in an empirical equation to estimate displacement. Two cases, a sloping ground model and a free-face model, are used. This prediction method is the least reliable in the small displacement range with the level of accuracy probably no better than 1 m. However, it does allow a relatively straightforward screening to be accomplished to identify the potential severity of lateral spreads. Several research projects are also presently in progress to enhance these empirical prediction models by improvements in approaches used in the regression analysis and the use of a larger database.

Newmark Time History Analyses

The Newmark method has been used extensively to study earthquake-induced displacements in dams (e.g., Makdisi and Seed, 1978) and natural slopes (e.g., Jibson, 1993). This approach involves the double integration of earthquake records above the yield acceleration. The yield acceleration (k_y) is determined by finding

method. These charts allow deformations during seismic loading to be estimated using relationships between the acceleration ratio (i.e., ratio of yield acceleration (k_y) to the peak ground acceleration (k_{max}) occurring at the base of the sliding mass) to ground displacement. The Martin and Qiu (1994) charts are recommended in this Appendix, as it included peak ground acceleration and peak ground velocity as additional regression parameters. This method does not include earthquake magnitude. Martin and Qiu note that magnitude was not a statistically significant parameter for the range of magnitudes M6 to M7.5) used in their evaluation.

- **Numerical Modeling:** The most rigorous approach to assessing liquefaction-induced lateral spread or slope deformations entails the use of dynamic finite element / finite difference programs coupled with effective stress based soil constitutive models. However, the use of such programs is normally beyond the scope of routine bridge design projects. Finn (1991; 1998) gives a summary of such approaches, and a recent case history has been described by Elgamel et al. (1998).

The decision between use of the Youd empirical approach and any one of several charts or numerical models will depend on a number of factors, including the level of seismic loading and the consequences of failure. Normally, the Youd empirical approach should be used only for screening of the potential for lateral spreading, as the uncertainty associated with this method of estimating displacements is generally assumed to be large. Although charts and numerical methods offer the capability of estimating displacements more accurately, these methods are often limited by the methods of characterizing the boundary conditions for the problem and on the selection of material properties. Extreme care must be exercised when any of these methods are used.

If lateral spreading is anticipated at a site, the geotechnical engineer should meet with the Owner and decide what approach offers the most appropriate method of estimating the magnitude of lateral spread.

the seismic coefficient that causes the factor of safety in a slope stability assessment to be 1.0. During the stability analyses, the liquefied layer is modeled with the residual strength of the soil. Other layers with partial buildup in porewater pressure can also be degraded in strength during the evaluation.

The earthquake records must be selected from the available catalogue of records, such that they are representative of the source mechanism, magnitude, and distance for the site. A minimum of three records from three independent earthquakes should be selected for the Newmark analyses. Often it is necessary to modify these records for local site effects, as the ground motion propagates through soil to the base of the sliding block.

A number of uncertainties are inherent in the approach due to the assumptions involved. In particular, for liquefaction-induced lateral spreads, uncertainties include:

- The point in the time history when cyclic strength degradation or liquefaction is triggered.
- The magnitude of the apparent post-liquefaction residual resistance as discussed above.
- The influence of the thickness of liquefied soil on displacement.
- Changes in values of yield acceleration (k_y) as deformations accumulate.
- The influence of a non-rigid sliding mass.
- The influence of ground motion incoherence over the length of the sliding mass.

Simplified Newmark Charts

The simplified chart correlations were developed by conducting Newmark analyses on a large number of earthquake records and then statistically analyzing the results. Of the various chart methods, the Martin and Qiu (1994) method is recommended for use on bridge design projects. Figure 3B.2.6-1 and Figure 3B.2.6-2 show the relationships developed by Martin and Qiu (1994). A velocity-to-acceleration ratio of 60 is used if the epicentral distance is less than 15 km; a velocity-to-acceleration ratio of 30 is used for distances greater than 30 km; and values are interpolated

between these distances. These figures are appropriate for magnitudes between 6 and 7.5. If magnitudes exceed 7.5, the deformation should be determined using other methods, such as by conducting Newmark time history analyses or 2-dimensional numerical modeling.

The Franklin and Chang (1977) procedure, which was given in earlier editions of the AASHTO Standard Guidelines, is now thought to overestimate displacements, partly because it was developed by bounding all data and partly because the database had some artificially high records. The Hynes and Franklin (1984) charts used the same database as did Martin and Qiu, and therefore the mean values from the Hynes and Franklin chart are normally similar to the values estimated by the Martin and Qiu method. The Wong and Whitman (1982) provides the smallest estimate of displacements, and appears to be unconservative at times.

To use these charts, the yield acceleration is determined by finding the seismic coefficient that causes the factor of safety in a slope stability assessment to be 1.0. As noted for the Newmark Time History Analyses, the liquefied layer is modeled with the residual strength of the soil. Other layers with partial buildup in porewater pressure can also be degraded in strength during the evaluation. With the yield acceleration and the peak ground acceleration at the base of the failure surface (k_{max}), it is a simple matter to enter the chart and determine the estimated amount of displacement.

These simplified chart methods are limited by the database used in their development. Typically few records greater than magnitude 7.5 were available for analysis, and therefore, use of the methods for larger magnitudes must be done with caution. Other limitations are similar to those presented for the Newmark Time History Analyses.

Numerical Modeling

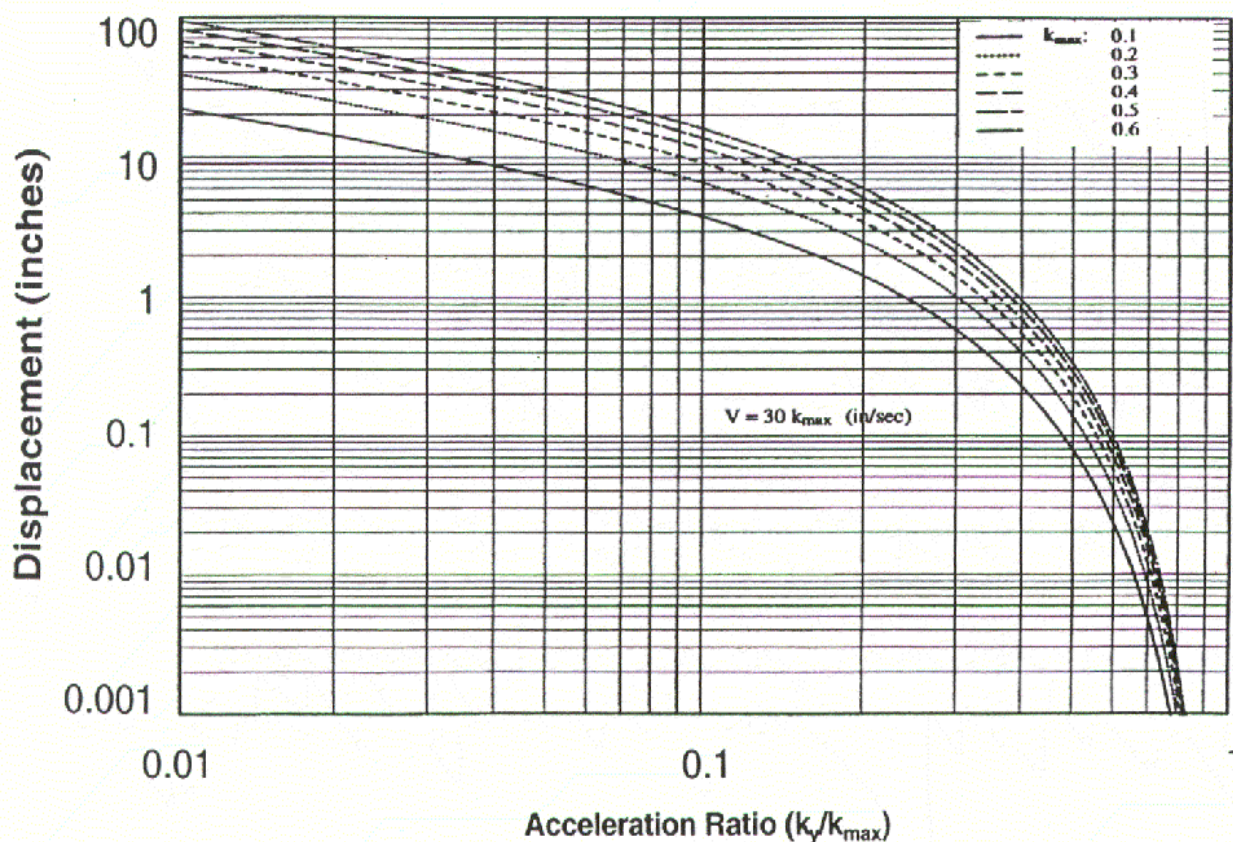
Various two-dimensional, nonlinear computer programs have been used to perform these analyses. For realistic modeling, these programs must be able to account for large displacements, nonlinear soil properties, and changes in effective stress during seismic modeling. One computer program seeing increasing use for this type of modeling is FLAC (Itasca, 1998). This program has been used on a number of bridge-related projects, including the Alaskan Way Viaduct in downtown Seattle, Washington (Kramer et al., 1995).

As with any rigorous modeling method,

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considerable experience and judgment are required when using a program such as FLAC to model soil-pile-structure interaction during earthquake-induced liquefaction. Good practice when using these methods is to compare the results to results of empirically-based simplified methods or to laboratory experimental data, such as produced in the centrifuge.

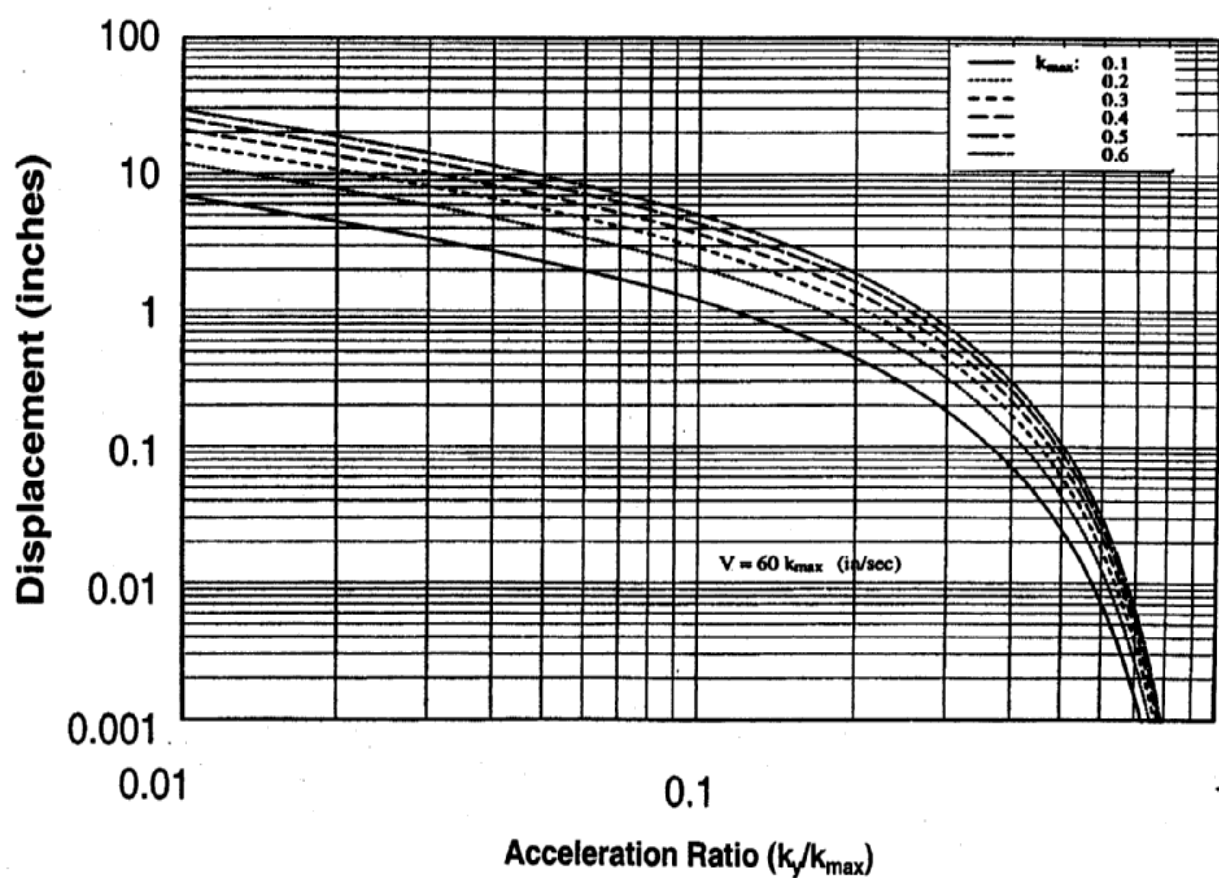


NOTE: DISPLACEMENTS LESS THAN SEVERAL INCHES ARE SHOWN FOR PRESENTATION PURPOSES ONLY. THE ACCURACY OF THE PREDICTIVE METHOD IS SUCH THAT PREDICTED DEFORMATIONS LESS THAN SEVERAL INCHES SHOULD NOT BE USED.

Figure 3B.2.6-2. Martin and Qiu (1994) Simplified Displacement Chart for Velocity-Acceleration Ratio of 30

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NOTE: DISPLACEMENTS LESS THAN SEVERAL INCHES ARE SHOWN FOR PRESENTATION PURPOSES ONLY. THE ACCURACY OF THE PREDICTIVE METHOD IS SUCH THAT PREDICTED DEFORMATIONS LESS THAN SEVERAL INCHES SHOULD NOT BE USED.

Figure 3B.2.6-2. Martin and Qiu (1994) Simplified Displacement Charts for Velocity-Acceleration Ratio of 60

3B.2.6.3. Settlement

Another consequence of liquefaction resulting from an earthquake is the volumetric strain caused by the excess porewater pressures generated in saturated granular soils by the cyclic ground motions. The volumetric strain, in the absence of lateral flow or spreading, results in settlement. Liquefaction-induced settlement could lead to collapse or partial collapse of a structure, especially if there is significant differential settlement between adjacent structural elements. Even without collapse, significant settlement could result in damage.

In addition to the settlement of saturated deposits, the settlement of dry and/or unsaturated granular deposits due to earthquake shaking should also be considered in estimating the total seismically induced settlements.

C3B.6.3

The Tokimatsu and Seed (1987) procedures for both saturated and dry (or unsaturated) sands is the most common of the procedures currently used to estimate the magnitude of settlement. Figure 3B.2.6-3 shows the relationship between the cyclic stress ratio (τ_{av}/σ'_o) and volumetric strain for different values of $(N_1)_{60}$. It should also be noted that the settlement estimates are valid only for level-ground sites that have no potential for lateral spreading. If lateral spreading is likely at a site and is not mitigated, the settlement estimates using the Tokimatsu and Seed method will likely be less than the actual values.

The settlement of silty sand and silt requires adjustments of the cyclic strength for fines content. Ishihara (1993) recommends increasing the cyclic shear strength of the soils if the Plasticity Index (PI) of the fines is greater than 10. This increases the factor of safety against liquefaction and decreases the seismically-induced settlement estimated using the Ishihara and Yoshimine procedure. Field data suggest that the Tokimatsu and Seed procedure without correcting the SPT values for fines content could result in overestimation of seismically-induced settlements (O'Rourke et al., 1991; Egan and Wang, 1991). The use of an appropriate fines-content correction will depend on whether the soil is dry/unsaturated or saturated and if saturated whether it is completely liquefied (i.e., post-liquefaction), on the verge of becoming liquefied (initial liquefaction), or not liquefied. SCEC (1999) suggests that for 15 percent fines, the SPT correction value ranges from 3 to 5 and for 35 percent fines it ranges from 5 to 9.

Although the Tokimatsu and Seed procedure for estimating liquefaction- and seismically-induced settlements in saturated sand is applicable for most level-ground cases, caution is required when using this method for stratified subsurface conditions. Martin et al. (1991) demonstrated that for stratified soil systems, the SPT-based method of liquefaction evaluation outlined by Seed et al. (1983) and Seed et al. (1985) could over-predict (conservative) or under-predict (unconservative) excess porewater pressures developed in a soil layer depending on the location of the soil layer in the stratified system. Given the appropriate boundary conditions, Martin et al. (1991) shows that thin, dense layers of soils could liquefy if sandwiched

between liquefiable layers. For this situation the estimated settlement using the Tokimatsu and Seed procedure (which is based on the SPT values and excess porewater pressures generated in the individual sand layers) therefore, may be over-predicted or under-predicted.

The Tokimatsu and Seed (1987) method can be used to estimate settlement in layered deposits by accounting for settlement of non-liquefiable layers. One approach to estimate the settlement of such a non-liquefiable soil layer is to use Figure 3B.2.6-3 in combination with Figure 3B.2.6-4 to determine if the layer will be affected by the layer below. (i.e.,); If $H_c > H_b$, then the settlement of the nonliquefied layer can be estimated by assuming that the volumetric strain in the layer will be approximately 1.0 percent (1.0 percent seems to be the volumetric strain corresponding to initial liquefaction), given that the non-liquefiable layer (H_b) meets ALL of the following criteria:

- Thickness of the layer is less than or equal to 1.5 m.
- Corrected SPT value $(N_1)_{60}$ less than 30 or CPT tip resistance normalized to 100 kPa (q_{c1N}) less than 160.
- Soil type is sand or silty sand with fines content less than or equal to 35 percent.
- Magnitude of design earthquake is greater than or equal to 7.0.

The logic for using these four criteria is that the migration of porewater pressure into and subsequent settlement of the non-liquefiable layer depends on factors such as the thickness, density (SPT or CPT tip value), and permeability (soil type) of the layer and the duration of earthquake shaking (magnitude). It should be noted that the criteria are only guidelines to allow the Designer to be aware of the potential settlement contributions from certain non-liquefiable soil layers present in a layered system.

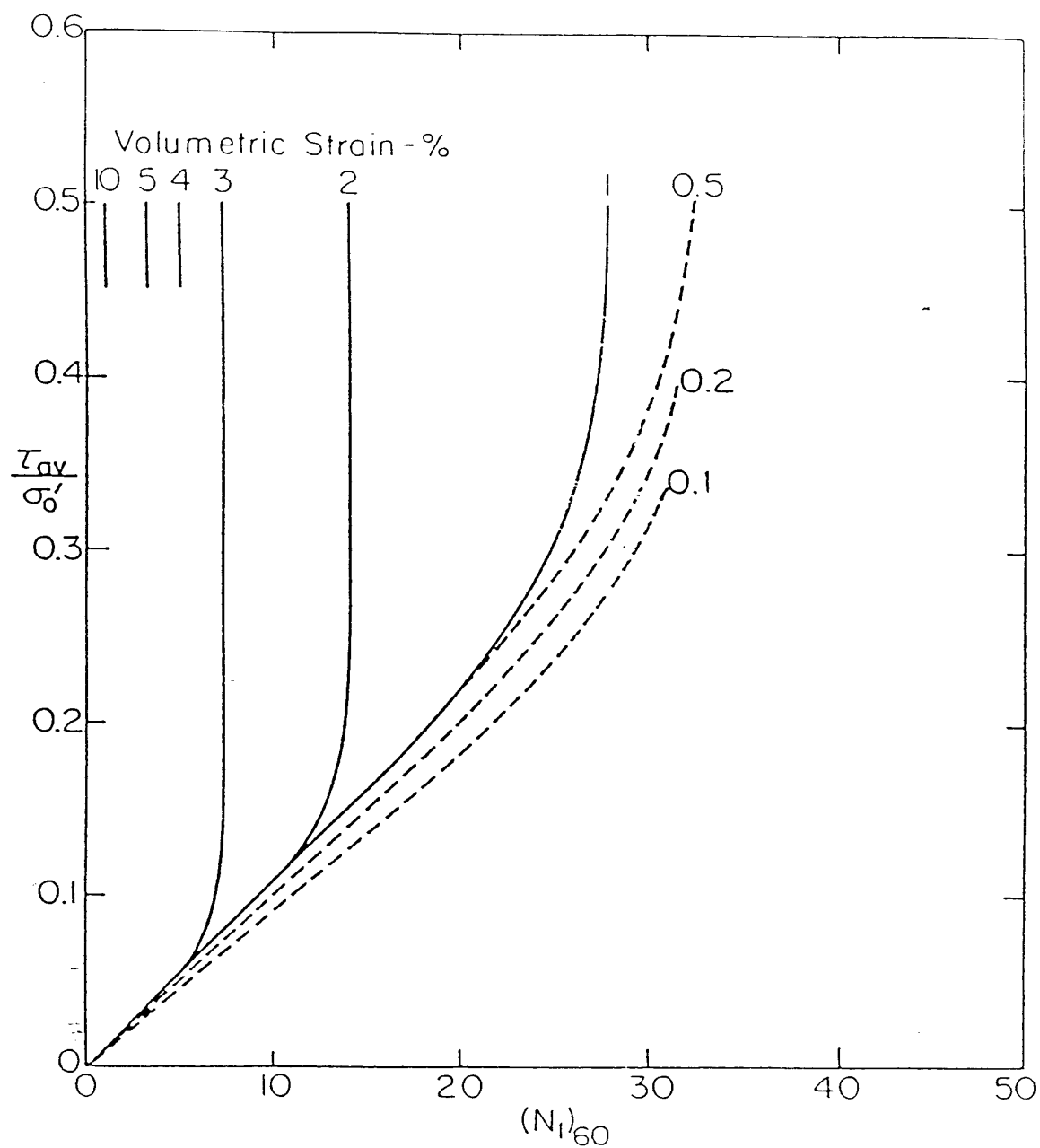


Figure 3B.2.6-3. Relationship Between Cyclic Stress Ratio, $(N_1)_{60}$ and Volumetric Strain for Saturated Clean Sands and Magnitude = 7.5 (after Tokimatsu and Seed, 1987)

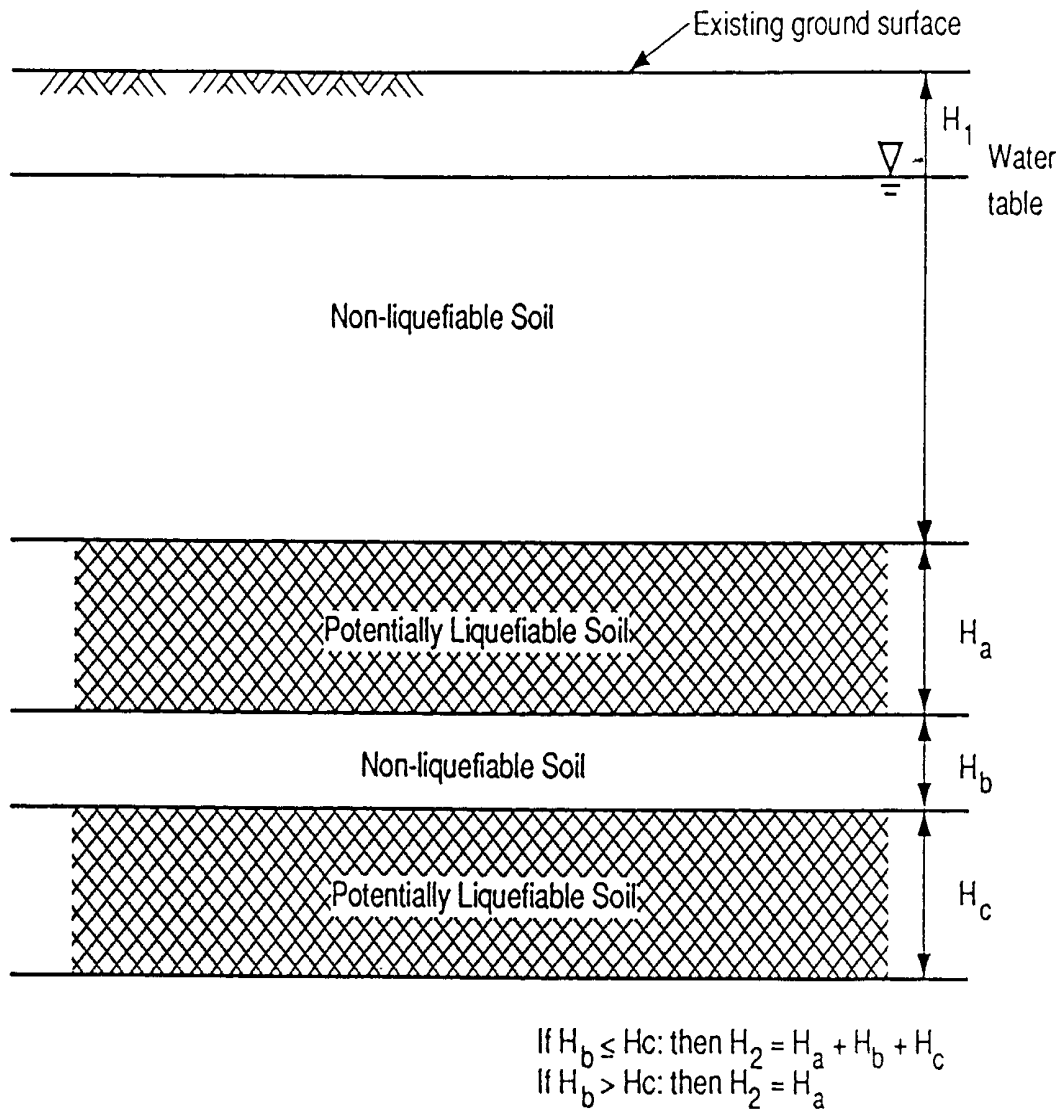


Figure 3B.2.6-4. Schematic Diagram for Determination of H_1 and H_2 Used in Figure 3.10.6-5 (after Ishihara, 1985)

3B.3 Other Collateral Hazards

The potential risk to bridges located in SDR 3 and higher from collateral hazards not associated with liquefaction must also be considered. These other collateral hazards include fault rupture, landsliding, differential compaction, and flooding or inundation.

If the risk of the ground displacement hazard from one or more of these sources is determined to be unacceptable by the Owner for the desired performance level, then the hazard should be mitigated through use of ground improvement methods or by selecting an alternate bridge location.

3B.3.1 Fault Rupture

Ground displacements generally are expected to reoccur along preexisting fault traces. The development of a new fault or reactivation of a very old (pre-Quaternary) fault is uncommon and generally does not need to be considered for typical bridges. Faults are generally considered active and present a potential risk to a bridge if they have displaced in the past 11,000 years. Bridges should not be constructed across active faults, unless specialized studies are performed to quantify the amount of potential fault movement and to determine the consequences of this movement to the bridge.

3B.3.2 Landsliding

Earthquake-induced landsliding represents a significant hazard to roadways in seismically active areas, and can be a hazard to bridges. Damage can be in the form of ground movement either at the abutment or extending to the central piers of a bridge. Sites that are most susceptible to earthquake-induced landslides include locations with slopes of 18 degrees or greater, or a history of rock falls, avalanches, or debris torrents. -

C3B.3

With the exception of flooding and inundation, these other collateral hazards involve ground displacements. These ground displacement hazards can sometimes be very large, on the order of meters, and quantification of the amount of displacement can be difficult. Detailed geotechnical explorations and analyses are usually required to identify the potential for and the consequences of these displacement hazards.

C3B.3.1

To evaluate the potential hazards of surface fault rupture, a number of evaluations are necessary, including determination of the location of fault traces, the nature and amount of near-surface deformations, and the history of deformations. Maps showing the location of active faults have been developed by many state geological agencies and by the United States Geological Survey. The potential amount of movement can be estimated from empirical relationships between magnitude of the seismic event on the fault and displacement (e.g., Wells and Coppersmith, 1994).

The evaluation of fault displacement involves skills and techniques not commonly used in geotechnical or geologic investigations, and therefore should be done by an individual or organization with specific expertise in making these estimates. The Owner must consider the uncertainty in these estimates and the consequences of incorrect estimates when deciding whether to locate a bridge across a fault.

C3B.3.2

Pseudo-static stability methods are often used to evaluate the potential for landsliding at soil sites (in the absence of liquefaction). These methods involve conducting slope stability analyses using a seismic coefficient equal to two-thirds to one-half the predicted peak ground acceleration. Conditions are normally considered acceptable if the computed factor of safety under the imposed loads is 1.0 or higher. If the factor of safety is less than 1.0, a sliding block analysis

using the Newmark (1965) method, as discussed in Article 3B.2.6..2, is conducted to estimate the magnitude of displacement during the landslide. A detailed discussion of seismic-induced landslides is presented in MCEER (2000).

Where cliffs or steep slopes occur, earthquake-induced rock fall hazards may exist. The Colorado Rock Fall Simulation Program (Pfeiffer and Higgins, 1991) can be used to evaluate the potential danger from this mechanism.

Numerous more rigorous two and three-dimensional computer methods, which model the nonlinear response of the soil or rock, can be used to investigate the potential for landsliding, pending the Owner's approval. In some cases these more rigorous methods may be the only reasonable method for making the evaluation.

3B.3.3 Differential Compaction

Loose cohesionless soil above the water table will tend to densify during the period of earthquake ground shaking. This potential should be considered when evaluating the potential for differential displacement between the bridge abutment and the closest central pier or between central piers in a multiple bridge.

3B.3.4 Flooding or Inundation

Tsunamis and seiches can be triggered by earthquakes, causing wave impact and inundation. Failure of reservoirs or aqueducts, and canals located upslope of the bridge can also result in flooding. With the exception of coastal areas in western United States, the risk associated with these mechanisms is low for most bridge sites.

3B.4 Designing for Collateral Hazards

Collateral hazards discussion described in the previous paragraphs of this Appendix identify methods for quantifying the occurrence of collateral hazards. In most cases it is also possible to quantify the amount of displacement associated with the hazards. These estimates are normally made assuming free-field conditions, and therefore don't consider the effects on or from a bridge structure located on the hazards. In some cases the foundations of the structure will either limit or prevent the amount of predicted displacement.

C3B.3.3

Procedures describe by Tokimatsu and Seed (1987) can be used to estimate the amount of settlement. The Tokimatsu and Seed procedure for estimating seismically-induced settlements in dry (and unsaturated) sand requires that the settlement estimates be multiplied by a factor of 2.0 to account for the effect of multidirectional shaking, as discussed by Tokimatsu and Seed (1987).

C3B.3.4

For some performance levels in SDR 3, 4, 5, and 6, it may be desirable to confirm that flooding and inundation will not jeopardize the bridge. Maps have been developed for some areas, such as the west coast of the United States, showing areas where tsunamis danger exists. Most states also have identified possible areas of inundation from failure of reservoirs.

C3B.4

The occurrence of a collateral hazards is normally determined by an engineering geologist and a geotechnical engineer. Often results are presented in terms of a factor of safety or an estimated amount of deformation. The bridge designer is then left with the decision on how this information should be used in the selection and design of the bridge foundation system. Too often, little communication occurs between the geotechnical engineer/geologist and the bridge designer regarding the uncertainties and

Procedures for evaluating the effects of soil movement are summarized in the following paragraphs. Additional requirements for foundations and abutments are presented in Sections 10 and 11, respectively, of the Specifications.

3B.4.1 Spread Footing Foundations

Spread footing foundations located above liquefiable layers must consider the potential for loss in bearing support and for liquefaction-induced settlement if liquefaction is predicted below the foundation. Either of these occurrences can result in displacements of the bridge support system that lead to damage of the structure.

3B.4.1.1 Loss of Bearing Support for Spread Footings

Liquefaction can cause the loss of bearing capacity beneath spread footing foundations supported on “stable” strata above the liquefiable soils. In view of the possible loss in support, spread footing foundations for bridge structures are not recommended above liquefiable soil layers, except in SDR 1 and SDR 2. For SDR 3 and above the liquefiable layer should be at least two foundation widths below the bottom of the footing. At this depth the induced vertical stress in the soil from the footing is less than 10 percent of the bearing pressure imposed at the base of the foundation. Even with the low overburden stress increase, the potential for settlement should be determined.

Spread footing foundations typically should not be used when lateral spreading or flow failures that would load the foundations are predicted. In most cases the spread footing will move with the soil, resulting in excessive bending and possible collapse of the column supported by the footing.

implications associated with the prediction and quantification of the hazard. This approach to seismic design is poor practice in general, and potentially incorrect practice in the area of seismic hazards design. The best and most efficient design for handling the collateral seismic hazards described above will be achieved only if the geotechnical and bridge engineers work as a team.

C3B.4.1

The state-of-the-practice for predicting the consequences of liquefaction, whether it is loss in bearing support or settlement, is one of the least precise of the predictions made by geotechnical engineers. This imprecision reflects the complexity of the overall liquefaction mechanisms and the uncertainties on how these will affect a spread footing foundation. For this reason spread footing foundations are normally discouraged if liquefaction is predicted below the footing.

If liquefaction is predicted to occur below a planned spread footing foundation, this potential should be brought to the attention of the Owner, and a decision made as to the appropriateness of the spread footing foundation in this particular situation.

C3B.4.1.1

Spread footings supporting bridge structures should not normally be used above layers that will liquefy in SDR 3, 4, 5, and 6 because of the potential for loss in bearing capacity and post-earthquake settlement as porewater pressures dissipate. As bearing pressure is lost the foundation will displace downward, likely resulting in differential settlement between column supports. While numerical methods can be used to predict the amount of settlement, the accuracy of the numerical prediction is not usually sufficient to make accurate estimates of distortion between columns. At least part of the difficulty in making these predictions, either numerically or by simple methods, is the inherent variability of soils.

For non-critical spread footing foundations, it is possible to design the footing for the occurrence of liquefaction. For these situations, Ishihara's method of analysis (Ishihara, 1993) for surface manifestation can be used for shallow footings, using the elevation of the bottom of the

footing as the top of the surface layer. If Ishihara's criteria cannot be met, consideration should be given to alternative mitigation methods. In the event that an explicit bearing capacity analysis is performed, the undrained residual strength of liquefied layers can be used in assessing the bearing capacity.

If spread footing foundations must be used above liquefiable layers, whether it is for an SDR 3 or an SDR 6 site, another alternative to consider is to improve the ground below the footing using stone columns, compaction grouting, or a similar improvement procedure. The area improved should extend a distance from the footprint of the footing such that liquefaction of surrounding soils will not cause loss in bearing capacity for the footing. Mitchell et al. (1998) provide guidance in designing liquefaction mitigation methods.

3B.4.1.2 Settlement of Spread Footing

Settlement of spread footings located above loose granular soils should be quantified using the procedures identified in Articles 3B.2.6.2.1 and 3B.3.3. These evaluations should be made whenever liquefaction is predicted to occur below the footing or, in the case of dry or unsaturated soils that are expected to liquefy, if the $(N_1)_{60}$ value is less than 30.

Where there are relatively uniform conditions at a site with deep sediments (if demonstrated by the field program), minimum differential settlement of less than one-half of the total settlement may be used in the design. When the subsurface condition varies significantly in lateral directions and/or the thickness of soil deposit (Holocene deposits and artificial fills) varies within the site, a minimum value of one-half to two-thirds of the total settlement is suggested. Once again, it should be noted that the settlement and differential settlement estimates are valid only for level-ground sites that have no potential for lateral spread. If lateral spreading is likely at a site and is not mitigated, the differential settlements could be much greater than the above-suggested values.

3B.4.2 Deep Foundations

Deep foundations extending through liquefiable soils will require special considerations. The lateral capacities of piles or drilled shafts may be reduced if the surrounding soils liquefy. Lateral spreading or flow slides can also result in the imposition of significant additional lateral demands on the deep

C3B.4.1.2

The differential settlement between adjacent columns, or distortion, is a more useful parameter for the structural designers than the differential settlement estimate. However, a more detailed (and therefore, more expensive) site investigation may be required for making good estimates of site-specific settlements. Therefore, it is suggested that the differential settlement estimates for the site be used as representative of the minimum differential settlement between adjacent supports, unless a more detailed site investigation is performed to obtain specific estimates.

C3B.4.2

If the effects of liquefaction cannot be adequately accommodated in deep foundation design, consideration should be given to alternative mitigation methods. Liquefaction effects on deep foundations can be mitigated by the implementation of ground improvement

foundations. Liquefaction also can result in settlement of the liquefied strata and the strata above the liquefied strata. This settlement will cause downdrag or negative friction to be imposed on the deep foundations. The potential for these must be addressed for bridges located in SDR 3, 4, 5, and 6.

3B.4.2.1 Loss in Lateral Support for Deep Foundations

Although a well-designed pile foundation should extend beyond the deepest depth of liquefaction, liquefaction of a layer above the toe of the pile can result in loss of lateral support of the pile. This can reduce the stiffness of the soil-pile system if the loss in lateral support occurs within 10 pile diameters of the bottom of the pile cap or the ground surface. The effects of this loss should be quantified in accordance with procedures given in Section 10 of the Specifications.

3B.4.2.2 Loads from Lateral Spreading/Flow

If lateral flow or spreading of the ground is predicted during a seismic event, piles that would be loaded by the deforming ground need to be designed to withstand the loads from the moving soil. The recommended design approach for evaluating this condition involves the following four steps:

1. Slope stability analyses are conducted to determine the yield acceleration. This step may include the pinning effects of the piles or the increased resistance of soil that has been improved by some type of ground improvement method.
2. Newmark sliding block analyses are performed to estimate displacements of the soil-pile system.

techniques prior to, or after deep foundation installation.

C3B.4.2.1

The change in stiffness of a pile extending through liquefied soil can be determined by conducting a lateral pile analysis using a beam-column-type computer software. Common examples of these software are LPILE+ and COM624. These programs allow modeling of individual layers within the soil profile. Liquefied layers are assigned a residual strength and treated as a cohesive soil. The strain necessary to mobilize 50 percent of ultimate resistance (ϵ_{50}) is assumed to be 0.02.

If a cohesionless layer does not liquefy but the factor of safety against liquefaction is less than 1.5, a reduced soil friction angle and a reduced subgrade modulus should be used. It is suggested that the reduced friction angle be taken as 10 degrees for FS of 1.0 and should be interpolated for FS between 1.0 and 1.5. Modulus of subgrade reaction values are reduced in a similar manner with the modulus at FS of 1.0 equal to the modulus of a soft clay.

C3B.4.2.2

A flowchart of the proposed methodology for evaluating spreading is given in Figure 3B.4.2-1. Key components of this methodology are numbered in the flowchart, and this chart along with the following commentary provide a 'roadmap' to the recommended procedure for lateral spreading resistance design. The primary feature of the proposed methodology is the use of passive piles to restrict the movement of soil and foundations to levels that are tolerable by the structure.

- Step 1: The soil layers that are likely to liquefy are identified.
- Step 2: A stability analysis is conducted to determine the likelihood of soil movements,

3. The passive force that can ultimately develop ahead of a pile or foundation as soil movement occurs is estimated, and
4. The likely plastic mechanisms that may develop in the foundations and substructure are evaluated.

The rationale behind the proposed method is to determine the likely magnitude of lateral soil movement and assess the ability of the structure to both accommodate this movement and/or potentially limit the movement.

The concept of considering a plastic mechanism in the foundation under the action of spreading forces is tantamount to accepting substantial damage in the foundation. This is a departure from seismic design for vibration alone, and the departure is felt reasonable because it is unlikely that the formation of a mechanism in the foundation will lead to structure collapse. The reasoning behind this is that lateral spreading is essentially a displacement-controlled process. Thus the estimated soil displacements represent a limit on the structure displacement, excluding the phenomena of buckling of the piles or shafts below grade and the continued displacement that could be produced by large $P-\Delta$ effects. Buckling should be checked, and methods that include the soil residual resistance should be used. Meyersohn, et al. (1992) provide a method for checking buckling as an example. The effects of $P-\Delta$ amplification are discussed later in this section.

and to determine the extent of such movements. This would include the depths of soil likely to move and the plan extent of the likely soil failure block. Assessment of the impacts to a bridge structure can then be made by considering the proximity of the failure block to the foundation system.

- Step 3: The maximum displacement of the soil is estimated. This can be accomplished using the simplified Newmark charts or the Newmark Time History Analysis described in Article 3B.2.6.2. The Designer is permitted to apply more advanced techniques if the benefits justify the additional engineering costs and with the concurrence of the Owner. In some cases, substantial improvements and reduction in overall estimated displacements can be achieved.
- Step 4: An assessment is made whether soil flows around the foundation or movement of the foundation will occur. The assessment requires a comparison between the estimated passive soil forces that can be exerted on the foundation system and the ultimate structural resistance that can be developed by the structure, itself. This assessment requires estimating the forces that can develop if soil is to actually flow around the foundation system and comparing them with the likely resistance the structure will provide. In cases where a crust of non-liquefied material exists at or near the ground surface, the full structural resistance is likely to be less than the flow-induced passive forces, and in such cases the foundation is likely to move with the soil. In many cases, it may be immediately obvious which condition, soil flow or foundation movement, is more likely. Qualitative illustrations of the two scenarios are given in Figure 3B.4-2 and Figure 3B.4-3.
- Step 5: If flow of soil around the structure is indicated, then the foundation is designed to withstand the passive pressures created by the soil flowing around the structure. The induced forces are effectively the largest forces that the structure will experience, and for this reason it is conservative to design a structure for such forces.
- Step 6: If on the other hand, the assessment indicates that movement of the

foundation is likely, then the structure must be evaluated for adequacy at the maximum expected displacement. This check is shown in Step 6. The implication of this assessment is that for relatively large ground movements, soil displacements are likely to induce similar magnitude movements of the foundation. In this context, “large” is taken relative to the structural yield resistance. The resulting induced movements of the foundations may produce substantial plasticity in the foundations, and may induce relatively large reactions in the superstructure. Guidelines for the acceptable rotation are provided in the Article 5.16 of the Specifications. For an upper level event, the recommended acceptance criterion is a plastic rotation of 0.05 radians. The allowance of plasticity in the foundation is believed to be reasonable, even though plasticity may occur below grade, because damage in the foundation is not likely to pose a collapse hazard.

- Step 7: If deformations are not acceptable, there are realistically only two ways to restrict the foundation and substructure forces to acceptable values. The first method is to design the foundations to resist the forces that would accompany passive flow of the soil around the foundations. The other method would be to limit the ground movement by providing either ground and/or structural remediation. It is the structural option that provides the simplest first option, and this makes use of the “pinning” or dowel action that pile or shaft foundations contribute as they cross the potential failure plane of the moving soil mass.
- Step 8: The determination of the plastic mechanism that is likely to occur in the presence of spreading should be done in a reasonable manner. Due to the range of inherent uncertainties, great precision in the determination may not produce more accuracy. Thus a simple estimate of the mechanism and its corresponding lateral resistance capability is often adequate. For instance, one method is to use the upper bound method of plasticity and postulate potential mechanisms, then using judgment assess the mechanism that is likely to

control. The acceptance criteria are basically the structural deformation criteria for SDAP E, which uses the push-over-method. In fact, the piles are the elements that limit the acceptable displacements of the system.

The lateral shear that produces the plastic mechanism can be adjusted downward to account for the driving effect of the P- Δ effect. The lateral soil force that produces a plastic mechanism in the foundation/substructure system is required; therefore, the reduction in shear required to produce a mechanism due to P- Δ should be considered. Figure 3B.4-4 and Figure 3B.4-5 illustrate a first-order correction for P- Δ effects for a stub abutment and for an intermediate pier with piles and pile cap.

A more precise method of determining the plastic mechanism would be to use an approach that ensures compatibility of deformations between the soil and piles (e.g., similar to LPILE) and which accounts for plastic deformations in the piles themselves. This second requirement could be satisfied by using software that is capable of performing push-over-analysis, then using p-y curves from a program such as LPILE to produce boundary support elements that ensure compatibility.

- Step 9: The system then must be assessed for a prescribed displacement field to represent the likely soil spreading deformation. From this analysis, an estimate of the likely shear resistance the foundation will provide is estimated and this shear can then be incorporated back into the stability analysis.
- Step 10: If substantial resistance is provided, then its effect on limiting the instability driven movement of the soil block should be introduced into the stability analysis. This step is typically not included in current assessments of potential foundation movements, although inclusion of this resistance could improve the expected performance of the structure.
- Step 11 and 12: The overall displacement

is re-calculated with the revised resistance levels considered. Once a realistic displacement is calculated, then the foundation and structural system can be assessed for this movement. It is at this point that more permissive displacements than for substructure design can be relied upon. This implies that plastic rotations, and potentially large ones, may be allowed to occur in the foundation under such conditions.

- **Step 13:** If the behavior of the structure is acceptable then the design is complete; if not, then the Designer must assess whether to try to produce adequacy either through additional piles or shafts, and these may not need to connect to the foundation (passive piles). Alternately ground improvement approaches may be considered, for instance stone columns. The selection of structural or geotechnical remediation methods is based on the relative economy of the system being used.

The process is repeated by returning to Step 8 and modifying the available resistance until the slope is stabilized. The fact that inelastic deformations may occur below grade during the upper level seismic event and that these may be difficult to detect and inspect should be considered. However, typically the presence of large ground movements induced by earthquake motions is discernible. Thus it should be possible to postulate whether inelastic deformations have occurred from the post-earthquake inspection information. Additionally, inclinometer tubes could be installed in selected elements of deep foundations to allow quantitative assessment of pile/shaft movement following an earthquake.

3B.4.2.3 Settlement and Downdrag

Deep foundations should also be designed for settlement that occurs during the seismic event. The settlement can be estimated based on settlement below the neutral plane of the pile. Procedures given in Section 10 can be used to estimate the location of the neutral plane. The Tokimatsu and Seed (1987) method described in Article 3B.2.6.3 can be used to estimate the settlement.

Drag loads will be imposed on a pile as liquefied

C3B.4.2.3

The drag load will develop along the side of the pile from settlement of all layers above the bottom of the liquefied layer. The drag load in non-liquefied layers will be the same as the ultimate side resistance developed under compressive loading. The drag load along the portion of the pile that is in liquefied soil will initially be the residual strength of the liquefied soil, but then increase gradually as porewater pressures dissipate. For design purposes it is

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layers settle. These loads should be used to estimate the total settlement of the pile (i.e., added to the settlement estimated by the Tokimatsu and Seed (1987) method, as the structural capacity of the pile under the drag loads.

conservative to assume that maximum drag occurs at the end of porewater pressure dissipation, when the soil strength has returned to its initial condition.

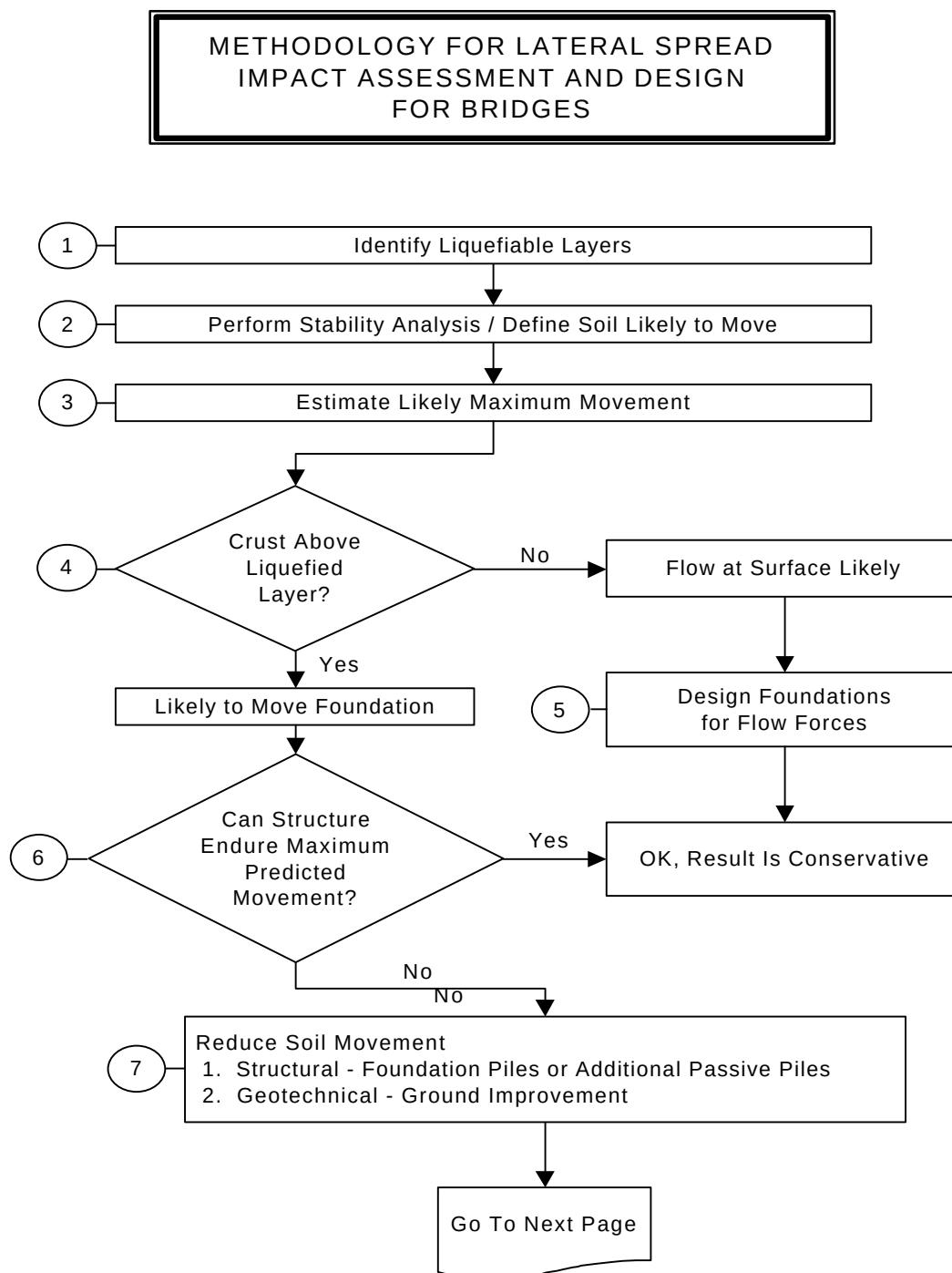


Figure 3B.4.2-1. Flowchart Showing Process for Evaluating the Effects of Lateral Spread and Flows on a Bridge Foundation

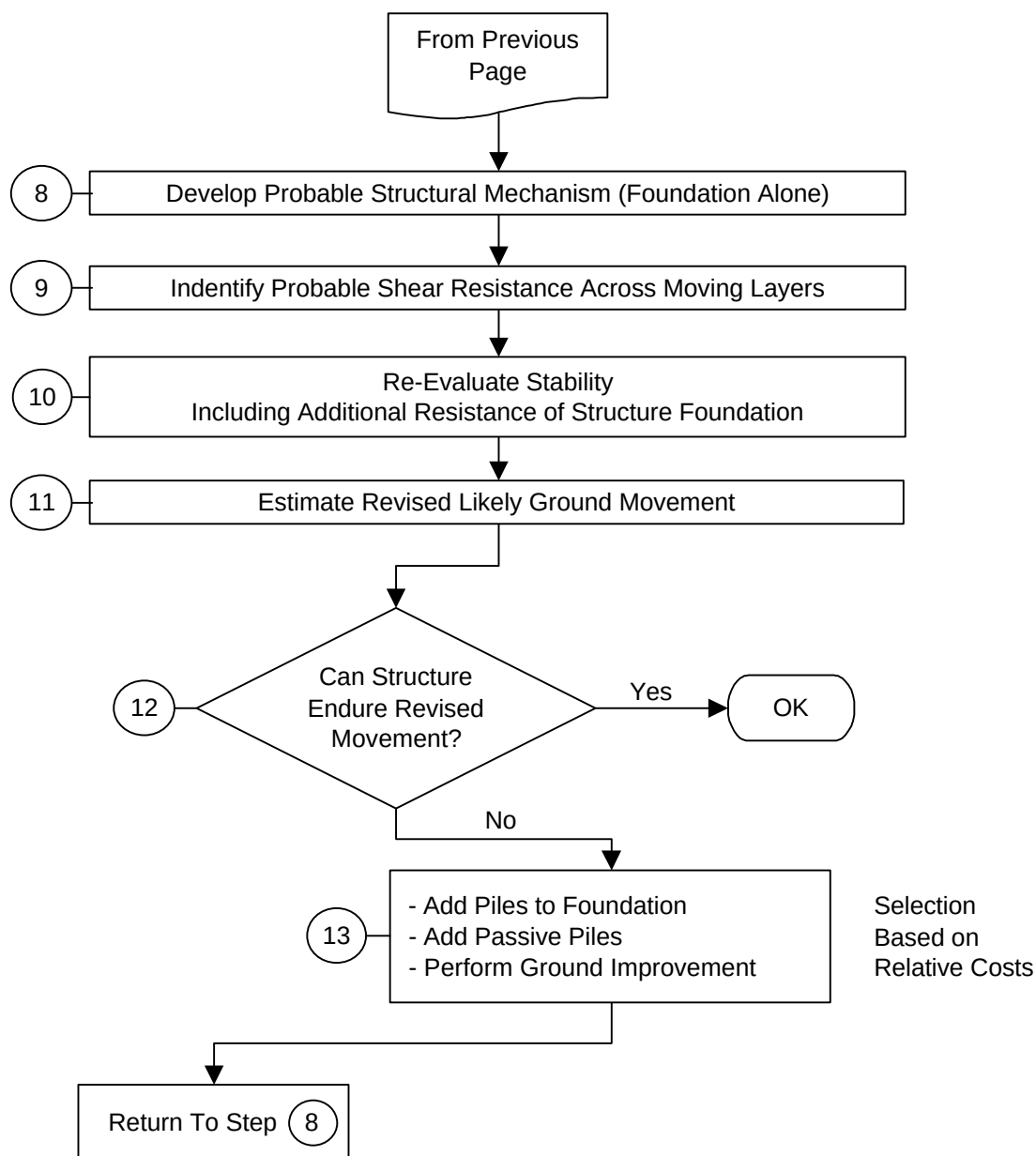


Figure 3B.4.2-1. Flowchart Showing Process for Evaluating the Effects of Lateral Spread and Flows on a Bridge Foundation (cont.)

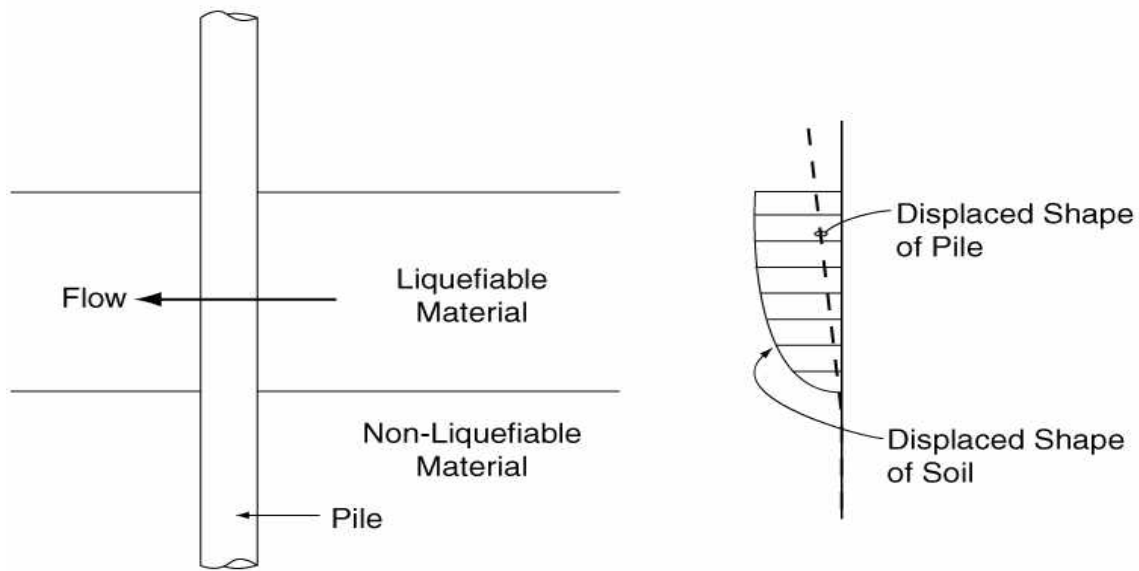


Figure 3B.4-2. Flow of Liquefied Soil Past Pile

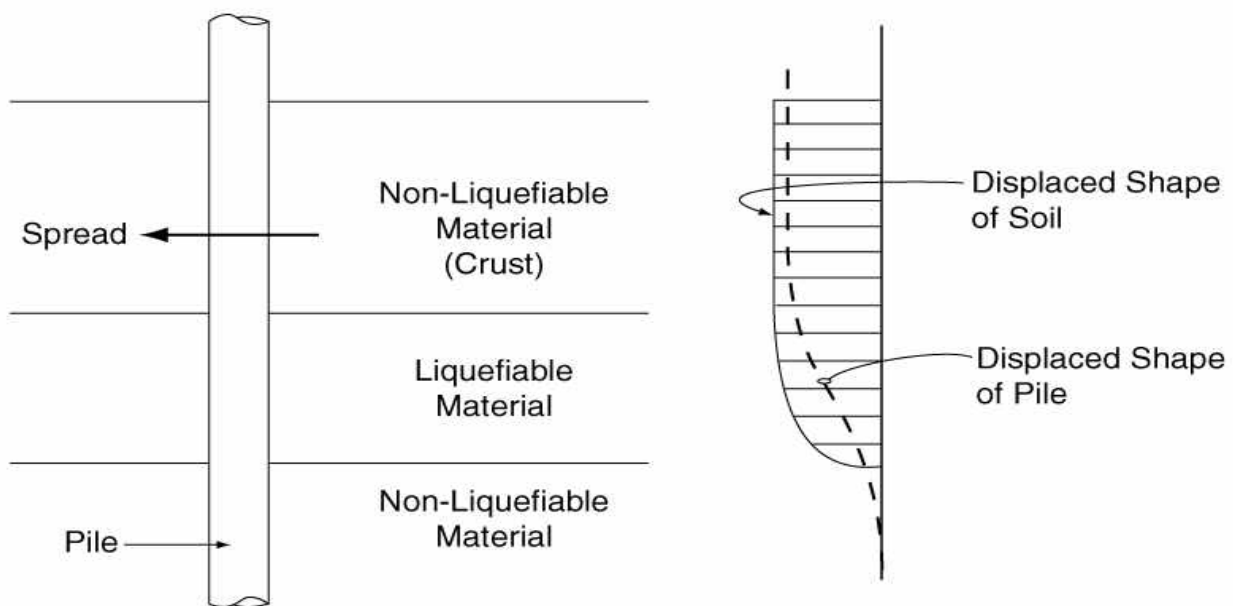


Figure 3B.4-3. Flow of Liquefied Soil with Crust Past Pile

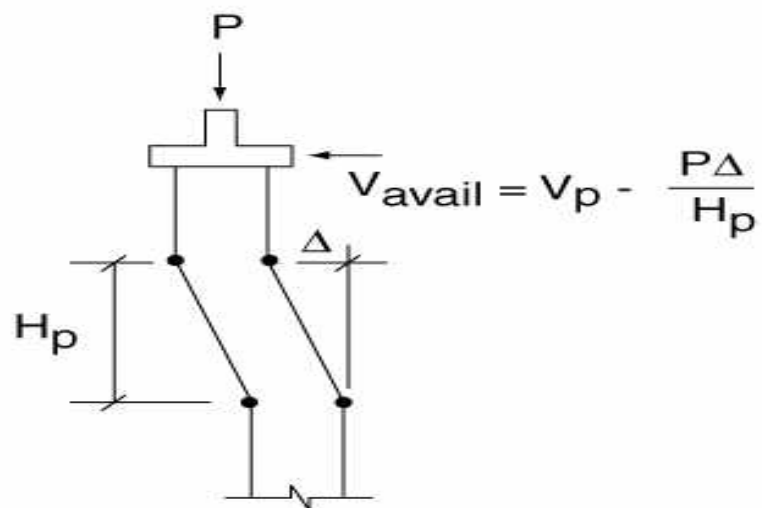


Figure 3B.4-4. P- Δ Effects to Stub Abutment

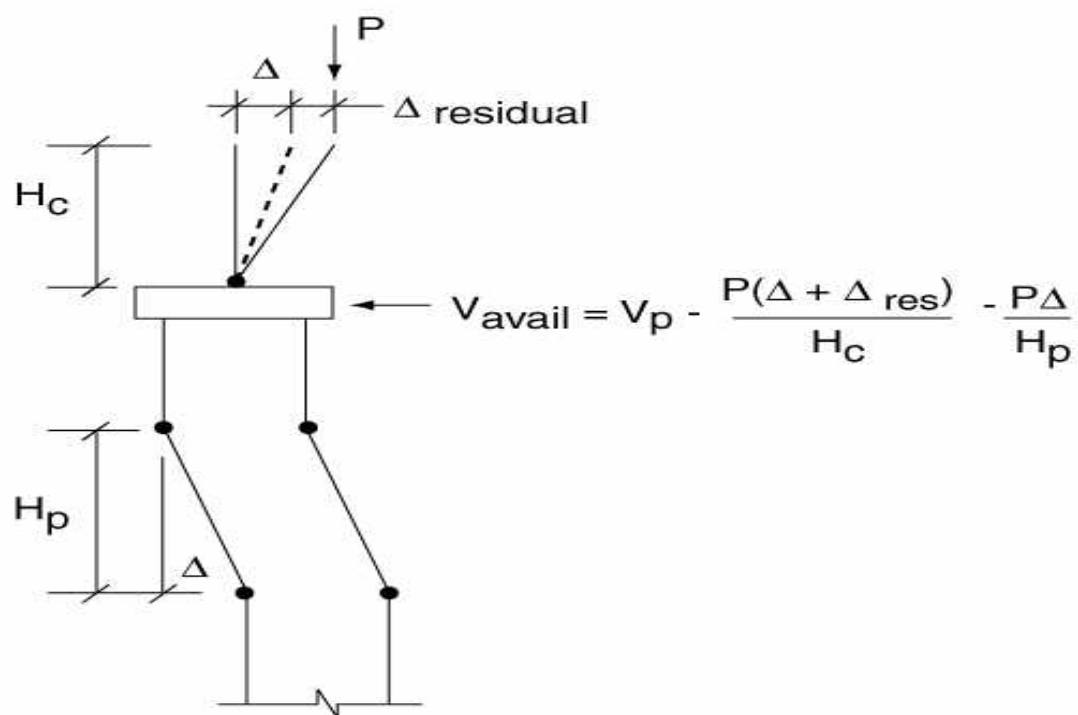


Figure 3B.4-5. P- Δ Effects for an Intermediate Pier with Piles and Pile Cap

3B.4.3 Ground Improvement

Ground improvement methods can be implemented to mitigate the effects of liquefaction. A number of these methods are available, including grouting (compaction, permeation, and jet), vibro systems (vibratory probe, vibro-compaction, vibro-replacement), surcharge and buttress fills, reinforcement and containment (root piles, mixed-in-place walls and columns) and drains. Cooke and Mitchell (1999) provide detailed guidelines for liquefaction of bridge sites. The suitability of these methods will depend on the soil conditions at the site, the location of the ground water, and project logistics.

A critical phase in any ground improvement method is confirmation that the ground improvement goals have been achieved. Pre- and post field explorations are required using SPT or CPT methods to confirm that required ground improvements have been achieved. In many cases it will be desirable to conduct a test program using before the actual ground improvement program to confirm that the proposed improvement methods will work in the particularly conditions occurring at the project site.

C3B.4.3

Two of the more common procedures for accomplishing this remediation are described below:

- Vibro-Replacement: The most widely used densification method is the vibro-replacement technique. This method involves the repeated insertion and withdrawal of a large vibrating probe in the soil, to the desired depth of densification. As vibration-induced liquefaction occurs, crushed stone backfill is placed around the vibrator leading to the development of a stone column approximately 1 m in diameter. The stone column provides for an increased effectiveness of vibration transmission, and facilitates drainage of excess pore water pressures as densification occurs. The procedure is repeated at grid spacing of 7 to 12 feet. Relative densities of the order of 80 percent, can be accomplished by the method. The method has been shown to be effective if sands to be densified contain less than 15 to 20 percent fines, although the use of wick drains placed at the midpoints of stone column grid points to aid drainage, can potentially lead to densification of sandy silts (Luehring et. al., 1998). Details on design information and equipment applications can be found in many publications such as Baez (1995, 1997), Hayden and Baez (1994), and Martin (1998).
- Compaction Grouting: This method involves pumping a stiff mix of soil, cement, and water into the ground under high pressure to compress or densify the soil. For sites where vibratory techniques may be impractical, compaction grouting can be used. Typically, a very stiff (25 to 50 mm slump) soil-cement-water mixture is injected into the soil, forming grout bulbs which displace and potentially densify the surrounding ground, without penetrating the soil pores. A grid or network of grout columns formed by bottom up grouting, results in improved liquefaction resistance over a required areal extent, similar to the use of a network of stone columns described above. An overview of this approach is documented by Boulanger and Hayden (1995).

2B.4.3.1 Bearing Capacity and Settlement

Ground improvement methods can be used to limit settlements of approach fills and improve bearing capacity or lateral capacity of soil that is predicted to liquefy. The amount of improvement is determined by the type and extent of improvement. Cooke and Mitchell (1999) provide guidance on evaluating these improvement methods.

When used to improve the bearing capacity for spread footings or the lateral capacity of piles footings, the ground is usually improved to a level that won't liquefy during the seismic event. However, material beyond the improved zone will likely liquefy. Porewater pressures in the liquefied zone can migrate into the improved area, reducing the capacity of the improved zone. Similarly, loss in strength in the liquefied zone can lead to loss in either vertical or lateral support within the improved ground, due to loss soil reaction in the liquefied zone. This loss in capacity can lead to increased vertical or lateral displacements. The placement of a zone with a radius of 1.5 to 2 times the thickness of the liquefiable layer can be used to eliminate post liquefaction downdrag on a pile, and the potential effects of cyclic ground lurch (progressive unidirectional movement of soil due to high ground accelerations).

The improved ground will also propagate ground motions more effectively than liquefied zone. Site conditions following ground improvement will likely be stiffer than what existed before ground improvement. This increased stiffness should be considered when defining the site category for determining peak ground and spectral accelerations.

These factors must be considered during the design process.

3B.4.3.2 Lateral Spreading and Flow

Ground improvement methods can be used to control or limit the amount of lateral flow or spreading. The approach used in design is to increase the strength of the ground enough that it either causes the liquefied soil to flow around the improved ground or provides sufficient resistance to stop the lateral spread or flow. In most bridge designs the goal will be to prevent movement of the approach fill, either transverse or in line with the bridge alignment. Conventional slope stability methods are used to make these assessments. Initially, the potential for flow failure should be evaluated, with the improved ground characterized by a higher strength. If the resulting factor of safety is greater than 1.0, then either the Newmark Charts or the Newmark Time History Analyses can be conducted to determine the amount of ground deformation. Procedures described in Article 3B.4.2.2 can then be used to evaluate whether the resulting deformations meet design criteria for the bridge structure and foundation.

A Newmark approach can be used to determine the buttress width that leads to acceptable displacement performance of abutment or bridge pier piles in the failure zone. This involves determining the yield acceleration for slope movement through the improved ground, and then using the simplified charts, equations, or integrated earthquake records to revise the displacement procedure. As the width of the improved zone increases, the amount of deformation will decrease. This relationship allows a cost-benefit study to be conducted to determine the minimum area of improved ground (minimum costs) that will result in deformations that can be tolerated by the bridge structure-foundation system.

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SPECIFICATIONS

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4.3 NOTATIONS

B_L	=	capacity spectrum response reduction factor for constant-velocity portion of design response spectrum curve
B_S	=	capacity spectrum response reduction factor for short-period portion of design response spectrum curve
C_S	=	seismic coefficient
C_{sm}	=	seismic coefficient from design response spectrum curve for uniform load method
D	=	effective depth of reinforced concrete column
El_{eff}	=	effective flexural rigidity, including effect of concrete cracking of reinforced concrete members
F	=	equivalent static lateral force for uniform load method
F_a	=	site coefficient for short-period portion of design response spectrum curve
F_v	=	site coefficient for long-period portion of design response spectrum curve
g	=	acceleration due to gravity, 32.2 ft/sec ² or 9.81 m/sec ²
K	=	lateral stiffness of bridge in uniform load method
K_{eff}	=	effective lateral stiffness at design displacement
L	=	length of bridge
M_n	=	nominal flexural strength of member
p_e	=	uniform load on superstructure for uniform load method for design response spectrum curve
p_0	=	unit uniform load on superstructure for uniform load method
S_s	=	0.2-second period spectral acceleration on Class B rock from national ground motion maps
S_1	=	1-second period spectral acceleration on Class B rock from national ground motion maps
T_{eff}	=	effective vibration period at design displacement
T_m	=	vibration period for uniform load method
$v_{s,max}$	=	maximum displacement of bridge under uniform load
W	=	weight of bridge
b	=	damping ratio in percent
e_y	=	yield strain of longitudinal reinforcing steel
Δ	=	displacement of superstructure

SECTION 4 – STRUCTURAL ANALYSIS AND EVALUATION

SPECIFICATIONS

COMMENTARY

4.8 SEISMIC ANALYSIS

4.8.1 General

When seismic analysis is required for Seismic Design and Analysis Procedure (SDAP) C, D, and E, the bridge shall be analyzed using a mathematical model that consider the geometry, boundary conditions, material behavior of the structure. The Engineer should consider the force and deformation effects being quantified and the accuracy required when defining a mathematical model.

A representation of the foundation and soil that supports the bridge may be included in the mathematical model of the foundations depending on the type of foundation, the Seismic Design and Analysis Procedure (SDAP), and the Seismic Detailing Requirement (SDR). When the foundations and abutments are included in the mathematical model, the assumed properties shall be consistent with the expected deformations of the soil.

In the case of seismic design, gross soil movement and liquefaction shall also be considered in the analysis when applicable.

4.8.2 Selection of Seismic Analysis Procedure

For seismic design the choice of the mathematical model and analysis procedure shall be based on the requirements of Article 3.10.3.

Table 3.10.3-2 identifies the Seismic Design and Analysis Procedure. When required, the Seismic Design and Analysis Procedures use the following seismic demand analysis and/or seismic displacement capacity verification procedures in order of increasingly higher-level of ability to represent structural behavior.

C4.8.1

Seismic analysis encompasses a demand analysis and a displacement capacity verification. The objective of a demand analysis is to estimate the forces and displacements induced by the seismic excitation. Depending on the design procedure, a verification of displacement capacity of piers or bents may be required. The objective of a displacement capacity verification is to determine the displacement of an individual pier or at which the deformation capacity of the inelastic earthquake resisting elements is reached. The displacement capacity must be greater than the displacement demand. The accuracy of the demand and capacity analyses depend on the assumption of the model related to the geometry, boundary conditions, material properties, and energy dissipation incorporated in the model. It is the responsibility of the Engineer to assess the reasonableness of a model in representing the behavior of the structure at the level of forces and deformations expected for the seismic excitation.

Very flexible bridges, e.g., suspension and cable-stayed bridges, shall be analyzed accounting for the nonlinear geometry.

The need for modeling of foundations and abutments depends on the sensitivity of the structure to foundation flexibility and associated displacements. This in turns depends on whether the foundation is a spread footing, pile footing with pile cap, a pile bent, or drilled shaft. Article 4.8.4.4 defines the requirements for the foundation modeling in the seismic analysis.

When gross soil movement or liquefaction is determined to be possible, the model shall represent the change in support conditions and additional loads on the substructure associated with soil movement.

For structures whose response is sensitive to the support conditions, such as in a fixed-end arch, the model of the foundation shall account for the conditions present.

C4.8.2

Bridges are designed to remain essentially elastic when subjected to earthquakes with a high-probability of occurrence (50% exceedance in 75 years). For low-probability earthquakes (3% exceedance in 75 years) and depending on the desired performance level, bridges are designed to dissipate energy through inelastic deformation in earthquake resisting elements. Depending of the type of analysis, the demand and capacity may be expressed in terms of forces (bending

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- *Capacity Spectrum Analysis* - Seismic response of a very regular structure is modeled as a single degree-of-freedom system, and the demand analysis and capacity evaluation are combined in a single procedure. The capacity spectrum analysis may be used for seismically isolated bridges.
- *Elastic Response Spectrum Analysis* - Seismic demands are determined by a response spectrum analysis using the spectrum defined in 3.10.2. For bridges with a regular configuration, the uniform load method may be used, otherwise a multi-mode dynamic analysis is required.
- *Nonlinear Static Displacement Capacity Verification ("Pushover" Analysis)* - The displacement capacity of individual piers or bents is determined by a lateral load-displacement analysis accounting for the nonlinear behavior of the inelastic components.
- *Nonlinear Dynamic Analysis* – Nonlinear dynamic analysis using earthquake ground motion records to evaluate the displacement and force demands accounting for the inelastic behavior of the components.

A higher level analysis may be used in place of a lower-level analysis. The displacements from any demand analysis must satisfy the requirements in Article 3.10.3.10.

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moments in the plastic hinge zones or shear forces in isolation bearings) and/or displacements of the structure at the centroid of the mass.

In specifying the minimum Seismic Design and Analysis Procedure (SDAP), two principles are followed. First, as the seismic hazard increases, improved modeling and analysis for seismic demands is necessary because the behavior may be sensitive to the maximum demands. Secondly, as the complexity of the bridge increases, more sophisticated models are required for seismic demand and displacement capacity evaluation. No seismic analysis is necessary for regular bridges in SDAP B because minimum ductile detailing and capacity design principles provide sufficient displacement capacity for the hazard levels and performance requirements in which SDAP B is permitted. For bridges with a very regular configuration, a single degree-of-freedom model is sufficiently accurate to represent the seismic response. For these types of bridges, the capacity spectrum method in SDAP C combines the demand and capacity evaluation. The capacity spectrum method is appropriate for most structures with seismic isolation systems.

For structures that do not satisfy the requirements for a capacity spectrum analysis, an elastic response spectrum analysis, SDAP D, must be used to determine the displacement demands and the forces in the plastic hinge of structural components. Two elastic response spectrum analyses methods are permitted: the uniform load method, or the multi-mode response spectrum method depending on the configuration of the structure.

The uniform load method is suitable for structures with regular configuration. Long bridges, or those with significant skew or horizontal curvature, have dynamic characteristics that shall be represented in a multi-mode dynamic analysis.

The model for an elastic response spectrum analysis is linear, and as such it does not represent the inelastic behavior of earthquake resisting elements under strong ground motion. However, with the proper representation of the inelastic elements and interpretation of responses, an elastic analysis provides reasonable estimates of seismic demands. The model must be based on cracked section properties for concrete components and secant stiffness coefficients for the foundations, abutments, and seismic isolation components that are consistent with the expected level of deformation of the element. The only forces that are meaningful from an elastic response spectrum analysis are the forces in the earthquake resisting substructure elements, such as the bending moment at a plastic hinge in a column. The elastic forces in the earthquake resisting elements are reduced a factor that accounts for ductility of the earthquake resisting system. The displacements at the center of mass, generally the superstructure, can be used to estimate the

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A nonlinear dynamic analysis is required for structures with seismic isolation systems and (1) an effective vibration period greater than 3 seconds, or (2) effective damping greater than 30 percent.

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displacement demand of the structure including the effect of inelastic behavior in the earthquake resisting elements.

For SDAP E, a displacement capacity evaluation is required. The displacement capacity evaluation involves determining the displacement at which the first component reaches its inelastic deformation capacity. All non-ductile components shall be designed using capacity design principles to avoid brittle failure. For simple piers or bents, the displacement capacity can be evaluated by hand calculations using the geometry of displaced shapes and forces and deformation at the plastic capacity. For more complicated piers or bents, particularly when foundations and abutments are included in the model, a nonlinear static ("pushover") analysis may be used to evaluate the displacement capacity. It is recommended that the nonlinear static analysis continue beyond the displacement at which the first component reaches its inelastic deformation capacity in order to understand the behavior beyond the displacement capacity.

The displacement capacity is compared against the displacement demand determined from an elastic response spectrum analysis. The displacement capacity must exceed the demand by at least 50%. There are several reasons for this requirement. While on average the displacement of the elastic model, using a design response spectrum, should be approximately equal to the inelastic displacement, a significant difference is possible because of variability of the ground motion and its effect on inelastic behavior. Secondly, the demand analysis is performed on a three-dimensional model, whereas the displacement capacity verification is performed for individual bents or piers in the longitudinal and transverse directions separately. In Article 3.10.3.10.5, the displacement demand is multiplied by 1.5 to account for ground motion variability and the differences in the demand and capacity models and analysis methods.

A nonlinear dynamic analysis is the most general analysis method because the effect of inelastic behavior is included in the demand analysis. Depending on the mathematical model, the deformation capacity of the inelastic elements may or may not be included in dynamic analysis. A nonlinear dynamic analysis requires a suite of time histories (Article 3.10.2.5) of earthquake ground motion that are representative of the hazard and conditions at the site. Because of the complexity involved with nonlinear dynamic analysis, it is best used in conjunction with SDAP E.

Seismically isolated structures with very long periods or large damping ratios require a nonlinear dynamic analysis because the analysis procedures using an effective stiffness and damping may not properly represent the effect of isolation units on the response of the structure. The model for nonlinear

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analysis shall represent the hysteretic relationships for the isolator units.

4.8.3 Seismic Lateral Load Distribution

4.8.3.1 APPLICABILITY

These provisions shall apply to decks, girders, diaphragms (cross-frames), lateral bracing, and connections between the superstructure and the substructure, which are part of the earthquake resisting system in structures with Seismic Detailing Requirements (SDR) 2, 3, 4, 5, and 6. These provisions do not apply in Seismic Detailing Requirement 1.

C4.8.3.1

4.8.3.2 DESIGN CRITERIA

The Engineer shall demonstrate that a clear, straightforward load path from the superstructure to the substructure exists and that all components and connections are capable of resisting the imposed load effects consistent with the chosen load path.

If the overstrength forces are chosen for use in the design of the superstructure, then the elastic force distribution in the superstructure obtained from an elastic response spectrum analysis is not appropriate for use in the superstructure design. Unless a more refined analysis is made when using the overstrength forces in the superstructure design, the inertial forces expected to act on the superstructure may be assumed to vary linearly along the superstructure, and they shall produce both translational and rotational equilibrium when combined with the plastic mechanism forces from the substructure.

The flow of forces in the assumed load path must be accommodated through all affected components and details including, but not limited to, flanges and webs of main beams or girders, cross-frames, connections, slab-to-girder interfaces, and all components of the bearing assembly from top flange interface through the confinement of anchor bolts or similar devices in the substructure.

The analysis and design of end diaphragms and cross-frames shall consider horizontal supports at an appropriate number of bearings. Slenderness and connection requirements of bracing members that are part of the lateral force resisting system shall comply with applicable provisions specified for main member design.

Members of diaphragms and cross-frames identified by the Designer as part of the load path carrying seismic forces from the superstructure to the bearings shall be designed and detailed to remain elastic, based on the applicable gross area criteria, under all design

If the forces from the substructure corresponding to the overstrength condition are used to design the superstructure, it shall be recognized that the distribution of these forces may not be the same as that of the elastic demand analysis forces. The Engineer may calculate a more refined distribution of the inertial forces present when a full mechanism has developed. However, in lieu of such a calculation, the simpler linear distribution may be used, so long as the applied forces are in equilibrium with the plastic substructure forces. The vertical spatial relationship between location of the substructure plastic resistance and the location of the superstructure inertia force application shall also be considered in this analysis

Diaphragms, cross-frames, lateral bracing, bearings, and substructure elements are part of a earthquake resisting system in which the lateral loads and performance of each element are affected by the strength and stiffness characteristics of the other elements. Past earthquakes have shown that when one of these elements responded in a ductile manner or allowed some movement, damage was limited. In the strategy taken herein, it is assumed that ductile plastic hinging in substructure or seismic isolator units are the primary source of energy dissipation.

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earthquakes, regardless of the type of bearings used. The applicable provisions for the design of main members shall apply.

However, if elements of the earthquake resisting system are explicitly intended and designed to respond inelastically, then the previous paragraph does not apply to such elements. All other elements of the earthquake resisting system shall either be capacity-protected or designed for the elastic forces.

If elements of the earthquake resisting system are designed to fuse (i.e. breakaway) in the larger earthquake, then the redistribution of force that occurs with such alteration of the seismic load path shall be accounted for in the analysis.

All load-resisting elements shall have sufficient deformation capacity at the displacement of the center of mass of structure as determined from the seismic analysis.

4.8.3.3 LOAD DISTRIBUTION

A viable load path shall be established to transmit seismic loads to the substructure based on the stiffness characteristics of the deck, girders, diaphragms – end, intermediate and pier – (often referred to as cross-frames in steel bridges), lateral bracing, and connections between the superstructure and substructure. Unless a more refined analysis is made, an approximate load path shall be assumed as noted below.

In bridges with:

- A concrete deck that can provide horizontal diaphragm action, or
- A horizontal bracing system in the plane of the deck,

the lateral loads applied to the deck shall be assumed to be transmitted directly to the bearings through end diaphragms and/or pier diaphragms. The development and analysis of the load path through the deck or through the lateral bracing, if present, shall utilize assumed structural actions analogous to those used for the analysis of wind loading.

In bridges that have:

- Decks that cannot provide horizontal diaphragm action and
- No lateral bracing in the plane of the deck,

the lateral loads applied to the deck shall be distributed through the intermediate diaphragms to the bottom lateral bracing or the bottom flange, and then to the bearings, and through the end diaphragms and pier diaphragms in proportion to their relative rigidity and the respective tributary mass of the deck.

If a lateral bracing system is not present, and the bottom flange is not adequate to carry the imposed force effects, the first procedure shall be used, and the deck shall be designed and detailed to provide the necessary

COMMENTARY

Even if a component does not participate in the load path for seismic forces it must deform under the seismic loads. Such components must be checked that they have deformation capacity sufficient to maintain their load resistance under seismic induced deformations.

C4.8.3.3

A continuous path is necessary for the transmission of the superstructure inertia forces to the substructure. Concrete decks have significant rigidity in their horizontal plane, and in short to medium slab-on-girder spans, their response approaches rigid body motion. Therefore, the lateral loading of the intermediate diaphragms is minimal, consisting primarily of local tributary inertia forces from the girders, themselves.

Bearings do not usually resist load simultaneously, and damage to only some of the bearings at one end of a span is not uncommon. When this occurs, high load concentrations can result at the location of the other bearings, and this effect shall be taken into account in the design of the end and pier diaphragms. Also, a significant change in the load distribution among end and pier diaphragm members may occur.

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horizontal diaphragm action.

4.8.4 Modeling Requirements of Seismic Analysis

4.8.4.1 GENERAL

C4.8.4.1

For the dynamic analysis of structures subjected to earthquakes, the geometric configuration, strength, stiffness, mass, and energy dissipation mechanisms of the structural components and footings shall be included in the mathematical model.

Depending on the seismic analysis method different types of approximations may be used for modeling the strength, stiffness, and energy dissipation mechanisms. One-dimensional beam-column elements are sufficient for dynamic analysis of structures due to earthquake ground motion (referred to as “spine” models or “stick” models). For seismic analysis, grid or finite element analysis are generally not necessary. They greatly increase the size of the model and complicate the understanding of the force and deformation distribution through the substructure because of the large number of vibration modes.

The geometry of skew, horizontal curvature, and joint size shall be included in the model. However, two-dimensional models are adequate for bridges with skew less than 30 degrees and a subtended angle of horizontal curvature less than 20 degrees. When skew is included in a three-dimensional model, the geometry and boundary conditions at the abutments and bearing shall be represented in order to determine the forces and displacements at these locations. Short columns or piers may be modeled with a single element, but tall columns may have two or more elements, particularly if they have significant mass, in the case of concrete, or are modeled as framed substructures.

Bridges with multiple frames may be analyzed using models of a partial number of frames. Each model shall represent the geometry, mass, stiffness, and boundary conditions for the frames included in the model.

For bridges with multiple frames, separated by expansion bearings or hinges, it is unnecessary to model and analyze the entire bridge for seismic loads. Each frame shall have sufficient strength to resist inertia loads from the mass of the frame. However, when adjacent frames have large differences in vibration period, the frame with the longer period may increase the seismic load on the frame with the shorter period by impact across the bearing or hinge or by transverse forces through shear keys. To account for these effects, the number of frames included in a model depends on the ratio of vibration period of the frames. For bridges in which the period ratio of adjacent frames is less than 0.70 (shortest period frame divided by longest period frame), it is recommended to limit a model to five frames. The first and fifth frames in the model are considered to be boundary frames, representing the interaction with the remainder of the structure. The response of the three interior frames can be used for design of those frames. For a bridge with more than five frames, several different models are then used in the design.

For bridges with period ratios of frames between 0.70 and 1.0, fewer than five frames may be used in a

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The seismic analysis shall consider the two horizontal ground motion components.

The combination of loads from different horizontal and vertical components is given in Article 3.10.2.4.

The effect of the vertical component ground motion on bridges within 10 km of an active fault shall be included according to the requirements in Article 3.10.2.6.

4.8.4.2 DISTRIBUTION OF MASS

The modeling of mass shall be made with consideration of the degree of discretization in the model and the anticipated motion due to seismic excitation.

The number of degrees-of-freedom shall be selected to represent the total mass and mass distribution of the structure.

4.8.4.3 STIFFNESS AND STRENGTH

4.8.4.3.1 General

The mathematical model shall represent the stiffness of individual structural elements considering the materials, section dimensions, and force transfer between elements. For ductile earthquake resisting elements the stiffness shall be representative of the stiffness near the yield deformation. For capacity protected elements, including the superstructure, the elastic stiffness shall be represented in the mathematical model.

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model.

A common practice is to define the longitudinal direction as the chord connecting the ends of the bridge, and the transverse direction orthogonal to the longitudinal direction.

Bridges within 10 km of active fault require a site specific study and inclusion of vertical ground motion in the seismic analysis. For bridges located more than 10 km from active fault the procedures in Article 3.10.2.6 are used to account for the response to vertical ground motion in lieu of including the vertical component in the seismic analysis. If the vertical ground motion component is not included in the dynamic analysis, the forces from the analysis must be modified to account for the effect. For bridges with long, flexible spans, C-bents, or other large eccentricity in the load path for vertical loads, it is recommended to include vertical ground motion in the dynamic analysis.

C4.8.4.2

The distributions of stiffness and mass are included in the model for dynamic analysis. The discretization of the model shall account for geometric and material variation in stiffness and mass. Most of the mass of a bridge is in the superstructure. Four to five elements per span are generally sufficient to represent the mass and stiffness distribution of the superstructure. For spine models of the superstructure, the line of elements shall be located at the mass centroid. Rigid links can be used to represent the geometric location of mass relative to the spine elements in the model.

For single column piers, C-bents, or other unusual configurations, the rotational mass moment of inertia of the superstructure about the longitudinal axis shall be included.

The inertia of live loads need not be included in the seismic analysis. However, the probability of a large live load being on the bridge during an earthquake shall be considered when designing bridges with high live-to-dead load ratios that are located in metropolitan areas where traffic congestion is likely to occur.

C4.8.4.3.1

For elastic analysis methods, there is a significant approximation in representing the force-deformation relationship of inelastic structural elements by a single linearized stiffness. For inelastic columns or other inelastic earthquake resisting elements, the common practice is to use an elastic stiffness for steel elements and cracked stiffness for reinforced concrete elements. However, the stiffness of seismic isolator units, abutments, and soil in foundations are represented by a secant stiffness consistent with the maximum

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For Displacement Capacity Verification (nonlinear static analysis), the mathematical model shall include the strength based on nominal material properties. For nonlinear dynamic analysis, the models shall represent the stiffness, strength, and hysteretic behavior of the inelastic seismic resisting elements under cyclic loads.

4.8.4.3.2 Substructure

The flexural stiffness of columns and pier walls shall consider the effect of axial load. For reinforced concrete columns and pier walls, the stiffness shall represent the effects of cracking.

For Displacement Capacity Verification (inelastic static analysis), the strength of structural steel components in the model shall be based on the nominal plastic capacity. The flexural strength of reinforced and prestressed elements shall be based on nominal material properties of the steel and concrete.

The stiffness of capacity protected elements shall be based on elastic properties, including the effects of concrete cracking.

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deformation. The Engineer shall consider the distribution of displacements from an elastic analysis to verify that they are consistent with the inelastic behavior of the earthquake resisting elements.

C4.8.4.3.2

Seismic design procedures have been calibrated using stiffness that is representative of deformations close to the yield deformations. At these levels of deformation reinforced concrete elements will have cracked. The effects of cracking on the stiffness depend on the cross-section, longitudinal reinforcement ratio, axial load, and amount of bond slip. The cracked flexural stiffness of a reinforced concrete member can be obtained by moment-curvature analysis of the cross section, with modifications for bond-slip. In lieu of a moment-curvature analysis, the cracked section stiffness may be estimated by:

$$EI_{eff} = \frac{M_n}{(2e_y/D')}$$

where M_n is the nominal flexural strength of the section considering axial load, e_y is the yield strain of the reinforcement, and D' is the effective depth of the column. If the flexural strength has not been selected, the effective stiffness may be approximated by $EI_{eff} = 0.50EI_g$ for columns and pier walls (in the weak direction), where EI_g is the cross-sectional stiffness based on gross geometry and nominal material properties.

Where the load path depends on torsion of a reinforced concrete column or substructure element, the cracked torsional stiffness may be taken as one-fifth of the uncracked torsional stiffness.

The objective of the nonlinear displacement capacity verification is to determine the displacement at which the inelastic components reach their deformation capacity. The deformation capacity is the sum of elastic and plastic deformations. The plastic deformation is expressed in terms of the rotation of the plastic hinges. A nonlinear analysis using nominal strengths of the components gives larger plastic deformations than an analysis including overstrength. Hence, it is appropriate to use the nominal strength of the components when estimating the displacement capacity.

The stiffness of pier caps shall be included in the model. Pile caps and joints in reinforced concrete substructures may be assumed to be rigid. The strength

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4.8.4.3.3 Superstructure

The stiffness of the superstructure shall be consistent with the load path identified accordance with Article 4.6.2.8.3, including composite behavior between girders and decks and effective width of the superstructure that are monolithic with piers.

C4.8.4.3.3

For a spine or stick model of the superstructure, the stiffness is represented by equivalent section properties for axial deformation, flexure about two-axes, torsion, and possibly shear deformation in two directions. The calculation of the section stiffness shall represent reasonable assumptions about the three-dimensional flow of forces in the superstructure, including composite behavior.

The effects of skew can be neglected in the model of the superstructure. However, for large skew angles, the geometry of the piers with respect to the superstructure, and connections between the two, must be included in the model.

For reinforced box girders the effective stiffness may be based on three-quarters of the gross stiffness to account for cracking. For prestressed box girders, the full gross stiffness shall be used. The torsional stiffness may be based on a rationale shear flow without reduction due to cracking.

The flexural stiffness of the superstructure about a transverse axis is reduced near piers when there is a moment transfer between the superstructure and pier because of shear lag effects. The reduced stiffness shall be represented in the model of the superstructure.

4.8.4.4 FOUNDATIONS

Foundations may be modeled using the Foundation Modeling Method (FMM) defined in Table 4.8.4.4-1. Section 10 of the Specifications provides the requirements for estimating the depth to fixity and foundation springs.

C4.8.4.4

A wide range of methods for modeling foundations for seismic analysis are possible. Generally a refined model is unnecessary for seismic analysis. For many cases the assumption of a rigid foundation is adequate. Flexibility of a pile bent or shaft can be estimated using an assumed point of flexibility associated with the stiffness estimate of the pile or shaft and the soil. Spread footings and piles can be modeled with rotational and translational springs.

The requirement for including soil springs for Foundation Modeling Method II depends on the contribution of the foundation to the elastic displacement of the pier. Foundation springs for a pier are required when the foundation increases the elastic displacement of the pier by more than 20%. This comparison may be made on individual piers using estimates of the pier stiffness with hand calculations. If the contributions exceeds 15% for a majority of piers in a bridge, then it is recommended that foundation springs be included in all piers for the seismic analysis.

This approach is based on judgement that the forces and displacements from a seismic analysis with and without foundation springs that contribute less than 20% of the displacement of a pier will be comparable for design. More flexible spread and pile footings should be

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The required foundation modeling method depends on the Seismic Detailing Requirement (SDR) and the Seismic Design and Analysis Procedure (SDAP).

For SDR 3, Foundation Modeling Method I (FMM I) is required for any SDAP.

For SDR 4, 5, and 6, Foundation Modeling Method I may be used for SDAP C. SDAP D and E require the use of Foundation Modeling Method II (FMM II).

For SDAP E, FMM II is required in the Displacement Capacity Verification if it is used in the multi-mode dynamic analysis for displacement demand. The foundation models in the multi-mode dynamic analysis and Displacement Capacity Verification shall be consistent and representative of the footing behavior.

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modeled and included in the seismic analysis.

If foundation springs are included in the multi-mode dynamic analysis, they must be included in the pushover analysis so the two models are consistent for the displacement comparison.

For most spread footings and piles with pile cap a secant stiffness for the soil springs is adequate. If the design limits for spread or pile footings are exceeded, according to the requirements in Article 10, bi-linear soil springs are required for the pushover analysis.

For pile bents and drilled shafts, an estimated depth to fixity is generally adequate for representing the relative flexibility of the soil and pile or shaft. Soil springs with secant stiffness may be used to provide a better representation based a P-y curves for the footing and soil. Bi-linear springs may be used in the pushover analysis if there is particular concern with depth of the plastic hinge and effective depth of fixity.

If bi-linear springs are used in a pushover analysis, a secant stiffness typical of the expected level of soil deformation is used in the multi-mode dynamic analysis for valid comparison of displacement demand and capacity.

Table 4.8.4.4-1 Definition of Foundation Modeling Method

Foundation Type	FMM I	FMM II
Spread Footing	Rigid	Rigid for Soil Types A and B. For other soil types, foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Footing with Pile Cap	Rigid	Foundation springs required if footing flexibility contributes more than 20% to pier displacement.
Pile Bent/Drilled Shaft	Estimated depth to fixity	Estimated depth to fixity or soil-springs based on P-y curves.

For sites identified as susceptible to liquefaction or lateral spread, the model of the foundations and structures shall consider the nonliquefied and liquefied conditions using the procedures specified in Article 3.10.4.1.

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4.8.4.5 ABUTMENTS

C4.8.4.5

The model of the abutment shall reflect the expected behavior of the abutment under seismic loads in each orthogonal direction. Resistance of structural components shall be represented by cracked section properties for multi-mode response spectrum analysis. The resistance from passive pressure shall be represented by a value for the secant stiffness consistent with the maximum displacement. For the Displacement Capacity Verification, the strength of each component in the abutment, including soil, shall be included.

Articles 11.6.5.1.1 and 11.6.5.1.2 provide requirements for the modeling of abutments in the longitudinal and transverse directions, respectively. The iterative procedure with secant stiffness coefficients defined in those articles are included in the mathematical model of the bridge to represent the resistance of the abutments in an elastic analysis. The load-displacement behavior of the abutment may be used in a static nonlinear analysis when the resistance of the abutment is included in the design of the bridge.

4.8.4.6 SEISMIC ISOLATOR UNITS

C4.8.4.6

Seismic isolator units shall be modeled by an effective stiffness based on the properties of the isolator unit.

To simplify the nonlinear behavior of the isolator unit, a bilinear simplification may be used. The analysis shall be repeated using upper-bound properties in one analysis and lower-bound properties in another as specified in Article 15.4. The purpose of the upper- and lower-bound analyses is to determine the maximum forces in the substructure and maximum displacement of the isolation system.

The upper- and lower-bound analyses are not required if the displacements, using equation (4.7.4.2-1), do not vary from the design values by more than 15 percent when the maximum and minimum values of the isolator unit properties are used (Article 15.4). For these simplified calculations, damping ratios greater than 30 percent may be used to establish the 15 percent limit.

The requirements for analysis of bridges with seismic isolation systems are specified in Article 15.4 and are based on the 1999 AASHTO *Guide Specifications for Seismic Isolation Design*, which provide requirements for modeling seismic isolator units, including the use of property modification factors as given in Article 15.5.

The force-deformation characteristics can be idealized as a bilinear relationship with two key variables: second slope stiffness and characteristic strength. The area under the bilinear curve is energy dissipated by hysteretic work during cyclic loading. For design, the force-deformation relationship can be represented by an effective stiffness based on the secant and a damping coefficient.

The requirements for determining the upper-bound and lower-bound properties is provided in Article 15.4.

4.8.4.7 HINGES

C4.8.4.7

Two models shall represent expansion bearings and intermediate hinges. The compression model assumes the superstructure at the bearing or hinge is closed and can transfer longitudinal forces. The tension model assumes the bearing or hinge is open and cannot transfer longitudinal forces. The stiffness of restraining devices, if any, shall be included in the tension model.

A compression model need not be considered for expansion bearings if it can be demonstrated by calculation that longitudinal forces cannot be transferred through the superstructures at the bearing location.

The use of compression and tension models is expected to provide a reasonable bound on forces (compression model) and displacements (tension model).

4.8.4.8 DAMPING

C4.8.4.8

Energy dissipation in the structure, including, footings and abutments, may be represented by viscous

Damping may be neglected in the calculation of natural frequencies and associated nodal

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damping. The selection of the viscous damping ratio depends on the type of dynamic analysis and the configuration of the bridge.

For elastic response spectrum analysis, the viscous damping ratio is based on the energy dissipation due to small and moderate deformation of the members and soil.

4.8.5 Seismic Analysis Procedures

The regularity requirements that permit use of the Capacity Spectrum Analysis Method are given in Article 3.10.3.4.2. The regularity requirements for using the Uniform Load Method and Multi-mode Methods of Analyses are given in Article 4.8.5.3.1.

4.8.5.1 CAPACITY SPECTRUM ANALYSIS

The lateral strength of each pier in the longitudinal and transverse directions shall be at least C_s times the tributary weight for the pier.

The lesser of the following equations shall be used to assess C_s for the 50% in 75 year and 3% in 75 year earthquake loadings:

$$C_s \Delta = \left(\frac{F_v S_1}{2p B_L} \right)^2 g \quad (4.8.5.1-1)$$

$$C_s = \frac{F_a S_s}{B_s} \quad (4.8.5.1-2)$$

where B_s and B_L are response reduction factors for short and long period structures, respectively, and are defined in Table 4.8.5.1-1. The response spectrum values and soil

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displacements. The effects of damping shall be considered when the dynamic response for seismic loads is considered.

Suitable damping values may be obtained from field measurement of induced free vibration or by forced vibration tests. In lieu of measurements, the following values may be used for the equivalent viscous damping ratio:

- Concrete construction: 5 percent
- Welded and bolted steel construction: 2 percent
- Timber: 5 percent

For one or two-span bridges with abutments designed to activate significant passive pressure in the longitudinal direction, a damping ratio of up to 10 percent may be used for longitudinal vibration modes.

Equivalent viscous damping may be considered to represent the energy dissipation due to cyclic loading of yielding members. Equivalent damping shall only be used with a secant stiffness estimate for the entire structure. For single degree-of-freedom models the equivalence can be established within a satisfactory degree of accuracy. For bridges with seismic isolation or other seismic protection components, the equivalence is established in an approximate manner. Equivalent viscous damping shall not be used to represent inelastic energy dissipation for any other model or method of dynamic analysis.

C4.8.5.1

The capacity spectrum analysis may be used for bridges that are designed to respond to earthquake ground motion as a single degree-of-freedom system in the longitudinal and transverse direction. Very regular bridges that satisfy the special requirements are expected to respond as a single degree-of-freedom system and the capacity spectrum approach may be used for such cases.

The capacity spectrum analysis uses the elastic response spectrum defined in Article 3.10.2.1. The elastic spectrum is reduced to account for dissipation of energy in the inelastic earthquake resisting elements. The reduced elastic spectrum is evaluated at the effective vibration period, which is based on an effective stiffness equal to the design strength divided by the maximum displacement. An advantage of the capacity

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Table 4.8.5.1-1. The response spectrum values and soil factors, $F_v S_1$ and $F_a S_s$, are defined in Article 3.10.2. In Equation 4.8.5.1-1, Δ is the displacement of the pier.

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spectrum method is that the vibration period does not need to be calculated because it is implicit in equations 4.8.5.1-1 and 4.8.5.1-2. Equation 4.8.5.1-1 will govern for most bridges, and as a result the Designer has several choices in selecting the lateral strength and maximum displacement as described in Article C3.10.3.4.

For stiff bridges, the maximum displacement may give a seismic coefficient C_s greater than required by Equation 4.8.5.1-2. In such cases the strength need not be greater than the value defined by Equation 4.8.5.1-2.

The basis of the capacity spectrum method is to linearize nonlinear structural behavior by determining a "secant" period and effective damping factor based on hysteretic response. This approach was originally proposed by Gulkan and Sozen (1974) and called the "Substitute Structure Method".

Assuming the peak response of the nonlinear structure is equal to the displacement of an equivalent (substitute) SDOF system, the effective period is given by

$$T_{eff} = 2p \sqrt{\frac{m}{K_{eff}}} = 2p \sqrt{\frac{W/g}{F_y/\Delta_{max}}} = 2p \sqrt{\frac{\Delta_{max}}{C_c g}} \quad (C4.8.5.1-1)$$

in which m = structure mass; W = seismic structure weight; F_y and D_{max} are the idealized response force and maximum displacement shown in Figure C2.5.6-3; C_c = normalized base shear given by $C_c = F_y/W$; g = gravitational acceleration.

The seismic demand ($C_d = F_{elastic}/W$ where $F_{elastic}$ = elastic design force) can be expressed in terms of the design spectrum with the appropriate damping as used for seismic isolation such that the lesser of the following governs

$$C_d = \frac{F_a S_s}{B_s} \quad (C4.8.5.1-2)$$

$$C_d = \frac{F_v S_1}{T_{eff} B_L} \quad (C4.8.5.1-3)$$

in which $F_a S_s$ and $F_v S_1$ are obtained from Article 3.10.2, and B_s and B_L are modification factors for the short and long period portions of the design spectra that account for hysteretic damping effects, given by

$$B_s = \left(\frac{x_{eff}}{0.05} \right)^{0.5} \quad \text{and} \quad B_L = \left(\frac{x_{eff}}{0.05} \right)^{0.3} \quad (C4.8.5.1-4)$$

where for an equivalent elasto-plastic system

$$x_{eff} = 0.05 + \frac{2}{p} h \left(1 - \frac{1}{m} \right) \quad (C4.8.5.1-5)$$

in which m = displacement ductility factor; h = energy absorption efficiency factor.

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Based on extensive experimental calibration, h may be taken as follows:

- seismically detailed reinforced concrete elements

$$h = 0.35 - 0.4$$

- poorly detailed (non-ductile) reinforced concrete

$$h = 0.25$$

- For timber structures

$$h = 0.1 - 0.15$$

- For steel structures

$$h = 0.70$$

Assuming the capacity is equal to the reduced demand and taking equation (C4.8.5.1-1) and substituting it into (C4.8.5.1-3) and rearranging, gives for long period structures:

$$F_v S_1 = 2p \sqrt{\frac{C_c^* \Delta^*}{g}} B_L \quad (\text{C4.8.5.1-6})$$

for short period structures

$$F_a S_s = C_c B_s \quad (\text{C4.8.5.1-7})$$

Note the *greater* of the above two equations governs.

In the above, $C_c^* = C_c / a_2$ and $\Delta^* = \Delta / a_1$ where a_1 and a_2 are transformation factors that account for converting a MDOF system into a substitute SDOF structure. These are defined as

$$a_1 = f_{mn} \frac{\sum_{i=1}^N w_i f_{im}}{\sum_{i=1}^N w_i f_{im}^2} \quad (\text{C4.8.5.1-8})$$

$$a_2 = \frac{\left[\sum_{i=1}^N w_i f_{im} \right]^2}{\sum_{i=1}^N w_i \sum_{i=1}^N w_i f_{im}^2} \quad (\text{C4.8.5.1-9})$$

where $\sum_{i=1}^N w_i = W$ = total seismic weight; w_i = tributary weight at location i ; and f_{mn} = m^{th} mode shape at the n^{th} location.

It should be noted that if the bridge structure has a simple configuration such that the deck is subjected to pure translation (that is there is no substantial deck

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When equation 4.8.5.1-1 governs for the 3% in 75-year earthquake, the displacement of the superstructure, D , shall satisfy the requirements of Article 3.10.3.10. When equation 4.8.5.1-1 governs for the 50% in 75 year earthquake, D shall be taken as 1.3 times the yield displacement of the pier.

4.8.5.2 CAPACITY SPECTRUM ANALYSIS FOR STRUCTURES WITH SEISMIC ISOLATION SYSTEMS

The capacity spectrum analysis procedure may be used for structures with seismic isolation systems that meet the regularity requirements for the Uniform Load Method of Article 4.8.5.3.2 and the effective vibration period is 3 seconds or less, and the effective damping is less than or equal to 30 percent of critical. Article 15.4 specifies other required analysis procedures.

The displacement, Δ , (meters) of the superstructure (including the substructure and bearing unit deformation) is given by

$$\Delta = \frac{0.25F_v S_1 T_{eff}}{B} \text{ (meters)} \quad (4.8.5.2-1)$$

$$\Delta = \frac{10F_v S_1 T_{eff}}{B} \text{ (inches)} \quad (4.8.5.2-2)$$

$$T_{eff} = 2p \sqrt{\frac{W}{K_{eff} g}} \quad (4.8.5.2-3)$$

The damping coefficient, B , is based on the percentage of critical damping according to Table 4.8.5.2-1. The percentage of critical damping depends on the energy dissipation by the isolation system, which shall be determined by test of the isolation systems characteristics, and substructure as specified in Article 15.10. The damping coefficient may be determined by linear interpolation of the values in Table 4.8.5.2-1.

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bending due to favorable support conditions), then the structure will behave in a single-degree-of-freedom fashion, thus a_1 and a_2 are set to unity. Such a condition can be orchestrated by design, particularly when all the piers have a similar stiffness and the deck is uncoupled from the abutments through the use of low stiffness bearing supports as required for the application of this analysis method.

The maximum displacement of the superstructure for the 3% in 75-year earthquake is limited by the plastic deformation capacity of the substructure, taken as $\Delta = q_p H$ with $q_p = 0.035$ for reinforced concrete and the P- Δ limitation in Article 3.10.3.10.4. The maximum displacement of the superstructure for the 50% in 75-year earthquake is limited to 1.3 times the elastic displacement of the substructure.

C4.8.5.2

The requirements of Article 7.1 in the AASHTO *Guide Specifications for Seismic Isolation Design* (1999) is the capacity spectrum method. Using the capacity spectrum equation in the velocity-controlled region of the spectrum (4.8.5.1-1), the maximum displacement is

$$\Delta = \left(\frac{F_v S_1}{2p B} \right)^2 g \frac{1}{C_s} \quad (C4.8.5.2-1)$$

In the capacity spectrum method, the effective period is defined by the maximum displacement and seismic coefficient:

$$T_{eff} = 2p \sqrt{\frac{\Delta}{C_s g}} \quad (C4.8.5.2-2)$$

With the effective stiffness expressed as $K_{eff} = C_s W / \Delta$, the effective period is

$$T_{eff} = 2p \sqrt{\frac{W}{K_{eff} g}} \quad (C4.8.5.2-3)$$

Solving (C4.8.5.2-2) for the seismic coefficient, substituting into (C4.8.5.2-1) and simplifying gives

$$\Delta = \frac{g}{(2p)^2} \frac{F_v S_1 T_{eff}}{B} \quad (C4.8.5.2-4)$$

In meter units the coefficient for the expression is 0.25, and in inches units the coefficient is 10. This is the same as (3a) and (3b) in the 1999 *Guide Specifications* with AS_1 replaced by $F_v S_1$ for the 3% in 75 year earthquake loading. In the *Guide Specifications*, the reduction factor B is defined for the long-period range as is B in this article.

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Alternatively, the seismic coefficient evaluated at the effective period and reduced for the effects of energy dissipation is:

$$C_s = \frac{F_v S_1}{T_{eff} B} \quad (C4.8.5.2-5)$$

This is the same as equation (2a) in the *Guide Specifications* with AS_i replaced by $F_v S_1$ for the 3% in 75 year earthquake loading and the B values from the 1999 Guide Specifications are given in Table 4.8.5.2-1.

Table 4.8.5.1-1 Capacity Spectrum Response Reduction Factors for Bridges with Ductile Piers

(a) 50% in 75 Year Earthquake Loading

<i>Performance Level</i>	B_S	B_L
Operational	1	1
Life Safety	1	1

(b) 3% in 75 Year Earthquake Loading

<i>Performance Level</i>	B_S	B_L
Operational	1	1
Life Safety	2.3	1.6

Table 4.8.5.2-1 Capacity Spectrum Response Reduction Factors for Bridges with Seismic Isolation Systems

	Damping (as percentage of critical)						
	=2	5	10	20	30	40	50
B	0.8	1.0	1.2	1.5	1.7	1.9	2.0

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4.8.5.3 ELASTIC RESPONSE SPECTRUM ANALYSIS

4.8.5.3.1 Selection of Elastic Response Spectrum Analysis Method

C4.8.5.3.1

The uniform load method may be used for structures satisfying the requirements in Table 4.8.5.3.1-1. For structures not satisfying the regularity requirements, the multi-mode dynamic analysis shall be used.

Table 4.8.5.3.1-1 Requirements for Uniform Load Method

Parameter	Value				
Number of Spans	2	3	4	5	6
Maximum subtended angle for a curved bridge	20°	20°	30°	30°	30°
Maximum span length ratio from span to span	3	2	2	1.5	1.5
Maximum bent/pier stiffness ratio from span to span, excluding abutments	---	4	4	3	2

4.8.5.3.2 Uniform Load Method

C4.8.5.3.2

The uniform load method shall be based on the fundamental mode of vibration in the longitudinal or transverse direction. The period of this mode of vibration shall be taken as that of an equivalent single mass-spring oscillator. The stiffness of this equivalent spring shall be calculated using the maximum displacement that occurs when an arbitrary uniform lateral load is applied to the bridge. The seismic coefficient, C_{sm} , specified in Article 3.10.2.1 shall be used to calculate the equivalent uniform seismic load from which seismic force effects are found. However, for periods less than T_s , the seismic coefficient shall be equal to S_{DS} .

The uniform load method, described in the following steps, may be used for both transverse and longitudinal earthquake motions. It is essentially an equivalent static method of analysis that uses a uniform lateral load to approximate the effect of seismic loads. The method is suitable for regular bridges that respond principally in their fundamental mode of vibration. The capacity spectrum analysis is similar to the uniform load method, in that they are both appropriate for bridges whose dynamic response can be represented by an equivalent single degree-of-freedom system. Capacity spectrum analysis may only be used for bridges in which abutments do not resist significant longitudinal or transverse seismic forces. For such bridges, the vibration mode shape is essentially a rigid body displacement of the superstructure, providing a uniform lateral load.

Whereas displacements are calculated with reasonable accuracy, the method can overestimate the transverse shears at the abutments by up to 100 percent. Consequently, the columns may have inadequate lateral strength because of the overestimate of abutment forces. A multi-mode dynamic analysis is recommended to avoid unrealistic distributions of seismic forces.

The steps in the uniform load method are:

1. Calculate the static displacements $v_s(x)$ due to an assumed uniform load p_o , as shown in Figure C1. The uniform loading p_o is applied over the length of the bridge; it has dimension of force/unit length and may be arbitrarily set equal to 1.0. The static displacement $v_s(x)$ has the dimension of length.
2. Calculate the bridge lateral stiffness, K , and total weight, W , from the following expressions:

$$K = \frac{p_o L}{V_{s,MAX}} \quad (C4.8.5.3.2-1)$$

$$W = \int_0^L w(x) dx \quad (C4.8.5.3.2-2)$$

where:

L = total length of the bridge

$v_{s,MAX}$ = maximum value of $v_s(x)$

$w(x)$ = nominal, unfactored dead load of the bridge superstructure and tributary substructure.

The weight shall take into account structural elements and other relevant loads including, but not limited to, pier caps, abutments, columns, and footings. Other loads, such as live loads, may be included.

3. Calculate the period of the bridge, T_m , using the expression:

$$T_m = 2p \sqrt{\frac{W}{Kg}} \quad (C4.8.5.3.2-3)$$

where:

g = acceleration of gravity

4. Calculate the equivalent static earthquake loading p_e from the expression:

$$p_e = \frac{C_{sm} W}{L} \quad (C4.8.5.3.2-4)$$

where:

C_{sm} = the dimensionless elastic seismic response coefficient according to Article 3.10.2.1 with the coefficient taken as S_{DS} for short periods.

p_e = equivalent uniform static seismic loading per unit length of bridge applied to represent the primary mode of vibration.

5. Calculate the displacements and member forces for use in design either by applying p_e to the structure

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4.8.5.3.3 Uniform Load Method for Structures with Seismic Isolation Systems

C4.8.5.3.3

The statically equivalent seismic force is given by

$$F = C_s W \quad (4.8.5.3.3-1)$$

The elastic seismic response coefficient, C_s , used to determine the equivalent force is given by

$$C_s = \frac{K_{eff} d}{W} \quad (4.8.5.3.3-2a)$$

$$C_s = \frac{F_v S_1}{T_{eff} B_L} \quad (4.8.5.2.3-2b)$$

The statically equivalent seismic force shall be used with the uniform load method in Article 4.8.5.3.2.

4.8.5.3.4 Multi-mode Dynamic Analysis Method

C4.8.5.3.4

The elastic multi-mode dynamic analysis method shall be used for bridges in which coupling occurs in more than one of the three coordinate directions within each mode of vibration. As a minimum, linear dynamic analysis using a three-dimensional model shall be used to represent the structure.

The number of modes included in the analysis shall be at least three times the number of spans in the model for regular bridges.

The elastic seismic response spectrum as specified in Article 3.10.2.1 shall be used for each mode. The spectrum at the vibration periods shall be scaled for damping ratios other than 5 percent.

For structures with seismic isolation the scaling shall apply only for periods greater than $0.8T_{eff}$. The 5 percent response spectrum shall be used for other modes.

and performing a second static analysis or by scaling the results of the first step above by the ratio ρ_e/ρ_o .

Vibration modes are convenient representation of dynamic response for response spectrum analysis. Enough modes shall be included to provide sufficient participation for bending moments in columns, or other components with inelastic deformation. Dynamic analysis programs, however, usually only compute participation factors for base shear, often expressed as a percentage of total mass. For regular bridges the guideline of including 90% of the modal mass for horizontal components generally provides sufficient number of modes for accurate estimate of forces in lateral load resisting components. For irregular bridges, or large models of multiple-frame bridges, the participating mass may not indicate the accuracy for forces in specific components. It is for this reason that the models of long bridges are limited to five frames.

The response spectrum in Article 3.10.2.1 is based on 5 percent damping. The spectrum must be modified when other damping values are used, such as subject to Article 4.8.4.8 for bridges without seismic isolation. For bridges with seismic isolation the additional damping from the seismic isolator units applies only to the isolated vibration modes. Other vibration modes have damping defined in Article 4.8.4.8.

A suitable modification of the 5 percent response

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The member forces and displacements due to a single component of ground motion may be estimated by combining the respective response quantities (moment, force, displacement, or relative displacement) from the individual modes by the Complete Quadratic Combination (CQC) method.

4.8.5.4 SEISMIC DISPLACEMENT CAPACITY VERIFICATION

The displacement capacity verification analysis shall be applied to individual piers or bents to determine the lateral load-lateral displacement behavior of the pier or bent. The capacity evaluation shall be performed for individual piers or bents in the longitudinal and transverse direction separately.

The evaluation shall identify the component in the pier or bent that first reaches its inelastic deformation capacity as given in Articles 5.16 and 6.15.6. The displacement at which the first component reaches deformation capacity defines the displacement capacity for the pier or bent and this shall exceed the demand given in Article 3.10.3.9.5. The model shall represent all components providing seismic load resistance.

When required by Article 4.8.4.4, the model for the foundation shall include soil springs or an estimated depth to fixity.

The model for the displacement capacity verification is based on nominal capacities of the inelastic components. Stiffness and strength degradation of

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spectrum is to divide the spectrum by:

$$\left(\frac{b}{5}\right)^{0.3}$$

for vibration periods greater than T_s and divide by

$$\left(\frac{b}{5}\right)^{0.5}$$

for vibration periods less than or equal to T_s , where b is the damping ratio in percent up to 30 percent.

Member forces and displacements obtained using the CQC combination method are generally adequate for most bridge systems (Wilson et al. 1981).

If the CQC method is not readily available, alternative methods include the square root of the sum of the squares method (SRSS), but this method is best suited for combining responses from modes with well-separated frequencies. For closely spaced modes, the absolute sum of the modal responses shall be used.

C4.8.5.4

The objective of the displacement capacity verification analysis is to determine the displacement at which the earthquake resisting elements achieve their inelastic deformation capacity. Damage states are defined by local deformation limits, such as plastic hinge rotation, footing settlement or uplift, or abutment displacement. Displacement may be limited by loss of capacity such as degradation of strength under large inelastic deformations or *P-D* effects.

For simple piers or bents, the maximum displacement capacity can be evaluated by hand calculations using the defined mechanism and the maximum allowable deformations of the plastic hinges. If axial force-moment interaction is significant, iteration is necessary to determine the mechanism.

For more complicated piers or foundations, displacement capacity can be evaluated using a nonlinear static analysis procedure, commonly known as a pushover analysis.

Displacement capacity verification is required for individual piers or bents. Although it is recognized that force redistribution may occur as the displacement increases, particularly for frames with piers of different stiffness and strength, the objective of the capacity verification is to determine the maximum displacement capacity of each pier. The displacement capacity is to be compared with an elastic demand analysis, which considers the effects of different stiffness and is specified in Article 3.10.3.9.5

Nominal inelastic capacities are used for the displacement capacity verification. Although the displacement capacity verification considers a

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inelastic components and effects of loads acting through the lateral displacement shall be considered.

The maximum displacement of a pier or bent is achieved when a component reaches the maximum deformation. Maximum plastic hinge rotations for structural components are specified in Articles 5.16 and 6.15.6. The maximum deformation for foundation and abutments are limited by geometric constraints on the structure.

The model of the foundation for the displacement capacity evaluation shall be consistent with the demand analysis.

For the purpose of this Article, the displacement is the displacement at the center of mass for superstructure supported by the pier or bent under consideration.

4.8.5.5 NONLINEAR DYNAMIC ANALYSIS PROCEDURE

Nonlinear dynamic analysis provides displacements and member actions (forces and deformations) as a function of time for a specified earthquake ground motion. All loads in Extreme Load Case I shall be included in the analysis.

The ground motion time histories shall satisfy the requirements of Article 3.10.2.5.

A minimum of three ground motions, representing the design event, shall be used in the analysis. Each ground motion shall include two horizontal components and a vertical component. The maximum action for the three ground motions shall be used for design. If more than seven ground motions are used, the design action may be the mean of the actions for the individual ground motions.

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monotonically increasing displacement, the effects of cyclic loading must be considered when selecting an appropriate model and establishing a maximum inelastic deformation. This includes strength and stiffness degradation and low-cycle fatigue.

Generally, the center of mass is at the elevation of the mass centroid of the superstructure.

C4.8.5.5

The nonlinear dynamic analysis procedure is normally only used for the 3% in 75 year earthquake. The structure is expected to remain essentially elastic for the 50% in 75 year earthquake, hence a multi-mode response spectrum analysis is adequate.

The results of a nonlinear dynamic analysis should be compared with the a multi-mode response spectrum analysis as a check for reasonableness of the nonlinear model.

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5.2 NOTATION

(SUPPLEMENTAL NOTATION RELATED TO SECTION CHANGES)

A_b = area of longitudinal reinforcing bars being restrained by rectilinear hoops and/or cross ties

A_{bh} = bar area of the transverse hoops or ties restraining the longitudinal steel

A_{cc} = confined core area

A_{sh} = total area of transverse reinforcement along the axis of bending in the direction of the applied shear

A'_{sh} = total area of transverse reinforcement perpendicular to direction of the applied shear

A_{st} = total area of longitudinal steel

A_v = shear area of concrete

b_w = the web width resisting shear in a rectangular section

b_{je} = the effective joint width, found using a 45-degree spread from the column boundaries.

D = diameter of circular column

D' = the distance between the outer layers of the longitudinal reinforcement on opposite faces of the member, equal to the pitch circle diameter for a circular section

D'' = centerline section diameter/width of the perimeter spiral/hoops

d_b = diameter of the main longitudinal reinforcing bars.

f_{yh} = transverse reinforcement yield stress

f_y = yield stress of the longitudinal reinforcement

f_h = the average axial stresses in the horizontal direction within the plane of the connection under

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consideration

f_{su} = ultimate tensile strength of the longitudinal reinforcement

f_v = the average axial stresses in the vertical direction within the plane of the connection under consideration

h_b = the cap beam or footing depth

h_c = the column lateral dimension in the direction considered

H_c = the height of the cap beam/joint.

K_{shape} = plastic strength factor that depends on the shape of the section

L_p = effective plastic hinge length give by

M_y = yield moment of the section

M_{po} = plastic overstrength moment

M_p = the maximum plastic moment

N_f = number of cycles of loading expected at the maximum displacement amplitude

P_e = factored axial load including seismic effects

s = the center-to-center spacing of hoopsets or the pitch the spiral steel

U_{sf} = strain energy capacity (modulus of toughness) of the transverse reinforcement

v_{hv} = the average shear stress within the plane of the connection.

V_p = the contribution due to arch action given by

V_c = the tensile contribution of the concrete

V_s = the contribution of shear resistance provided by transverse reinforcement

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r_v = ratio of transverse reinforcement for shear inside the potential plastic hinge zone.

r_v^* = ratio of transverse reinforcement for shear outside the potential plastic hinge zone.

e_y = yield strain of the longitudinal reinforcement

f = resistance factor for seismic shear (0.85)

r_t = volumetric ratio of longitudinal reinforcement

r_s = ratio of transverse reinforcement

Λ = fixity factor

q = angle of the principal crack plane

a = geometric aspect ratio angle

q_p = plastic rotational capacity of hinge zones

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5.4.3.1 GENERAL

C5.4.3.1

Reinforcing bars, deformed wire, cold-drawn wire, welded plain wire fabric, and welded deformed wire fabric shall conform to the material standards as specified in Article 9.2 of the AASHTO LRFD Bridge Construction Specifications.

Reinforcement shall be deformed, except that plain bars or plain wire may be used for spirals, hoops, and wire fabric.

The nominal yield strength shall be the minimum as specified for the grade of steel selected, except that yield strengths in excess of 520 MPa shall not be used for design purposes except as permitted herein. The yield strength or grade of the bars or wires shall be shown in the contract documents. Bars with yield strengths less than 400 MPa shall be used only with the approval of the Owner.

High strength high alloy bars, with an ultimate tensile strength of up to 1600 MPa, may be used for longitudinal column reinforcement for seismic loading providing it can be demonstrated through tests that the low cycle fatigue properties is not inferior to normal reinforcing steels with yield strengths of 520 MPa or less.

Wire rope or strand may be used for spirals in columns in SDR 3, 4, 5 and 6 if it can be shown through tests that the modulus of toughness exceeds 100MPa.

Where ductility is to be assured or where welding is required, steel conforming to the requirements of ASTM A 706, "Low Alloy Steel Deformed Bars for Concrete Reinforcement," should be specified.

High strength reinforcement reduces congestion and cost as demonstrated by Mander and Cheng (1999), and Dutta, Mander and Kokorina, (1999). However it is important to ensure that the cyclic fatigue life is not inferior when compared to ordinary mild steel reinforcing bars. Mander, Panthaki, and Kasalanati, (1994) have shown that modern high alloy prestressing threadbar steels can have sufficient ductility to justify their use in seismic design.

The *Modulus of Toughness* is defined as the area beneath the monotonic tensile stress-strain curve from initial loading (zero stress) to fracture.

A 706 reinforcement should be considered for seismic design because of the greater quality control by which unanticipated overstrength is limited.

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5.7.4.2 LIMITS FOR REINFORCEMENT

C5.7.4.2

Additional limits on reinforcement for compression members in SDR 3 and above shall be considered as specified in Article 5.10.11.4.1a.

The maximum area of prestressed and nonprestressed longitudinal reinforcement for noncomposite compression components shall be such that:

$$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} < 0.04 \quad (5.7.4.2-1a)$$

for elements participating in the earthquake resisting system (ERS) and

$$\frac{A_s}{A_g} + \frac{A_{ps} f_{pu}}{A_g f_y} \leq 0.08 \quad (5.7.4.2-1b)$$

for all other elements

and

$$\frac{A_{ps} f_{pe}}{A_g f'_c} \leq 0.30 \quad (5.7.4.2-2)$$

The minimum area of prestressed and nonprestressed longitudinal reinforcement for noncomposite compression components shall be such that:

$$\frac{A_s f_y}{A_g f'_c} + \frac{A_{pu} f_{pu}}{A_g f'_c} \geq 0.108 \quad (5.7.4.2-3)$$

where:

A_s = area of nonprestressed tension steel (mm²)

A_g = gross area of section (mm²)

A_{ps} = area of prestressing steel (mm²)

f_{pu} = specified tensile strength of prestressing steel (MPa)

f_y = specified yield strength of reinforcing bars (MPa)

f'_c = specified compressive strength of concrete (MPa)

f_{pe} = effective prestress (MPa)

The present steel volumetric ratio limits are an artifact of old elastic design and are excessively high for ductile seismic elements. It is for this reason that the total effective limit should be halved to 4% to give better inherent ductility to components.

According to current ACI codes, the area of longitudinal reinforcement for nonprestressed noncomposite compression components should be not less than 0.01 A_g . Because the dimensioning of columns is primarily controlled by bending, this limitation does not account for the influence of the concrete compressive strength. To account for the compressive strength of concrete, the minimum reinforcement in flexural members is shown to be proportional to f_n/f_y in Article 5.7.3.3.2. This approach is also reflected in the first term of Equation 5.7.4.2-3. For fully prestressed members, current codes specify a minimum average prestress of 1.6 MPa. Here also the influence of compressive strength is not accounted for. A compressive strength of 35 MPa has been used as a basis for these provisions, and a weighted averaging procedure was used to arrive at the equation.

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The minimum number of longitudinal reinforcing bars in the body of a column participating in the ERS shall be six in a circular arrangement and eight in a rectangular arrangement. The minimum size of bar shall be 16 mm.

5.7.4.6 SPIRALS AND TIES

The area of steel for spirals and ties in bridges in SDR 3 and above shall comply with the requirements specified in Article 5.10.11.

Where the area of spiral and tie reinforcement is not controlled by:

- Seismic requirements,
- Shear or torsion as specified in Article 5.8, or
- Minimum requirements as specified in Article 5.10.6,

the ratio of spiral reinforcement to total volume of concrete core, measured out-to-out of spirals, shall not be less than:

$$r_s = 0.45 \left(\frac{A_g}{A_c} - 1 \right) \frac{f'_c}{f_{yh}} \quad (5.7.4.6-1)$$

where:

A_g = gross area of concrete section (mm²)

A_c = area of core measured to the outside diameter of the spiral (mm²)

f'_c = specified strength of concrete at 28 days, unless another age is specified (MPa)

f_{yh} = specified yield strength of spiral reinforcement (MPa)

Other details of spiral and tie reinforcement shall conform to the provisions of Articles 5.10.6 and 5.10.11.

5.8.3 Sectional Design Model

5.8.3.1 GENERAL

The sectional design model may be used for shear

COMMENTARY

Where columns are pinned to their foundations, a small number of central bars have sometimes been used as a connection between footing and column.

At least eight longitudinal bars are required in a rectangular column with all of those bars restrained against buckling with transverse hoops and/or cross ties. This is necessary to provide proper confining of the core concrete.

C5.7.4.6

Equation (5.7.4.6-1) has historically been used for confining concrete columns. It has also been used for seismic resistant columns. The basis of equation (5.7.4.6-1) is to provide confinement to the core concrete to ensure the axial load carrying capacity of the column is preserved after the cover concrete spalls off. Bridge columns rarely have very high levels of axial loads, and it is for this reason it should not be used for establishing the confinement requirements for seismic resistant columns. The equation, however, is necessary for those columns or piles that may experience pure axial compression loads under construction; for example, pile driving.

C5.8.3.1

In the sectional design approach, the component is

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design where permitted in accordance with the provisions of Article 5.8.1.

In lieu of the methods specified herein, the resistance of members in shear or in shear combined with torsion may be determined by satisfying the conditions of equilibrium and compatibility of strains and by using experimentally verified stress-strain relationships for reinforcement and for diagonally cracked concrete. Where consideration of simultaneous shear in a second direction is warranted, investigation shall be based either on the principles outlined above or on a three-dimensional strut-and-tie model.

5.10.2.2 SEISMIC HOOKS

Seismic hooks shall consist of a 135°-bend, plus an extension of not less than the larger of $10.0 d_b$ or 75 mm. Seismic hooks shall be used for transverse reinforcement in regions of expected plastic hinges. Such hooks and their required locations shall be detailed in the contract documents.

5.10.6 Transverse Reinforcement for Compression Members

5.10.6.1 GENERAL

The provisions of Article 5.10.11 shall also apply to design and detailing in SDR 3, and above.

Transverse reinforcement for compression members may consist of either spirals, hoops or ties.

COMMENTARY

investigated by comparing the factored shear force and the factored shear resistance at a number of sections along its length. Usually this check is made at the tenth points of the span and at locations near the supports.

See Article 5.10.11.4.1c for additional requirements for Seismic Zones 3 and 4.

An appropriate nonlinear finite element analysis or a detailed sectional analysis would satisfy the requirements of this article. More information on appropriate procedures and a computer program that satisfies these requirements are given by Collins and Mitchell (1991). One possible approach to the analysis of biaxial shear and other complex loadings on concrete members is outlined in Rabbat and Collins (1978), and a corresponding computer-aided solution is presented in Rabbat and Collins (1976). A discussion of the effect of biaxial shear on the design of reinforced concrete beam-to-column joints can be found in Paulay and Priestley (1992).

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5.10.6.2 SPIRALS

Spiral reinforcement for compression members other than piles shall consist of one or more evenly spaced continuous spirals of either deformed or plain bar or wire with a minimum diameter of 9.5 mm. The reinforcement shall be arranged so that all primary longitudinal reinforcement is contained on the inside of, and in contact with, the spirals.

The clear spacing between the bars of the spiral shall not be less than either 25 mm or 1.33 times the maximum size of the aggregate. The center-to-center spacing shall not exceed 6.0 times the diameter of the longitudinal bars or 150 mm.

Except as specified in Article 5.10.11.4.1 for SDR 3 and above, spiral reinforcement shall extend from the footing or other support to the level of the lowest horizontal reinforcement of the supported members.

Anchorage of spiral reinforcement shall be provided by 1.5 extra turns of spiral bar or wire at each end of the spiral unit. For SDR 3 and above the extension of transverse reinforcement into connecting members shall meet the requirements of Article 5.10.11.4.3.

Splices in spiral reinforcement may be one of the following:

- Lap splices of 48.0 uncoated bar diameters, 72.0 coated bar diameters, or 48.0 wire diameters; lap splices shall not be used in potential plastic hinge zones;
- Approved mechanical connectors; or
- Approved welded splices.

5.10.6.3 HOOPS AND TIES

C5.10.6.3

In compression members, all longitudinal bars shall be enclosed by perimeter hoops. Ties shall be used to provide lateral restraint to intermediate longitudinal bars within the reinforced concrete cross section.

Transverse hoops and ties that shall be equivalent to:

- No. 10 bars for No. 29 or smaller bars,
- No. 16 bars for No. 36 or larger bars, and
- No. 16 bars for bundled bars.

The spacing of transverse hoops and ties shall not exceed the least dimension of the compression member or 300 mm. Where two or more bars larger than No. 36 are bundled together, the spacing shall not exceed half the least dimension of the member or 150 mm.

Deformed wire, wire rope or welded wire fabric of

The spacing of hoops and ties will generally be considerably closer than. The maximum spacing specified in this article may govern outside potential plastic hinge zones.

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equivalent area may be used instead of bars.

Hoops and ties shall be arranged so that every corner and alternate longitudinal bar has lateral support provided by the corner of a tie having an included angle of not more than 135°. Except as specified herein, no bar shall be farther than 150 mm center-to-center on each side along the tie from such a laterally supported bar.

Where the column design is based on plastic hinging capability, no longitudinal bar shall be farther than 150 mm clear on each side along the tie from such a laterally supported bar. Where the bars are located around the periphery of a circle, a complete circular tie may be used if the splices in the ties are staggered.

Ties shall be located vertically not more than half a tie spacing above the footing or other support and not more than half a tie spacing below the lowest horizontal reinforcement in the supported member.

5.10.11 Provisions for Seismic Design

5.10.11.1 GENERAL

The provisions of these articles shall apply only to the extreme event limit state.

In addition to the other requirements specified in Article 5.10, reinforcing steel shall also conform to the seismic resistance provisions specified herein.

Bridges subjected to Seismic Hazard Levels III & IV (Seismic Hazard Level II and above for the Operational Performance Level) shall satisfy both the requirements specified in Article 5.10.11.3 for SDR 2 and the requirements specified in Article 5.10.11.4 for SDR 3 and above.

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Columns in SDR 3 and above shall be detailed for plastic hinging. The plastic hinge zone is defined in Article 5.10.11.4.1c. Additional requirements for transverse reinforcement for bridges in SDR 4 and above are specified in Article 5.10.11.4.1. Plastic hinging may be used as a design strategy for other extreme events, such as ship collision.

C5.10.11.1

. Bridge Designers working with sites subjected to Seismic Hazard Levels III and IV are encouraged to avail themselves of current research reports and other literature to augment these Specifications.

The 1989 Loma Prieta and 1994 Northridge earthquakes confirmed the vulnerability of columns with inadequate transverse reinforcement and inadequate anchorage of longitudinal reinforcement. Also of concern:

- Lack of adequate reinforcement for positive moments that may occur in the superstructure over monolithic supports when the structure is subjected to longitudinal dynamic loads;
- Lack of adequate shear strength in joints between columns and bent caps under transverse dynamic loads; and
- Inadequate reinforcement for torsion, particularly in outrigger-type bent caps.
- Inadequate transverse reinforcement for shear and restraint against global buckling of longitudinal bars ("bird caging")

The purpose of the additional design requirements of this article is to increase the probability that the design of the components of a bridge are consistent with the principles of "Capacity Design", especially for bridges located in Seismic Hazard Levels II to IV, and that the potential for failures observed in past earthquakes is minimized. The additional column design requirements of this article for bridges located in Seismic Hazard Levels III and IV are to ensure that a column is provided with reasonable ductility and is forced to yield in flexure and that the potential for a shear, compression failure

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due to longitudinal bar buckling, buckling, or loss of anchorage mode of failure is minimized. See also Articles 2.5.6 and 3.10.3.8 for further explanation. The actual ductility demand on a column or pier is a complex function of a number of variables, including:

- Earthquake characteristics, including duration, frequency content and near field (pulse) effects.
- Design force level,
- Periods of vibration of the bridge,
- Shape of the inelastic hysteresis loop of the columns, and hence effective hysteretic damping.
- Elastic damping coefficient,
- Contributions of foundation and soil conditions to structural flexibility, and
- Spread of plasticity (plastic hinge length) in the column.

The damage potential of a column is also related to the ratio of the duration of strong motion shaking to the natural period of vibration of the bridge. This ratio will be an indicator of the low cycle fatigue demand on the concrete column hinge zones.

5.10.11.2 SDR 1

No consideration of seismic forces shall be required for the design of structural components, except for the design of the connection of the superstructure to the substructure as specified in Article 3.10.3.2.

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5.10.11.3 SDR 2

For columns, and pile bents or drilled shafts with in-ground hinging, transverse reinforcement shall be provided as specified by the “Implicit Method” for shear in Article 5.10.11.4.1(c).

For piles the top three-diameters (3D) shall be provided with transverse reinforcement required by the “Implicit Method” in Article 5.10.11.4.1(c). The angles shall be set at $q = \alpha = 35^\circ$ and $\Lambda = 1$.

5.10.11.4 SDR 3 AND ABOVE

5.10.11.4.1 Column Requirements

For the purpose of this article, a vertical support shall be considered to be a column if the ratio of the clear height to the maximum plan dimensions of the support is not less than 2.5. For a flared column, the maximum plan dimension shall be taken at the minimum section of the flare. For supports with a ratio less than 2.5, the provisions for piers of Article 5.10.11.4.2 shall apply.

A pier may be designed as a pier in its strong direction and a column in its weak direction.

The piles of pile bents as well as drilled shaft and caissons shall be regarded as columns for design and detailing purposes.

If architectural flares or other treatments are provided to columns adjacent to potential plastic hinge zones, they shall be either “structurally isolated” in such a way that they do not add to the flexural strength capacity of the columns or the column and adjacent structural elements shall be designed to resist the forces generated by increased flexural strength capacity.

The size of the gap required for structural separation is 0.05 times the distance from the center of the column to the extreme edge of the flare, or 1.5 times the calculated plastic rotation from the pushover analysis times the distance from the center of the column to the extreme edge of the flare. Equation 5.16.1-4 provides an estimate of the reduced plastic hinge length at this location.

COMMENTARY

C5.10.11.3

Bridges in SDR 2 have a reasonable probability of being subjected to seismic forces that will cause yielding of the columns. Thus, it is deemed necessary that columns have some limited ductility capacity, although it is recognized that the ductility demand will not be as great as for columns of bridges in SDR 3 and above. The most important provision is to ensure additional shear capacity is provided. This is to ensure dependable shear strength is maintained when the shear strength degrades under cyclic loading and the concrete contribution (V_c) vanishes. Another important region is the potential plastic hinge zones at the top of piles in pile foundations that may be subjected to hinging. This is to ensure some level of ductility is provided by the transverse reinforcement in the event of a partial mechanism forming in the foundation. This requirement is necessary because in SDR 2 full capacity design is not needed, but ductility must be assured.

C5.10.11.4.1

The definition of a column in this article is provided as a guideline to differentiate between the additional design requirements for a wall-type pier and the requirements for a column. If a column or pier is above or below the recommended criterion, it may be considered to be a column or a pier, provided that the appropriate R-Factor of Article 3.10.3.7 and the appropriate requirements of either Articles 5.10.11.4.1 or 5.10.11.4.2 are used. For columns with an aspect ratio less than 2.5, the forces resulting from plastic hinging will generally exceed the elastic design forces; consequently, the forces of Article 5.10.11.4.2 would not be applicable.

Certain oversize columns exist for architectural/aesthetic reasons. These columns, if fully reinforced, place excessive moment and/or shear demands on adjoining elements. The designer should strive to “structurally isolate” those architectural elements that do not form part of the primary energy dissipation system that are located either within or in close proximity to plastic hinge zones. Nevertheless, the architectural elements must remain serviceable throughout the life of the structure. For this reason, minimum steel for temperature and shrinkage should be provided. Note that, when architectural flares are not isolated, Article 3.10.3.8 requires that the design shear force for a flared column be the worst case calculated using the overstrength moment of the oversized flare or the shear generated by

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For oversized or architectural portions of piers or columns, minimum longitudinal and transverse reinforcement that complies with temperature and shrinkage requirements elsewhere in these specifications shall be provided.

5.10.11.4.1a Longitudinal Reinforcement

The area of longitudinal reinforcement shall not be less than 0.008 or more than 0.04 times the gross cross-section area A_g .

5.10.11.4.1b Flexural Resistance

The biaxial strength of columns shall not be less than that required for flexure, as specified in Article 3.10.3.7. The column shall be investigated for both extreme load cases, as specified in Article 3.10.2.4, at the extreme event limit state. The resistance factors of Article 5.5.4.2 shall be replaced for both spirally and tied reinforcement columns by the value $\phi = 1.0$, providing other member actions have been designed in accordance with the principles of capacity design.

5.10.11.4.1c Column Shear and Transverse Reinforcement

Provision of transverse reinforcement for shear shall be determined by one of the following two methods: implicit approach or an explicit approach. The implicit approach may be used for all Seismic Hazard Levels. However, for Seismic Hazard Level IV with a two-step design (SDAP E), the shear strength shall be checked using the explicit approach.

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a plastic hinge at the bottom of the flare.

C5.10.11.4.1a

This requirement is intended to apply to the full section of the columns. The 0.8 percent lower limit on the column reinforcement reflects the traditional concern for the effect of time-dependent deformations as well as the desire to avoid a sizable difference between the flexural cracking and yield moments. The 4 percent maximum ratio is to avoid congestion and extensive shrinkage cracking and to permit anchorage of the longitudinal steel, but most importantly, the less the amount of longitudinal reinforcement, the greater the ductility of the column. Note that Section 3.10.3.8 requires that the design shear force for a flared column be calculated using the worst case of the moment of the oversized flare or the shear generated by a plastic hinge at the bottom of the flare.

C5.10.11.4.1b

Columns are required to be designed biaxially and to be investigated for both the minimum and maximum axial forces. Resistance factors of unity may be used wherever moments and axial loads are derived from a plastic mechanism.

C5.10.11.4.1c

The implicit method is conservative and is most appropriate when a shear demand has not been calculated, e.g., SDR 2 and piles. The explicit method should result in less reinforcement and is recommended if the shear demand is available.

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Method 1: Implicit Shear Detailing Approach

(a) In potential plastic hinge zones (Article 3.10.3.9)

- For circular sections
- For rectangular sections

$$r_v = K_{shape} \Lambda \frac{r_t}{f} \frac{f_{su}}{f_{yh}} \frac{A_g}{A_{cc}} \tan \alpha \tan \phi \quad (5.10.11.4.1c-1)$$

in which

r_v = ratio of transverse reinforcement
given by either (5.10.11.4.1-2) or (5.10.11.4.1-3).

- for rectangular sections

$$r_v = \frac{A_{sh}}{b_w s} \quad (5.10.11.4.1c-2)$$

and

- for circular columns

$$r_v = \frac{r_s}{2} = \frac{2A_{bh}}{sD''} \quad (5.10.11.4.1c-3)$$

where

A_{sh} = the area of the transverse hoops and cross-ties
transverse to the axis of bending

A_{bh} = the area of one spiral bar or hoop in a circular section

S = the center-to-center spacing of hoopsets or the pitch
the spiral steel

b_w = the web width resisting shear in a rectangular section

D'' = spiral diameter in a circular section

The terms in equation (5.10.11.4.1-1) are defined below:

K_{shape} = factor that depends on the shape of the section
and shall be taken as

- for circular sections $K_{shape} = 0.32$
- for square sections with 25 percent of the longitudinal reinforcement placed in each face $K_{shape} = 0.375$
- for walls with strong axis bending $K_{shape} = 0.25$
- for walls with weak axis bending $K_{shape} = 0.5$

Λ = fixity factor,

$\Lambda = 1$ fixed-free (pinned one end)

$\Lambda = 2$ fixed-fixed

COMMENTARY

This implicit shear detailing approach assumes that

$$fV_u = V_c + V_p + V_s \geq \frac{\Lambda M_p^o}{H_c}$$

in which $V_c = 0$ (the contribution of shear carried by the concrete tensile section). This shear demand at plastic overstrength (M_p^o) is implicitly resisted by arch action

(V_p) which is carried by a corner-to-corner diagonal

strut in the concrete, and truss action (V_s) which is

resisted by the transverse reinforcement. The

overstrength demand for the transverse steel comes

solely from the presence of the longitudinal

reinforcement. It is for this reason the transverse steel

(r_v) is directly proportional to the longitudinal steel (r_t).

Thus, if steel congestion results for a chosen column

size, one viable solution is to enlarge the column and

reduce the longitudinal steel volume.

For a derivation of the implicit shear detailing approach, refer to the recent research by Dutta and Mander (1998).

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f_{su} = the ultimate tensile stress of the longitudinal reinforcement. If f_{su} is not available from coupon tests, then it shall be assumed that $f_{su} = 1.5 f_y$. For SDR 2 f_{su} may be taken as f_y .

q = angle of the principal crack plane given by

$$\tan q = \left(\frac{1.6 r_v A_v}{\Lambda r_t A_g} \right)^{0.25} \quad (5.10.11.4.1c-4)$$

with $q \geq 25^\circ$ and $q \geq a$

a = geometric aspect ratio angle given by

$$\tan a = \frac{D'}{L}$$

where D' = pitch circle diameter of the longitudinal reinforcement in a circular section, or the distance between the outer layers of the longitudinal steel in other section shapes.

A_v = shear area of concrete which may be taken as $0.8A_g$ for a circular section, or $A_v = b_w d$ for a rectangular section.

The spacing of the spirals or hoopsets shall not exceed 250mm or one-half the member width.

(b) Outside the Potential Plastic Hinge Zone

Outside the potential plastic hinge zone (Article 3.10.3.9) the transverse reinforcement may be reduced to account for some contribution of the concrete in shear resistance. The required amount of transverse reinforcement, outside the potential plastic hinge zone r_v^* , shall be given by

$$r_v^* = r_v - 0.17 \frac{\sqrt{f'_c}}{f_{yh}} \quad (5.10.11.4.1c-5)$$

where r_v = the steel provided in the potential plastic hinge zone.

r_v^* shall not be less than the minimum amount of transverse reinforcement required elsewhere in these specifications based on non-seismic requirements.

COMMENTARY

This clause assumes the concrete is capable of sustaining a concrete stress of $v_c = 0.17 \sqrt{f'_c} \cot q$. The basis of equation (5.10.11.4.1c-5) follows

Shear in end zones = shear outside end zones

$$V_s = V_s^* + V_c$$

where V_s^* = shear carried by the transverse steel outside the plastic hinge zone. Expanding both sides gives

$$r_v A_v f_{yh} \cot q = r_v^* A_v f_{yh} \cot q + 0.17 \sqrt{f'_c} \cot q A_v$$

Solving for r_v^* , the required amount of transverse reinforcement outside the potential plastic hinge zone, gives equation (5.10.11.4.1c-5). Note that if r_v^* is negative, this means the concrete alone is theoretically adequate for strength, although the minimum steel is still required if this occurs.

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Method 2: Explicit Approach

The design shear force, V_u , on each principal axis of each column and pile bent shall be determined from considerations of the flexural overstrength being developed at the most probable locations of critical sections within the member, with a rational combination of the most adverse end moments.

In the end regions, the shear resisting mechanism shall be assumed to be provided by a combination of truss (V_s) and arch (strut) action (V_p) such that

$$fV_s \geq V_u - (V_p + V_c) \quad (5.10.11.4.1c-6)$$

where V_p = the contribution due to arch action given by

$$V_p = \frac{\Lambda}{2} P_e \tan \alpha \quad (5.10.11.4.1c-7)$$

where

$$\tan \alpha = \frac{D'}{L} \quad (5.10.11.4.1c-8)$$

P_e = compressive axial force including seismic effects

D' = pitch circle diameter of the longitudinal reinforcement in a circular column, or the distance between the outermost layers of bars in a rectangular column

L = column length

Λ = fixity factor defined above

V_c = the tensile contribution of the concrete towards shear resistance. At large displacement ductilities only a minimal contribution can be assigned as follows

$$V_c = 0.05 \sqrt{f'_c} b_w d \quad (5.10.11.4.1c-9)$$

Outside the plastic hinge zone

$$V_c = 0.17 \sqrt{f'_c} b_w d \quad (5.10.11.4.1c-10)$$

where

f'_c = concrete strength in MPa,

b_w = web width of the section, and

d = effective depth

V_s = the contribution of shear resistance provided by transverse reinforcement given by:

The shear strength model is based on the concept that the total shear strength is given by the following design equation:

$$V_u < V_s + V_p + V_c$$

The concrete tensile contribution to shear, V_c , is assumed to significantly diminish under high ductilities and cyclic loading.

The requirements of this article are intended to avoid column shear failure by using the principles of "capacity protection". The design shear force is specified as a result of the actual longitudinal steel provided, regardless of the design forces. This requirement is necessary because of the potential for superstructure collapse if a column fails in shear.

A column may yield in either the longitudinal or transverse direction. The shear force corresponding to the maximum shear developed in either direction for noncircular columns should be used for the determination of the transverse reinforcement.

For a noncircular pile, this provision may be applied by substituting the larger cross-sectional dimension for the diameter.

As a starting point for initial design, assume $q = 35^\circ$. The actual crack angle should be estimated based on the provided transverse reinforcement using equation (5.10.11.4.1c-14). From this the shear strength should be checked based on the provided steel.

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(i) for circular columns:

$$V_s = \frac{p}{2} \frac{A_{bh}}{s} f_{yh} D'' \cot q \quad (5.10.11.4.1c-12)$$

(ii) for rectangular sections

$$V_s = \frac{A_v}{s} f_{yh} D'' \cot q \quad (5.10.11.4.1c-13)$$

where

A_{bh} = area of one circular hoop/spiral reinforcing bar

A_{sh} = total area of transverse reinforcement in one layer in the direction of the shear force

f_{yh} = transverse reinforcement yield stress

D'' = centerline section diameter/width of the perimeter spiral/hoops

q = principal crack angle/plane calculated as follows:

$$\tan ? = \left(\frac{1.6 r_v A_v}{\Lambda r_t A_g} \right)^{0.25} \geq \tan \alpha \quad (5.10.11.4.1c-14)$$

where

r_v = volumetric ratio of shear reinforcement given by

$$r_v = \frac{A_{sh}}{b_w s} \quad \text{for rectangular section}$$

$$r_v = \frac{r_s}{2} = \frac{2A_{bh}}{sD''} \quad \text{for circular columns.}$$

and A_v = shear area of concrete which may be taken as $0.8A_v$ for a circular section, or $A_v = b_w d$ for a rectangular section.

Extent of Shear Steel

Shear steel shall be provided in all potential plastic hinge zones as defined in Article 3.10.3.9.

The Explicit shear approach defined herein is similar to the shear model of Priestley, Verma and Xiao (1994). Based on a survey of empirical observations, Priestley et al. recommended that the crack angle be taken as $q = 35^\circ$ and 30° for design and analysis, respectively.

The crack angle computed in equation (5.10.11.4.1c-14) is more general. The associated theory is based on research by Kim and Mander (1999). In their approach an energy minimization of shear-flexure deflections was used on a truss model of a beam-column element to find an analytical expression for the crack angle. This theoretical crack angle equation was then validated against a wide variety of experimental observations.

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5.10.11.4.1d Transverse Reinforcement for Confinement at Plastic Hinges

The core concrete of columns and pile bents shall be confined by transverse reinforcement in the expected plastic hinge regions. The spacing shall be taken as specified in Article 5.10.11.4.1f.

For a circular column, the volumetric ratio of spiral reinforcement, r_s , shall not be less than:

a) for circular sections

$$r_s = 0.008 \frac{f'_c}{U_{sf}} \left[12 \left(\frac{P_e}{f'_c A_g} + r_t \frac{f_y}{f'_c} \right)^2 \left(\frac{A_g}{A_{cc}} \right)^2 - 1 \right] \quad (5.10.11.4.1d-1)$$

b) for rectangular sections

$$\frac{A_{sh}}{sB} + \frac{A'_{sh}}{sD} = 0.008 \frac{f'_c}{U_{sf}} \left[15 \left(\frac{P_e}{f'_c A_g} + r_t \frac{f_y}{f'_c} \right)^2 \left(\frac{A_g}{A_{cc}} \right)^2 - 1 \right] \quad (5.10.11.4.1d-2)$$

where:

f'_c = specified compressive strength of concrete at 28 days, unless another age is specified (MPa)

f_y = yield strength of reinforcing bars (MPa)

P_e = factored axial load (N) including seismic effects

U_{sf} = strain energy capacity (modulus of toughness) of the transverse reinforcement = 110 MPa.

$r_s = \frac{4A_b}{D's}$ = ratio of transverse reinforcement where

D' = center-to-center diameter of perimeter hoop for spiral.

Within plastic hinge zones, splices in spiral reinforcement shall be made by full-welded splices or by full-mechanical connections.

s = vertical spacing of hoops, not exceeding 100 mm (mm)

COMMENTARY

C5.10.11.4.1d

Plastic hinge regions are generally located at the top and bottom of columns and pile bents. should govern; these requirements are not in addition to those of Article 5.10.11.4.1c.

These equations ensure that the concrete is adequately confined so that the transverse hoops will not prematurely fracture as a result of the plastic work done on the critical column section. For typical bridge columns with low levels of axial load, these equations rarely govern, but must be checked. The equations were developed by Dutta and Mander (1998), with experiments demonstrating that they work well for both regular mild steel spirals as well as high strength steel in the form of wire rope (see Dutta et al, 1999). Note the latter should not be used for hoops, ties or stirrups with bent hooks.

Loss of concrete cover in the plastic hinge zone as a result of spalling requires careful detailing of the confining steel. It is clearly inadequate to simply lap the spiral reinforcement. If the concrete cover spalls, the spiral will be able to unwind. Similarly, rectangular hoops should be anchored by bending ends back into the core.

Figures C5.10.11.4.1d-1 through C5.10.11.4.1d-4 illustrate the use of Equations 5.10.11.4.1d-1 and -2. The required total area of hoop reinforcement should be determined for both principal axes of a rectangular or oblong column, and the greater value should be used.

While these Specifications allow the use of either spirals, hoops or ties for transverse column

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A_{cc} = area of column core concrete, measured to the centerline of the perimeter hoop or spiral (mm²)

A_g = gross area of column (mm²)

A_{sh} = total area of transverse reinforcement in the direction of the applied shear

A'_{sh} = total area of transverse reinforcement perpendicular to direction of the applied shear

B'' & D''
= core dimension of tied column in the direction under consideration (mm)

Transverse hoop reinforcement may be provided by single or overlapping hoops. Cross-ties having the same bar size as the hoop may be used. Each end of the cross-tie shall engage a peripheral longitudinal reinforcing bar. All cross-ties shall have seismic hooks as specified in Article 5.10.2.2.

Transverse reinforcement meeting the following requirements shall be considered to be a cross-tie:

- The bar shall be a continuous bar having a hook of not less than 135°, with an extension of not less than six diameters but not less than 75 mm at one end and a hook of not less than 90° with an extension not less than six diameters at the other end for SDR 2 and above.
- Hooks shall engage all peripheral longitudinal bars.
- 90° hooks of two successive cross-ties engaging the same longitudinal bars shall be alternated end-for-end are permitted for SDR 1 and 2.

Transverse reinforcement meeting the following requirements shall be considered to be a hoop:

- The bar shall be closed tie or continuously wound tie.
- A closed tie may be made up of several reinforcing elements with 135° hooks having a six diameter but not less than a 75 mm extension at each end.
- A continuously wound tie shall have at each end a 135° hook with a six diameter but not less than a 75 mm extension that engages the longitudinal reinforcement.

COMMENTARY

reinforcement, the use of spirals is recommended as the more effective and economical solution. Where more than one spiral cage is used to confine an oblong column core, the spirals should be interlocked with longitudinal bars as shown in Figure C5.10.11.4.1.d-3. Spacing of longitudinal bars of a maximum of 200 mm center-to-center is also recommended to help confine the column core.

Examples of transverse column reinforcement are shown herein.

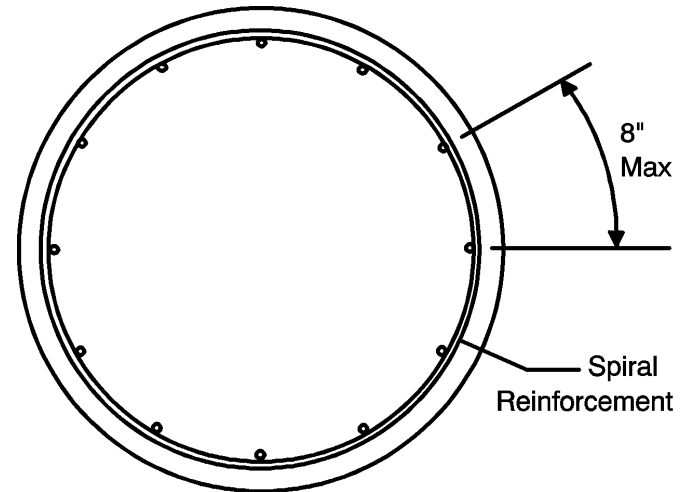


Figure C5.10.11.4.1d-1 - Single Spiral

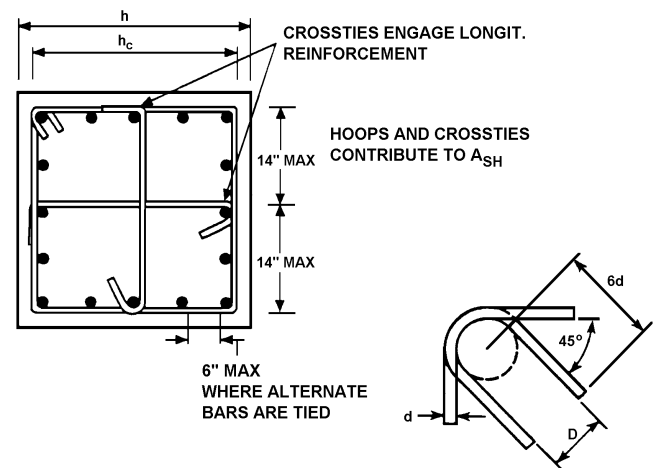


Figure C5.10.11.4.1d-2 - Column Tie Details

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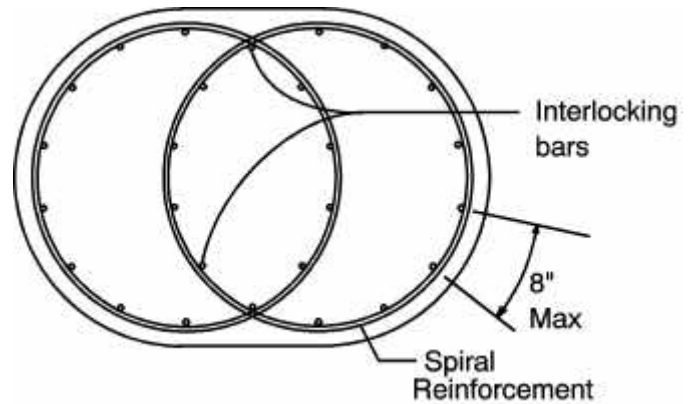


Figure C5.10.11.4.1d-3 - Column Interlocking Spiral Details

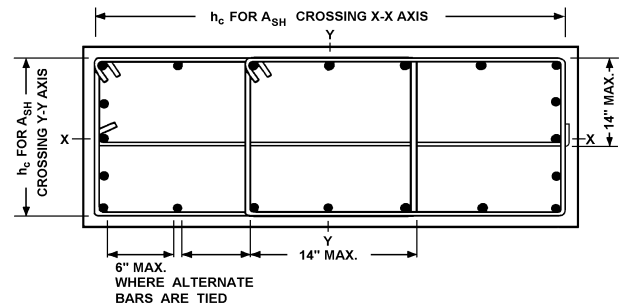


Figure C5.10.11.4.1d-4 - Column Tie Details

5.10.11.4.1e Transverse Reinforcement for Longitudinal Bar Restraint in Plastic Hinges

The longitudinal reinforcement in the potential plastic hinge zone shall be restrained by antibuckling steel as follows:

- (i) $s \leq 6d_b$
- (ii) For circular sections confined by spirals or circular hoops

$$r_s = 0.016 \left(\frac{D}{s} \right) \left(\frac{s}{d_b} \right) r_t \frac{f_y}{f_{yh}} \quad (5.10.11.4.1e-1)$$

(iii) for rectangular sections confined by transverse hoops and/or cross ties the area of the cross tie or hoop legs (A_{bh}) shall be:

$$A_{bh} = 0.09 A_b \frac{f_y}{f_{yh}} \quad (5.10.11.4.1e-2)$$

C5.10.11.4.1e

Longitudinal reinforcing bars in potential plastic hinge zones may be highly strained in compression to the extent they may buckle. Buckling may either be

- (a) local between two successive hoop sets or spirals, or
- (b) global and extend over several hoop sets or spirals.

Criteria (i) and (ii) are required to ensure the yield capacity of the longitudinal reinforcement is maintained. This is a life-safety requirement. If global buckling of the longitudinal reinforcing is to be inhibited to ensure post-earthquake repairability, then it is recommended the following be adopted:

$$r_s = 0.024 \left(\frac{D}{d_b} \right) r_t \frac{f_y}{f_{yh}}$$

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where

$$r_s = \text{ratio of transverse reinforcement} \left(r_s = \frac{4A_{bh}}{sD'} \right)$$

D = diameter of circular column

d_b = diameter of longitudinal reinforcing bars being restrained by circular hoop or spiral

A_b = area of longitudinal reinforcing bars being restrained by rectilinear hoops and/or cross ties

A_{bh} = bar area of the transverse hoops or ties restraining the longitudinal steel

r_t = volumetric ratio of longitudinal reinforcement

f_y = yield stress of the longitudinal reinforcement

f_{yh} = yield stress of the transverse reinforcing bars

5.10.11.4.1f Spacing of Transverse Reinforcement for Confinement and Longitudinal Bar Restraint

Transverse reinforcement for confinement and longitudinal bar retention (Articles 5.10.11.4.1d and 5.10.11.4.1e shall be provided at all plastic hinge zones as defined in Article 3.10.3.9 except that the requirements of Article 5.10.11.4.1e need not apply to the pile length from 3D to 10D below the pile cap.

The spacing of transverse reinforcement shall not be less than:

$$\frac{M}{V} \left(1 - \frac{M_y}{M_{po}} \right) \quad (5.10.11.4.1f-1)$$

The spacing of transverse reinforcement shall not exceed one-quarter of the minimum member dimension or 150 mm center-to-center.

5.10.11.4.1g Splices

The provisions of Article 5.11.5 shall apply for the design of splices.

Lap splices in longitudinal reinforcement shall be used only within the center half of column height, and the splice length shall not be less than 400mm or 60.0-bar diameters.

The spacing of the transverse reinforcement over the length of the splice shall not exceed one-quarter of the minimum member dimension.

Full-welded or full-mechanical connection splices conforming to Article 5.11.5 may be used, provided that not more than alternate bars in each layer of longitudinal reinforcement are spliced at a section, and the distance between splices of adjacent bars is greater than 450mm measured along the longitudinal axis of the column.

COMMENTARY

and

$$A_{bh} = 0.25 A_b \frac{f_y}{f_{yh}}$$

Criteria (ii) may lead to congestion of hoops/spirals in circular columns with large columns of longitudinal reinforcement. One way to overcome this is to use wire rope or prestressing strand as transverse reinforcement with a high yield strain.

An alternate approach to relieve transverse reinforcement congestion arising from these antibuckling requirements is to use two concentric rings of longitudinal steel. The antibuckling requirements need only apply to the outer ring of longitudinal bars.

This requirement ensures all inelastic portions of the column are protected by confining steel.

C5.10.11.4.1g

It is often desirable to lap longitudinal reinforcement with dowels at the column base. This is undesirable for seismic performance because:

- The splice occurs in a potential plastic hinge region where requirements for bond is critical, and
- Lapping the main reinforcement will tend to concentrate plastic deformation close to the base and reduce the effective plastic hinge length as a result of stiffening of the column over the lapping region. This may result in a severe local curvature demand.

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5.10.11.4.1h Flexural Overstrength

Article 3.10.3.8 provides several alternate methods for calculating the flexural moment overstrength capacity (M_{po}) for columns/ piles/ drilled shafts that are part of the ERS. The plastic moment-axial load interaction formula developed by Mander, Dutta and Goel (1997) may be used to calculate the overstrength moment of a column or drilled shaft:

COMMENTARY

C5.10.11.4.1h Flexural Overstrength

The simplified method for calculating an overstrength moment-axial load interaction diagram (Mander, et. al, 1997) involves a parabolic curve fit to (M_{bo} , P_b) and (0, P_{to}) given by Equation C5.10.11.4.1h-1.

$$\frac{M_{po}}{f'_c A_g D} = \left(\frac{M_{bo}}{f'_c A_g D} \right) \left[1 - \left(\frac{\frac{P_e}{f'_c A_g} - \frac{P_b}{f'_c A_g}}{\frac{P_{to}}{f'_c A_g} - \frac{P_b}{f'_c A_g}} \right)^2 \right] \quad (C5.10.11.4.1h-1)$$

where:

$\frac{P_e}{f'_c A_g}$ = axial stress ratio on the column based on gravity load and seismic (framing) actions

$\frac{P_{to}}{f'_c A_g} = -r_t \frac{f_{su}}{f'_c}$ = normalized axial tensile capacity of the column

$\frac{P_b}{f'_c A_g} = 0.425 b_1$ = normalized axial load capacity at the maximum nominal (balanced) moment on the section where b_1 = stress block factor (≤ 0.85)

$$\frac{M_{bo}}{f'_c A_g D} = \left(K_{shape} r_t \frac{f_{su}}{f'_c} \frac{D'}{D} + \frac{P_b}{f'_c A_g} \frac{1 - k_o}{2} \right) \quad (C5.10.11.4.1h-2)$$

D' = pitch circle diameter of the reinforcement in a circular section, or the out-to-out dimension of the reinforcement in a rectangular section, this generally may be assumed as $D' = 0.8D$.

f_{su} = ultimate tensile strength of the longitudinal reinforcement.

K_{shape} should be taken defined in Article 5.10.11.4.1c.

k_o = a factor related to the stress block centroid which should be taken as 0.6 and 0.5 for circular and rectangular sections, respectively.

5.10.11.4.2 Limited Ductility Requirements for Wall-Type Piers

C5.10.11.4.2

These limited ductility provisions, herein specified, shall apply to the design for the strong direction of a pier.

The requirements of this article are based on limited data available on the behavior of piers in the inelastic

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COMMENTARY

Providing ductile detailing is used, either direction of a pier may be designed as a column conforming to the provisions of Article 5.10.11.4.1, with the response modification factor for columns used to determine the design forces. If the pier is not designed as a column in either direction, then the limitations for factored shear resistance herein specified shall apply.

The minimum reinforcement ratio, both horizontally, r_h , and vertically, r_v , in any pier shall not be less than 0.0025. The vertical reinforcement ratio shall not be less than the horizontal reinforcement ratio.

Reinforcement spacing, either horizontally or vertically, shall not exceed 450 mm. The reinforcement required for shear shall be continuous and shall be distributed uniformly.

The factored shear resistance, V_r , in the pier shall be taken as the lesser of:

$$V_r = 0.253\sqrt{f_c'}bd \quad (5.10.11.4.2-1)$$

$$V_r = fV_n \quad (5.10.11.4.2-2)$$

for which:

$$V_n = \left[0.063\sqrt{f_c'} + r_h\gamma_y \right] bd \quad (5.10.11.4.2-3)$$

Horizontal and vertical layers of reinforcement should be provided on each face of a pier. Splices in horizontal pier reinforcement shall be staggered and splices in the two layers shall not occur at the same location.

5.12 MOMENT-RESISTING CONNECTION BETWEEN MEMBERS (COLUMN/BEAM JOINTS AND COLUMN/FOOTING JOINTS)

5.12.1 Implicit Approach: Direct Design

C5.12.1

Flexural reinforcement in continuous, restrained, or cantilever members or in any member of a rigid frame shall be detailed to provide continuity of reinforcement at intersections with other members to develop the nominal moment resistance of the joint.

In SDR 3 and above, joints shall be detailed to resist shears resulting from horizontal loads through the joint.

Transverse reinforcement in cap beam-to-column or pile cap-to-column joints should consist of the greater of:

- (a) Confinement reinforcement given by clause 5.10.11.4.1d;
- (b) Antibuckling reinforcement given by clause

range. Consequently, the R-Factor of 2.0 for piers is based on the assumption of minimal inelastic behavior.

The requirement that $r_v \geq r_h$ is intended to avoid the possibility of having inadequate web reinforcement in piers which are short in comparison to their height. Splices should be staggered in an effort to avoid weak sections.

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COMMENTARY

5.10.11.4.1e; this clause can be waived if the longitudinal bars framing into the joint is surrounded by sufficient concrete to inhibit bar buckling. For the purpose of waiving this clause cover to the longitudinal steel shall be taken as the greater of 150 mm or 6 longitudinal bar diameters.

- (c) Shear reinforcement given by clause 5.10.11.4.1c where the principal crack angle q is given by the aspect ratio of the member and is defined by the joint dimensions as follows

$$\tan q = \tan a = \frac{D}{H_c}$$

where

D = width or diameter of the column framing into the joint

H_c = the height of the cap beam/joint. Thus the joint shear horizontal (transverse) reinforcement is given by:

For circular columns with spirals or circular hoops

$$r_s \geq 0.76 \frac{r_t}{f} \frac{f_{su}}{f_{yh}} \frac{A_g}{A_{cc}} \tan^2 a. \quad (5.12.1-1)$$

for rectangular sections with rectilinear hoops and/or ties

$$\frac{A_{sh}}{sB''} \geq 1.2 \frac{B'/D' + 0.5}{2B'/D' + 2} \frac{r_t}{f} \frac{f_{su}}{f_{yh}} \frac{A_g}{A_{cc}} \tan^2 a \quad (5.12.1-2)$$

If the above equations lead to congested steel placement details, then alternative details may be adopted through the use of rational strut and tie models as given in clause 5.12.2

where

$$r_s = \text{ratio of transverse hoops/spirals} \left(r_s = \frac{4A_{bh}}{sD'} \right)$$

r_t = ratio of longitudinal reinforcement area to gross area of section

A_{sh} = area of transverse reinforcement in the direction of the applied shear

f_{su} = yield strength of transverse reinforcement

A_g = gross area of section

A_{cc} = confined core area (take as $0.8A_g$ for a circular section)

f = resistance factor for seismic shear (0.85)

Shear steel will often govern in connections due to the increased shear demand at flexural overstrength arising from a smaller shear span within the joint compared to the columns framing into the connection. If this results in considerable congestion, particularly when large volumes of longitudinal steel exist, then design method 2 might give some relief. This is because methods 2 permits some of the joint reinforcement to be placed outside the joint in the adjacent cap beam.

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5.12.2 Method 2: Explicit Detailed Approach

5.12.2.1 DESIGN FORCES AND APPLIED STRESSES

Moment-resisting connections between members shall be designed to transmit the maximum forces applied by the connected members. Connection forces shall be based on the assumption of maximum plastic moment.

Forces acting on the boundaries of connections shall be considered to be transmitted by mechanisms involving appropriate contributions by concrete and reinforcement actions. Mechanisms shall be based on an analysis of force-transfer within the connection, and shall be supported by relevant test results.

Principal stresses in any vertical plane within a connection shall be calculated in accordance with Eq. (5.12.2.1-1) and (5.12.2.1-2)

Principal tension stress is given by:

$$p_t = \frac{(f_h + f_v)}{2} - \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \quad (5.12.2.1-1)$$

Principal compression stress is given by:

$$p_c = \frac{(f_h + f_v)}{2} + \sqrt{\left(\frac{f_h - f_v}{2}\right)^2 + v_{hv}^2} \quad (5.12.2.1-2)$$

where

f_h and f_v = the average axial stresses in the horizontal and vertical directions within the plane of the connection under consideration (compression stress positive) and

v_{hv} = the average shear stress within the plane of the connection.

COMMENTARY

C5.12.2

The designer may consider the following means to improve constructability:

- prestressing the joint as a means of reducing reinforcing steel,
- placing vertical shear reinforcement within the joint and/or in the cap beam adjacent to the joint region.

C5.12.2.1 DESIGN FORCES AND APPLIED STRESSES

The stresses f_h and f_v in Eq. 5.12.2.1-1 and 5.12.2.1-2 are nominal compression stresses in the horizontal and vertical directions, respectively. In a typical joint f_v is provided by the column axial force P_e . An average stress at midheight of the cap beam, or mid-depth of the footing, should be used, assuming a 45-degree spread away from the boundaries of the column in all directions. The horizontal axial stress f_h is based on the mean axial force at the center of the joint, including effects of cap beam prestress, if present.

The joint shear stress v_{hv} can be estimated with adequate accuracy from the expression

$$v_{hv} = \frac{M_p}{h_b h_c b_{ji}} \quad (C5.12.2.1-1)$$

where

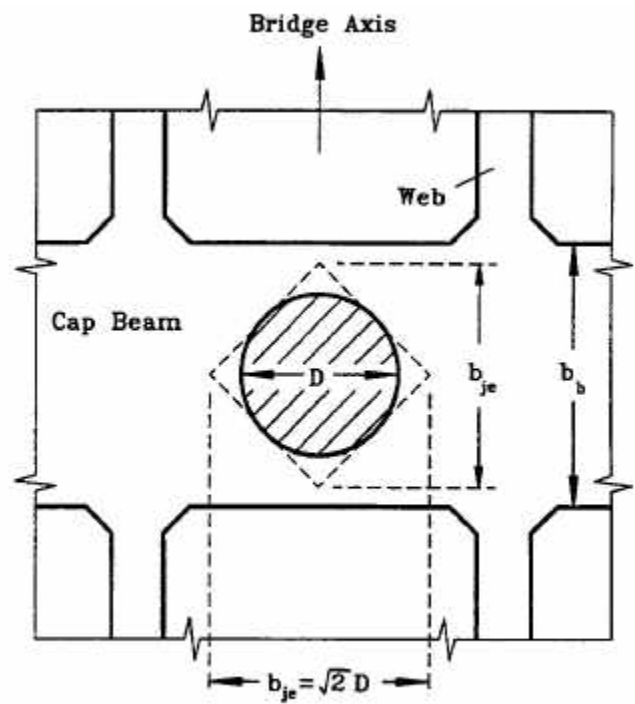
M_p = the maximum plastic moment

h_b = the cap beam or footing depth

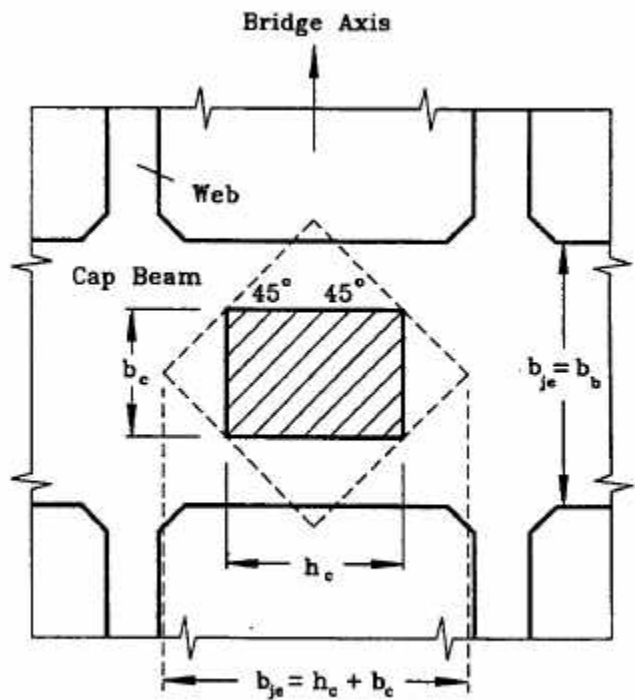
h_c = the column lateral dimension in the direction considered (i.e., $h_c = D$ for a circular column)

b_{ji} = the effective joint width, found using a 45-degree spread from the column boundaries.

Figures 5.12.1 (Priestley, Seible and Calvi, 1996) clarify the quantities to be used in this calculation.



(a) Circular Column



(b) Rectangular Column

Figure C5.12.1 Effective joint width for shear stress calculations.

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5.12.2.2 MINIMUM REQUIRED HORIZONTAL REINFORCEMENT

When the principal tension stress is less than $P_t = 0.29\sqrt{f'_c}$ MPa, the minimum amount of horizontal joint shear reinforcement to be provided shall be capable of transferring 50 percent of the cracking stress resolved to the horizontal direction. For circular columns, or columns with intersecting spirals, the volumetric ratio of transverse reinforcement in the form of spirals or circular hoops to be continued into the cap or footing shall not be less than

$$r_s = \frac{0.29\sqrt{f'_c}}{f_{yh}} \quad (5.12.2.2-1)$$

where

f_{yh} = yield stress of horizontal hoop/tie reinforcement in the joint.

5.12.2.3 Maximum Allowable Compression Stresses

Principal compression stress in a connection, calculated in accordance with Eq. (5.12.2.1-2) shall not exceed

$$p_c = 0.25f'_c.$$

5.12.3 Reinforcement for Joint Force Transfer

5.12.3.1 ACCEPTABLE REINFORCEMENT DETAILS

Where the magnitude of principal tension stress values (calculated in accordance with Eq. 5.12.2.1-1), exceed

$r_t = 0.29\sqrt{f'_c}$ MPa, vertical and horizontal joint reinforcement, placed in accordance with Articles 5.12.3.2, 5.12.3.3 and 5.12.3.4 is required.

5.12.3.2 VERTICAL REINFORCEMENT

5.12.3.2.1 Stirrups

On each side of the column or pier wall, the beam member that is subject to bending forces shall have vertical stirrups, with a total area $A_{jv} = 0.16A_{st}$ located within a distance $0.5D$ or $0.5h$ from the column or pier wall face. These vertical stirrups shall be distributed over

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C5.12.2.2 MAXIMUM REQUIRED HORIZONTAL REINFORCEMENT

The need to include spiral reinforcement to aid in joint force transfer has become obvious as a result of the poor performance of moment-resisting connections in recent earthquakes and in large-scale tests. Theoretical consideration (Priestley, Seible and Calvi, 1996), and experimental observation (Sriharan and Priestley et al., 1994a); Sriharan and Priestley, 1994b; Priestley et al. 1992), indicate that unless the nominal principal tension stress in the connection (joint region) exceeds $0.29\sqrt{f'_c}$ MPa, diagonal cracking in the connection will be minimal. Equation (5.12.2.2-1) requires placement of sufficient hoop reinforcement to carry 50 percent of the tensile force at $0.29\sqrt{f'_c}$ MPa, nominal tensile stress, resolved into the horizontal plane. This is minimum level of reinforcement.

C5.12.2.3 Maximum Allowable Compression Stresses

The principal compression stress in a connection is limited to $0.25f'_c$. This limits the shear stress to less than $0.25f'_c$. It is felt that the level of nominal principal compression stress is a better indicator of propensity for joint crushing than is the joint shear stress.

C5.12.3 Reinforcement for Joint Force Transfer

C5.12.3.1 ACCEPTABLE REINFORCEMENT DETAILS

A "rational" design is required for joint reinforcement when principal tension stress levels exceed $0.29\sqrt{f'_c}$ MPa. The amounts of reinforcement required are based on the mechanism shown in Figure C5.12.2 which primarily uses external reinforcement for joint resistance to reduce joint congestion.

C5.12.3.2 VERTICAL REINFORCEMENT

C5.12.3.2.1 Stirrups

Figure C5.12.2 is intended to clarify this clause. A_{ST} is the total area of column reinforcement anchored in the joint. Reinforcement A_{jv} is required to provide the tie force T_s resisting the vertical component of strut D2 in Figure C5.12.2. This reinforcement should be placed

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a width not exceeding $2D$.

where

A_{st} = total area of longitudinal steel

D = diameter of circular column

h = depth of rectangular column

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close to the column cage for maximum efficiency. In addition, it will be recognized that the cap beam top reinforcement or footing bottom reinforcement may have severe bond demands, since stress levels may change from close to tensile yield on one side of the joint to significant levels of compression stress on the other side. The required $0.08 A_{ST}$ vertical ties inside the joint are intended to help provide this bond transfer by clamping the cap-beam rebar across possible splitting cracks. Similar restraint may be required for superstructure top longitudinal rebar.

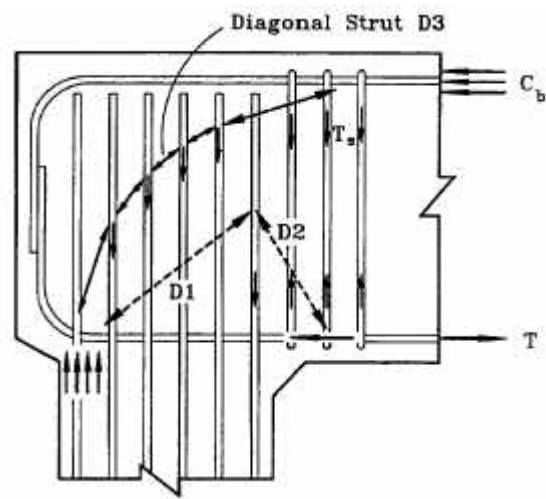


Figure C5.12.2 External vertical joint reinforcement for joint force transfer.

When the cap beam and/or superstructure is prestressed, the bond demands will be much less severe and the clamping requirement can be relaxed. It can also be shown theoretically (Priestley, Seible and Calvi, 1996) that the volumetric ratio of hoop reinforcement can be proportionately reduced to zero as the prestress force approaches $0.25T_c$.

Figure C5.12.3 shows each of the areas within which the reinforcement required by this clause must be placed. For an internal column of a multi-column bent, there will be four such areas, overlapping, as shown in Figure C5.12.3(a). For an exterior column of a multi-column bent, there will be three such areas (Figure C5.12.3(b)). For a single-column bent with monolithic column/cap beam connection, there will be two such areas corresponding to longitudinal response (Figure C5.12.3(c)). Where these areas overlap, vertical joint reinforcement within the overlapping areas may be considered effective for both directions of response. Where shear reinforcement exists within a given area and is not fully utilized for shear resistance in the

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direction of response considered, that portion not needed for shear resistance may be considered to be vertical joint reinforcement. Since cap beam shear reinforcement is normally dictated by conditions causing cap beam negative moment (gravity and seismic shear are additive) while the external joint reinforcement discussed in this section applies to cap beam positive moment (when gravity and seismic shear are in opposition), it is normal to find that a considerable portion of existing cap beam shear reinforcement adjacent to the joint can be utilized.

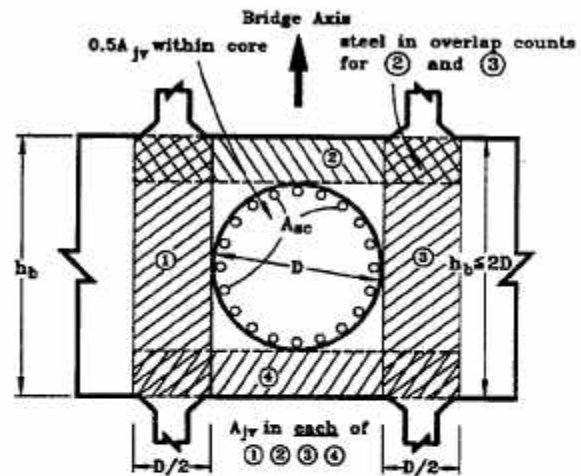
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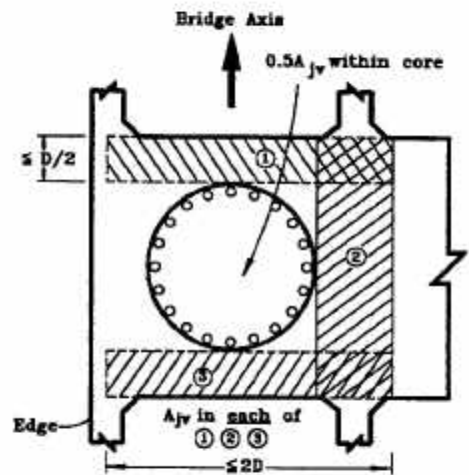
5.12.3.2.2 Clamping Reinforcement

Longitudinal reinforcement contributing to cap beam or footing flexural strength (i.e., superstructure top reinforcement, cap top reinforcement, footing bottom reinforcement) shall be clamped into the joint by vertical bars providing a total area of $0.08A_{ST}$. These bars shall be hooked around the restrained longitudinal reinforcement and extend into the joint a distance not less than two-thirds of the joint depth. If more than 50 percent of the superstructure moment capacity and/or cap-beam moment capacity is provided by prestress, this reinforcement may be omitted, unless needed for the orthogonal direction of response.

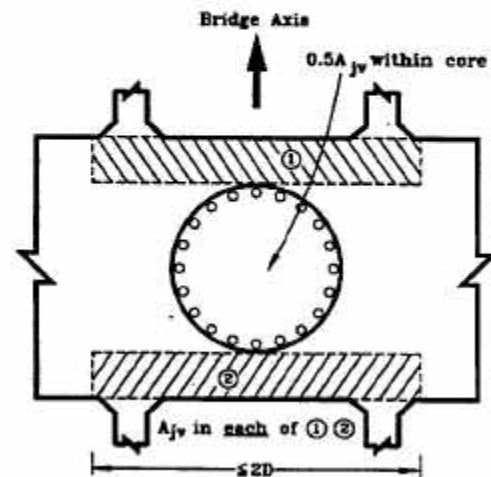
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(a) Internal Column: Multicolumn Bent



(b) Exterior Column: Multicolumn Bent



(c) Single Column Bent

Figure C5.12.3 Locations for vertical joint reinforcement.

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5.12.3.2.2 HORIZONTAL REINFORCEMENT

Additional longitudinal reinforcement in the cap beam, superstructure, and footing of total amount $0.08A_{ST}$ over and above the required for flexural strength, shall be placed in the face adjacent to the column (i.e., bottom of cap beam or superstructure; top of footing), extending through the joint and for a sufficient distance to develop its yield strength at a distance of $0.5D$ from the column face, as shown in Figure 5.12.1

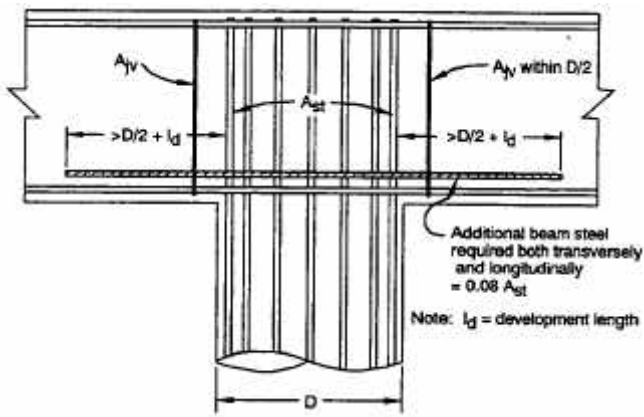


Figure 5.12.1 Additional cap beam bottom reinforcement for joint force transfer.

5.12.2.3 HOOP OR SPIRAL REINFORCEMENT

The required volumetric ration of column joint hoop or spiral reinforcement to be carried into the cap or footing shall not be less than

$$r_s \geq \frac{0.4A_{ST}}{\ell_{ac}^2} \quad (5.12.1.2-1)$$

5.12.4 Footing Strength

5.12.4.1 FLEXURAL STRENGTH FOR GROUP VII LOADS

In determining the flexural strength of footings resisting gravity plus seismic overloads, with monolithic column/footing connections, the effective width of the footing shall not be taken to be greater than the width of the column plus a tributary footing width, equal to the effective depth of the footing, on either side of the column.

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C5.12.2.3 HORIZONTAL REINFORCEMENT

Additional cap-beam bottom reinforcement of area $0.08A_{ST}$ is required to provide the horizontal resistance of the strut D2 in Figure C5.12.2.

Special care is needed for knee joints as represented by Figure C5.12.3(b). For moment tending to close the joint, force transfer must be provided between the top cap beam reinforcement and the column outer reinforcement. When the cap beam does not extend significantly past the column, this is best effected by making the cap beam top and bottom reinforcement into a continuous loop outside the column cage, as shown in Figure C5.12.2.

If a cap-beam cantilever is provided, with cap-beam reinforcement passing beyond the joint, additional vertical shear reinforcement outside the joint, as for Figure C5.12.3, will be required.

Moment-resisting connections designed according to these requirements have performed well in experiments (Seible et al., 1994; Sritharan and Priestley, 1994a; Sritharan and Priestley, 1994b).

This reinforcement may be omitted in prestressed or partially prestressed cap beams if the prestressed design force is increased by the amount needed to provide an equivalent increase in cap-beam moment capacity to that provided by this reinforcement.

C5.12.2.4

The hoop or spiral reinforcement of Eq. (5.12.1.2-1) is required to provide adequate confinement of the joint, and to resist the net outward thrust of struts D1 and D2 in Figure C5.12.2.

C5.12.4 Footing Strength

C5.12.4.1 FLEXURAL STRENGTH FOR GROUP VII LOADS

Under extreme seismic loading, it is common for the footing to be subjected to positive moments on one side of the column and negative moments on the other. In this case, shear lag considerations show that it is unrealistic to expect footing reinforcement at lateral distances greater than the footing effective depth to effectively participate in footing flexural strength. Tests

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on footings (Xiao et al., 1994) have shown that a footing effective width complying with this clause will produce a good prediction of maximum footing reinforcement stress. If a larger effective width is adopted in design, shear lag effects will result in large inelastic strains developing in the footing reinforcement adjacent to the column. This may reduce the shear strength of the footing and jeopardize the footing joint force transfer mechanisms. Since the reinforcement outside the effective width is considered ineffective for flexural resistance, it is permissible to reduce the reinforcement ratio in such regions to 50 percent of that within the effective width unless more reinforcement is required to transfer pile reactions to the effective sections.

5.12.4.2 FOOTING SHEAR STRENGTH

C5.12.4.2 FOOTING SHEAR STRENGTH

5.12.4.2.1 Effective Width

C5.12.4.2.1 Effective Width

The effective width for determining the shear strength of footings for gravity plus seismic overloads shall be as for flexural overstrength

Arguments similar to those for moment apply to the effective width for shear strength estimation.

5.12.4.2.2 Shear Reinforcement

When the nominal shear strength in footings arising from the maximum flexural overstrength, vertical stirrups or ties shall be provided to carry the deficit in shear strength. These stirrups shall be placed within the effective width as defined by clause 5.12.2.2.1.

5.14.4 Concrete Piles

5.14.4.1 GENERAL

C5.14.4.1

All loads resisted by the footing and the weight of the footing itself shall be assumed to be transmitted to the piles. Piles installed by driving shall be designed to resist driving and handling forces. For transportation and erection, a precast pile should be designed for not less than 1.5 times its self-weight.

The material directly under a pile-supported footing is not assumed to carry any of the applied loads.

Any portion of a pile where lateral support adequate to prevent buckling may not exist at all times, shall be designed as a column.

Locations where such lateral support does not exist include any portion of a pile above the anticipated level of scour or future excavation as well as portions that extend above ground, as in pile bents.

The points or zones of fixity for resistance to lateral loads and moments shall be determined by an analysis of the soil properties, as specified in Article 10.7.4.2.

Concrete piles shall be embedded into footings or pile caps, as specified in Article 10.7.1.5. Anchorage reinforcement shall consist of either an extension of the pile reinforcement or the use of dowels. Uplift forces or stresses induced by flexure shall be resisted by the

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reinforcement. The steel ratio for anchorage reinforcement shall not be less than 0.005, and the number of bars shall not be less than four. The reinforcement shall be developed sufficiently to resist a force of $1.25 f_y A_s$.

In addition to the requirements specified in Articles 5.14.4.1 through 5.14.4.5, piles used in the seismic zones shall conform to the requirements specified in Article 5.14.4.6.

5.14.4.2 SPLICES

Splices in concrete piles shall develop the axial, flexural, shear, and torsional resistance of the pile. Details of splices shall be shown in the contract documents.

C5.14.4.2

AASHTO LRFD Bridge Construction Specifications has provisions for short extensions or "buildups" for the tops of concrete piles. This allows for field corrections due to unanticipated events, such as breakage of heads or driving slightly past the cutoff elevation.

5.14.4.3 PRECAST REINFORCED PILES

5.14.4.3.1 Pile Dimensions

Precast concrete piles may be of uniform section or tapered. Tapered piling shall not be used for trestle construction, except for that portion of the pile that lies below the ground line, or in any location where the piles are to act as columns.

Where concrete piles are not exposed to salt water, they shall have a cross-sectional area measured above the taper of not less than 90 000 mm². Concrete piles used in salt water shall have a cross-sectional area of not less than 142 000 mm². The corners of a rectangular section shall be chamfered.

The diameter of tapered piles measured 600 mm from the point shall be not less than 200 mm where, for all pile cross-sections, the diameter shall be considered as the least dimension through the center of cross-section.

C5.14.4.3.1

A 25 mm connection chamfer is desirable, but smaller chamfers have been used successfully. Local experience should be considered.

5.14.4.3.2 Reinforcing Steel

Longitudinal reinforcement shall consist of not less than four bars spaced uniformly around the perimeter of the pile. The area of reinforcing steel shall not be less than 1.5 percent of the gross concrete cross-sectional area measured above the taper.

The full length of longitudinal steel shall be enclosed with spiral reinforcement or equivalent hoops. The spiral reinforcement shall be as specified in Article 5.14.4.4.3.

5.14.4.4 PRECAST PRESTRESSED PILES

5.14.4.4.1 Pile Dimensions

Prestressed concrete piles may be octagonal,

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square, or circular and shall conform to the minimum dimensions specified in Article 5.14.4.3.1.

Prestressed concrete piles may be solid or hollow. For hollow piles, precautionary measures, such as venting, shall be taken to prevent breakage due to internal water pressure during driving, ice pressure in trestle piles, or gas pressure due to decomposition of material used to form the void.

The wall thickness of cylinder piles shall not be less than 125 mm.

5.14.4.4.2 Concrete Quality

The compressive strength of the pile at the time of driving shall not be less than 35 MPa. Air-entrained concrete shall be used in piles that are subject to freezing and thawing or wetting and drying.

5.14.4.4.3 Reinforcement

Unless otherwise specified by the Owner, the prestressing strands should be spaced and stressed to provide a uniform compressive stress on the cross-section of the pile after losses of not less than 5 MPa.

The full length of the prestressing strands shall be enclosed with spiral reinforcement as follows:

For piles not greater than 600 mm in diameter:

- Spiral wire not less than W3.9,
- Spiral reinforcement at the ends of piles having a pitch of 75 mm for approximately 16 turns,
- The top 150 mm of pile having five turns of additional spiral winding at 25 mm pitch, and
- For the remainder of the pile, the strands enclosed with spiral reinforcement with not more than 150 mm pitch.

for piles greater than 600 mm in diameter:

- Spiral wire not less than W4.0,
- Spiral reinforcement at the end of the piles having a pitch of 50 mm for approximately 16 turns,
- The top 150 mm having four additional turns of spiral winding at 38 mm pitch, and
- For the remainder of the pile, the strands enclosed with spiral reinforcement with not more than 100 mm pitch.

C5.14.4.4.3

The purpose of the 5 MPa compression is to prevent cracking during handling and installation. A lower compression may be used if approved by the Owner.

For noncircular piles, use the least dimension through the cross-section in place of the "diameter."

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5.14.4.5 CAST-IN-PLACE PILES

Piles cast in drilled holes may be used only where soil conditions permit.

Shells for cast-in-place piles shall be of sufficient thickness and strength to hold their form and to show no harmful distortion during driving or after adjacent shells have been driven and the driving core, if any, has been withdrawn. The contract documents shall stipulate that alternative designs of the shell need be approved by the Engineer before any driving is done.

5.14.4.5.1 Pile Dimensions

Cast-in-place concrete piles may have a uniform section or may be tapered over any portion if cast in shells or may be bell-bottomed if cast in drilled holes or shafts.

The area at the butt of the pile shall be at least 64 500 mm². The cross-sectional area at the tip of the pile shall be at least 32 300 mm². For pile extensions above the butt, the minimum size shall be as specified for precast piles in Article 5.14.4.3.

5.14.4.5.2 Reinforcing Steel

The area of longitudinal reinforcement shall not be less than 0.8 percent of A_g , with spiral reinforcement not less than 5 mm diameter at a pitch of 150 mm. The reinforcing steel shall be extended 3000 mm below the plane where the soil provides adequate lateral restraint.

Shells that are more than 3 mm in thickness, may be considered as part of the reinforcement. In corrosive environments, a minimum of 1.5 mm shall be deducted from the shell thickness in determining resistance.

C5.14.4.5

Cast-in-place concrete piles include piles cast in driven steel shells that remain in place and piles cast in unlined drilled holes or shafts.

The construction of piles in drilled holes should generally be avoided in sloughing soils, where large cobblestones exist or where uncontrollable groundwater is expected. The special construction methods required under these conditions increase both the cost and the probability of defects in the piles.

The thickness of shells should be shown in the contract documents as "minimum." This minimum thickness should be that needed for pile reinforcement or for strength required for usual driving conditions: e.g., 3.5 mm minimum for 355 mm pile shells driven without a mandrel. AASHTO LRFD Bridge Construction Specifications requires the Contractor to furnish shells of greater thickness, if necessary, to permit his choice of driving equipment.

Caution should be taken in counting the casing as longitudinal reinforcement. In so doing, there are several seismic and constructional ramifications. If a casing is considered to be part of the longitudinal reinforcement, proper account must be made of its contribution to flexural overstrength—failure to recognize the high flexural strength may lead to unaccounted shear force demands being transferred into connections and elsewhere in the structure. Also, if shells are considered as flexural reinforcement, then delays during construction may be expected due to additional time needed for the inspection of site welds.

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5.14.4.6 SEISMIC REQUIREMENTS

5.14.4.6.1 SDR 1

No additional design provisions need be considered for Zone 1.

5.14.4.6.2 SDR 2

5.14.4.6.2a General

Piles for structures in SDR 2 may be used to resist both axial and lateral loads. The minimum depth of embedment and axial and lateral pile resistances required for seismic loads shall be determined by means of design criteria established by site-specific geological and geotechnical investigations.

Concrete piles shall be anchored to the pile footing or cap by either embedment of reinforcement or anchorages to develop uplift forces equal to 1.5 times the nominal uplift capacity of the pile or the maximum uplift demand calculated according to Articles 10.7.5 and 10.8.5. The embedment length shall not be less than the development length required for the reinforcement specified in Article 5.11.2.

Concrete-filled pipe piles shall be anchored with steel dowels as specified in Article 5.14.4.1, with a minimum steel ratio of 0.008 . Dowels shall be embedded as required for concrete piles. Timber and steel piles, including unfilled pipe piles, shall be provided with anchoring devices to develop any uplift forces. The uplift force shall be taken to be equal to 1.5 times the nominal uplift capacity of the pile or the maximum uplift demand calculated according to Articles 10.7.5 and 10.8.5.

The designer may consider the following means to improve constructability:

- Prestressing the joint as a means of reducing reinforcing steel,
- Placing vertical shear reinforcement within the joint and/or in the cap beam adjacent to the joint region

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5.14.4.6.2b *Cast-in-Place and Precast Concrete Piles*

For cast-in-place and precast concrete piles, longitudinal steel shall be provided in the upper end of the pile for a length not less than either one-third of the pile length or 2400 mm, with a minimum steel ratio of 0.008 provided by at least four bars. Spiral reinforcement or equivalent ties of not less than No. 10 bars shall be provided at pitch not exceeding one-fourth the pile diameter or minimum width within a length not less than 600 mm or 1.5 pile diameters below the soffit of the pile cap. Within these potential plastic hinge zones, the transverse reinforcement shall be detailed for shear reinforcement as required by the implicit approach of Article 5.10.11.4.1c.

5.14.4.6.3 SDR 3 and above

5.14.4.6.3a *General*

In addition to the requirements specified for SDR 2, piles in SDR 3 and above shall conform to the provisions specified herein.

5.14.4.6.3b *Transverse Reinforcement Requirements for Piles*

The upper end of every pile shall be reinforced and confined as a potential plastic hinge region as specified in Article 3.10.3.9, except where it can be established that there is no possibility of any significant lateral deflection in the pile. If an analysis of the bridge and pile system indicates that a plastic hinge can form at a lower level, the plastic hinge zone shall extend 3D below the point of maximum moment. The transverse reinforcement in the top 3D of the pile shall be detailed for the maximum of shear, confinement, and longitudinal bar restraint as for concrete columns described in Article 5.10.11.4.1. The top 10D of the pile shall be detailed for the maximum of shear and confinement as for concrete columns and described in Articles 5.10.11.4.1c and 5.10.11.4.1d.

5.14.4.6.3c *Volumetric Ratio of Transverse Reinforcement for Piles*

In lieu of a precise soil structure interaction analysis to ascertain the shear demand, a value of $\alpha = 25$ degrees may be assumed for use in the implicit shear design equations.

C5.14.4.6.3b

Note the special requirements for pile bents given in Article 5.10.11.4.1

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5.14.4.6.3d Cast-in-Place and Precast Concrete Piles

For cast-in-place and precast concrete piles, longitudinal steel shall be provided for the full length of the pile. In the upper two-thirds of the pile, the longitudinal steel ratio, provided by not less than four bars, shall not be less than 0.008.

5.16 PLASTIC ROTATIONAL CAPACITIES

The plastic rotational capacity shall be based on the appropriate performance limit state for the bridge. In lieu of this prescriptive values given below, the designer may determine the plastic rotational capacity from tests and/or a rational analysis.

5.16.1 Life-Safety Performance

The plastic rotational capacity of hinges shall be based on

$$q_p = 0.11 \frac{L_p}{D'} (N_f)^{-0.5} \quad rad \quad (5.16.1-1)$$

in which

N_f = number of cycles of loading expected at the maximum displacement amplitude which may be estimated from

$$N_f = 3.5 (T_n)^{-1/3} \quad (5.16.1-2)$$

$$2 \leq N_f \leq 10$$

where T_n = natural period of vibration of the structure.

For liquifiable soils and piled foundation assessment, use $N_f = 2$

L_p = effective plastic hinge length give by

$$L_p = 0.08 \frac{M}{V} + 4400 e_y d_b \quad (5.16.1-3)$$

where

M/V = shear span of the member (M = end moment
 V = shear force)

e_y = yield strain of the longitudinal reinforcement;

When an isolation gap of length L_g is provided between a structurally separated flare and an adjacent structural element, the plastic hinge length is given by

$$L_p = L_g + 8800 e_y d_b \quad (15.16.1-4)$$

where L_g is the gap between the flare and the adjacent element.

A moment-curvature analysis based on strain compatibility and nonlinear stress-strain relations can be used to determine plastic limit states. From this a rational analysis is used to establish the rotational capacity of plastic hinges.

If a section has been detailed in accordance with the transverse reinforcement requirement of these provisions, then the section is said to be 'capacity protected' against undesirable modes of failure such as shear, buckling of longitudinal bars, and concrete crushing due to lack of confinement. The one remaining failure mode is low cycle fatigue of the longitudinal reinforcement. The fatigue life depends on the fatigue capacity [Chang and Mander, 1994a, (NCEER 94-0006)] versus demand [Chang and Mander, 1994b (NCEER 94-0013)].

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D' = the distance between the outer layers of the longitudinal reinforcement on opposite faces of the member, equal to the pitch circle diameter for a circular section.

d_b = diameter of the main longitudinal reinforcing bars.

In lieu of the precise analysis given above, a conservative value of $q_p = 0.035 \text{ rad}$ shall be assumed.

For life-safety assessment of pile foundations that are potentially liquifiable, then $q_p = 0.055 \text{ rad}$

This rotational capacity ensures a dependable fatigue-life for all columns, regardless of the period-dependent cyclic demand.

5.16.2 Operational Performance Limit State

To ensure the immediate use of the bridge structure following a design ground motion, the maximum rotational capacity should be limited to $q_p = 0.01 \text{ rad}$.

5.16.3 In-Ground Hinges

5.16.3.1 Ordinary Soils

The maximum rotational capacity for in-ground hinges shall be restricted to $q_p = 0.02 \text{ rad}$.

C5.16.3.1 In-ground hinges are necessary for certain types of bridge substructures. These may include, but not restricted to:

- Pile bents
- Pile foundations with strong pier walls
- Drilled shafts
- Piled foundations with oversized columns.

It is necessary to restrict these plastic hinge rotations in order to limit the crack width and plastic strains. This limit is expected to reduce plastic strains to less than 40 percent of their above-ground counterpart (with $q_p = 0.035 \text{ rad}$.) This is because the plastic hinge length of in-ground hinges is typically two pile diameters due to the reduced moment gradient in the soil.

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5.16.3.2 Liquifiable Soils

The rotational capacity for in-ground hinges for liquifiable soil layers that may lead to a mechanism in the pile or shaft foundation shall be restricted to $q_p = 0.07rad$

COMMENTARY

C5.16.3.2 This requirement is for the life-safety assessment only of pile foundations where the liquifiable layer forces a mechanism in the piles or drilled shafts. This near-upper bound value is intended to sustain only one or two cycles of gross ground movement.

SECTION 5 – CONCRETE STRUCTURES

SPECIFICATIONS

COMMENTARY

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SECTION 6 – STEEL STRUCTURES

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SECTION 6 – STEEL STRUCTURES

SPECIFICATIONS

6.1 SCOPE

This section covers the design of steel components, splices and connections for beam and girder structures, frames, trusses and arches, cable-stayed and suspension systems, and metal deck systems, as applicable.

Curved girder structures are not included.

A brief outline for the design of steel girder bridges is presented in Appendix B.

COMMENTARY

C6.1

Most of the provisions for proportioning main elements are grouped by structural action:

- Tension and combined tension and flexure (Article 6.8)
- Compression and combined compression and flexure (Article 6.9)
- Flexure and flexural shear:
 - I-sections (Article 6.10)
 - Box sections (Article 6.11)
 - Miscellaneous sections (Article 6.12)

Provisions for connections and splices are contained in Article 6.13.

Article 6.14 contains provisions specific to particular assemblages or structural types, e.g., through-girder spans, trusses, orthotropic deck systems, and arches.

6.2 DEFINITIONS

To be added to existing definitions in Section 6.

Capacity protected element – Parts of the structure that is either connected to a critical element or within its load path and that is prevented from yielding by virtue of having the critical member limit the maximum force that can be transmitted to the capacity protected element.

Critical elements – Parts of the structure that are expected to absorb energy, undergo significant inelastic deformations while maintaining their strength and stability.

Nominal resistance - Resistance of a member, connection or structure based on the expected yield strength (F_{ye}), other specified material properties, and the nominal dimensions and details of the final section(s) chosen, calculated with all material resistance factors taken as 1.0.

Overstrength Capacity - Resistance of a member, connection or structure based on the nominal dimensions and details of the final section(s) chosen, calculated accounting for the expected development of large strains and associated stresses larger than the minimum specified yield values.

6.3 NOTATION

To be added to existing notation in Section 6.

B = factor that sets the shape of the interaction diagram for concrete-filled steel pipe, as defined in Article 6.15.4.3.4.b

F_{ye} = Expected yield strength of steel to be used (MPa)

R_y = Ratio of the expected yield strength F_{ye} to the minimum specified yield strength F_y

M_{rc} = factored moment resistance of a concrete filled steel pipe for Article 6.15.4.3.4.2 (kN-m)

P_{ro} = factored compressive resistance of concrete-filled steel pipe (Articles 6.9.2.1 and 6.9.5.1) with $\lambda = 0$ (kN)

P_{rc} = factored compressive resistance of the concrete core of a concrete-filled steel pipe (Articles 6.9.2.1 and 6.9.5.1) with $\lambda = 0$ (kN)

θ_p = maximum rotational capacity

6.7.5 Lateral Bracing**6.7.5.1 GENERAL**

The need for lateral bracing shall be investigated for all stages of assumed construction procedures and the final condition.

Where required, lateral bracing should be placed either in or near the plane of a flange or chord being braced. Investigation of the requirement for lateral bracing shall include, but not be limited to:

- Transfer of lateral wind loads to the bearings as specified in Article 4.6.2.7,
- Transfer of lateral loads as specified in Article 4.6.2.8, and
- Control of deformations during fabrication, erection, and placement of the deck.

Lateral bracing required for conditions other than the final condition may be removed.

If permanent lateral bracing is included in the structural model used to determine force effects, it shall be designed for all applicable limit states. The provisions of Articles 6.8.4 and 6.9.3 shall apply.

Connection plates for lateral bracing shall satisfy the requirements specified in Article 6.6.1.3.2.

When lateral bracing is designed for seismic loading, the provisions of Articles 4.8.3 and 3.10.3.12 shall apply.

Articles 4.8.3 and 3.10.3.12 require the engineer to ensure that a clear load path exists from the seismically induced inertia forces at deck level, down to the foundation. Although the articles are applicable to all bridges, they are particularly relevant for steel bridges.

To comply with Articles 4.8.3 and 3.10.3.12, the engineer should ensure appropriate load-transfer mechanisms at the interface between the concrete slab and steel superstructure. Although the bond between the concrete and steel may be sufficient to provide the needed force transfer in some bridges, there is no evidence to prove or disprove the adequacy of this bond. Shear studs are required as an effective low-cost measure to provide load-transfer in new bridges, but the lack of experimental evidence may not justify the higher cost required to add such studs during the seismic retrofit of existing bridges.

Article 3.10.3.14 contains specific requirements applicable to bearings located along this load path.

Viable load path options other than those described in Articles 4.8.3 and 3.10.3.12 may be considered, if demonstrated to be appropriate by the engineer. Research on the seismic behavior of integral bent-caps may result in satisfactory design and detailing requirements for that purpose.

Note that non-fatal damage along the load path may be expected following the rare earthquake (e.g. joints may be damaged, shear studs may have induced cracking in the slab, etc.). However, yielding is not permitted in anchor bolts, base plates, and other capacity protected members.

6.15 PROVISIONS FOR SEISMIC DESIGN (S.I. Units)**6.15.1 General**

The provisions of Article 6.15 shall apply only to a limited number of specially detailed steel components designed to dissipate hysteretic energy during earthquakes. Article 6.15 does not apply to steel members that are designed to remain elastic during earthquakes.

For the few specially designed steel members that are within the scope of Article 6.15, the other requirements of Section 6 are also applicable (unless superseded by more stringent requirements in Article

C6.15.1

It is essential to realize that most components of steel bridges are not expected to behave in a cyclic inelastic manner during an earthquake. The provisions of Article 6.15 are only applicable to the limited number of components (such as specially detailed ductile substructures or ductile diaphragms) whose stable hysteretic behavior is relied upon to ensure satisfactory bridge seismic performance. The seismic provisions of Article 6.15 are not applicable to the other steel members expected to remain elastic during seismic

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6.15).

Continuous and clear load path or load paths shall be assured. Proper load transfer shall be considered in designing foundations, substructures, superstructures and connections.

Welds shall be designed as capacity protected elements. Partial penetration groove welds shall not be used in ductile substructures.

Abrupt changes in cross sections of members in ductile substructures are not permitted in plastic hinge zones unless demonstrated acceptable by analysis and supported by research results.

COMMENTARY

response. Note that in most steel bridges, the steel superstructure is expected (or can be designed) to remain elastic.

Until recently, only a few steel bridges had been seriously damaged in earthquakes. One span of the San Francisco-Oakland Bay Bridge collapsed due to loss of support at its bearings during the 1989 Loma Prieta earthquake, and another bridge suffered severe bearing damage (EERI, 1990). The end diaphragms of some steel bridges suffered damage in a subsequent earthquake in Northern California (Roberts, 1992). During the 1994 Northridge earthquake some steel bridges, located very close to the epicenter, sustained damage to either their reinforced concrete abutments, connections between concrete substructures and steel superstructures, steel diaphragms or structural components near the diaphragms (Astaneh-Asl et al, 1994). However, a large number of steel bridges were damaged by the 1995 Hyogoken-Nanbu (Kobe) earthquake. The concentration of steel bridges in the area of severe ground motion was considerably larger than for any previous earthquake and some steel bridges collapsed. Many steel piers, bearings, seismic restrainers and superstructure components suffered significant damage (Bruneau, Wilson and Tremblay, 1996). This experience emphasizes the importance of ductile detailing in the critical elements of steel bridges.

Research on the seismic behavior of steel bridges (e.g. Astaneh-Asl, Shen and Cho, 1993; Dicleli and Bruneau, 1995a, 1995b; Dietrich and Itani, 1999; Itani et al., 1998a; McCallen and Astaneh-Asl, 1996; Seim, Ingham and Rodriguez, 1993; Uang et al., 2000; Uang et al., 2001; Zahrai and Bruneau 1998) and findings from recent seismic evaluation and rehabilitation projects (e.g. Astaneh and Roberts, 1993, 1996; Ballard et al., 1996; Billings et al, 1996; Dameron et al., 1995; Donikian et al., 1996; Gates et al., 1995; Imbsen et al., 1997; Ingham et al., 1996; Jones et al., 1997; Kompfner et al., 1996; Maroney 1996; Prucz et al., 1997; Rodriguez and Ingham, 1996; Schamber et al., 1997; Shirolé and Malik, 1993; Vincent et al., 1997) further confirm that seismically induced damage is likely in steel bridges subjected to large earthquakes and that appropriate measures must be taken to ensure satisfactory seismic performance.

The intent of Article 6.15 is to ensure the ductile response of steel bridges during earthquakes. First, effective load paths must be provided for the entire structure. Following the concept of capacity design, the load effect arising from the inelastic deformations of part of the structure must be properly considered in the design of other elements that are within its load path.

Second, steel substructures must be detailed to ensure stable ductile behavior. Note that the term

SECTION 6 – STEEL STRUCTURES

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“substructure” here refers to structural systems exclusive of bearings (Article 3.10.3.14) and articulations, which are considered in other Sections. Steel substructures, although few, need ductile detailing to provide satisfactory seismic performance.

Third, considerations for other special ductile systems is introduced, and described in the commentary.

Special consideration may be given to slip-critical connections that may be subjected to cyclic loading. Some researchers have expressed concern that the Poisson effect may cause steel plate thickness to reduce when yielding on net section occurs during seismic response, which may translate into a reduced clamping action on the faying surfaces after the earthquake. This has not been experimentally observed, nor noted in post-earthquake inspections, but the impact of such a phenomenon would be to reduce the slip-resistance of the connection, which may have an impact on fatigue resistance. This impact is believed to be negligible for a Category C detail for finite life, and a Category D detail for infinite life. Design to prevent slip for the Expected Earthquake should be also considered.

6.15.2 Materials

Ductile Substructure Elements and ductile end-diaphragms, as defined in Article 6.15, shall be made of either:

- (a) M270 (ASTM 709M) Grade 345 and Grade 345W steels
- (b) ASTM A992 steel, or
- (c) A500 Grade B or A501 steels (if structural tubing or pipe).

Other steels may be used provided that they are comparable to the approved Grade 345 steels.

In Article 6.15, nominal resistance is defined as the resistance of a member, connection or structure based on the expected yield strength (F_{ye}), other specified material properties, and the nominal dimensions and details of the final section(s) chosen, calculated with all material resistance factors taken as 1.0.

Overstrength capacity is defined as the resistance of a member, connection or structure based on the nominal dimensions and details of the final section(s) chosen, calculated accounting for the expected development of large strains and associated stresses larger than the minimum specified yield values.

The expected yield strength shall be used in the calculation of nominal and probable resistances, where expected yield strength is defined as $F_{ye} = R_y F_y$ where R_y shall be taken as 1.1 for the permitted steels listed above.

Welding requirements shall be compatible with

C6.15.2

To ensure that the objective of capacity design is achieved, Grade 250 steel is not permitted for the components expected to respond in a ductile manner. Grade 250 is difficult to obtain and contractors often substitute it with a Grade 345 steel. Furthermore it has a wide range in its expected yield and ultimate strength and very large overstrength factors to cover the anticipated range of property variations. The common practice of dual-certification for rolled shapes, recognized as a problem in the perspective of capacity design following the Northridge earthquake, is now becoming progressively more common also for steel plates. As a result, only Grade 345 steels are allowed within the scope of Article 6.15.2, with a R_y of 1.1.

In those instances when Grade 250 must be used, capacity design must be accomplished assuming a Grade 345 steel (i.e., with a R_y of 1.5 applied to the F_y of 250 Mpa), but R-factor design and deformation limits shall be checked using Grade 250's yield strength of 250 Mpa.

The use of A992 steel is explicitly permitted. Even though this ASTM grade is currently designated for “shapes for buildings”, there is work currently being done to expand applicability to any shapes. ASTM 992 steel, recently developed to ensure good ductile seismic performance, is specified to have both a minimum and maximum guaranteed yield strength,

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AWS/ASSHTO D1.5-96 Structural Bridge Welding Code. However, under-matched welds are not permitted for special seismic hysteretic energy dissipating systems (such as ductile substructures and ductile diaphragms).

Steel members expected to undergo significant plastic deformations during a seismic event shall meet the toughness requirements of A709/A709M Supplementary Requirement S84 (Fracture Critical). Welds metal connecting these members shall meet the toughness requirements specified in the AWS D1.5 Bridge Specification for Zone III.

6.15.3 Sway Stability Effects

The sway effects produced by the vertical loads acting on the structure in its displaced configuration shall be determined from a second-order analysis. Alternatively, recognized approximate methods for P- Δ analysis, or the provisions in Article 3.10.3.9.4, can be used.

6.15.4 Steel Substructures

Article 6.15.4 is for the detailing of steel substructures only, and is not applicable to energy dissipating systems implemented in bridge superstructures.

COMMENTARY

and may be worthy of consideration for ductile energy dissipating systems in steel bridges.

Note that since other steels may be used provided that they are comparable to the approved Grade 345 steels, High Performance Steel (HPS) Grade 345 would be admissible, but not HPS Grade 485 (or higher). This is not a detrimental restriction for HPS steel, as the scope of Article 6.15 encompasses only a few steel members in a typical steel bridge. (Note that, based on very limited experimental data available, it appears that HPS Grade 485 has a lower rotational ductility capacity and may not be suitable for “ductile fuses” in seismic applications).

When other steels are used for energy dissipation purposes, it is the responsibility of the designer to assess the adequacy of material properties available and design accordingly.

Other steel members expected to remain elastic during earthquake shall be made of steels conforming to Article 6.4.

Steel members and weld materials shall have adequate notch toughness to perform in a ductile manner over the range of expected service temperatures. The A709/A709M S84 “Fracture-Critical Material Toughness Testing and Marking” requirement, typically specified when the material is to be utilized in a fracture-critical application as defined by the American Association of State Highway and Transportation Officials (AASHTO), is deemed to be appropriate to provide the level of toughness sought for seismic resistance. For weld metals, note that the AWS D1.5 Bridge Specification requirement for Zone III, familiar to the bridge engineering community, is similar to the 20 ft-lbs at -20F requirement proposed by the SAC Joint Venture for weld metal in welded moment frame connections in building frames.”

The capacity design philosophy and the concept of capacity-protected element are defined in Article 3.10.3.8.

C6.15.4

Although the proposed seismic provisions focus primarily on ductile substructures as energy dissipation systems (in Article 6.15.4), alternative approaches and innovative strategies are possible and encouraged (in Article 6.15.5) to achieve the design intent.

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Article 6.15.4 applies to single level bridges within the scope described in Article 3.10.1. (Double-decker bridges or other complex multi-level configurations are not single level bridges). For other structures, Article 6.15.5.4 may be applied.

For ductile steel columns, frames and bents, as well as ductile and nominally ductile braced frames, following the same philosophy adopted for concrete frames, seismic energy dissipation is to take place in the substructure, namely by plastic hinging of steel columns/piers in moment frames (except for multi-tier frame bents as described later), and by axial yielding of the braces in braced frames.

For braced frames, all references to “inelastic hinging of the column” in other seismic requirements elsewhere in the Specifications should be interpreted as “brace yielding”.

Note that for analysis, the rigidity of the connections in steel substructures should be taken into account in the modeling. This would be particularly significant at the base of columns where details sometimes used can behave more like semi-rigid connections. The engineer should carefully assess this flexibility in modeling of the structure.

6.15.4.1 SDR 1

No specific ductile details are required for steel substructures beyond the minimum seismic detailing requirements specified in Article 3.10.3.1, Table 3.10.3-2, and Article 3.10.3.9.2.

6.15.4.2 SDR 2

The ductile details specified in this Article for steel substructures are in addition to the minimum seismic detailing requirements specified in Article 3.10.3.1, Table 3.10.3-2, and Article 3.10.3.9.2.

Design of capacity-protected elements should be accomplished considering the nominal resistance of the ductile energy-dissipating element instead of their overstrength capacity.

C6.15.4.2

In conformance with the general requirements, the design requirements for SDR 2 are somewhat less stringent than those stipulated for higher SDRs. One particular such relaxation allows the design of capacity-protected elements using the nominal resistance of the ductile energy-dissipating element instead of their overstrength capacity. This reflects the lower level of inelastic response expected for structures in lower seismic zones.

Even when the bridge configuration makes it eligible for the “no analysis” option in Section 4, the steel energy dissipating elements shall be detailed following the requirements specified in Article 6.15.4.2.

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6.15.4.2.1 Ductile Moment-Resisting Frames and Bents

6.15.4.2.1.a General

Ductile moment-resisting frames and bents shall meet the requirements of Article 6.15.4.3.1, as modified in accordance with this article.

6.15.4.2.1.b Columns

Columns shall be designed as Ductile Substructure Elements.

The maximum axial compressive load limit of Article 6.15.4.3.1.b shall be replaced by $0.40A_gF_y$.

6.15.4.2.1.c Beams, Panel Zones and Connections

Beams, panel zones, moment resisting connections, and column base connections shall be designed as Capacity Protected Elements as defined in Articles C3.10.3.8.1 and C6.15.4.3.

The nominal flexural resistance of the column shall be determined from Article 6.15.4.3.1.c.

6.15.4.2.2 Ductile Concentrically Braced Frames

Ductile concentrically braced frames and bents shall meet the requirements of Article 6.15.4.3.2.

6.15.4.2.3 Concentrically Braced Frames and Bents with Nominal Ductility

Concentrically braced frames and bents with nominal Ductility shall meet the requirements of Article 6.15.4.3.4 except braces in chevron braced frames need not conform to Article 6.15.4.3.3.c but shall meet the requirements of Article 6.15.4.3.3.f.

6.15.4.2.4 Other Framing Systems

Other framing systems shall meet the requirements of Article 6.15.5.

6.15.4.3 SDR 3 AND ABOVE

Steel substructures in SDR 3 and above shall conform to Article 6.15.4.3 as well as the pertinent requirements of Article 3.10.3.1, Table 3.10.3-2, and Article 3.10.3.9.2.

C6.15.4.2.1.b

This is an arbitrary increase in the permitted maximum axial load, due to the lower ductility demands expected in SDR 2.

This ensures that braces have connections able to develop gross axial yielding of the brace, but does not impose limits on the width-to-thickness ratio and slenderness of the braces. This is acceptable in light of the low R-factor assigned to this system, and the smaller duration and intensity of seismic excitations expected in SDR 2.

C6.15.4.3

Critical elements are the parts of the structure that are expected to absorb energy, undergo significant inelastic deformations while maintaining their strength and stability. Other parts that are either connected to a critical element or within its load path should be either: (i) proportioned and detailed as critical elements; (ii) designed to resist full elastic loads, or; (iii) they should be capacity-protected using the forces

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of Article 3.10.3.8. Applying the concept of capacity design, an element is generally considered to be capacity-protected if the critical element reaches its capacity before the capacity-protected element experiences the corresponding load effect exceeding its resistance. The probable capacity of an element is equal to its nominal capacity increased to account for overstrength due to higher yield than specified yield and strain-hardening effects. The load combinations for design of capacity-protected elements under this article represent loads due to the extreme event earthquake combined with the permanent loads. Alternatively, they can be designed to resist full elastic seismic load, calculated using $R = 1$. However, this alternative usually results in higher loads.

6.15.4.3.1 Ductile Moment-Resisting Frames and Single Column Structures

6.15.4.3.1.a General

This article applies to ductile moment-resisting frames and bents, constructed with I-shape beams and columns connected with their webs in a common plane. Except as noted in Article 6.15.4.3.1.5, columns shall be designed as ductile structural elements, while the beams, the panel zone at column-beam intersections and the connections shall be designed as Capacity Protected Elements.

C6.15.4.3.1.a

It is believed that properly detailed fully welded column-to-beam or beam-to-column connections in the moment-resisting frames that would typically be used in bridges can exhibit highly ductile behavior and perform adequately during earthquakes (contrary to what was observed in buildings following Northridge). As a result, strategies to move plastic hinges away from the joints are not required in the Specifications.

However, the engineer may still elect to provide measures (such as haunches at the end of yielding members) to locate plastic hinges some distance away from the welded beam-to-column or column-to-beam joint (FEMA 1995, 1997, 2000).

Although beams, columns and panel zones can all be designed, detailed and braced to undergo severe inelastic straining and absorb energy, the detailing requirements of Article 6.15 address common bridge structures with deep non-compact beams much stiffer flexurally than their supporting steel columns, and favors systems proportioned so that plastic hinges form in the columns. This is consistent with the philosophy adopted for concrete bridges.

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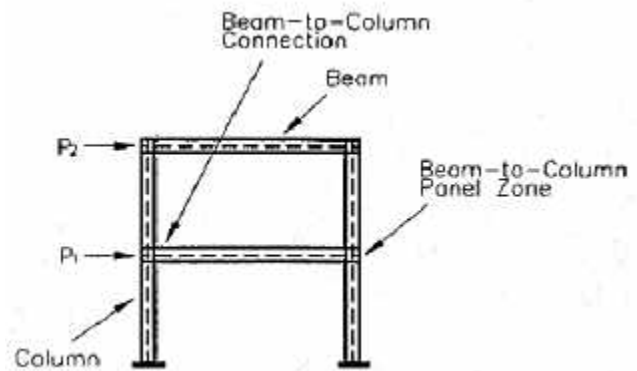


Figure C6.15.4.3.1.a-1: Example of moment frame/bent.

Even though some bridges could be configured and designed to develop stable plastic hinging in beams without loss of structural integrity, the large gravity loads that must be simultaneously resisted by those beams also make plastic hinging at mid-span likely as part of the plastic collapse mechanism. The resulting deformations can damage the superstructure (diaphragms, deck, etc.).

The special case of multi-tier frames is addressed in Article 6.15.4.3.1.5.

6.15.4.3.1.b Columns

Width-to-thickness ratios of compression elements of columns shall be in compliance with Table 6.15.1. Full penetration flange and web welds are required at column-to-beam (or beam-to-column) connections.

The resistance of columns to combined axial load and flexure shall be determined in accordance with Article 6.9.2.2. The factored axial compression due to seismic load and permanent loads shall not exceed $0.20A_gF_y$.

The shear resistance of the column web shall be determined in accordance with Article 6.10.7.

The potential plastic hinge zones (Article 3.10.3.9), near the top and base of each column, shall be laterally supported and the unsupported distance from these locations shall not exceed $17250r_y/F_y$. These lateral supports shall be provided either directly to the flanges or indirectly through a column web stiffener or a continuity plate. Each column flange lateral support shall resist a force of not less than 2% of the nominal column flange strength (btF_y) at the support location. The possibility of complete load reversal shall be considered.

When no lateral support can be provided, the column maximum slenderness shall not exceed 60 and transverse moments produced by the forces otherwise resisted by the lateral bracing (including the second

C6.15.4.3.1.b

At plastic hinge locations, members absorb energy by undergoing inelastic cyclic bending while maintaining their resistance. Therefore, plastic design rules apply, namely, limitations on width-to-thickness ratios, web-to-flange weld capacity, web shear resistance, lateral support, etc.

Axial load in columns is also restricted to avoid early deterioration of beam-column flexural strengths and ductility when subject to high axial loads. Tests by Popov et al. (1975) showed that W-shaped columns subjected to inelastic cyclic loading suffered sudden failure due to excessive local buckling and strength degradation when the maximum axial compressive load exceeded $0.50A_gF_y$. Tests by Schneider et al. (1992) showed that moment-resisting steel frames with hinging columns suffer rapid strength and stiffness deterioration when the columns are subjected to compressive load equal to approximately $0.25A_gF_y$. Note that most building codes set this limit at $0.30A_gF_y$.

The requirement for lateral support is identical to Equation 6.10.4.1.7-1 with a moment (M) of zero at one end of the member, but modified to ensure inelastic rotation capacities of at least four times the elastic rotation corresponding to the plastic moment (resulting in a coefficient of 17250 instead of the approximately 25000 that would be obtained for

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order moment due to the resulting column displacement) shall be included in the seismic load combinations.

Splices that incorporate partial joint penetration groove welds shall be located away from the plastic hinge zones as defined in Article 3.10.3.9 at a minimum distance equal to the greater of:

- (a) one-fourth the clear height of column;
- (b) twice the column depth; and
- (c) one metre.

6.15.4.3.1.c Beams

The Factored Resistance of the beams shall be determined in accordance with Article 6.10.4. At a joint between beams and columns the sum of the Factored Resistances of the beams shall not be less than the sum of the Probable Resistances of the column(s) framing into the joint. The probable flexural resistance of columns shall be taken as the product of the overstrength factor (defined in Article 3.10.3.9) times the columns nominal flexural resistance determined either in accordance to Article 6.9.2.2, or by

$$M_{rx} = 1.18M_{px} \left[1 - \frac{P_u}{AF_{ye}} \right] \leq M_{px} \quad (6.15.4.3.1c-1)$$

unless demonstrated otherwise by rational analysis, and where M_{px} is the column plastic moment under pure bending calculated using F_{ye} .

6.15.4.3.1.d Panel Zones and Connections

Column-beam intersection panel zones, moment resisting connections and column base connections shall be designed as Capacity Protected Elements.

Panel zones shall be designed such that the vertical shearing resistance is determined in accordance with Article 6.10.7.2.

Beam-to-column connections shall have resistance not less than the resistance of the beam stipulated in Article 6.15.4.3.1.c.

Continuity plates shall be provided on both sides of the panel zone web and shall finish with total width of at least 0.8 times the flange width of the opposing flanges. Their b/t shall meet the limits for projecting elements of Article 6.9.4.2. These continuity plates shall be proportioned to meet the stiffener requirements stipulated in Article 6.10.8.2 and shall be connected to both flanges and the web.

Flanges and connection plates in bolted connections shall have a factored net section ultimate resistance calculated by Equation 6.8.2.1-2, at least equal to the factored gross area yield resistance given

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approximately 25000 that would be obtained for Equation 6.10.4.1.7-1). Consideration of a null moment at one end of the column accounts for changes in location of the inflexion point of the column moment diagram during earthquake response. Figure 10.27 in Bruneau et al. (1997) could be used to develop other unsupported lengths limits.

Built-up columns made of fastened components (bolted, riveted, etc.) are beyond the scope of Article 6.15.

C6.15.4.3.1.c

Since plastic hinges are not expected to form in beams, beams need not conform to plastic design requirements.

The requirement for beam resistance is consistent with the outlined capacity-design philosophy. The beams should either resist the full elastic loads or be capacity-protected. In the extreme load situation, the capacity-protected beams are required to have nominal resistances of not less than the combined effects corresponding to the plastic hinges in the columns attaining their probable capacity and the probable companion permanent load acting directly on the beams. The columns' probable capacity should account for the overstrength due to higher yield than specified yield and strain hardening effects. The value specified in Article 6.9.2.2, used in conjunction with the resistance factor for steel beams in flexure, ϕ , of 1.00, (Article 6.5.4.2) is compatible with the AISC (1997) $1.1R_y$ used with a resistance factor, ϕ , of 0.9 (here R_y is embedded in F_{ye}).

C6.15.4.3.1.d

The panel zone should either resist the full elastic load (i.e. $R=1.0$) or be capacity-protected.

Column base connections should also resist the full elastic loads ($R=1.0$) or be capacity-protected, unless they are designed and detailed to dissipate energy.

Panel zone yielding is not permitted.

There is a concern that doubler plates in panel zones can be an undesirable fatigue detail. For plate-girder sections, it is preferable to specify a thicker web plate if necessary rather than use panel zone doubler plates.

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by Equation 6.8.2.1-1, with A_g and A_n in Article 6.8.2.1 taken here as the area of the flanges and connection plates in tension.

6.15.4.3.1.e Multi-tier Frame Bents

For multi-tier frame bents, capacity design principles as well as the equations of Article 6.15.4.3.1 may be modified by the engineer to achieve column plastic hinging only at the top and base of the column, and plastic hinging at the ends of all intermediate beams. Column plastic hinging shall not be forced at all joints at every tier.

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C6.15.4.3.1.e

Multi-tier frame bents are sometimes used, mostly because they are more rigid transversely than single-tier frame bents. In such multi-tier bents, the intermediate beams are significantly smaller than the top beam as they are not supporting the gravity loads from the superstructure.

As a result, in a multi-tier frame, plastic hinging in the beams may be unavoidable, and desirable, in all but the top beam. In fact, trying to ensure strong-beam weak-column design at all joints in multi-tier bents may have the undesirable effect of concentrating all column plastic hinging in one tier, with greater local ductility demands than otherwise expected in design.

Using capacity design principles, the equations and intent of Article 6.15.4.3.1 may be modified by the engineer to achieve column plastic hinging only at the top and base of the column, and plastic hinging at the ends of all intermediate beams, as shown in Figure C6.15.4.3.1.e-1.



Figure C6.15.4.3.1.e-1: Acceptable plastic mechanism for multi-tier bent.

6.15.4.3.2 Ductile Concentrically Braced Frames

6.15.4.3.2.a General

Braces are the Ductile Substructure Elements in ductile concentrically braced frames.

Concentrically braced frames are those in which the centerlines of diagonal braces, beams, and columns are approximately concurrent with little or no joint eccentricity. Inelastic straining must take place in bracing members subjected principally to axial load. Compression members can absorb considerable energy by inelastic bending after buckling and in subsequent straightening after load reversal but the

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amount is small for slender members. Local buckling or buckling of components of built-up members also limits energy absorption.

6.15.4.3.2.b Bracing Systems

Diagonal braces shall be oriented such that a nearly identical ultimate strength is achieved in both sway directions, when considering only the strength contribution of braces in tension. To achieve this, it is required that, at any level in any planar frame, the sum of the horizontal components of the strength of the braces in tension when the frame sway in one direction, shall be within 30% of the same value for sway in the other direction.

Article 6.15.4.3.2 is only applicable to braced frames for which all braces' action lines meet at beam-to-column intersection points (such as X-braces).

This requirement ensures some redundancy and also similarity between the load-deflection characteristics in the two opposite directions. A significant proportion of the horizontal shear is carried by tension braces so that compression brace buckling will not cause a catastrophic loss in overall horizontal shear capacity. Alternative wording sometimes encountered to express the same intent include:

- (a) Diagonal braces shall be oriented such that, at any level in any planar frame, at least 30% of the horizontal shear carried by the bracing system shall be carried by tension braces and at least 30% shall be carried by compression braces.
- (b) Along any line of bracing, braces shall be deployed in alternate directions such that, for either direction of force parallel to the bracing, at least 30 percent but no more than 70 percent of the total horizontal forced is resisted by tension braces.

This ensures that structural configurations that depend predominantly on the compression resistance of braces (such as case (a) in Figure C6.15.4.3.2.b-1) are avoided. Case (b) in that same figure is a better design that meets the above criteria.

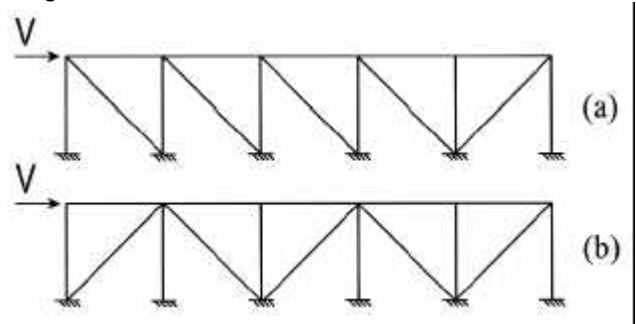


Figure C6.15.4.3.2.b-1: Examples of (a) Unacceptable and (b) Acceptable braced bent configurations.

This article also excludes bracing systems that have not exhibited the ductile behavior expected for ductile concentrically braced frames, such as:

- (a) Chevron bracing or V-bracing, in which pairs of braces are located either above or below a beam and meet the beam at a single point within the middle half of the span;
- (b) K-bracing, in which pairs of braces meet a column on one side near its mid-height; or
- (c) Knee-bracing.

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6.15.4.3.2.c *Design Requirements for Ductile Bracing Members*

Bracing members shall have a slenderness ratio, KL/r , less than $2600/\sqrt{F_y}$.

The width-to-thickness ratios of bracing members should be limited as indicated in Table 6.15.1. For back-to-back legs of double angle bracing members for which buckling out of the plane of symmetry governs, the width-to-thickness ratio shall not exceed $200/\sqrt{F_y}$.

In built-up bracing members, the slenderness ratio of the individual parts between stitches shall be not greater than 0.4 times the slenderness ratio of the member as a whole. When it can be shown that braces will buckle without causing shear in the stitches, the spacing of the stitches shall be such that the slenderness ratio of the individual parts does not exceed 0.75 times the slenderness ratio of the built-up member.

Until the late 1990's, for the ductile design of concentrically braced frames in buildings, the slenderness ratio limits for braces were approximately 75% of the value specified here. The philosophy was to design braces to contribute significantly to the total energy dissipation when in compression. Member slenderness ratio was restricted because the energy absorbed by plastic bending of braces in compression diminishes with increased slenderness. To achieve these more stringent KL/r limits, particularly for long braces, designers have almost exclusively used tubes or pipes for the braces. This is unfortunate as these tubular members are most sensitive to rapid local buckling and fracture when subjected to inelastic cyclic loading (in spite of the low width-to-thickness limits prescribed). Recent reviews of this requirement revealed that it may be unnecessary, provided that connections are capable of developing at least the member capacity in tension. This is partly because larger tension brace capacity is obtained when design is governed by the compression brace capacity, and partly because low-cycle fatigue life increases for members having greater KL/r . As a result, seismic provisions for buildings (AISC 1997; CSA 2001) have been revised to permit members having longer KL/r values. The proposed relaxed limits used here are consistent with the new recently adopted philosophy for buildings.

Early local buckling of braces prohibits the braced frames from sustaining many cycles of load reversal. Both laboratory tests and real earthquake observations have confirmed that premature local buckling significantly shortens the fracture life of HSS braces. The more stringent requirement on the b/t ratio for rectangular tubular sections subjected to cyclic loading is based on tests (Tang and Goel, 1987; Uang and Bertero, 1986). The b/t limit for circular sections is identical to that in the AISC plastic design specifications (AISC 1993; Sherman 1976).

6.15.4.3.2.d *Brace Connections*

The controlling overstrength capacity shall be taken as the axial tensile yield strength of the brace ($A_g F_{ye}$). Brace connections shall be designed as Capacity Protected Elements.

Connections must be designed to ensure that the bracing member is capable of yielding the gross section. Consequently, brace strength calculated based on tension rupture on the effective net section and block shear rupture, shall be greater than the design tensile strength of brace given by gross section yielding.

Eccentricities that are normally considered negligible (for example at the ends of bolted or welded angle members) may influence the failure mode of connections subjected to cyclic load (Astaneh, Goel and Hanson, 1986).

A brace which buckles out-of-plane will form a plastic hinge at mid-length and hinges in the gusset plate at each end. When braces attached to a single gusset plate buckle out-of-plane, there is a tendency for the plate to tear if it is restrained by its attachment to the adjacent frame members (Astaneh, Goel and

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Eccentricities in bracing connections shall be minimized.

Brace connections including gusset plates shall be detailed to avoid brittle failures due to rotation of the brace when it buckles. This ductile rotational behavior shall be allowed for, either in the plane of the frame or out of it, depending on the slenderness ratios.

The design of gusset plates shall also include consideration of buckling.

Stitches that connect the separate elements of built-up bracing members shall, if the overall buckling mode induces shear in the stitches, have a strength at least equal to the design tensile strength of each element. The spacing of stitches shall be uniform and not less than two stitches shall be used. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

6.15.4.3.2.e Columns, Beams, and Other Connections

Columns, beams, beam-to-column connections and column splices that participate in the lateral-load-resisting system shall be designed as Capacity Protected Elements with the following additional requirements:

- (a) Columns, beams and connections shall resist forces arising from load redistribution following brace buckling or yielding. The brace compressive resistance shall be taken as $0.3 \phi_c P_n$ if this creates a more critical condition.
- (b) Column splices made with partial penetration groove welds and subject to net tension forces due to overturning effects shall have Factored Resistances not less than 50% of the flange yield load of the smaller member at the splice.

6.15.4.3.3 Concentrically Braced Frames with Nominal Ductility

6.15.4.3.3.a General

Braces are the Ductile Substructure Elements in nominally ductile concentrically braced frames.

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Hanson, 1986). Provision of a clear distance, approximately twice the plate thickness, between the end of the brace and the adjacent members allows the plastic hinge to form in the plate and eliminates the restraint. When in-plane buckling of the brace may occur, ductile rotational behavior should be possible either in the brace or in the joint. Alternatively, the system could be designed to develop hinging in the brace, and the connections shall then be designed to have a flexural strength equal to or greater than the expected flexural strength $1.2R_y M_p$ of the brace about the critical buckling axis.

Buckling of double angle braces (legs back-to-back) about the axis of symmetry leads to transfer of load from one angle to the other, thus imposing significant loading on the stitch fastener (Astaneh, Goel and Hanson, 1986).

Columns and beams that participate in the lateral-load-resisting system must also be designed to ensure that a continuous load path can be maintained.

A reduced compressive resistance must be considered for this purpose. This takes into account the fact that, under cyclic loading, the compressive resistance of a bracing member rapidly diminishes. This reduction stabilizes after a few cycles to approximately 30% of the nominal compression capacity.

The unreduced brace compressive resistance must be used if it leads to a more critical condition, as it will be attained in the first cycle. However, redistributed loads resulting from the reduced buckled compressive brace loads must be considered in beams and columns as well as in connections, if it leads to a more critical condition.

Other connections that participate in the lateral-load-resisting system must also be designed to ensure that a continuous load path can be maintained. Therefore,

- (a) they must resist the combined load effect corresponding to the bracing connection loads and the permanent loads that they must also transfer; and
- (b) they must also resist load effect due to load redistribution following brace yielding or buckling.

Detailing requirements are relaxed for concentrically braced frames having nominal ductility

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6.15.4.3.3.b *Bracing Systems*

Diagonal braces shall be oriented such that a nearly identical ultimate strength is achieved in both sway directions, when considering only the strength contribution of braces in tension. To achieve this, it is required that, at any level in any planar frame, the sum of the horizontal components of the strength of the braces in tension when the frame sway in one direction, shall be within 30% of the same value for sway in the other direction.

The categories of bracing systems permitted by this Article includes:

- (a) tension-only diagonal bracing,
- (b) chevron bracing (or V-bracing) and,
- (c) direct tension-compression diagonal bracing systems of the geometry permitted in Article 6.15.4.3.2.2, but that do not satisfy all the requirements for ductile concentrically braced frames.

Tension-only bracing systems in which braces are connected at beam-to-column intersections are permitted in bents for which every column is fully continuous over the entire bent height, and where no more than 4 vertical levels of bracing are used along the bent height.

6.15.4.3.3.c *Design Requirements for Nominally Ductile Bracing Members*

Bracing members shall have a slenderness ratio, KL/r , less than $3750/\sqrt{F_y}$. This limit is waived for members designed as tension-only bracing.

In built-up bracing members, the slenderness ratio of the individual parts shall be not greater than 0.5 times the slenderness ratio of the member as a whole.

For bracing members having KL/r less than $2600/\sqrt{F_y}$, the width-to-thickness ratios of bracing members should be limited as indicated in Table 6.15.1. For bracing members that exceed that value, the width-to-thickness ratio limits can be obtained by linear interpolation between the values in Table 6.15.1 when KL/r is equal to $2600/\sqrt{F_y}$ and 1.3 times the values in Table 6.15.1 when KL/r is equal to $3750/\sqrt{F_y}$.

For back-to-back legs of double angle bracing members for which buckling out of the plane of

(a steel substructure having less stringent detailing requirements). They are consequently being designed to a greater force level.

This requirement ensures some redundancy and also similarity between the load-deflection characteristics in the two opposite directions. A significant proportion of the horizontal shear is carried by tension braces so that compression brace buckling will not cause a catastrophic loss in overall horizontal shear capacity.

Tension-only systems are bracing systems in which braces are connected at beam-to-column intersections and are designed to resist in tension 100% of the seismic loads.

K-braced frames, in which pairs of braces meet a column near its mid-height, and knee-braced frames shall not be considered in this category.

Systems in which all braces are oriented in the same direction and may be subjected to compression simultaneously shall be avoided.

Analytical and experimental research, as well as observations following past earthquakes, have demonstrated that K-bracing systems are poor dissipators of seismic energy. The members to which such braces are connected can also be adversely affected by the lateral force introduced at the connection point of both braces on that member due to the unequal compression buckling and tension yielding capacities of the braces.

Knee-braced systems in which the columns are subjected to significant bending moments are beyond the scope of this article.

Nominally ductile braced frames are expected to undergo limited inelastic deformations during earthquakes. Braces yielding in tension are relied upon to provide seismic energy dissipation. While frames with very slender braces (i.e. tension-only designs) are generally undesirable for multistoried frames in buildings, this is mostly because energy dissipation in such frames tend to concentrate in only a few stories, which may result in excessive ductility demands on those braces. However, non-linear inelastic analyses show that satisfactory seismic performance is possible for structures up to 4 stories with tension-only braces, provided that connections are capable of developing at least the member capacity in tension and that columns are continuous over the frame height (CSA 2001). The width-to-thickness ratios for the compression elements of columns can be relaxed for braces having KL/r

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symmetry governs, the width-to-thickness ratio limit can be taken as $200/\sqrt{F_y}$.

No width-to-thickness ratio limit is imposed for braces designed as tension-only members and having KL/r greater than $3750/\sqrt{F_y}$.

6.15.4.3.3.d *Brace Connections*

Brace connections shall be designed as Capacity Protected Elements. The controlling overstrength capacity shall be taken as the axial tensile yield strength of the brace ($A_g F_{ye}$).

For tension-only bracing the controlling probable resistance shall be multiplied by an additional factor of 1.10.

Connections must be designed to ensure that the bracing member is capable of yielding the gross section. Consequently, brace strength calculated based on tension rupture on the effective net section and block shear rupture, shall be less than the design tensile strength of brace given by gross section yielding.

Stitches that connect the separate elements of built-up bracing members shall, if the overall buckling mode induces shear in the stitches, have a strength at least equal to one-half of the design tensile strength of each element. The spacing of stitches shall be uniform and not less than two stitches shall be used. Bolted stitches shall not be located within the middle one-fourth of the clear brace length.

6.15.4.3.3.e *Columns, Beams and Other Connections*

Columns, beams, and connections shall be designed as Capacity Protected Elements.

6.15.4.3.3.f *Chevron Braced and V-Braced Systems*

Braces in chevron braced frames shall conform to the requirements of Article 6.15.4.3.3.c, except that bracing members shall have a slenderness ratio, KL/r , less than $2600/\sqrt{F_y}$. Tension-only designs are not permitted.

The beam attached to chevron braces or V-braces shall be continuous between columns and its top and bottom flanges shall be designed to resist a lateral load of 2% of the flange yield force ($F_y b_f t_{fb}$) at the point of intersection with the brace.

Columns, beams and connections shall be designed to resist forces arising from load redistribution following brace buckling or yielding, including the maximum unbalanced vertical load effect applied to the

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approaching 200, as members in compression do not yield at that slenderness.

The additional factor of 1.10 for tension-only bracing systems is to ensure, for the slender members used in this case, that the impact resulting when slack is taken up, does not cause connection failure. Details leading to limited zones of yielding, such as occur at partial joint penetration groove welds should be avoided.

Bracing at the beam-brace intersection in chevron and inverted-chevron frames is crucial to prevent lateral torsional buckling of the beam at that location. Effective lateral bracing requires structural elements framing transversely to the frame bent, which may be only possible in 4-column tower piers where horizontal members can be introduced to tie and brace all four faces of the tower pier. Alternatively, lateral bracing could be provided by a connection to the superstructure if proper consideration is given to fatigue and deformation compatibility.

Furthermore, geometry of the braced system must be chosen to preclude beam deformations that could translate into undesirable superstructure damage.

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beam by the braces. The brace compressive resistance shall be $0.3 \phi_c P_n$ if this creates a more critical condition.

A beam that is intersected by chevron braces shall be able to support its permanent dead and live loads without the support provided by the braces.

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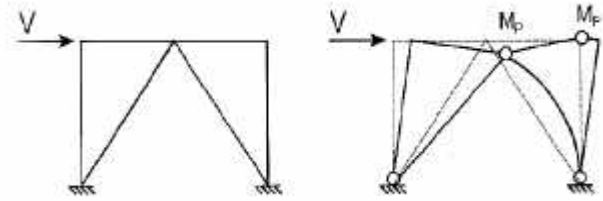


Figure 6.15.4.3.3.e-1: Plastic mechanism for a chevron braced bent configuration that would introduce undesirable superstructure damage (unless this bridge has only two girders that are located directly over the columns).

6.15.4.3.4 Concrete-filled Steel Pipes

6.15.4.3.4.a General

Concrete-filled steel pipes use as columns, piers, or piles expected to develop full plastic hinging of the composite section as a result of seismic response shall be designed in accordance with Articles 6.9.2.2, 6.9.5, 6.12.3.2.2, as well as the requirements in this Article 6.15.4.3.4.

C6.15.4.3.4

C6.15.4.3.4.a

This article is only applicable to concrete-filled steel pipes without internal reinforcement, and connected in a way that allows development of their full composite strength. It is not applicable to design a concrete-filled steel pipe that relies on internal reinforcement to provide continuity with another structural element, or for which the steel pipe is not continuous or connected in a way that enables it to develop its full yield strength. When used in pile bent, the full composite strength of the plastic hinge located below ground can only be developed if it can be ensured that the concrete fill is present at that location.

Recent research (e.g. Alfawakiri 1997, Bruneau and Marson 1999) demonstrates that the AASHTO equations for the design of concrete-filled steel pipes in combined axial compression and flexure (Articles 6.9.2.2, 6.9.5, and 6.12.3.2.2), provide a very conservative assessment of beam-column strength. Consequently, the calculated strength of concrete-filled steel pipes that could be used as columns in ductile moment resisting frames or pile-bents, could be significantly underestimated. This is not surprising given that these equations together are deemed applicable to a broad range of composite member types and shapes, including concrete-encased steel shapes. While these equations may be perceived as conservative in a non-seismic perspective, an equation that more realistically captures the plastic moment of such columns is essential in a capacity design perspective. Capacity-protected elements must be designed with adequate strength to elastically withstand plastic hinging in the columns. Underestimates of this hinging force translates into under-design of the capacity-protected elements; a column unknowingly stronger than expected will not yield before damage develops in the foundations or at other undesirable locations in the structure. This can

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6.15.4.3.4.b Combined Axial Compression and Flexure

Concrete-filled steel pipe members required to resist both axial compression and flexure and intended to be ductile substructure elements shall be proportioned so that:

$$\frac{P_u}{P_r} + \frac{BM_u}{M_{rc}} \leq 1.0$$

and

$$\frac{M_u}{M_{rc}} \leq 1.0$$

where P_r is defined in Articles 6.9.2.1 and 6.9.5.1, and M_{rc} is defined in Article 6.15.4.3.4.3

$$B = \frac{P_{ro} - P_{rc}}{P_{rc}}$$

P_{ro} = factored compressive resistance (Articles 6.9.2.1 and 6.9.5.1) with $\lambda = 0$

$$P_{rc} = \phi_c A_c f'_c$$

M_u is the maximum resultant moment applied to the member in any direction, calculated as specified in Article 4.5.3.2.2

6.15.4.3.4.c Flexural Strength

The factored moment resistance of a concrete filled steel pipe for Article 6.15.4.3.4.2 shall be calculated using either of the following two methods:

(a) Method 1 – Using Exact Geometry

$$M_{rc} = f_r [C_r e + C'_r e']$$

where

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be of severe consequences as the capacity protected elements are not detailed to withstand large inelastic deformations. The provisions of Article 6.15.4.3.4 are added to prevent this behavior.

Note that for analysis, as implied by Article 6.9.5, flexural stiffness of the composite section can be taken as $E_s I_s + 0.4 E_c I_c$, where I_c is the gross inertia of the concrete ($\pi D^4/16$), I_s is the inertia of the steel pipe, and E_s and E_c are respectively the steel and concrete modulus of elasticity.

C6.15.4.3.4.b.

This equation is known to be reliable up to a maximum slenderness limit D/t of $28000/F_y$, underestimating the flexural moment capacity by 1.25 on average. It may significantly overestimate columns strength having greater D/t ratios.

This new equation is only applicable to concrete-filled steel pipes. Other equations may be needed to similarly replace that of Article 6.9.2.2. for other types of composite columns (such as concrete-encased columns).

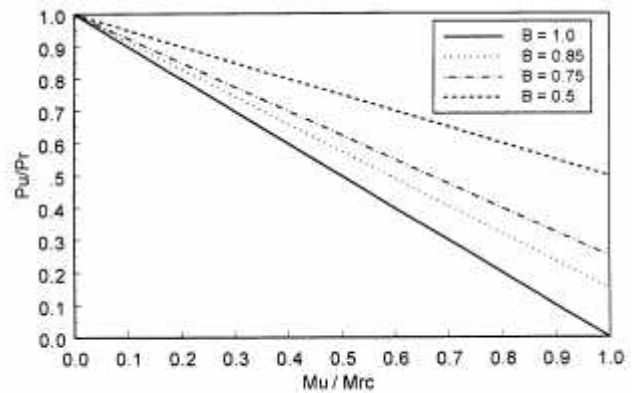


Figure C6.15.4.3.4.b-1: Interaction curves for concrete-filled pipes.

C6.15.4.3.4.c

When using these equations to calculate the forces acting on capacity protected members as a result of plastic hinging of the concrete-filled pipes, F_y should be replaced by F_{ye} , for consistency with the capacity design philosophy.

Figure C6.15.4.3.4.c-1 illustrates the geometric parameters used in this Article.

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$$C_r = F_y b \frac{Dt}{2}$$

$$C'_r = f'_c \left[\frac{bD^2}{8} - \frac{b_c}{2} \left(\frac{D}{2} - a \right) \right]$$

$$e = b_c \left[\frac{1}{(2p - b)} + \frac{1}{b} \right]$$

$$e' = b_c \left[\frac{1}{(2p - b)} + \frac{b_c^2}{1.5bD^2 - 6b_c(0.5D - a)} \right]$$

$$a = \frac{b_c}{2} \tan\left(\frac{b}{4}\right)$$

$$b_c = D \sin\left(\frac{b}{2}\right)$$

where β is in radians and found by the recursive equation:

$$b = \frac{A_s F_y + 0.25D^2 f'_c \left[\sin(b/2) - \sin^2(b/2) \tan(b/4) \right]}{(0.125 D^2 f'_c + Dt F_y)}$$

(b) Method 2 – Using Approximate Geometry

A conservative value of M_{rc} is given by

$$M_{rc} = f_t \left[(Z - 2th_n^2) F_y + \left[\frac{2}{3} (0.5D - t)^3 - (0.5D - t) h_n^2 \right] f'_c \right]$$

where

$$h_n = \frac{A_c f'_c}{2Df'_c + 4t(2F_y - f'_c)}$$

and Z is the plastic modulus of the steel section alone.

For capacity design purposes, in determining the force to consider for the design of capacity protected elements, the moment calculated by this approximate method shall be increased by 10%.

COMMENTARY

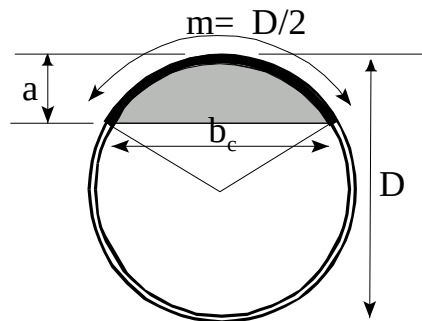
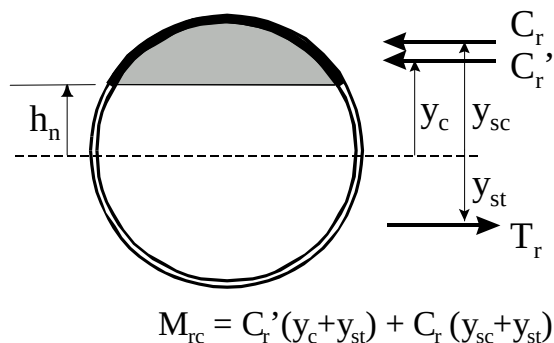


Figure C6.15.4.3.4.c-1: Flexure of concrete-filled pipe; shaded area is concrete in compression above the neutral axis.

Moment resistance is calculated assuming the concrete in compression at f'_c , and the steel in tension and compression at F_y . The resulting free-body diagram is shown in Figure C6.15.4.3.4.c-2, where e is equal to $y_{sc} + y_{st}$, e' is equal to $y_c + y_{st}$, and y_c is the distance of the concrete compressive force (C'_r) from the center of gravity, and y_{st} and y_{sc} are the respective distances of the steel tensile (T_r) and compressive forces (C_r) from the center of gravity.



$$M_{rc} = C'_r (y_c + y_{st}) + C_r (y_{sc} + y_{st})$$

Figure C6.15.4.3.4.c-2: Free-body diagram used to calculate moment resistance of concrete-filled pipe.

In Method 2, a geometric approximation is made in calculating the area of concrete in compression by subtracting the rectangular shaded area shown in Figure C6.15.4.3.4.c-3 from the total area enclosed by the pipe (and dividing the result by 2). Neutral axis is at height h_n .

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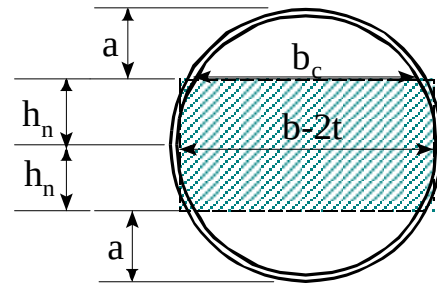


Figure C6.15.4.3.4.c-3: Flexure of concrete-filled pipe – illustrates approximation made in Method 2.

Method 2 (using approximate geometry) gives smaller moments compared to Method 1 (exact geometry). The requirement to increase the calculated moment by 10% for capacity design when using the approximate method was established from the ratio of the moment calculated by both methods for a D/t of 10. That ratio decreases as D/t increases.

6.15.4.3.4.d Beams and Connections

Capacity-protected members must be designed to resist the forces resulting from hinging in the concrete-filled pipes calculated from Article 6.15.4.3.4.2.

Recent experimental work by Bruneau and Marson (1999), Shama et al. (2001), Azizinamini et al. (1999), provide examples of full fixity connection details. Note that, in some instances, full fixity may not be needed at both ends of columns. Concrete-filled steel pipes, when used in pile bents, only require full moment connection at the pile-cap.

6.15.5 Special Systems

This Article provides minimum considerations that must be addressed for the design of special systems.

Article 6.15.5, Special Systems, contains systems less familiar to bridge engineers. Eccentrically braced substructures are included in this section partly for that reason, but also because most configurations of this system would introduce beam deformations that are undesirable in bridges as this could translate into superstructure damage. Furthermore, bracing of the links may be a difficult design issue that requires special consideration in bridge bents.

The engineer must take the necessary steps to ensure that special systems will provide a level of safety comparable to that provided in these Specifications. This may require review of published research results, observed performance in past earthquakes, and/or special investigations.

6.15.5.1 DUCTILE ECCENTRICALLY BRACED FRAMES

Ductile eccentrically braced frames for bents and towers may be used provided that the system, and in particular the eccentric link and link beam, can be

Note that the scope of 6.15.5.1 is for eccentrically braced frames used as ductile substructure, not as part of ductile diaphragms.

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demonstrated to remain stable up to the expected level of inelastic response. This demonstration of performance shall be preferably achieved through full-scale cyclic tests of specimens of size greater or equal to that of the prototype.

Seismic design practice for eccentrically braced frames used in buildings can be used to select width-to-thickness ratios, stiffeners spacing and size, and strength of the links, as well as to design diagonal braces and beams outside of the links, columns, brace connections, and beam-to-column connections.

Only the eccentric brace configuration in which the eccentric link is located in the middle of a beam is permitted.

COMMENTARY

Eccentrically braced frames have been extensively tested and implemented in numerous buildings, but, at the time of this writing, few new bridges have been built relying on shear links for seismic energy dissipation. An obvious difficulty in bridge applications arises because the eccentric link cannot be easily laterally braced to prevent movement out of the plane of the braced bent. Nonetheless, the bents of the Richmond-San Raphael bridge near San Francisco have been retrofitted using eccentrically braced frames. For that bridge, multiple adjacent frames were used to be able to provide proper bracing of the shear links. Large scale testing was conducted to validate that retrofit concept (Vincent 1996; Itani et al, 1998b). Furthermore, the tower of the new east bay crossing of the Bay Bridge between San Francisco and Oakland is connected by shear links, albeit not in an eccentrically braced frame configuration (Tang et al., 2000).

While effective eccentrically braced bents are possible, only details that have been tested with the same lateral bracing considerations as in the prototype must be used. Other details must be experimentally validated. Note that size effects have not been fully investigated. Although it is preferable to use links of sizes no greater than those validated by full-scale tests, in some instances, this may not be possible.

Extensive detailing requirements are not provided within these specifications. However, the engineer could follow the detailing practice used for buildings, modified to address the above concerns regarding lateral bracing.

The scope of this article is restricted to eccentrically braced frame of split-V configuration. Eccentrically braced frames configurations in which the ductile link is adjacent to a beam-column connection are prohibited, unless it can be demonstrated by tests of specimens of size greater or equal to the prototype that the connection can develop the required strength and hysteretic ductility.

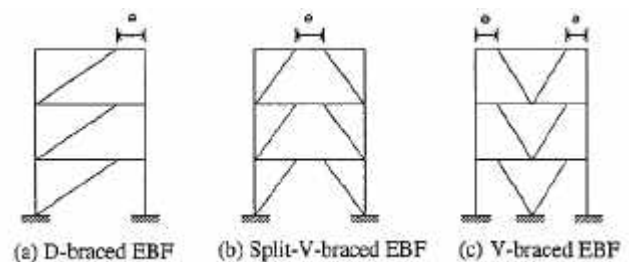


Figure C6.15.5.1-1: Eccentrically braces frames configurations, the scope of C6.15.5.1 being restricted to split-V configuration (case b).

Furthermore, geometry of the eccentrically braced system must be chosen to preclude beam deformations that could translate into undesirable superstructure damage,. As such, the configuration shown in Figure 6.15.15.5.1-1 would introduce undesirable superstructure damage, unless this bridge has only two girders that are located directly over the columns. In most cases, alternative configurations would be required.

For eccentrically braced frames, all references to “inelastic hinging of the column” in other seismic requirements elsewhere in the Specifications should be interpreted as “yielding of the eccentric link”.

6.15.5.2. DUCTILE END-DIAPHRAGMS IN SLAB-ON-GIRDER BRIDGES

Ductile end-diaphragms in slab-on-girder bridges can be designed to be the ductile energy dissipating elements for seismic excitations in the transverse directions of straight bridges provided that:

- (a) Specially detailed diaphragms capable of dissipating energy in a stable manner and without strength degradation upon repeated cyclic testing are used;
- (b) Only ductile energy dissipating systems whose adequate seismic performance has been proven through cycling inelastic testing are used;
- (c) Design considers the combined and relative stiffness and strength of end-diaphragms and girders (together with their bearing stiffeners) in establishing the diaphragms strength and design forces to consider for the capacity protected elements;
- (d) The response modification factor to be considered in design of the ductile diaphragm is given by:

$$R = \left(\frac{m + \frac{K_{DED}}{K_{SUB}}}{1 + \frac{K_{DED}}{K_{SUB}}} \right)$$

where μ is the ductility capacity of the end-diaphragm itself, and K_{DED}/K_{SUB} is the ratio of the stiffness of the ductile end-diaphragms and substructure; unless the engineer can demonstrate otherwise, μ should not be taken greater than 4;

- (e) All details/connections of the ductile end-diaphragms are welded.
- (f) The bridge does not have horizontal wind-bracing connecting the bottom flanges of girders, unless the last wind bracing panel before each support is designed as a ductile panel equivalent and in parallel to its adjacent

The ductile diaphragm strategy is not effective when the substructure is significantly more flexible than the superstructure. This is addressed by Article 6.15.5.2.d. Bridges having wide piers, wall-piers, or other substructure elements of similar limited ductility, would be good candidates for the implementation of the ductile diaphragm system. In these examples, the ductile diaphragms could also be designed to yield instead of the bridge piles, thus preventing the development of damage below ground level where it cannot be inspected following an earthquake.

The contribution of girders can be significant and cannot be neglected, as indicated in Article 6.15.5.2.c. For that reason, ductile diaphragm are generally more effective in longer span bridges, and may be of limited benefit for short span bridges.

Note that the inertia forces attributable to the mass of the pier-cap will be resisted by the substructure, in spite of the presence of ductile diaphragms. Refined analyses should consider this condition if that mass is a significant portion of the total superstructure mass.

For ductile end-diaphragms, all references to “inelastic hinging of the column” in other seismic requirements elsewhere in the Specifications should be interpreted as “yielding of the ductile diaphragm”.

A detailed procedure for the design of ductile diaphragms is presented in Appendix 6A, along with illustrations of systems that would satisfy the restrictions of Articles 6.15.5.2.a and 6.15.5.2.b.

vertical end-diaphragm.

- (g) An effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the slab to the diaphragm.

Overstrength factors to be used to design the capacity-protected elements depend on the type of ductile diaphragm used, and shall be based on available experimental research results.

6.15.5.3. DUCTILE END-DIAPHRAGMS IN DECK TRUSS BRIDGES

Ductile end-diaphragms in deck-truss bridges can be designed to be the ductile energy dissipating elements for seismic excitations in the transverse directions of straight bridges provided that:

- (a) Specially detailed diaphragms capable of dissipating energy in a stable manner and without strength degradation upon repeated cyclic testing are used;
- (b) Only ductile energy dissipating systems whose adequate seismic performance has been proven through cycling inelastic testing are used;
- (c) The last lower horizontal cross-frame before each support is also designed as a ductile panel equivalent and in parallel to its adjacent vertical end-diaphragm;
- (d) Horizontal and vertical energy dissipating ductile panels are calibrated to have a ratio of stiffness approximately equal to their strength ratio;
- (e) The concrete deck is made continuous between supports (and end-diaphragms), and an effective mechanism is present to ensure transfer of the inertia-induced transverse horizontal seismic forces from the deck to the diaphragms.;
- (h) The response modification factor to be considered in design of the ductile diaphragm is given by:

$$R = \frac{\left(m + \frac{K_{DED}}{K_{SUB}} \right)}{1 + \frac{K_{DED}}{K_{SUB}}}$$

where μ is the ductility capacity of the end-diaphragm itself, and K_{DED}/K_{SUB} is the ratio of the stiffness of the ductile end-diaphragms and substructure; unless the engineer can demonstrated otherwise, μ should not be taken greater than 4;

- (i) All capacity-protected members are demonstrated able to resist without damage or

Articles 6.15.5.3. and 6.15.5.2 share much conceptual similarities, but seismic forces in deck-trusses follow a more complex and redundant load-path. This requires the use of ductile diaphragms vertically over the supports as well as horizontally in the last lower horizontal cross-frame before each support.

For ductile end-diaphragms, all references to “inelastic hinging of the column” in other seismic requirements elsewhere in the Specifications should be interpreted as “yielding of the ductile diaphragm”.

Further research may allow to relax the limits imposed by Articles 6.15.5.3.d and 6.15.5.3.e

A detailed procedure for the design of ductile diaphragms is presented in Appendix 6B.

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instability the maximum calculated seismic displacements.

Overstrength factors to be used to design the capacity-protected elements depend on the type of ductile diaphragm used, and shall be based on available experimental research results.

6.15.5.4 OTHER SYSTEMS

Other framing systems and frames that incorporate special bracing, active control, or other energy-absorbing devices, or other types of special ductile superstructure elements shall be designed on the basis of published research results, observed performance in past earthquakes, or special investigation, and provide a level of safety comparable to those in these AASHTO Specifications.

6.15.6 PLASTIC ROTATIONAL CAPACITIES

The plastic rotational capacity shall be based on the appropriate performance limit state for the bridge. In lieu of the prescriptive values given below, the designer may determine the plastic rotational capacity from tests and/or a rational analysis.

6.15.6.1 LIFE-SAFETY PERFORMANCE

A conservative values of $\theta_p=0.035$ radians may be assumed.

6.15.6.2 IMMEDIATE USE LIMIT STATE

To ensure the immediate use of the bridge structure following a design ground motion, the maximum rotational capacity should be limited to $\theta_p=0.005$ radians.

6.15.6.3 IN GROUND HINGES

The maximum rotational capacity for in-ground hinges should be restricted to $\theta_p=0.01$ radians.

COMMENTARY

Note that many other "special systems" may emerge in the future, such as friction-braced frames, shock transmission units, other approaches of superstructure plastic hinging, marine bumpers etc.

A moment-curvature analysis based on strain compatibility and non-linear stress-strain relations can be used to determine plastic limit states. From this, a rational analysis is used to establish the rotational capacity of plastic hinges.

In-ground hinges are necessary for certain types of bridge substructures. These may include, but not restricted to:

- Pile bents
- Pile foundations with strong pier walls
- Drilled shafts
- Piled foundations with oversized columns

It is necessary to restrict these plastic hinge rotations in order to limit plastic strains. This limit is expected to reduce plastic strains to less than 10 percent of their above-ground counterpart (with $\theta_p=0.035$ radians), due to the increased plastic hinge length of in-ground hinges.

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Table 6.15.1 Limiting Width-to-Thickness Ratios

Description of element	Width-to-thickness ratio (b/t) ¹	Limiting width-to-thickness ratio $\frac{l_p^2}{\sqrt{F_y}}$	Limiting width-to-thickness ratio k^3
Flanges of I-shaped sections and channels in compression	$\frac{b_f}{2t_f}$	$\frac{135}{\sqrt{F_y}}$	0.30
Webs in combined flexural and axial compression	$\frac{h_c}{t_w}$	For $\frac{P_u}{\Phi_b P_y} \leq 0.125$ $\frac{1365}{\sqrt{F_y}} \left(1 - \frac{1.54 P_u}{\Phi_b P_y} \right)$ For $\frac{P_u}{\Phi_b P_y} > 0.125$ $\frac{500}{\sqrt{F_y}} \left(2.33 - \frac{P_u}{\Phi_b P_y} \right) \geq \frac{665}{\sqrt{F_y}}$	For $\frac{P_u}{\Phi_b P_y} \leq 0.125$ $3.05 \left(1 - \frac{1.54 P_u}{\Phi_b P_y} \right)$ For $\frac{P_u}{\Phi_b P_y} > 0.125$ $1.12 \left(2.33 - \frac{P_u}{\Phi_b P_y} \right) \geq 1.48$
Hollow circular sections (pipes)	$\frac{D}{t}$	$\frac{8950}{F_y}$	$\frac{200}{\sqrt{F_y}}$
Unstiffened rectangular tubes	$\frac{b}{t}$	$\frac{300}{\sqrt{F_y}}$	0.67
Legs of angles	$\frac{b}{t}$	$\frac{145}{\sqrt{F_y}}$	0.32

1. Width-to-thickness ratios of compression elements – Note that these are more stringent for members designed to dissipate hysteretic energy during earthquake than for other members (Article 6.9.4.2).
2. Limits expressed in format to satisfy the requirement $\frac{b}{t} \leq l_p$
3. Limits expressed in format to satisfy the requirement $\frac{b}{t} \leq k \sqrt{\frac{E}{F_y}}$
4. Note: In the above, b_f and t_f are respectively the width and thickness of an I-shaped section, h_c is the depth of that section and t_w is the thickness of its web.

6A1.1 DESIGN PROCEDURE

A seismic design strategy that relies on ductile end-diaphragms inserted in the steel superstructure can be, in some instances, an effective alternative to energy dissipation in the substructure. This could be the case, for example, when stiff wall-piers that can difficultly be detailed to have a stable ductile response are used as a substructure. The ductile diaphragms considered in this Article are therefore those that can be specially designed and calibrated to yield before the strength of the substructure is reached (substructural elements, foundation, and bearings are referred generically as “substructure” here). Many types of systems capable of stable passive seismic energy dissipation could be used for this purpose. Among those, eccentrically braced frames (EBF) (e.g. Malley and Popov 1983; Kasai and Popov 1986), shear panel systems (SPS) (Fehling et al. 1992; Nakashima 1995), and steel triangular-plate added damping and stiffness devices (TADAS) (Tsai et al. 1993), popular in building applications, have been studied for bridge applications (Zahrai and Bruneau 1999a, 1999b). These are illustrated in Figures 6A1-1 to 6A1-3. Although concentrically braced frames can also be ductile, they are not admissible in Article 6.15.5.2 because they can often be stronger than calculated, and their hysteretic curves can exhibit pinching and some strength degradation.

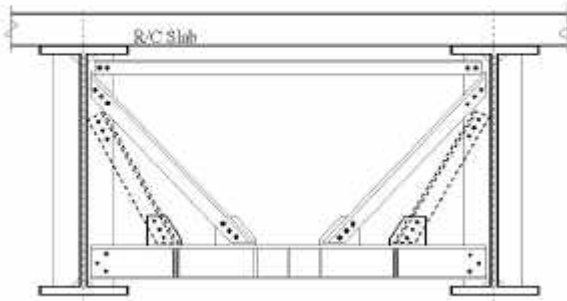


Figure 6A1-1 EBF Ductile Diaphragms

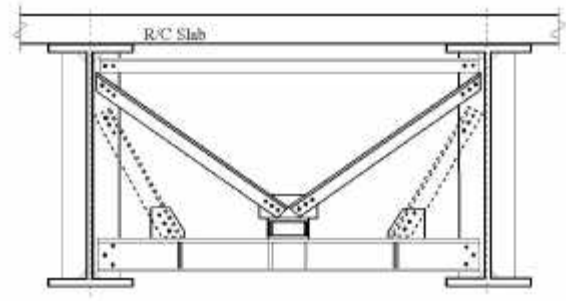


Figure 6A1-2 SPS Ductile Diaphragms

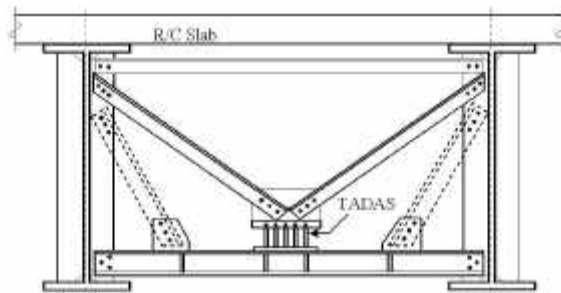


Figure 6A1-3 TADAS Ductile Diaphragms

Note that the plate girders can also contribute to the lateral load resistance, making the end-diaphragm behave as a dual system. Therefore, the lateral stiffness of the stiffened girders, SK_g , must be added to the stiffness of the ductile diaphragms, SK_{DD} (usually much larger than the former), to obtain the lateral stiffness of the bridge end-diaphragms (adding the stiffnesses of both ends of the span), K_{ends} , i.e:

$$K_{ends} = \sum K_{DD} + \sum K_g \quad (6A1.1-1)$$

The stiffness contribution of a plate girder is obviously a function of the fixity provided to its top and bottom flanges by the deck slab and bearing respectively. If full fixity is provided at both flanges of the plate girder,

$$K_g = \frac{12EI_g}{h_g^3} \quad (6A1.1-2)$$

where I_g is the moment of inertia of the stiffened stub-girder (mainly due to the bearing web stiffeners) in the lateral direction, and h_g is its height. If one end is fully fixed, the other one pinned,

$$K_g = \frac{3EI_g}{h_g^3} \quad (6A1.1-3)$$

If both ends effectively behave as pin supports, $K_g=0$. Full fixity at the deck level in composite bridges is possible if shear studs are closely spaced and designed to resist the pull-out forces resulting from the moments developed at the top of the girders under lateral seismic forces. As for fixity at the bearing level, it obviously depends on the type of bearings present. However, even when infinitely rigid bearings are present, full fixity is still difficult to ensure due to flexibility of the girder flanges, as revealed by finite element analyses of subassemblies at the girder-to-bearing connection point.

It is the engineer's responsibility to determine the level of fixity provided at the ends of the girders. However, contrary to conventional design, the most conservative solution is not obtained when zero fixity is assumed because fixity also adds strength to the diaphragms, and the role of the ductile diaphragms is to limit the magnitude of the maximum forces that can develop in the substructure.

The lateral stiffness of the ductile diaphragms, K_{DD} , depends on the type of ductile device implemented. For example, if a ductile SPS is used, the stiffness of one such end-diaphragm in a slab-on-girder bridge, K_{SPS} , can be obtained by:

$$K_{SPS} = \frac{E}{\frac{l_b}{2A_b \cos^2 a} + \frac{L_s}{4A_{bb}} + \left(\frac{h_l^3}{3I_l} + \frac{2.6h_l}{A_{s,l}} \right) + \frac{L_s (h_l + d_{bb}/2)^2}{12I_{bb}} + \frac{H \tan^2 a}{2A_g}} \quad (6A1.1-4)$$

where E is the modulus of elasticity, l_b and A_b are the length and area of each brace, a is the brace's angle with the horizontal, L_s is the girder spacing, d_{bb} , A_{bb} and I_{bb} are the depth, cross sectional area and moment of inertia for the bottom beam, h_l , I_l and $A_{s,l}$ are the length, moment of inertia and shear area of the link, and H and A_g are the height and area of the stiffened girders.

Similarly, lateral stiffness of the EBF and TADAS implemented as end-diaphragms of slab-on-girder bridges, K_{EBF} and K_{TADAS} , can be computed as follows:

$$K_{EBF} = \frac{E}{\frac{l_b}{2A_b \cos^2 a} + \frac{a}{2A_l} + \frac{e^2 H^2}{12L_s I_l} + \frac{1.3eH^2}{aL_s A_{s,l}} + \frac{H \tan^2 a}{2A_g}} \quad (6A1.1-5)$$

$$K_{TADAS} = \frac{E}{\frac{l_b}{2A_b \cos^2 a} + \frac{L_s}{4A_{bb}} + \frac{6h_T^3}{Nb_T t_T^3} + \frac{L_s(h_T + d_{bb}/2)^2}{12I_{bb}} + \frac{H \tan^2 a}{2A_g}} \quad (6A1.1-6)$$

where a is the length of the beam outside the link, e , l_b , A_b and $A_{s,l}$ are the length, moment of inertia, cross sectional and shear areas of the link, N , h_T , b_T , and t_T are the number, height, width and thickness of the TADAS plates, and all other parameters are as defined previously. Note that of the five terms in the denominator of Equations 6A1.1-4 to 6A1.1-6, the second and fifth which account for axial deformations of bottom beam and stiffened girders could be ignored, and the fourth (accounting for the rotation of bottom beam at midspan in SPS and TADAS) could have a small impact if the bottom beam was a deep and stiff beam, which is not however always the case.

For a bridge having a given number of girders, n_g , number of end-diaphragms implemented at each support, n_d , and girder spacing, L_s , the design procedure for a ductile diaphragm consists of the following steps (illustrated in Figure 6A.1-4):

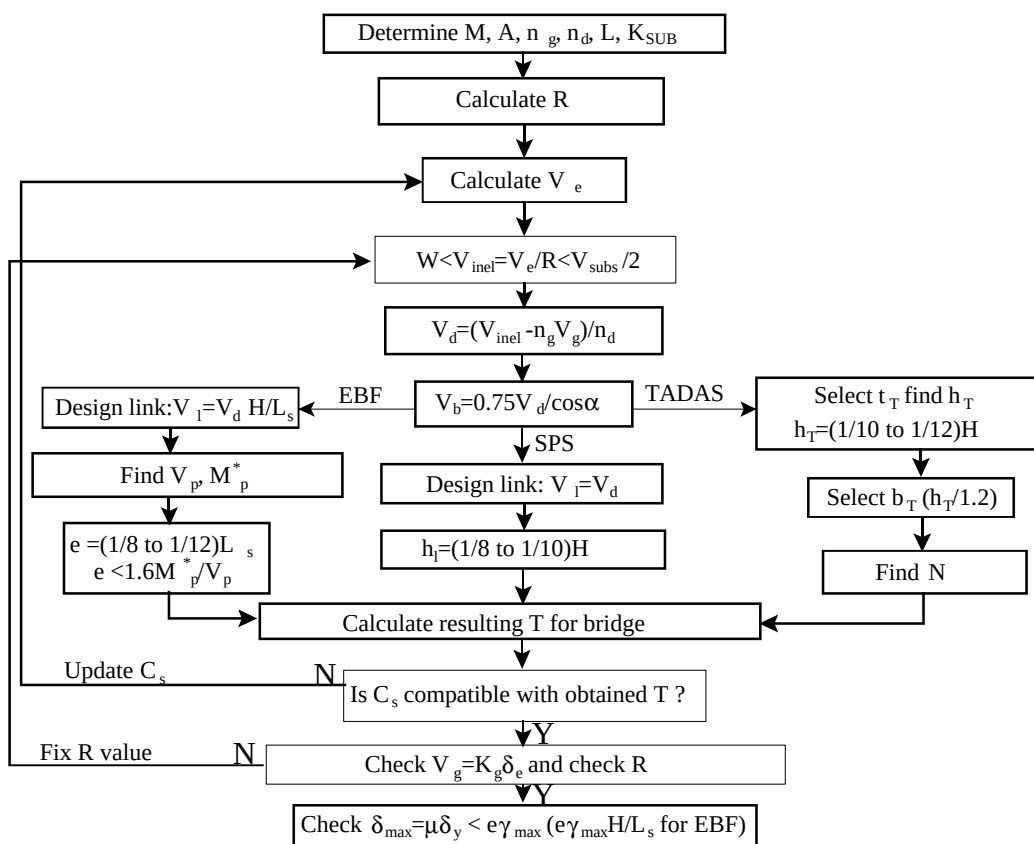


Figure 6A1-4: Flow Chart of Design Process for Ductile Diaphragm

- 1) Determine the elastic seismic base shear resistance, V_e , for one end of the bridge (half of equivalent static force).
- 2) Calculate $V_{inel} = V_e / R$, where V_{inel} is the inelastic lateral load resistance of the entire ductile diaphragm panel at the target reduction factor, and R is the force reduction factor calculated as indicated in Article 6.15.5.2. Note that m in that equation represents the ductility capacity of the

ductile diaphragm as a whole, not the local ductility of the ductile device that may be implemented in that diaphragm.

- 3) Determine the design lateral load, V_d , to be resisted by the energy dissipation device (e.g. link beam or TADAS) at the target ductility level, by:

$$V_d = \frac{V_{inel} - n_g V_g}{n_d} \quad (6A1.1-7)$$

where V_g is the lateral load resistance of one stiffened girder. Note that in short bridges, V_g can be a dominant factor that could overwhelm the resistance contribution provided by the special ductile diaphragm elements. In that perspective, it is recommended in this procedure that the bearing stiffeners at the support of these girders be trimmed to the minimum width necessary to satisfy the strength and stability requirements. Ideally, the braced diaphragm assembly should also be 5 to 10 times stiffer than the girders with bearing web stiffeners (even though ductility demand tends to be larger in stiffer structures) to prevent, or at least minimize, yielding in the main girders under transverse displacements. Note that in longer bridges, particularly those with a lesser number of girders per cross-section, the contribution of the girders to lateral load resistance is nearly insignificant.

- 4) Design all structural members and connections of the ductile diaphragm, with the exception of the seismic energy dissipation device, to be able to resist forces corresponding to $1.5V_d$ to account for potential overstrength of the ductile device due to strain hardening, strain rate effects and higher than specified yield strength. For example, braces should be designed to resist an axial compression force, V_b , equal to:

$$V_b = 1.5 \left(\frac{V_d}{2 \cos a} \right) = 0.75 \frac{V_d}{\cos a} \quad (6A1.1-8)$$

Likewise, for the SPS and TADAS systems, the bottom beam should be designed to resist a moment equal to $1.5 V_d h_l$ or $1.5 V_d h_T$. Moreover, for a given SPS or TADAS device, it is also advantageous to select a flexurally stiff bottom beam to minimize rigid-body rotation of the energy dissipating device and thus maximize hysteretic energy at a given lateral deck displacement.

- 5) Design the energy dissipating device. For the link beam in an EBF end-diaphragm, the shear force V_l in the link is:

$$V_l = \frac{H}{L_s} V_d \quad (6A1.1-9)$$

The plastic shear capacity V_p of a wide flange steel beam is given by Equation 6.10.7.3.3c-2:

$$V_p = 0.58 F_y t_w d_l \quad (6A1.1-10)$$

where F_y is the yield stress of steel, t_w is the web thickness, and d_l is the depth of the beam. The moment simultaneously applied to the link must be less than the reduced moment capacity, M_p^* , of the link yielding in shear and equal to (Malley and Popov 1983):

$$M_p^* = t_f b_f F_y (d_l - t_f) \quad (6A1.1-11)$$

Since shear links are more reliable energy dissipators than flexural links (Kasai and Popov 1986; American Institute 1992), shear links are favored and their length is therefore limited by the equation below:

$$e < e_{\max} = 1.6 \frac{M_p^*}{V_p} \quad (6A1.1-12)$$

A link length, e , of 1/8 to 1/12 of the girder spacing, L_s , is recommended for preliminary design, the less restrictive value preferred for practical reasons (i.e. detailing constraints) in presence of closely spaced girders. Deeper link beams are also preferred as the resulting larger flexural stiffness enhances the overall stiffness of the ductile device, ensuring that its yield displacement is reached much before onset of yielding of the stiffened girders.

For a SPS, the above procedure would be followed with the obvious exception that $V_f = V_d$ and the height of panel should be limited to half of the value obtained by the above equation since the yielding link is only in single curvature, as opposed to double curvature for the EBF. A link height of 1/8 to 1/10 of the girder depth is recommended for preliminary design. However, for a TADAS system, replace step 5 with step 6:

- 6) Select a small plate thickness, t_T , based on available plate size. The shear strength, V_T , and the stiffness, K_T , of a TADAS device can be determined from (Tsai et al. 1993):

$$V_T = \frac{Nb_T t_T^2 F_y}{4h_T} \quad (6A1.1-13)$$

$$K_T = \frac{NEb_T t_T^3}{6h_T^3} \quad (6A1.1-14)$$

where N , b_T , t_T and h_T are the number, base width, thickness and height of the triangular steel plates. The ratio of the above equations directly provides a relationship between h_T and t_T :

$$h_T = \sqrt{\frac{2Et_T V_T}{3F_y K_T}} \quad (6A1.1-15)$$

Here, $V_T = V_d$ and a h_T of $H/10$ to $H/12$ is recommended. Hence, if a reasonable estimate of the desirable K_T for the TADAS device is possible, t_T can be determined directly from h_T . In turn, b_T can be chosen knowing that triangular plates with aspect ratio, h_T/b_T , between 1 and 1.5 are better energy dissipators, based on experimental results (Tsai et al. 1993). Finally, N can then be calculated. Small adjustments to all parameters follow as N is rounded up to the nearest whole number. Incidentally, many different yet appropriate TADAS systems could be designed within these constraints. Systems with thinner steel plates perform better.

- 7) Calculate the stiffness of the ductile end-diaphragm by using the equation presented earlier in this commentary. Review the assumed lateral period of the bridge, T , and update calculation as necessary.
- 8) For the maximum lateral drift of the bridge at the diaphragm location, $d_{\max}$, check that the maximum ductility capacity of ductile device is not exceeded. For shear links, this is commonly expressed in terms of the maximum link deformation angle, $g_{\max}$ (easily obtained by dividing the maximum relative

displacements of link ends by the link length), the maximum drift for the SPS and EBF diaphragms is respectively limited to:

$$d_{\max} < eg_{\max} \quad (6A1.1-16)$$

$$d_{\max} < \frac{eH}{L_s} g_{\max} \quad (6A1.1-17)$$

with generally accepted $g_{\max}$ limits of 0.08 (AISC 1997). Note that, for the SPS diaphragms, the following alternative equation accounting for the rotation of bottom beam at the link connection may be more accurate when this factor has an important impact:

$$d_{\max} < e \left(g_{\max} + \frac{V_d L_s (h_l + d_{bb} / 2)}{12EI_{bb}} \right) \quad (6A1.1-18)$$

Should these limits be violated, modify the link's depth and length as well as the stiffness of the EBF or SPS diaphragm as necessary, and repeat the design process. Finally, a maximum drift limit of 2% of the girder height is also suggested here, at least until experimental evidence is provided to demonstrate that higher values are acceptable.

Note that the ductile energy dissipating elements should be laterally braced at their ends to prevent out-of-plane instability. These lateral supports and their connections should be designed to resist 6% of the nominal strength of the beam flange, i.e. $0.06F_y t_f b_f$ (AISC 1997). In addition, to prevent lateral torsional buckling of beams in the SPS, EBF, and TADAS end-diaphragms, the unsupported length, L_u , of these beams shall not exceed $200b_f // F_y$ where b_f is the width of beam flange in metre and F_y is the yield strength of steel in MPa.

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6B.1 DESIGN PROCEDURE

Similarly to the procedure described in Appendix 6C, a seismic design strategy that relies on ductile end-diaphragms inserted in the steel superstructure of deck-truss bridges can be, in some instances, an effective alternative to energy dissipation in the substructure. This could be the case, for example, when stiff wall-piers that can difficultly be detailed to have a stable ductile response are used as a substructure. The ductile diaphragms considered in this Article are therefore those that can be specially designed and calibrated to yield before the strength of the substructure is reached (substructural elements, foundation, and bearings are referred generically as “substructure” here).

Seismically generated inertia forces in deck-trusses can follow two possible load paths from the deck to the supports. As a result, to implement the ductile diaphragm strategy in such bridges, it is necessary to locate yielding devices in both the end-cross frames and in the lower end panels adjacent to the supports. This is illustrated in Figure 6B1-1.

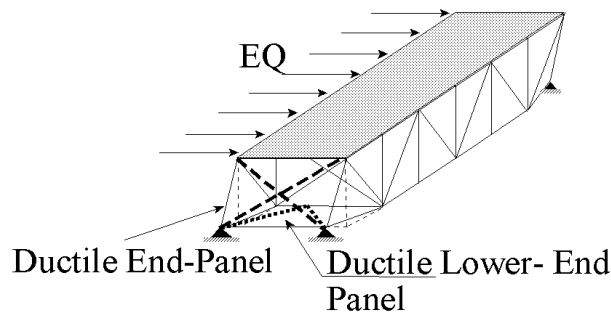


Figure 6B1-1: Ductile diaphragm concept in deck trusses

The methodology described in this Appendix is limited to simply supported spans of deck trusses. Until further research demonstrates otherwise, the design concept currently also requires stiffening of the top truss system, which can be achieved by making the concrete deck continuous and composite. This stiffening of the top truss system has two benefits. First, for a given deck lateral displacement at the supports, it reduces mid-span sway, resulting in lower forces in the interior cross-frames. Second, it increases the share of the total lateral load transferred through the top load path.

Note that the design strategy presented here only provides enhanced seismic resistance and substructure protection for the component of seismic excitation transverse to the bridge, and must be coupled with other devices that constraint longitudinal seismic displacements, such as simple bearings strengthening, rubber bumpers and the likes.

Under transverse earthquake excitation, end-diaphragms are designed to be the only energy dissipation elements in these bridges. The remaining structural components must be designed to remain elastic (i.e. capacity protected). Some restrictions on stiffness are necessary to prevent excessive ductility demands in the panels and excessive drift and deformations in other parts of the superstructure. The engineer must identify the displacement constraints appropriate to specific bridges; these will vary depending on the detailing conditions germane to the particular bridge under consideration. Generally, among those limits of important consequences, the maximum permissible lateral displacement of the deck must not exceed the values at which:

- P- Δ effects causes instability of the end verticals during sway of the end panel or damage to the connections of the end verticals;
- Unacceptable deformations start to develop in members or connections of the deck-truss, such as inelastic distortion of gusset plates, premature bolt or rivet failures, or damage to structural members;

- The energy dissipating devices used in the ductile panels reach their maximum deformation without loss of strength. This requires, for each type of energy dissipating devices considered, engineering judgement and experimental data on the device's ultimate cyclic inelastic performance, often expressed by a consensus opinion. For a given geometry, the ductility demand on the energy dissipating elements is related to the global ductility demand of the deck-truss. Therefore, global stiffness of the structure must be determined so as to keep global ductility and displacement demands within reasonable limits. Stiffness of the ductile devices has dominant effect on the overall stiffness, and this provides the control necessary for design.

Finally, it is recommended that the stiffness of the ductile panels be kept proportional to their respective capacity, as much as possible, to ensure that yielding in all ductile panels occurs nearly simultaneously. This should enhance energy dissipation capability and minimize the differences in the local ductility demands between the various yielding devices. It also helps prevent sudden changes in the proportion of the load shared between the two load paths, and minimize possible torsion along the bridge axis resulting from the instantaneous eccentricity that can develop when the end ductile panels yield first while the lower end ductile panels are still elastic.

General Design Methodology

Conceptually, any type of ductile energy dissipation system could be implemented in the end panels and lower end panels of the deck-truss, as long as its stiffness, ductility, and strength characteristics satisfy the requirements outlined in this appendix. The design methodology is iterative (initial properties must be assumed), and contains the following general steps.

1. Calculate Fundamental Period of Vibration

The fundamental period for the transverse mode of vibration is given by:

$$T = 2p \sqrt{\frac{M}{K_{Global}}} \quad (6B.1-1)$$

where M is the total mass of the deck, and K_{Global} , is given by:

$$K_{Global} = 2(K_{E,S} + K_{L,S}) \quad (6B.1-2)$$

where $K_{E,S}$ is the stiffness of the ductile end cross-frames, taking into account the contribution to stiffness of the braces, verticals, horizontal, and ductile energy dissipation device/system, and $K_{L,S}$ is given by:

$$K_{L,S} = \frac{K^* K_{L,E}}{K^* + K_{L,E}} \quad (6B.1-3)$$

where $K_{L,E}$ is the stiffness of the ductile last lower lateral panel, and

$$K^* = \frac{K_{C,B} + \sqrt{K_{C,B}^2 + 4K_{C,B}K_{L,B}}}{2} \quad (6B.1-4)$$

where $K_{L,B}$ represents the lateral stiffness of each panel of the lower lateral system (considering only the contribution of the braces to the panel stiffness) and $K_{C,B}$ represents the stiffness of the cross bracing panels (considering only the contribution of the braces to the panel stiffness).

The above equations are valid for a truss having at least 6 panels along its length. Otherwise, other equations can be derived following the procedure described in Sarraf and Bruneau (1998a).

2. Determine Design Forces

Although use of the capacity spectrum or push-over analysis is recommended for the design of such bridges, design is also possible using the R-factor approach. In that case, from the elastic seismic base shear resistance, V_e , for one end of the bridge (half of equivalent static force), it is possible to calculate $V = V_e / R$, where V is the inelastic lateral load resistance of the entire ductile diaphragm panel at the target reduction factor, and R is the force reduction factor calculated as indicated in Article 6.15.5.2. Note that μ in that equation represents the ductility capacity of the ductile diaphragm as a whole, not the local ductility of the ductile device that may be implemented in that diaphragm.

3. Determine Strength Constraints for Ductile Diaphragms in End Panels

The upper limit for the transverse shear capacity of each end cross-frame panel, $V_{E,S}$, can be determined from the following:

$$1.5V_{E,S} \leq \min\left(\frac{P_{cr}b}{h}, \frac{T_r b}{h}\right) \quad (6B.1-5)$$

where, P_{cr} is the critical buckling load of the end verticals including the effect of vertical gravity as well as vertical inertia force due to earthquake, T_r is the tensile capacity of the tie down device at each support, h , and b are height and width of the end cross-frame panel, respectively, and 1.5 is an overstrength factor.

4. Determine Strength Constraints for Ductile Diaphragms in Lower End Panels

Analyses showed that the force distribution in the interior cross-frames along the span is non-linear and of a complex shape. The model used to develop the equations presented here gives a conservative value of the lower end panel capacity, $V_{L,E}$, i.e. it ensures that $V_{L,E}$ is reached before any damage develops in any of the interior cross-frame.

The lower end panel capacity shall not exceed the maximum end-panel force attained when the first sway-frame force reaches its strength limit state, S_{cr} (corresponding to buckling of its braced members, fracture of a non-ductile connection, or other strength limit states), and defined by:

$$1.5V_{L,E} \leq \frac{\left(\sum_{i=1}^m (1-x)^{i-1} - m(1-x)^{m-1}\right) S_{cr}}{1 - (1-x)^{m-1}} \quad (6B.1-6)$$

where m is the number of interior cross-frames from the support to mid-span, 1.5 is the overstrength factor, and where:

$$x = \left(\frac{K_{C,B}}{K_{C,B} + \frac{K^* K_{L,B}}{K^* + K_{L,B}}} \right) \quad (6B.1-7)$$

Note that if the total number of interior cross-frames, k , in a deck-truss is an even number (i.e $m=(k+1)/2$, is not an integer), m can be conservatively taken as $k/2$.

Interior cross-frames shall be designed to resist the force R_1' , given by :

$$R_1' = 1.5Vx \left(1 - (1-x)^{m-1}\right) \quad (6B.1-8)$$

where V is the total seismic force at one end of the deck-truss superstructure.

5. Determine Total Superstructure Capacity

Given the above limits, the maximum total capacity of the superstructure will be the sum of the capacity of each ductile diaphragm, but not exceeding the substructure capacity, i.e:

$$1.5V_{\max} \leq \left[\text{Min} \left(2(V_{L,E} + V_{E,S}), 2V_{\text{sub}} \right) \right] \quad (6B.1-9)$$

where, V_{sub} is the largest shear that can be applied at the top of the abutment without damaging the substructure (connections, wind shoes, etc.), and 1.5 is the overstrength factor. The above equation can be easily modified for bridges having multiple simply-supported spans. Furthermore, a minimum strength, $V_{\min}$, must also be provided to resist the winds expected during life of the structure. Therefore, the yield capacity of the overall deck-truss system, R_{total} , should satisfy the following:

$$V_{\min} \leq R_{\text{total}} \leq V_{\max} \quad (6B.1-10)$$

6. Distributed Total System Capacity

The chosen total capacity of the system can then be divided proportionally between the lower end and end panels according to the following equations which ensure the same safety margin for both panels.

$$R_{L,E} = \frac{R_{\text{total}}}{V_{\max}} V_{L,E} \quad (6B.1-11)$$

$$R_{E,S} = \frac{R_{\text{total}}}{V_{\max}} V_{E,S} \quad (6B.1-12)$$

7. Define Capacity-Based Pseudo-Acceleration and Period Limits

A corresponding *Capacity-Based Pseudo Acceleration*, PSa_c , can be calculated as:

$$PSa_c = \frac{R_{\text{total}}}{M} \quad (6B.1-13)$$

This value can be drawn on a capacity spectrum, or compared with the required design values. Structural period of vibration directly ties this strength to the ductility and displacement demands. For example, in the intermediate period range, the ductility demand of systems having a constant strength decreases as the period increases (i.e. as stiffness decreases), while their displacement response increases. Therefore, a range of admissible period values can be located along the capacity-based pseudo-acceleration line, based on the permissible values of global ductility and displacement of the system corresponding to a particular ductile system.

Design iterations are required until a compatible set of strength and period are found to provide acceptable ductility and displacement demands. In other words, for a desired structural system strength, a range of limiting periods can be defined by a lower bound to the period, $T_{\min}$, to limit system ductility demands, and an upper bound, $T_{\max}$, to limit displacement demands (note that in some instances, $T_{\min}$ may not exist). As a result of these two constraints:

$$T_{\min} \leq T \leq T_{\max} \quad (6B.1-14)$$

Note that it may be more convenient to express these limits in terms of the global stiffness of the entire structural system, or of the end panel. Since:

$$K_{E,S} = \frac{K_{\text{Global}}}{a} \quad \text{where } a = 2 \left(1 + \frac{R_{L,E}}{R_{E,S}} \right) \quad (6B.1-15)$$

Then:

$$\frac{4p^2 M}{T_{\max}^2} \leq K_{Global} \leq \frac{4p^2 M}{T_{\min}^2} \quad (6B.1-16)$$

or for the end panel stiffness:

$$\frac{4p^2 M}{a T_{\max}^2} \leq K_{E,S} \leq \frac{4p^2 M}{a T_{\min}^2} \quad (6B.1-17)$$

This can be used to select proper values of stiffness for the end panel. To calculate the stiffness of the lower end ductile panel, $K_{L,E}$, stiffness of the lower load path system is first determined as:

$$K_{L,S} = \frac{(K_{Global} - 2K_{E,S})}{2} \quad (6B.1-18)$$

and $K_{L,E}$ is given by:

$$K_{L,E} = \frac{K^* K_{L,S}}{K_{L,S} - K^*} \quad (6B.1-19)$$

8. Design of Ductile Diaphragm Panels

As indicated in Appendix 6A, many types of systems capable of stable passive seismic energy dissipation could be used as ductile-diaphragms in deck-truss bridges. Among those, eccentrically braced frames (EBF) (e.g. Malley and Popov 1983; Kasai and Popov 1986), shear panel systems (SPS) (Fehling et al. 1992; Nakashima 1995), and steel triangular-plate added damping and stiffness devices (TADAS) (Tsai et al. 1993), popular in building applications, have been studied for bridge applications (Sarraf and Bruneau 1998a, 1998b). Although concentrically braced frames can also be ductile, they are not admissible in Article 6.15.5.2 because they can often be stronger than calculated, and their hysteretic curves can exhibit pinching and some strength degradation.

For convenience, the flexibility (i.e. inverse of stiffness) of panels having ductile diaphragms is provided below for a few types of ductile systems.

The flexibility of an eccentrically braced end panel, $f_{E,S}$, is expressed by:

$$f_{E,S} = \frac{h^2}{2EIb} \left(\frac{(a+e)^2}{3} - \frac{(b^2 - 2a^2)}{6} \right) + \frac{(a^2 + h^2)^{3/2}}{2EA_b a^2} + \frac{h^3}{2EA_{col} a^2} + \frac{(b-e)}{4EA_l} + \frac{eh^2}{2GA_s ab} \quad (6B.1-20)$$

where $a = (b-e)/2$, b is the panel width, h is the height, A_{col} is the cross-sectional area of a vertical panel member, A_b is the cross-sectional area of a bracing members, A_l , A_s , and I are respectively the cross-sectional area, shear area, and moment of inertia of the link beam, and e is the link length.

The flexibility, $f_{E,S}$, of a ductile VSL panel can be expressed by the following equation:

$$f_{E,S} = \frac{b(s+d/2)^2}{12EI} + \frac{2((h-s-d/2)^2 + b^2/4)^{3/2}}{EA_b b^2} + \frac{2h(h-s-d/2)^2}{EA_{col} b^2} + \frac{b}{4EA_l} + \frac{s}{A_s G} \quad (6B.1-21)$$

where, s is the height of the shear panel, I , is the bottom beam moment of inertia, and, d , is the depth of the bottom beam. The other parameters are as previously defined.

The required flexibility of the triangular plates alone for a TADAS system, f_T , expressed in terms of an admissible flexibility value of the end panel and other panel member properties, is given by:

$$f_T = f_{E,S} - \left(\frac{b(hh + d/2)^2}{12EI} + \frac{2\left(\left((1-h)h - d/2\right)^2 + (b/2)^2\right)^{3/2}}{EA_b b^2} + \frac{2h\left((1-h)h - d/2\right)^2}{EA_{col} b^2} + \frac{b}{4EA_l} \right) \quad (6B.1-22)$$

where η , is the ratio of height of triangular plates to the height of the panel and other parameters correspond to the panel members similar to those of VSL panel. Tsai, et.al. (1993) recommended using $\eta=0.10$.

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10.1 SCOPE

Provisions of this section shall apply for the design of spread footings, driven piles, and drilled shaft foundations. These provisions include guidance on the selection of resistance factors for design under both static and dynamic loading conditions. Other methods, especially when locally recognized and considered suitable for regional conditions, may be used if appropriate consideration is given to the uncertainty associated with estimating the response of the foundation system under the specific service, strength, or extreme event loading, and if these other methods are approved by the Owner.

This section includes provisions for seismic design of foundations. Use of the seismic provisions shall be coordinated with seismic requirements given in Section 3 - *Loads and Load Factors* and Section 4 - *Structural Analysis and Evaluation*. The foundation provisions for seismic design in Section 10 - *Foundations* are limited to geotechnical aspects of seismic design. Key seismic requirements for structural design are found in Section 5 - *Concrete Structures* and Section 6 - *Steel Structures*.

C10.1

Two significantly different loading conditions are covered in this provision.

- Static Loading: This type of loading typically involves either permanent loads or loads that are applied very slowly. These static loads include the dead weight of the structure and various combinations of other loads, as defined in Section 3 - *Loads and Load Factors*.
- Dynamic Loading: The second type of loading involves rapid or dynamic loading. The two primary sources of rapid or dynamic loads on bridges are ship or vessel impact and seismic loading. The duration of these loads typically will be for a few minutes or less, and hence geotechnical issues such as porewater pressures become important to the evaluation of soil response.

Significant differences exist in the methods used for dealing with uncertainty in soil behavior for static and some dynamic loads. For static loading resistance factors are used to distinguish between ultimate capacity and the maximum value that can be used for design. These resistance factors are intended to account for uncertainty in material property selection and in method of analysis. The resistance factor represents a portion of the factor of safety previously used in allowable stress design (ASD) methods. However, in ASD the factor of safety also included uncertainty due to the load. The reciprocal of the resistance factor represents what was formerly the factor of safety related to uncertainty in soil behavior and method of analysis. Significant efforts have been made to calibrate the resistance factors to conventional factors of safety in ASD and to the probability of failure (Appendix A of Barker et al., 1991; Withiam et al., 1998).

For dynamic or rapid loading some of the principles embodied within the resistance factors used for static design may not apply, at least in a simple prescriptive manner. This situation is particularly the case for earthquake loading, and in some cases may also be the case for ship impact loading. For example, during a seismic event one of the primary differences between static and dynamic design is that the stiffness of the foundation system has a significant effect on the loads that develop within the foundation system. Under seismic loading, selection of a lower resistance factor (higher factor of safety in ASD) can lead to unconservative estimates of foundation loads and response. In recognition of this, guidelines are presented in the seismic portions of this section for dealing with uncertainty.

As a final note, the specification of methods of analysis and calculation of resistance for foundations

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herein is not intended to imply that field verification and/or reaction to conditions actually encountered in the field are no longer needed. These traditional features of foundation design and construction are still practical considerations when designing in accordance with these Specifications.

10.2 DEFINITIONS

Batter Pile - Pile driven at an angle inclined to the vertical to provide higher resistance to lateral loads.

Bearing Pile - A pile whose purpose is to carry axial load through friction or point bearing.

Combination Point Bearing and Friction Pile - Pile that derives its capacity from contributions of both point bearing developed at the pile tip and resistance mobilized along the embedded shaft.

Combined Footing - A footing that supports more than one column.

Competent Rock – For non-seismic cases a rock mass with discontinuities that are open not wider than 3.2 mm. For seismic purposes, the competency of the rock is determined on the basis of the estimated shear wave velocity. Hard rock is rock with an average shear wave velocity in the upper 30 m of rock profile of greater than 1500 m/s; competent rock has a shear wave velocity between 760 and 1500 m/s; and soft rock has a shear wave velocity between 360 m/s and 760 m/s.

Deep Foundation - A foundation that derives its support by transferring loads to soil or rock at some depth below the structure by end bearing, adhesion or friction, or both.

Drilled Shaft - A deep foundation unit, wholly or partly embedded in the ground, constructed by placing fresh concrete in a drilled hole with or without steel reinforcement. Drilled shafts derive their capacity from the surrounding soil and/or from the soil or rock strata below its tip. Drilled shafts are also commonly referred to as caissons, drilled caissons, bored piles, or drilled piers.

Effective Stress - The net stress across points of contact of soil particles, generally considered as equivalent to the total stress minus the porewater pressure.

Friction Pile - A pile whose support capacity is derived principally from soil resistance mobilized along the side of the embedded pile.

Isolated Footing - Individual support for the various parts of a substructure unit; the foundation is called a footing foundation.

Length of Foundation - Maximum plan dimension of a foundation element.

Liquefaction - Process by which saturated granular soil loses strength and stiffness due to porewater pressure buildup.

Liquefaction-Induced Lateral Flow. – Lateral displacement of relatively flat slopes that occurs under the combination of gravity load and excess porewater pressure (without inertial loading from earthquake). Lateral flow often occurs after the cessation of earthquake loading.

Liquefaction-Induced Lateral Spreading – Incremental displacement of a slope that occurs from the combined effects of porewater pressure buildup, inertial loads from the earthquake, and gravity loads.

Overconsolidation Ratio (OCR) - Defined as the ratio of the preconsolidation pressure to the current vertical effective stress.

Pile - A relatively slender deep foundation unit, wholly or partly embedded in the ground, that is installed by driving, drilling, auguring, jetting, or otherwise and that derives its capacity from the surrounding soil and/or from the soil or rock strata below its tip.

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Pile Bent - A type of bent using piles as the column members.

Pile Shoe - A metal piece fixed to the penetration end of a pile to protect it from damage during driving and to facilitate penetration through very dense material.

Piping - Progressive erosion of soil by seeping water that produces an open pipe through the soil through which water flows in an uncontrolled and dangerous manner.

Plunging - A mode of behavior observed in some pile load tests, wherein the settlement of the pile continues to increase with no increase in load.

Point-Bearing Pile - A pile whose support capacity is derived principally from the resistance of the foundation material on which the pile tip rests.

RQD - Rock Quality Designation.

Shallow Foundation - A foundation that derives its support by transferring load directly to the soil or rock at shallow depth.

Slickensides - Polished and grooved surfaces in clayey soils or rocks resulting from shearing displacements along planes.

Total Stress - Total pressure exerted in any direction by both soil and water.

Width of Foundation - Minimum plan dimension of a foundation element.

10.3 NOTATION

The units shown after the description of each term are suggested units. Other units that are consistent with the expressions being evaluated may be used.

A	=	effective footing area for determination of elastic settlement of footing subjected to eccentric loads (mm^2) (10.6.2.2.3b)
A_p	=	area of pile point or base of drilled shaft (mm^2) (10.7.3.2)
A_s	=	surface area of pile shaft (mm^2) (10.7.3.2)
a_{si}	=	pile perimeter at the point considered (mm) (10.7.3.4.3c)
A_{soc}	=	area of drilled shaft socket in rock (mm^2) (C10.8.3.5)
A_u	=	uplift area of a belled drilled shaft (mm^2) (10.8.3.7.2)
B	=	footing width (mm); pile group width (mm) (10.6.3.1.2c)
B'	=	effective footing width (mm) (10.6.3.1.5)
C_{ae}	=	secondary settlement coefficient estimated from results of laboratory consolidation testing of undisturbed soil samples (DIM) (10.6.2.2.3c)
C_c	=	compression index (DIM) (10.6.2.2.3c)
C_{ce}	=	compression ratio (DIM) (10.6.2.2.3c)
C_{cr}	=	recompression index (DIM) (10.6.2.2.3c)
C_o	=	uniaxial compressive strength of rock (MPa) (10.6.2.3.2)
CPT	=	cone penetration test (10.5.5)
C_{re}	=	recompression ratio (DIM) (10.6.2.2.3c)
C_v	=	coefficient of consolidation (mm^2/YR) (10.6.2.2.3c)
C_{w1}, C_{w2}	=	correction factors for groundwater effect (DIM) (6.10.3.1.2c)
c	=	cohesion of soil (MPa); undrained shear strength (MPa) (10.6.3.1.2b)
$c_q, c_?$	=	soil compressibility factor (DIM) (10.6.3.1.2c)
c_1	=	undrained shear strength of the top layer of soil as depicted in Figure 3 (MPa) (10.6.3.1.2b)
c_2^*	=	shear strength of lower soil layer (MPa) (10.6.3.1.2b)
$\bar{c}$	=	reduced effective stress soil cohesion for punching shear (MPa) (10.6.3.1.2a)
D	=	pile width or diameter (mm); diameter of drilled shaft (mm) (10.7.3.4.2a) (10.8.3.3.2)
D'	=	effective depth of pile group (mm) (10.7.2.3.3)
D_b	=	depth of embedment of pile into a bearing stratum (mm) (10.7.2.1)

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D_f	= foundation embedment depth taken from ground surface to bottom of foundation (mm) (10.6.3.1.2b)
D_i	= pile width or diameter at the point considered (mm) (10.7.3.4.3c)
D_p	= diameter of the tip of a drilled shaft (mm); diameter of bell (mm) (10.8.3.3.2) (10.8.3.7.2)
d_q	= depth factor (DIM) (10.6.3.1.2c)
D_s	= diameter of socket when pile or drilled shaft is socketed into rock (mm) (10.7.3.5)
D_w	= depth to water surface taken from the ground surface (mm) (10.6.3.1.2c)
d	= depth factor for estimating tip capacity of piles in rock (DIM) (10.7.3.5)
E	= modulus of elasticity of pile (MPa) (10.7.4.2)
E_c	= modulus of elasticity of concrete (MPa) (C10.8.3.5)
E_i	= modulus of elasticity of intact rock (MPa) (C10.8.3.5)
E_m	= estimated rock mass modulus (MPa); rock mass modulus (MPa) (C10.6.2.2.3c) (10.6.2.2.3d)
E_o	= intact rock modulus (MPa) (10.6.2.2.3d)
E_p	= modulus of elasticity of pile (MPa) (10.7.4.2)
E_r	= modulus of elasticity of in-situ rock (MPa) (C10.8.3.5)
E_s	= soil modulus (MPa) (10.7.4.2)
e_B	= eccentricity of load parallel to the width of the footing (mm) (10.6.3.1.5)
e_L	= eccentricity of load parallel to the length of the footing (mm) (10.6.3.1.5)
e_o	= void ratio at initial vertical effective stress (DIM) (10.6.2.2.3c)
F_r	= reduction factor for point resistance of large diameter drilled shafts (DIM) (10.8.3.3.2)
f'_c	= 28-day compressive strength of concrete (MPa) (10.6.2.3.2)
f_s	= sleeve friction measured from a CPT (MPa) (10.7.3.4.3a)
f_{si}	= unit local sleeve friction resistance from CPT at the point considered (MPa) (10.7.3.4.3c)
g	= gravitational acceleration (m/s^2)
G	= shear modulus of soil at any shearing strain amplitude (kPa) (10.6.4.2.1)
G_{max}	= shear modulus of soil at shearing strain amplitudes equal 0.001 percent or lower (kPa) (10.6.4.2.1)
H	= horizontal component of inclined loads (N); distance from tips of piles to top of lowest stratum (mm) (10.6.3.1.3b)
H_c	= height of compressible soil layer (mm) (10.6.2.2.3c)
H_d	= height of longest drainage path in compressible soil layer (mm) (10.6.2.2.3c)
H_s	= height of sloping ground mass (mm); depth of embedment of pile or drilled shaft socketed into rock (mm) (10.6.3.1.2b) (10.7.3.5)
H_{s2}	= distance from bottom of footing to top of the second soil layer (mm) (10.6.3.1.2b)
h_i	= length interval at the point considered (mm) (10.7.3.4.3c)
I	= influence factor for the effective embedment of a pile group (DIM) (10.7.2.3.3)
I_x	= mass moment of inertia about x axis ($KN-mm-sec^2$) (10.6.4.2.1)
I_y	= mass moment of inertia about y axis ($KN-mm-sec^2$) (10.6.4.2.1)
I_p	= influence coefficient to account for rigidity and dimensions of footing (DIM); moment of inertia of pile (mm^4) (10.6.2.2.3d) (10.7.4.2)
I_2	= influence coefficient from Figure C10.8.3.5-1 (DIM)
i_q, i_2	= load inclination factors (DIM) (10.6.3.1.2c)
K	= load transfer factor (DIM) (10.8.3.4.2)
K_b	= coefficient for bearing on rock from pressuremeter test (DIM) (C10.8.3.5)
K_c	= correction factor for sleeve friction in clay (DIM) (10.7.3.4.3c)
K_e	= modulus modification ratio from Figure C10.8.3.5-3 (DIM) (C10.8.3.5)
K_s	= correction factor for sleeve friction in sand (DIM) (10.7.3.4.3c)
K_{sp}	= dimensionless bearing capacity coefficient (DIM) (10.7.3.5)
K_z'	= vertical stiffness (MN/mm) (10.6.4.2.1)
K_y'	= horizontal stiffness in y direction (MN/mm) (10.6.4.2.1)
K_x'	= horizontal stiffness in x direction (MN/mm) (10.6.4.2.1)
$K_{\theta x}'$	= rotation stiffness about x axis (MN/mm) (10.6.4.2.1)
$K_{\theta y}'$	= rotation stiffness about y axis (MN/mm) (10.6.4.2.1)
K_{sv}	= axial stiffness of pile (MN/mm) (10.7.4.3.1)
K_{rv}	= rotational stiffness of pile (MN/mm) (10.7.4.3.1)
k	= empirical bearing capacity coefficient from Figure 10.6.3.1.3d-1 (DIM) (10.6.3.1.3d)
L	= length of foundation (mm) (10.6.3.1.5)
L'	= effective footing length (mm) (10.6.3.1.5)
L_f	= depth to point considered when measuring sleeve friction (mm) (10.7.3.4.3c)

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L_i	=	depth to middle of length interval at the point considered (mm) (10.7.3.4.3c)
LL	=	liquid limit of soil (C10.8.1.9)
M_c	=	moment capacity of footing during seismic event (MN-mm) (10.6.4.2.1)
N	=	Standard Penetration Test (SPT) blow count (Blows/300 mm) (10.7.2.3.3)
$\bar{N}$	=	average (uncorrected) SPT blow count along pile shaft (Blows/300 mm) (10.7.3.4.2b)
N_c	=	bearing capacity factor (DIM) (10.6.3.1.2b)
N_q, N_{γ}	=	bearing capacity factors (DIM) (10.6.3.1.2c)
N_{cm}, N_{qm}	=	modified bearing capacity factors (DIM) (10.6.3.1.2b)
$N_{cm}, N_{qm}, N_{\gamma m}$	=	modified bearing capacity factors (DIM) (10.6.3.1.2b)
N_{corr}	=	corrected SPT blow count (Blows/300 mm) (10.7.2.3.3)
$\bar{N}_{corr}$	=	average value of corrected SPT blow count (Blows/300 mm) (10.6.3.1.3b)
N_m	=	bearing capacity factor (DIM) (10.6.3.1.2b)
N_{ms}	=	rock parameter (DIM) (10.6.2.3.2)
N_u	=	uplift adhesion factor for bell (DIM) (10.8.3.7.2)
$N_{\gamma m}$	=	modified bearing capacity factor (DIM) (10.6.3.1.2c)
N_1	=	SPT resistance, corrected for depth (Blows/300 mm); number of intervals between the ground surface and a point 8D below the ground surface (10.6.2.2.3b-1) (10.7.3.4.3c)
N_2	=	number of intervals between 8D below the ground surface and the tip of the pile (10.7.3.4.3c)
n_h	=	rate of increase of soil modulus with depth (MPa/mm) (10.7.4.2)
P	=	vertical load on footing during seismic event (MN) (10.6.4.2.1)
PL	=	plastic limit of soil (C10.8.1.9)
$*p_L$	=	limiting pressure obtained from pressuremeter test result (MPa) (10.6.3.1.3d)
p_o	=	total horizontal pressure at the depth where the pressuremeter test is performed (MPa) (10.6.3.1.3d)
p_1	=	limit pressure determined from pressuremeter tests averaged over a distance of 2.0 diameters above and below the base (MPa) (C10.8.3.5)
Q_{ep}	=	passive resistance of soil available throughout the design life of the structure (N) (10.6.3.3)
Q_g	=	nominal resistance of pile group (N) (10.7.3.10.1)
Q_L	=	nominal lateral resistance of single pile (N) (10.7.3.11)
Q_{Lg}	=	nominal lateral resistance of pile group (N) (10.7.3.11)
Q_n	=	nominal resistance (N) (10.6.3.3)
Q_p	=	nominal load carried by pile point (N) (10.7.3.2)
Q_R	=	factored resistance (N) (10.6.3.3)
Q_s	=	nominal load carried by pile shaft (N) (10.7.3.2)
Q_{sbell}	=	nominal uplift resistance of a belled drilled shaft (N) (10.8.3.7.2)
Q_{SR}	=	nominal side resistance of drilled shafts socketed in rock (N) (C10.8.3.5)
Q_{ug}	=	nominal uplift resistance of a pile group (N) (10.7.3.7.3)
Q_{ult}	=	total nominal bearing resistance (N) (10.7.3.2)
Q_t	=	maximum shear resistance between the foundation and the soil (N) (10.5.5)
q	=	net foundation pressure applied at $2D_b/3$ (MPa) (10.7.2.3.3)
q	=	P/BL during seismic event (MPa) (10.6.4.2.1)
q_c	=	static cone resistance (MPa); average static cone resistance over a depth B below the equivalent footing (MPa) (10.6.3.1.3c) (10.7.2.3.3)
q_{c1}	=	minimum average static cone resistance over a depth yD below a pile tip (MPa) (10.7.3.4.3b)
q_{c2}	=	minimum average static cone resistance over a distance 8D above the pile tip (MPa) (10.7.3.4.3b)
q_l	=	limiting point resistance (MPa) (10.7.3.4.2a)
q_n	=	nominal bearing resistance (MPa) (10.6.3.1.1)
q_o	=	vertical stress at base of loaded area (MPa) (10.6.2.2.3b)
q_p	=	nominal unit point resistance (MPa) (10.7.3.2)
q_{pr}	=	reduced nominal unit point resistance (MPa) (C10.8.3.3.2)
q_R	=	factored bearing resistance (MPa) (10.6.3.1.1)
q_s	=	unit shear resistance; nominal unit skin resistance (MPa) (10.6.3.3) (10.7.3.2)
q_{sbell}	=	nominal unit uplift resistance of a belled drilled shaft (MPa) (10.8.3.7.2)
q_u	=	average uniaxial compression strength of the rock core (MPa) (10.7.3.5)
q_{ult}	=	nominal bearing resistance (MPa) (10.6.3.1.1)
q_1	=	ultimate bearing capacity of footing supported in the upper layer of a two-layer system, assuming the upper layer is infinitely thick (MPa) (10.6.3.1.2a)

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q_2	=	ultimate bearing capacity of a fictitious footing of the same size and shape as the actual footing, but supported on surface of the second (lower) layer of a two-layer system (MPa) (10.6.3.1.2a)
R_a	=	radius adjustment factor (DIM) (10.6.4.2.1)
R_i	=	reduction factor accounting for the effect of load inclination (DIM) (10.6.3.1.3b)
r	=	radius of circular footing or B/2 for square footing (mm) (10.6.2.2.3d)
r_o	=	initial total vertical pressure at foundation level (MPa) (10.6.3.1.3d)
S_c	=	consolidation settlement (mm) (10.6.2.2.3a)
S_e	=	elastic settlement (mm) (10.6.2.2.3a)
S_n	=	distance between the nth pile and the axis of rotation (mm) (10.6.4.2.1)
SPT	=	standard penetration test (10.5.5)
S_s	=	secondary settlement (mm) (10.6.2.2.3a)
S_u	=	undrained shear strength (MPa) (10.6.3.1.2b)
$\bar{S}_u$	=	average undrained shear strength along pile shaft (MPa) (10.7.3.7.3)
S_c, S_q, S_γ	=	shape factors (DIM) (10.6.3.1.2b) (10.6.3.1.2c)
S_d	=	spacing of discontinuities (mm) (10.7.3.5)
T	=	time factor (DIM) (10.6.2.2.3c)
t	=	time for a given percentage of one-dimensional consolidation settlement (YR) (10.6.2.2.3c)
t_d	=	width of discontinuities (mm) (10.7.3.5)
t_1, t_2	=	arbitrary time intervals for determination of S_s (YR) (10.6.2.2.3c)
V	=	vertical component of inclined loads (N) (10.6.3.1.3b)
W_g	=	weight of block of soil, piles and pile cap (N) (10.7.3.7.3)
X	=	width of pile group (mm) (10.7.2.3.3)
Y	=	length of pile group (mm) (10.7.3.7.3)
Z	=	total embedded pile length (mm) (10.7.3.4.3c)
z	=	depth below ground surface (mm) (10.8.3.4.2)
a	=	adhesion factor applied to S_u (DIM) (10.7.3.3.2a)
a_E	=	reduction factor (DIM) (10.6.2.2.3d)
β	=	coefficient relating the vertical effective stress and the unit skin friction of a pile or drilled shaft (DIM) (10.7.3.3.2b)
β_m	=	punching index (DIM) (10.6.3.1.2b)
β_z	=	factor to account for footing shape and rigidity (DIM) (10.6.2.2.3d)
γ	=	density of soil (kg/m^3) (10.6.3.1.2b)
δ	=	angle of shearing resistance between soil and pile (DEG) (10.6.3.3)
η	=	efficiency factor for pile or drilled shaft group (DIM) (10.7.3.10.2)
λ	=	empirical coefficient relating the passive lateral earth pressure and the unit skin friction of a pile (DIM) (10.7.3.3.2c)
λ_v	=	empirical factor used to adjust resistance factors for ultimate capacity determination based on the method of construction supervision or monitoring during pile installation (DIM) (10.5.5)
μ_c	=	reduction factor for consolidation settlements to account for three-dimensional effects (DIM) (10.6.2.2.3c)
ν	=	Poisson's ratio (DIM) (10.6.4.2.1)
s	=	settlement of pile group (mm) (10.7.2.3.3)
s_{base}	=	settlement of the base of a drilled shaft (mm) (C10.8.3.5)
s_e	=	elastic shortening of a drilled shaft (mm) (C10.8.3.5)
SP_i	=	working load at the top of a rock socket (N) (C10.8.3.5)
s'_f	=	final vertical effective stress in soil at depth interval below footing (MPa) (10.6.2.2.3c)
s'_o	=	initial vertical effective stress in soil at depth interval below footing (MPa) (10.6.2.2.3c)
s'_p	=	maximum past vertical effective stress in soil at depth interval below footing (MPa) (10.6.2.2.3c)
s'_{pc}	=	current vertical effective stress in the soil, not including the additional stress due to the footing loads (MPa) (10.6.2.2.3c)
s_v	=	total vertical stress at the brace elevation (MPa) (C10.8.3.5)
s'_v	=	vertical effective stress (MPa) (C10.7.1.7)
f	=	resistance factor (10.5.5)
f_{ep}	=	resistance factor for passive pressure (10.6.3.3)
f_f	=	angle of internal friction of soil (DEG) (10.6.3.3)
f_g	=	resistance factor for the bearing capacity of a pile group failing as a unit consisting of the piles and the block of soil contained within the piles; group resistance factor (10.7.3.10.1)
f_L	=	pile group resistance factor for lateral loads (DIM) (10.7.3.11)

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f_q	=	resistance factor for the total bearing capacity of a pile for those methods that do not distinguish between total resistance and the individual contributions of tip resistance and shaft resistance (10.7.3.2)
f_{qs}	=	resistance factor for the shaft capacity of a pile for those methods that separate the resistance of a pile into contributions from tip resistance and shaft resistance (10.7.3.2)
f_{qp}	=	resistance factor for the tip capacity of a pile for those methods that separate the resistance of a pile into contributions from tip resistance and shaft resistance (10.7.3.2)
f_T	=	resistance factor for shear between soil and foundation (10.5.5)
f_u	=	resistance factor for the uplift capacity of a single pile (10.7.3.7.2)
f_{ug}	=	resistance factor for the uplift capacity of pile groups (10.7.3.7.3)
f'_1	=	effective stress angle of internal friction of the top layer of soil (DEG) (10.6.3.1.2c)
f^*	=	reduced effective stress soil friction angle for punching shear (DEG) (10.6.3.1.2a)
θ_z	=	stiffness embedment factor for vertical translation (DIM) (10.6.4.2.1)
θ_y	=	stiffness embedment factor for horizontal translation in y direction (DIM) (10.6.4.2.1)
θ_x	=	stiffness embedment factor for horizontal translation in x direction (DIM) (10.6.4.2.1)
$\theta_{\theta y}$	=	rotational embedment factor for horizontal translation in y direction (DIM) (10.6.4.2.1)
$\theta_{\theta x}$	=	rotational embedment factor for horizontal translation in x direction (DIM) (10.6.4.2.1)

10.4 DETERMINATION OF SOIL PROPERTIES

C10.4

Soil properties appropriate for use in the design of spread footings, driven piles, and drilled shafts shall be determined in accordance with procedures presented in Articles 2.3 and 2.4 and Appendix 2A in Section 2 of these Specifications. The choice of methods for determining material properties for design shall be made in consideration of (1) the type of soil into or on which the foundation is located, (2) the type and size of the likely foundation system, (3) the type of the design load (e.g., static or seismic), and (4) the type of analysis (e.g., service, strength, extreme event).

When selecting properties for use in design, consideration shall be given to the uncertainty in the determination of the soil property, resulting from the variability in geologic conditions at the site, the spatial extent of explorations, the quality of soil or rock samples recovered, the amount and quality of laboratory testing, and the methods used to interpret material properties. These considerations influence the selection of resistance factors for static loading conditions, as discussed in Article 10.5. For seismic loading conditions, the uncertainty in property determination defines the amount of property variation to be considered when computing foundation stiffness and capacity values required during seismic design.

The groundwater level and its possible variation shall be determined for the project location. This determination shall consider variations that could occur during the design life of the structure. If topographic variations occur at the site, the potential for artesian conditions shall also be considered.

Soil properties used in the design of foundations will differ depending on the type of soil at the site and the type of anticipated loading. For static loading it is usually appropriate to select properties representative of drained conditions if soils are cohesionless and undrained conditions if soils are cohesive. For drained conditions the friction angle of the soil (ϕ) should be used, while for undrained conditions the undrained strength (S_u) should be used. A combination of cohesion, c , and friction angle, ϕ , and can also be used to represent drained or undrained conditions for many soil types. In all cases the ϕ and c should be identified as either total stress or effective stress parameters, depending upon their method of determination.

For many seismic loading conditions, undrained soil behavior will occur, whether soils are cohesionless or cohesive. The undrained strength of soil depends on the rate of loading and the number of cycles of loading. These rate-of-loading effects are normally small for cohesionless materials. However, repeated cycles of undrained loading can result in liquefaction of saturated, loose cohesionless soils, as discussed in Article 3.10.5 and Appendix 3B. Liquefaction or partial liquefaction will result in a decrease in the strength of cohesionless soil and a reduction in the load-deformation (stiffness) response of the soil. The response of cohesive soil to seismic loads will depend on the consistency of the soil and the ratio of imposed shearing stress to the undrained strength of the soil. A cohesive soil will exhibit higher strength under its first cycle of rapid load, perhaps by as much as 40 percent. Subsequent cycles of load result in a progressive decrease in strength to a level that might be 80 percent of the initial static strength. Normal practice is to assume that the strength of a cohesive soil during seismic loading is equal to the static strength before cyclic loading, as discussed by Makdisi and Seed (1978). Additional information about the response of soil to seismic loading can be found in Kramer (1996).

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10.5.2 Service Limit States

Foundation design at the service limit state shall include:

- Settlements;
- Lateral displacements and rotation;
- Bearing resistance estimated using the presumptive bearing pressure, and
- Overall stability.

Consideration of settlement shall be based upon rideability and economy.

The evaluation of overall stability of earth slopes with or without a foundation unit shall be investigated at the Service Limit State based on the Service I Load Combination and an appropriate resistance factor. In lieu of better information, the resistance factor, ϕ , may be taken as:

- When the geotechnical parameters are well defined, and the slope does not support or contain a structural element 0.85
- When the geotechnical parameters are based on limited information, or the slope contains or supports a structural element 0.65

Higher resistance factors may be accepted in certain situations, depending on the type and extent of field explorations, laboratory testing, design methods, and general experience in the area. Such changes should be implemented only after detailed discussions with and approval from the Owner.

and Lam and Martin (1986).

Groundwater location is fundamental to the design of the foundation, whether loads are static or dynamic. In many areas the groundwater location changes with time of the year. The highest location of the groundwater should be used for most static analyses. For seismic analyses, the likelihood of the seismic event and the high groundwater location occurring simultaneously may be so low that an average groundwater location may be more suitable for design. The decision as to what groundwater level to use for seismic design should be determined after evaluating the amount and duration of water level changes, the potential influence on design, and the Owner's design strategy. As noted in Appendix 3B, the fluctuation of groundwater location should also be considered during the liquefaction assessment, as soils that are below the groundwater level for short periods of time are normally more resistant to liquefaction because of the low degree of saturation.

C10.5.2

In bridges where the superstructure and substructure are not integrated, correction of settlements can be made by jacking and shimming bearings. Article 2.5.2.3 provides jacking provisions for these bridges.

The cost of limiting foundation movements should be compared to the cost of designing the superstructure so that it can tolerate larger movements or of correcting the consequences of movements through maintenance to determine minimum lifetime cost. More stringent criteria may be established by the Owner.

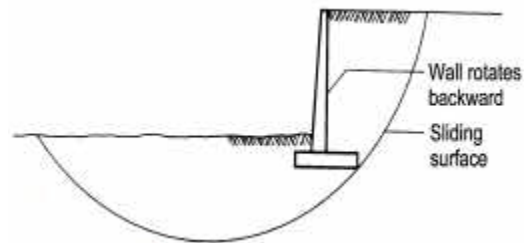


Figure C10.5.2-1 - Retaining Wall Overall Stability Failure

Figure C10.5.2-1 shows a retaining wall overall stability failure. Overall stability is a slope stability issue and, therefore, is considered a service limit state check. The check should be made for all slopes whose failure (movement) could affect the bridge abutments or piers, including end slopes, cut slopes, and fill slopes.

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10.5.4 Extreme Event Limit States

C.10.5.4

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10.5.4.1 GLOBAL SLOPE STABILITY

Global stability shall also be determined for the Extreme Event Limit State - Case I (Seismic Loading). Either pseudo static or Newmark-type analyses can be used for making these evaluations. If pseudo static methods are used, the resistance factor shall be 1.0 and the acceleration coefficient shall be 1/2 of the peak ground acceleration (i.e., $F_a S_s$). For these conditions, a capacity check shall be performed to confirm that the unfactored resistance ($\phi = 1.0$) is equal to or exceeds forces resulting from gravity and inertial loads (i.e., total factor of safety of 1.0 or more in ASD). If the capacity check determines that equilibrium is not achieved under these conditions, a Newmark-type analyses shall be conducted in accordance with procedures given in Appendix 3B to quantify the estimated amount of ground displacement.

If soils within or below the slope (i.e., in the case of an approach fill) are loose and cohesionless and located below the water table, the potential for porewater pressure build-up and associated liquefaction-induced lateral flow or spreading shall be evaluated in accordance with guidance given in Appendix 3B.

C10.5.4.1

Procedures for evaluating global slope stability under seismic loading are described in Kramer (1996). The pseudo static method is intended to provide a relatively simple approach for evaluating the stability of slopes under the Extreme Event from Seismic Loading. However, this approach provides no specific information about the consequences of seismic loading, other than a determination of whether or not the slope is in equilibrium. From past observations following large seismic events, deformations for slopes where unfactored resistance exceeds forces resulting from gravity and inertial loads (total factor of safety of 1.0 or more) have been very small (e.g., less than a 100 mm) when a seismic coefficient of 1/2 of the peak ground acceleration (PGA) and $\phi = 1.0$ were used in a pseudo static method of analysis.

The Newmark analysis allows the Designer to explicitly predict the level of deformation in cases when the above requirements for equilibrium are not satisfied. As discussed in Appendix 3B and by Kramer (1996), both simplified, chart-type and numerical methods exist for conducting a Newmark analysis. In the chart methods (e.g., Franklin and Chang, 1977; Hynes and Franklin, 1984; Wong and Whitman, 1982; and Martin and Qiu, 1994), the deformation is defined in terms of the magnitude of the seismic event (k_{max}) and the yield acceleration for the slope, which is defined as the seismic coefficient (k_y) which results in equilibrium between load and resistance (total factor of safety of 1.0). With the numerical methods deformations are estimated by double integrating seismic records above the yield acceleration of the slope. If this latter method is used, guidance given in Appendix 3A of Section 3 should be followed in selecting earthquake records to analyze. At least three records representative of the possible range of earthquake motions should be used.

For the both the simplified and the numerical approaches consideration also must be given to the potential amplification of ground motions at the site. Site response factors in Section 3 can be used to estimate the amount of amplification for many slopes. However, where abrupt changes in geometry occur, the potential for geometric amplification may have to be considered. Finite element or similar two-dimensional numerical methods may be required to perform these evaluations.

For simplified Newmark methods that require the earthquake magnitude to be defined, a mean magnitude can be obtained consistent with the desired probability of exceedance from deaggregation information in the USGS earthquake hazard website <http://geohazards.cr.usgs.gov/earthquake.shtml>.

The evaluation of global stability for sites that also involve liquefaction requires special consideration. Appendix 3B in Section 3 provides additional guidance for these situations.

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10.5.4.2 FOUNDATION DESIGN

Foundations shall be designed for extreme events as applicable. In some locations loading from extreme events will be the most critical type of loading. In these locations the Owner may have developed specific design approaches and requirements for the extreme event, particularly if the extreme event is the result of seismic loading. Before using the seismic provisions in these Specifications, the Designer shall confirm with the Owner that the procedures given in these Specifications are consistent with the Owner's design philosophy and requirements. The Designer should also be aware that the methodologies used to evaluate foundation response under some extreme events, such as seismic loading, continue to evolve. The methodologies given in these Specifications represent the state-of-the-practice in 2000. These provisions should not restrict the use of new and improved methods, if the Designer can demonstrate to the Owner that the use of the new or improved method is justified.

10.5.5 Resistance Factors

Resistance factors shall be used during the evaluation of Strength Limit State conditions as summarized in Article 10.5.3. Resistance factors for the Service Limit State and for seismic loading under Extreme Limit State shall be taken as 1.0.

Resistance factors for Strength Limit State design shall be as specified in Tables 10.5.5-1 through 10.5.5-3, unless required differently by the Owner. Where pile foundations are specified, the contract documents shall specify the level of field pile capacity verification required. The field verification specified shall be consistent with the value λ_v from Table 10.5.5-2 unless required otherwise by the Owner.

C10.5.4.2

Extreme events include flood, scour, vessel and vehicle collision, seismic loading, and other site-specific situations that the Designer determines should be included. This type of loading often involves unique structural and geotechnical considerations, which require close and repeated interaction between the geotechnical engineer and the structural engineer. Perhaps the best example of this is for seismic loading. If the site falls within Seismic Detailing Requirement (SDR) 3 and above, the structural engineer and the geotechnical engineer for the project should meet before design begins to discuss the potential consequences of seismic loading to the bridge-foundation system. From this meeting suitable foundation systems can be identified, and an approach for proceeding with field explorations and the geotechnical design can be developed. The approach should define the range of most-appropriate foundation types for the seismic environment, the type of geotechnical information required by the structural engineer for the seismic design, and the alternatives that might exist for other than structural solutions in the case where seismic-induced liquefaction appears possible.

For these situations the geotechnical engineer must be prepared to assess the uncertainties associated with seismic-induced ground motions, soil variability, and foundation performance, and then advise the structural engineer on what this might mean from the standpoint of foundation behavior. The structural engineer should make the geotechnical engineer aware of what loading conditions will occur and what the critical elements of structural response will be, in order for the geotechnical engineer to properly perform field explorations and develop foundation design information.

C.10.5.5

The resistance factors in Tables 10.5.5-1 to 10.5.5-3 should be used only for Strength Limit State loading conditions. These factors are intended to account for uncertainty in the determination of soil and rock properties and in the method of analysis used to estimate foundation behavior. For Strength Limit State, use of the resistance factor introduces a margin of safety into the capacity evaluation. Where statistical information was available, reliability theory tempered in some cases by judgment was used to derive the values of resistance factors given in Tables 10.5.5-1 through 10.5.5-3. In cases where there was insufficient information for calibration using reliability theory, values of resistance were chosen based on judgment, so that the design using LFRD procedures was

consistent with that using ASD procedures. Details of this approach are discussed in Appendix A of Barker et. al (1991) and by Withiam et al. (1998).

The Designer must use considerable care when selecting resistance factors. In some cases use of the resistance factor in Table 10.5.5.-1 though 10.5.5-3 is not appropriate. For example, the resistance factor for passive earth pressure associated with bearing capacity is taken as specified in Table 10.5.5-1 if a bridge component is pushed; e.g., backwall of deadman into the soil. On the other hand if passive earth pressure is used to determine force effects on other bridge components (e.g., the bending moments in the component of an integral abutment), it is conservative to assume that the maximum passive resistance is available; i.e., $\phi = 1.0$.

Resistance factors are not used for seismic design of the geotechnical elements of abutments and foundations. During revision of the seismic provisions in this Specification, consideration was given to using modified resistance factors for seismic loading. In this approach a higher resistance factor (relative to other load cases) might have been used because the likelihood of the seismic load would be less. The approach recommended in this Section is, however, to use a resistance factor of 1.0 for SDR 3 and above. By adopting a resistance factor of 1.0, the Designer is required to design for best-estimate soil properties. For low seismic zones (SDR 1 and SDR 2) the seismic provisions are limited to foundation capacity check under Strength Limit State only, and resistance factors are consistent with Strength Limit State. For these seismic zones static design normally results in adequate seismic design.

For the higher seismic zones (SDR 3 and above), uncertainty in material response and methods of analysis can be introduced though the use of upper and lower bound stiffness and capacity values. This approach is necessary to account for the amplification or attenuation that can occur within a structure depending on the ratio of the fundamental period of the foundation-structure system to the fundamental period of the seismic motion. This amplification phenomenon is not adequately captured in a simple resistance factor approach, and in fact can lead to un-conservative designs.

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Table 10.5.5-1 - Resistance Factors for Strength Limit State for Shallow Foundations

METHOD/SOIL/CONDITION			RESISTANCE FACTOR
Bearing Capacity and Passive Pressure		Sand	
		- Semiempirical procedure using SPT data	0.45
		- Semiempirical procedure using CPT data	0.55
		- Rational Method -- using f_f estimated from SPT data	0.35
		using f_f estimated from CPT data	0.45
		Clay	
		- Semiempirical procedure using CPT data	0.50
Sliding		- Rational Method -- using shear resistance measured in lab tests	0.60
		using shear resistance measured in field vane tests	0.60
		using shear resistance estimated from CPT data	0.50
		Rock	
		- Semiempirical procedure, Carter and Kulhawy (1988)	0.60
		Plate Load Test	0.55
		Precast concrete placed on sand using f_f estimated from SPT data using f_f estimated from CPT data	0.90 0.90
		Concrete cast-in-place on sand using f_f estimated from SPT data using f_f estimated from CPT data	0.80 0.80
		Sliding on clay is controlled by the strength of the clay when the clay shear is less than 0.5 times the normal stress and is controlled by the normal stress when the clay shear strength is greater than 0.5 times the normal stress (see Figure 1, which is developed for the case in which there is at least 150 mm of compacted granular material below the footing).	

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METHOD/SOIL/CONDITION			RESISTANCE FACTOR
	f_t	Clay (where shear resistance is less than 0.5 times normal pressure)	
		using shear resistance measured in lab tests	0.85
		using shear resistance measured in field tests	0.85
		using shear resistance estimated from CPT data	0.80
		Clay (where the resistance is greater than 0.5 times normal pressure)	0.85
		Soil on soil	1.0
	f_{ep}	Passive earth pressure component of sliding resistance	0.50

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Table 10.5.5-2 - Resistance Factors for Geotechnical Strength Limit State in Axially Loaded Piles

METHOD/SOIL/CONDITION		RESISTANCE FACTOR
Ultimate Bearing Resistance of Single Piles	Skin Friction: Clay a-method (Tomlinson 1987) β-method (Esrig & Kirby 1979 and Nordlund method applied to cohesive soils) γ-method (Vijayvergiya & Focht 1972)	0.70 γ_v 0.50 γ_v 0.55 γ_v
	End Bearing: Clay and Rock Clay (Skempton 1951) Rock (Canadian Geotechnical Society 1985)	0.70 γ_v 0.50 γ_v
	Skin Friction and End Bearing: Sand SPT-method CPT-method	0.45 γ_v 0.55 γ_v
	Wave equation analysis with assumed driving resistance Load Test	0.65 γ_v 0.80 γ_v
Block Failure	Clay	0.65
Uplift Resistance of Single Piles	a-method β-method γ-method SPT-method CPT-method Load Test	0.60 0.40 0.45 0.35 0.45 0.80
Group Uplift Resistance	Sand Clay	0.55 0.55
Method of controlling installation of piles and verifying their capacity during or after driving to be specified in the contract documents		Value of γ_v
Pile Driving Formulas, e.g., ENR, equation without stress wave measurements during driving		0.80
Bearing graph from wave equation analysis without stress wave measurements during driving		0.85
Stress wave measurements on 2% to 5% of piles, capacity verified by simplified methods, e.g., the pile driving analyzer		0.90

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Stress wave measurements on 2% to 5% of piles, capacity verified by simplified methods, e.g., the pile driving analyzer and static load test to verify capacity	1.00
Stress wave measurements on 2% to 5% of piles, capacity verified by simplified methods, e.g., the pile driving analyzer and CAPWAP analyses to verify capacity	0.95
Stress wave measurements on 10% to 70% of piles, capacity verified by simplified methods, e.g., the pile driving analyzer	0.95

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Table 10.5.5-3 – Resistance Factors for Geotechnical Strength Limit State in Axially Loaded Drilled Shafts

METHOD/SOIL/CONDITION			RESISTANCE FACTOR
<u>Ultimate Bearing Resistance of Single-Drilled Shafts</u>	<u>Side Resistance in Clay</u>	<u>a-method</u> <u>(Reese & O'Neill 1988)</u>	<u>0.65</u>
	<u>Base Resistance in Clay</u>	<u>Total Stress</u> <u>(Reese & O'Neill 1988)</u>	<u>0.55</u>
	<u>Side Resistance in Sand</u>	<u>Touma & Reese (1974)</u> <u>Meyerhof (1976)</u> <u>Quiros & Reese (1977)</u> <u>Reese & Wright (1977)</u> <u>Reese & O'Neill (1988)</u>	<u>See Discussion</u> <u>in Article 10.8.3.4</u>
	<u>Base Resistance in Sand</u>	<u>Touma & Reese (1974)</u> <u>Meyerhof (1976)</u> <u>Quiros & Reese (1977)</u> <u>Reese & Wright (1977)</u> <u>Reese & O'Neill (1988)</u>	<u>See Discussion</u> <u>in Article 10.8.3.4</u>
	<u>Side Resistance in Rock</u>	<u>Carter & Kulhawy (1988)</u> <u>Horvath & Kenney (1979)</u>	<u>0.55</u> <u>0.65</u>
	<u>Base Resistance in Rock</u>	<u>Canadian Geotechnical Society (1985)</u> <u>Pressure Method (Canadian Geotechnical Society 1985)</u>	<u>0.50</u> <u>0.50</u>
	<u>Side Resistance and End Bearing</u>	<u>Load Test</u>	<u>0.80</u>
<u>Block Failure</u>	<u>Clay</u>		<u>0.65</u>
<u>Uplift Resistance of Single-Drilled Shafts</u>	<u>Clay</u>	<u>a-method</u> <u>(Reese & O'Neill 1988)</u> <u>Belled Shafts</u> <u>(Reese & O'Neill 1988)</u>	<u>0.55</u> <u>0.50</u>
	<u>Sand</u>	<u>Touma & Reese (1974)</u> <u>Meyerhof (1976)</u> <u>Quiros & Reese (1977)</u> <u>Reese & Wright (1977)</u> <u>Reese & O'Neill (1988)</u>	<u>See Discussion</u> <u>in Article 10.8.3.7</u>
	<u>Rock</u>	<u>Carter & Kulhawy (1988)</u> <u>Horvath & Kenney (1979)</u>	<u>0.45</u> <u>0.55</u>
		<u>Load Test</u>	<u>0.80</u>
<u>Group Uplift Resistance</u>		<u>Sand</u> <u>Clay</u>	<u>0.55</u> <u>0.55</u>

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10.6 SPREAD FOOTINGS

10.6.1 General Considerations

Provisions of this Article shall apply to design of isolated footings and, where applicable, to combined footings. Special attention shall be given to footings on fill.

C10.6.1

Problems with insufficient bearing and/or excessive settlements in fill can be significant, particularly if poor (e.g., soft, wet, frozen, or nondurable) material is used, or if the material is not properly compacted. Settlement of improperly placed or compacted fill around piers can cause substantial increases in footing loads resulting from the downward drag or friction force exerted on the pier by the settling fill, i.e., negative skin friction. Even properly placed and compacted backfill undergoes some amount of settlement or swelling depending on the material type, moisture conditions, method of placement, and method and degree of compaction.

10.6.1.4 PRESSURE DISTRIBUTION

For Service Limit State and Strength Limit State loading, footings shall be designed so that the pressure under the footing is as nearly uniform as practicable. The distribution of soil pressure shall be consistent with properties of the soil or rock and the structure and with established principles of soil and rock mechanics.

For seismic loading under the Extreme Event Limit State, non-uniform pressure distributions will develop as the foundation system is subjected to overturning moments. These non-uniform stress distributions are permissible as long as the constraints given in Article 10.6.4.2.1 are satisfied.

10.6.1.4 GROUNDWATER

For Service Limit State and Strength Limit State, footings shall be designed in consideration of the highest anticipated groundwater table. The influences of groundwater table on the bearing capacity of soils or rocks and on the settlement of the structure shall be considered. In cases where seepage forces are present, they shall also be included in the analyses.

For seismic loading under the Extreme Limit State the average height of the groundwater table shall be used in design for SDR 3 and above, subject to the Owner's approval. The highest groundwater location shall be used for SDR 1 and SDR 2.

C10.6.1.4

The likelihood that the highest groundwater table will occur at the same time as the design earthquake is usually very small. For this reason an average groundwater elevation should usually be used in design for SDR 3 and above to obtain the best estimate of foundation stiffness and capacity. However, the decision to use the average groundwater location should be made after considering the likelihood of the highest level occurring during the seismic event, the potential consequences if a higher level is not used, and after discussing this approach with the Owner. The highest groundwater table elevation is acceptable for SDR 1 and SDR 2, inasmuch as only capacity checks are conducted for these categories.

As noted in previous articles, for most cases it is not necessary to use the highest groundwater elevation for liquefaction checks unless the groundwater remains at the highest level for an

10.6.4 Seismic Design at the Extreme Limit State

Seismic design and detailing requirements for spread footing foundations shall be determined in accordance with Section 3. Article 3.10.3 identifies minimum Seismic Design and Analysis Procedures (SDAP's) and minimum Seismic Detailing Requirements (SDR's) for all bridges based on the characteristics of the site and the structure, and the performance objectives for the bridge.

Procedures outlined in the following articles shall be followed in accordance with the SDAP and SDR designation.

extended period of time. The average groundwater level is usually appropriate for two reasons: (1) it represents the most likely case, and (2) the liquefaction potential in zones where groundwater is not permanently located is generally low due to the partially saturated conditions that tend to prevail.

C.10.6.4

During a seismic event, the inertial response of the bridge deck results in a transient horizontal force at the abutments and central piers. This inertial force is resisted by (1) the abutments, (2) the interior piers, or (3) some combination of the two. Forces imposed on the interior columns or piers result in both horizontal shear force and an overturning moment being imposed on the footing. The footing responds to this load by combined horizontal sliding and rotation. The amount of sliding and rotation depends on the magnitude of imposed load, the size of the footing, and the characteristics of the soil.

For seismic design of spread footings, the response of the footing to shear forces and moment is normally treated independently; i.e., the problem is de-coupled. The overturning component of the column load results in an increase in pressures on the soil. Since the response to moment occurs as a rotation, pressure is highest at the most distant point of the footing, referred to as the toe. This pressure can temporarily exceed the ultimate bearing capacity of the soil. As the overturning moment continues to increase, soil yields at the toe and the heel of the footing can separate from the soil, which is referred to as liftoff of the footing. This liftoff is temporary. As the inertial forces from the earthquake change direction, pressures at the opposite toe increase and, if moments are large enough, liftoff occurs at the opposite side. Bearing failure occurs when the force induced by the moment exceeds the total reactive force that the soil can develop within the area of footing contact. Soil is inherently ductile, and therefore, yielding at the toe and liftoff at the heel of the footing are acceptable phenomena, as long as (1) global stability is preserved and (2) settlements induced by the cyclic loading are small.

The shear component of column load is resisted by two mechanisms: (1) the interface friction between the soil and the footing along the side and at the base of the footing, and (2) the passive resistance at the face of the footing. These resistances are mobilized at different deformations. Generally, it takes more displacement to mobilize the passive pressure. However, once mobilized, it normally provides the primary resistance to horizontal loading.

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10.6.4.1 SDR 1 AND SDR 2

Spread footing foundations for SDR 1 and SDR 2 shall be designed for non-seismic loads in accordance with Strength Limit State requirements given in Article 10.6.3. Special design for seismic loads is not required.

The simplified approach to design for SDR 1 and SDR 2 shall be used for regular bridges located at sites where normal performance requirements are acceptable. If the geotechnical and structural engineers for the project determine that special conditions exist, then minimum requirements described for SDR 3 and above may be applicable.

10.6.4.2 SDR 3

Spread footing foundations for SDR 3 shall be designed using column loads developed by capacity design principles or elastic seismic loads, in accordance with Strength Limit State requirements given in Article 10.6.3. It will not normally be necessary to define spring constants for displacement evaluations or moment-rotation and horizontal force-displacement behavior of the footing-soil system (Article 4.8.4.4). Checks shall also be made to confirm that flow slides and loss of bearing support from liquefaction do not occur (Article 3.10.4).

10.6.4.2.1 Moment and Shear Capacity

The overturning capacity of the spread footings shall be evaluated using 1.0 times the nominal moment capacity of the column (Article 3.10.3.8) or the elastic seismic design force within the column, whichever is less. Procedures for Strength Limit State Design given in Article 10.6.3 shall be used when performing this evaluation.

A triangular stress distribution within the soil shall be used. The peak bearing pressure soil for the triangular distribution shall not exceed the ultimate bearing capacity of the soil at the toe of the footing. The width of maximum liftoff shall be no greater than 1/2 of the footing width for moment loading in each of the two directions treated separately.

COMMENTARY

C10.6.4.1

Shear forces and overturning moments developing for seismic loads will normally be small in locations where SDR 1 and SDR 2 apply. Except in special circumstances, the load and resistance factors associated with the Strength Limit State design using non-seismic loads will control the dimensions of the footing.

The potential for liquefaction and liquefaction-induced flow failures is also small and can normally be disregarded for these SDRs. The small potential for liquefaction and flow failure results from the low peak ground accelerations and small earthquake magnitudes normally occurring in these categories. Article 3.10.5.1 and Appendix 3B provide further discussion about liquefaction and the potential for flow failures for sites with low seismicity (SDR 1 and SDR 2) versus sites that have higher levels of seismicity (SDR 3 and above).

C10.6.4.2

Inertial response of a bridge deck results in a horizontal shear force and a moment at the connection of the column to the footing. The footing should not undergo permanent rotation, sliding, or appreciable settlement under these loads. Any permanent displacement that occurs should be constrained by the limits required to preserve the service level of the bridge as suggested in Table C3.10.1-2.

C10.6.4.2.1

The shear component of loading should not be included during the overturning check; i.e., a decoupled approach should be used in treating the two loads. Experience has shown that use of inclination factors to represent the combined horizontal load and moment in simplified bearing capacity equations can result in unreasonably sized footings for seismic loading.

Unfactored resistance is used for the moment capacity check for two reasons: (1) the potential for the design seismic load is very small, and (2) the peak load will occur for only a short duration. The distribution and magnitude of bearing stress, as well as liftoff of the footing, are limited to control settlement of the footing from the cycles of load.

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If a non-triangular stress distribution occurs or if the liftoff is greater 1/2 of the footing, either the footing shall be re-sized to meet the above criteria or special studies shall be conducted to demonstrate that non-triangular stress pressure distribution or larger amounts of liftoff will not result in excessive permanent settlement during seismic loading. The special studies shall include push-over analyses with nonlinear foundation springs for SDAP E conditions.

No shear capacity evaluation of the footing will normally be required for SDR 3.

10.6.4.2.2 Liquefaction Check for SDR 3

An evaluation of the potential for liquefaction within near-surface soil shall be made in accordance with requirements given in Article 3.10.5.1 and Appendix 3B of these Specifications. If liquefaction is predicted to occur for the design earthquake, the following additional requirements shall be satisfied:

Liquefaction without Lateral Flow or Spreading

For sites that liquefy but do not undergo lateral flow or spreading, the bottom of the spread footing shall be located either below the liquefiable layer or at least twice the minimum width above the liquefiable layer. If liquefaction occurs below the footing, settlements resulting from the dissipation of excess porewater pressures shall be established in accordance with procedures given in Appendix 3B.

If the depth of the liquefiable layer is less than twice the minimum foundation width, spread footing foundations shall not be used, unless

- ground improvement is performed to mitigate the occurrence of liquefaction, or
- special studies are conducted to demonstrate that the occurrence of liquefaction will not be detrimental to the performance of the bridge support system.

Before initiating any evaluations of ground improvement alternatives or before conducting special studies, the potential applicability of deep foundations as an alternative to spread footings shall be discussed with the Owner.

COMMENTARY

Non-triangular stress distributions or greater than 50 percent liftoff are allowed if studies can show that soil settlement from cyclic shakedown does not exceed amounts that result in damage to the bridge or unacceptable movement of the roadway surface. By limiting stress distribution and the liftoff to the specified criteria, the amount of shakedown will normally be small under normal seismic loading conditions.

No special check is required for the shear component of column loads for SDR 3 because the maximum horizontal load induced by the seismic event will normally be less than the friction mobilized at the base of the footing for this seismic category.

C10.6.4.2.2

Liquefaction below a spread footing foundation can result in three conditions that lead to damage or failure of a bridge:

- loss in bearing support which causes large vertical movement,
- horizontal forces on the footing from lateral flow or lateral spreading of the soil, and
- settlements of the soil as porewater pressures in the liquefied layers dissipate.

Most liquefaction-related damage during past earthquakes has been related to lateral flow or spreading of the soil. In the case of lateral flow and spreading, ground movements could be a meter or more. If the spread footing foundation is located above the water table, as often occurs, it will be very difficult to prevent the footing from being displaced with the moving ground. This could result in severe column distortion and eventual loss of supporting capacity.

In some underwater locations, it is possible that the flowing ground could move past the footing without causing excessive loading; however, these cases will be limited. For these situations special studies are required to evaluate the magnitude of forces that will be imposed on the foundation and to confirm that these forces will not result in large lateral movement of the footing.

Additional discussion of the consequences of liquefaction is provided in Appendix 3B to these

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Liquefaction with Lateral Flow or Spreading

If lateral flow or lateral spreading is predicted to occur, the amount of displacement associated with lateral flow or lateral spreading shall be established in accordance with procedures given in Appendix 3B. Once the deformation has been quantified, the following design approach shall be used.

- Determine whether the spread footings can be designed to resist the forces generated by the lateral spreading without unusual size or design requirements.
- If the footing cannot resist forces from lateral spreading or flow, assess whether the structure is able to tolerate the anticipated movements and meet the geometric and structural constraints of Table 3.10.1-2. The maximum plastic rotation shall be as defined in Article 5.16.
- If the structure cannot meet the performance requirements of Table 3.10.1-1, assess the costs and benefits of various mitigation measures to minimize the movements to a level that will meet the desired performance objective. If a higher performance is desired so that the spread footings will not have to be replaced, the allowable plastic rotations for concrete columns given in Article 5.16 shall be met.

The Owner shall be apprised of and concur with the approach used for the design of spread footing foundations for lateral flow or lateral spreading conditions.

10.6.4.3 SDR 4, 5, & 6

The design of spread footing foundations located in SDR 4, 5, and 6 shall be based on column moments and shears developed using capacity design principles as described in Section 3.10.3.8.

Foundation flexibility (Article 4.8.4.4) shall be modeled for Soil Types C, D, and E if foundation flexibility results in more than a 20 percent change in response. For Soil Types A and B, soil flexibility does not need to be considered because of the stiffness of the soil or rock. The potential for and effects of liquefaction and dynamic settlement shall also be determined for spread footing foundations subject to

Specifications. A flow chart showing the methodology for addressing the moving soil case is given in Figure 3B.4.2-1.

C10.6.4.3

Various approaches are available to evaluate the response of the bridge-footing system during the design event. In most cases the bridge designer will use equivalent linear springs to represent the soil-footing system. Guidance provided in these Specifications focuses on these simple procedures.

For critical or irregular bridges more rigorous modeling is sometimes used. These methods can involve use of two- and three-dimensional finite element or finite difference modeling methods. This approach to modeling involves considerable expertise in developing a model that represents the

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SDR 4 and above. Normally, spread footings shall not be located at sites within SDR 4, 5, and 6 where liquefaction is predicted to occur, unless:

- The foundation is located below the liquefiable layer.
- It can be demonstrated by special studies that liquefaction and its effects are very limited, or
- The ground will be improved such that liquefaction will not occur.

Owner approval shall be obtained before proceeding with a spread footing design at a site where liquefaction is predicted to occur.

10.6.4.3.1 Spring Constants for Footing (Nonliquefiable Sites)

When required to represent foundation flexibility, spring constants shall be developed for spread footing using equations given in Tables 10.6.4-1 and 10.6.4-2. Alternate procedures given in FEMA 273 (1997) are also suitable for estimating spring constants. These computational methods are appropriate for sites that do not liquefy or lose strength during earthquake loading. See Article 10.6.4.3.3 for sites that are predicted to liquefy.

The shear modulus (G) used to compute the stiffness values in Table 10.6.4-1 shall be determined by adjusting the low-strain shear modulus ($G_{\max}$) for the level of shearing strain using the following strain adjustment factors, unless other methods are approved by the Owner.

$$F_v S_1 \leq 0.40$$

- $G/G_{\max} = 0.50$ for 50% in 75-year event
- $G/G_{\max} = 0.25$ for 3% in 75-year event

$$F_v S_1 > 0.40$$

- $G/G_{\max} = 0.25$ for 50% in 75-year event
- $G/G_{\max} = 0.10$ for 3% in 75-year event

Uplift shall be allowed for footings subject to SDR 4, 5, and 6. The following area adjustment factors (R_a)

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soil-structure system. Close cooperation is required between the structural engineer and the geotechnical engineer when developing the model; each discipline has to be familiar with the limitations associated with the development of the model. Without this cooperative approach, it is very easy to obtain very precise results that have little relevance to likely performance during the design earthquake.

Liquefaction represents a special design problem for spread footings because of the potential for loss in bearing support, lateral movement of the soil from flow or lateral spreading, and settlements following an earthquake as porewater pressures in liquefied soil dissipate. Nonlinear, effective stress methods are normally required to adequately replicate these conditions in computer models. Such modeling methods are limited in number and required significant expertise. They are usually applicable for bridge design projects only in special circumstances.

C10.6.4.3.1

A Winkler spring model is normally used to represent the vertical and moment-rotation curve in the analysis. A uniformly distributed rotational stiffness can be calculated by dividing the total rotational stiffness of the footing by the moment of inertia of the footing in the direction of loading. Similar methods are used for vertical stiffness.

Strain and Lutoff Adjustment Factors

Equations given in Tables 10.6.4-1 and 10.6.4-2 are based on elastic halfspace theory. These equations were originally developed for very low levels of dynamic loading associated with machine foundations. For these levels of loading, it is possible to use the low-strain shear modulus ($G_{\max}$) of the soil, and the footing remains in full contact with the soil. During seismic loading, at least two different phenomena occur which are inconsistent with the assumptions used in the original development of these equations. These differences involve (1) the nonlinear response of the soil from both free field earthquake wave propagation and from local strain amplitude effects and (2) the lutoff of the footing.

- Strain Amplitude Effects: The strain amplitude effects reflect the inherent nonlinearity of soil, even at very low shearing strain amplitudes. As

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shall be applied to the equivalent area to account for geometric nonlinearity introduced by uplift, unless the Owner approves otherwise.

$$F_v S_1 \leq 0.40$$

- $R_a = 1.0$ for the 50% in 75-year event
- $R_a = 0.75$ for the 3% in 75-year event

$$F_v S_1 > 0.40$$

- $R_a = 0.75$ for the 50% in 75-year event
- $R_a = 0.5$ for the 3% in 75-year event

Values of $G_{\max}$ shall be determined by seismic methods (e.g., crosshole, downhole, or SASW), by laboratory testing methods (e.g., resonant column with adjustments for time), or by empirical equations (Kramer, 1996). The uncertainty in determination of $G_{\max}$ shall be considered when establishing strain adjustment factors.

No special computations are required to determine the geometric or radiation damping of the foundation system. Five percent system damping shall be used for design, unless special studies are performed and approved by the Owner.

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the seismic wave propagates through the soil, the soil softens, resulting in a reduced shear modulus. Both field measurements and numerical modeling have shown this softening, as discussed by Kramer (1996). A second source of soil nonlinearity also must be considered. As the footing responds to inertial loading from the bridge column, local soil nonlinearities occur around the footing as the soil is subjected to stress from the shear forces and overturning moments. While various procedures exist for estimating the free-field effects of wave propagation, simple methods for estimating the local strain effects have yet to be developed. Nonlinear finite element or finite difference methods can be used to evaluate these effects; however, for most bridge studies such modeling cannot be justified. In recognition of the need for simple guidelines, $G/G_{\max}$ adjustment factors were estimated. This approach for dealing with soil nonlinearity involves considerable judgment, which may warrant modification on a case-by-case basis.

- Liftoff Effects: The consequence of uplift during seismic loading will be that the effective area of the footing will be less than if full contact were to occur. The amount of uplift is expected to be larger in a higher seismic zone and during an event with a longer return period. The area adjustments for liftoff were made by recognizing that the maximum liftoff allowed under the extreme loading condition will usually be one-half uplift of the footing. It was also recognized that the maximum uplift would only occur for a short period of time, and that during most of the earthquake, the maximum loading might be from 50 to 70 percent of the peak value. For this reason the effective uplift would not be as much as the peak uplift. Values shown were selected after discussing the potential values of effective area that might occur and then applying considerable engineering judgment.

Uncertainty in Spring Constant Determination

Stiffness constants developed in the manner described in this Article involve uncertainty. A prudent Designer will account for this uncertainty by evaluating stiffness for upper and lower bound modulus values, in addition to the best-estimate shear modulus. The upper and lower bound values are used to account for (1) the variability of shear modulus that is likely to occur in the field, (2) the uncertainty in adjustments being used for shearing

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strain and geometric nonlinearities, and (3) limitations in the equation for determining stiffness.

The range of modulus variation used by the Designer in a sensitivity evaluation is expected to change, depending on the characteristics of the site, the details of the site characterization process, and the type of analysis. Common practice is often to assume that the lower bound shear modulus is approximately 50 percent of the best estimate and the upper bound is approximately 100 percent greater than the best estimate. If the resulting upper and lower bound values of stiffness are such that significant differences in bridge response is possible, then consideration should be given to either (1) evaluating bridge response for the range of stiffness values or (2) performing additional site characterization studies to reduce the range used in defining the upper and lower bound.

Geometric or Radiation Damping

The conventional approach during the use of elastic halfspace methods accounts for energy loss within the foundation system through a spectral damping factor. The spectral damping factor is typically defined as 5 percent, and is intended to represent the damping of the structure-foundation system. This damping differs from the geometric or radiation damping of a foundation. For translational modes of loading, the foundation damping can be in excess of 20 percent. The 5 percent spectral damping used in the modal analysis procedures is intended to account for the geometric damping within the foundation system, as well as damping in the bridge structure. While it may be possible to increase the spectral damping of the overall system to a higher level to account for the high geometric damping within the foundation, in view of the liftoff that is allowed to occur during the design earthquake, it is generally not prudent to count on the high levels of foundation damping, at least without special studies that properly account for the liftoff of the foundation.

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Table 10.6.4-1. Surface Stiffnesses for a Rigid Plate on a Semi-Infinite Homogeneous Elastic Half-Space (adapted from Gazetas, 1991)¹

Stiffness Parameter	Rigid Plate Stiffness at Surface, K_i'
Vertical Translation, K_z'	$\frac{GL}{1-n} \left[0.73 + 1.54 \left(\frac{B}{L} \right)^{0.75} \right]$
Horizontal Translation, K_y' (toward long side)	$\frac{GL}{2-n} \left[2 + 2.5 \left(\frac{B}{L} \right)^{0.85} \right]$
Horizontal Translation, K_x' (toward short side)	$\frac{GL}{2-n} \left[2 + 2.5 \left(\frac{B}{L} \right)^{0.85} \right] - \frac{GL}{0.75-n} \left[0.1 \left(1 - \frac{B}{L} \right) \right]$
Rotation, K_{qx}' (about x axis)	$\frac{G}{1-n} I_x^{0.75} \left(\frac{L}{B} \right)^{0.25} \left(2.4 + 0.5 \frac{B}{L} \right)$
Rotation, K_{qy}' (about y axis)	$\frac{G}{1-n} I_y^{0.75} \left[3 \left(\frac{L}{B} \right)^{0.15} \right]$

1. See Figure 10-6.4-1 for definitions of terms

Table 10.6.4-2. Stiffness Embedment Factors for a Rigid Plate on a Semi-Infinite Homogeneous Elastic Half-Space (adapted from Gazetas, 1991)¹

Stiffness Parameter	Embedment Factors, e_i
Vertical Translation, e_z	$\left[1 + 0.095 \frac{D}{B} \left(1 + 1.3 \frac{B}{L} \right) \right] \left[1 + 0.2 \left(\frac{(2L+2B)}{LB} d \right)^{0.67} \right]$
Horizontal Translation, e_y (toward long side)	$\left[1 + 0.15 \left(\frac{2D}{B} \right)^{0.5} \right] \left\{ 1 + 0.52 \left[\frac{\left(D - \frac{d}{2} \right) 16 (L+B) d}{BL^2} \right]^{0.4} \right\}$
Horizontal Translation, e_x (toward short side)	$\left[1 + 0.15 \left(\frac{2D}{L} \right)^{0.5} \right] \left\{ 1 + 0.52 \left[\frac{\left(D - \frac{d}{2} \right) 16 (L+B) d}{LB^2} \right]^{0.4} \right\}$
Rotation, e_{qx} (about x axis)	$1 + 2.52 \frac{d}{B} \left(1 + \frac{2d}{B} \left(\frac{d}{D} \right)^{-0.20} \left(\frac{B}{L} \right)^{0.50} \right)$
Rotation, e_{qy} (about y axis)	$1 + 0.92 \left(\frac{2d}{L} \right)^{0.60} \left(1.5 + \left(\frac{2d}{L} \right)^{1.9} \left(\frac{d}{D} \right)^{-0.60} \right)$

Note. Embedment factors multiplied by spring

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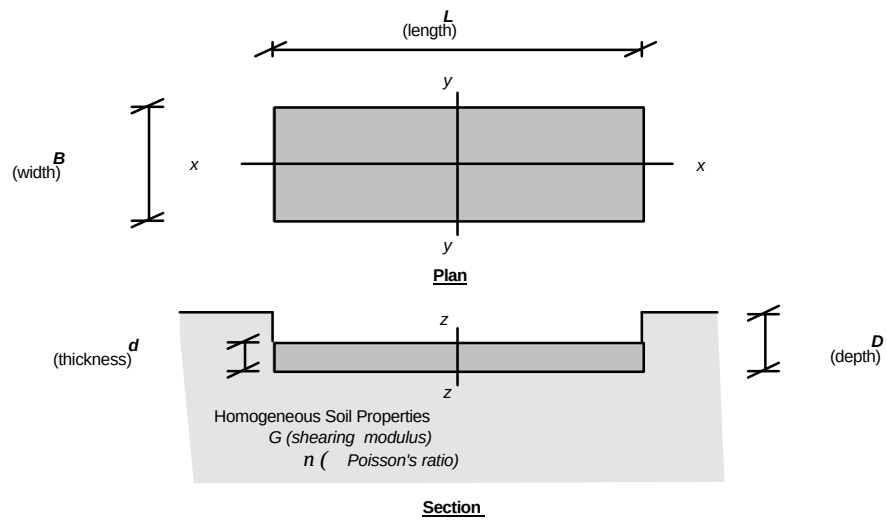


Figure 10.6.4-1. Properties of a Rigid Plate on a Semi-infinite Homogeneous Elastic Half-Space for Stiffness Calculations

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10.6.4.3.2 Moment-Rotation and Shear-Displacement Relationships for Footing (Nonliquefiable Sites)

The moment and shear capacity of the foundation shall be confirmed for design loads equal to 1.5 times the moment capacity of the column and for the plastic shear capacity of the column. Moment-rotation and shear force-displacement relationships shall be developed as required by Article 3.10.3.8. Unless approved otherwise by the Owner, the moment-rotation curve for SDAP E shall be represented by a bilinear, moment-rotation curve. The initial slope of the bi-linear curve shall be defined by the rotational spring constant given in Article 10.6.4.3.1.

The maximum resisting force (i.e., plastic cap) on the force-deformation curve shall be defined for the best-estimate case. The footing liftoff shall be no more than 50 percent at peak displacement during the push-over analysis, unless special studies are performed and approved by the Owner. A bilinear force displacement relationship shall also be developed for the shear component of resistance.

This approach shall not be used at sites that will liquefy during seismic loading. See Article 10.6.4.3.3 for sites that liquefy.

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C10.6.4.3.2

The foundation capacity in SDR 3 and above requires an evaluation of the soil to resist the overturning moment and the shear force from the column. Vertical loading to the footing will also change during seismic loading, and this change also needs to be considered.

The initial slope of the moment-rotation curve should be established using the best-estimate rotational spring constant defined in the previous article. Checks can be performed for the upper and lower bound of the initial slope; however, these variations will not normally be important to design.

It is critical during determination of the moment capacity for the moment-rotation curve to use the ultimate bearing capacity for the footing without use of a resistance factor (i.e., use $\phi = 1.0$). The determination of ultimate bearing capacity should not be limited by settlement of the footing, as is often done for static bearing capacity determination. The ultimate capacity for the moment-rotation relationship should be defined for the best-estimate soil conditions.

For important bridges, the Designer should consider use of upper and lower bounds for bearing capacity to account for uncertainties. The range for the upper and lower bound will depend on the variability of soils at the site and the extent of field explorations and laboratory testing. Common practice is often to assume that the lower bound capacity is approximately 50 percent of the best estimate and the upper bound is approximately 100 percent greater than the best estimate.

Shear-Displacement

During horizontal shear loading, the resisting force comprises the resistance developed along the base and the sides of the footing and from the passive pressure at the face of the footing. The passive pressure will often provide most of the reaction during a seismic event. For simplicity it can be assumed that the maximum resistance (passive + base + two sides) is developed at a deformation equal to 2 percent of the footing thickness.

The shear resistance on the base and side of the footing should be determined using an interface shear strength. For most cast-in-place concrete foundations, a value of interface friction of 0.8 times the friction angle of the soil will be appropriate. Displacements to mobilize this resistance will normally be less than 10 to 20 mm. The passive pressure at the face of the footing should

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be computed assuming an interface friction angle equal to 50 percent of the friction angle of the backfill material. The log spiral or Caquot-Kerisel (1948) methods, as given in Article 3.11 should be used for determining the ultimate passive pressure. If the backfill material changes within twice the height of the footing, the effects of the second material should be included in the computation of the passive pressure. A method of slices similar to a slope stability analysis offers one method of accomplishing this computation.

Deformations needed to mobilize the ultimate passive resistance of the face of a footing could easily exceed 25 mm for a typical footing thickness. The potential consequences of this movement relative to column behavior will usually be evaluated during the soil-structure interaction analysis. The uncertainty in computing deformations associated with ultimate passive resistance determination is such that a variation of -50 percent and +100 percent would not be unusual. If this variation has a significant effect on, say, the push-over-analysis, the Designer may want or modify the foundation or the soil conditions to reduce the uncertainty or limit the deformations.

As discussed by Kramer (1996), evidence exists that the available ultimate passive resistance during seismic loading could be reduced by the seismic response of the ground. This condition occurs if the direction of loading from the inertial response of the bridge structure is the same as the motions in the ground. These two loadings normally occur at dissimilar frequencies, and therefore, the coincidence of the directions of loading is usually for only a moment in time. When the movements are out of phase, the loading increases. It was felt that reducing the passive ultimate resistance for the short periods of coincidence would underestimate the effective passive capacity of the foundation (i.e., low ultimate resistance), and therefore the approach taken in this Specification is to ignore this potential effect. This approach clearly involves considerable judgment, and therefore, an alternate approach that includes the reduction in passive resistance could be used, subject to the Owner's approval.

Vertical Load Capacity

For most designs it is unnecessary to consider increases in vertical forces on the footing during seismic loading, as these forces will normally be a fraction of the gravity load. However, if the bridge site is located in proximity to an active fault, vertical accelerations could become important, as discussed in Article 3.10.2.6. For these situations the potential displacement should be checked using the spring constants given in Table 10.6.4-1 together with the increase in vertical column

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10.6.4.3.3 Liquefaction and Dynamic Settlement

An evaluation of the potential for liquefaction within near-surface soil shall be made in accordance with requirements given in Article 3.10.5.1 and Appendix 3B of these Specifications. If liquefaction is predicted to occur under the design ground motion, spread footings foundations shall not be used unless

- the footing is located below the liquefiable layer
- ground improvement is performed to mitigate the occurrence of liquefaction, or
- special studies are conducted to demonstrate that the occurrence of liquefaction will not be detrimental to the performance of the bridge support system.

The Owner's approval shall be obtained before initiating ground improvement or special studies.

10.6.5 Structural Design

The structural design of footings shall comply with the requirements given in Article 5.12.4.

10.7 DRIVEN PILES

10.7.1 General

10.7.1.1 USE

Piling shall be considered when spread footings cannot be founded on rock, stiff cohesive, or granular foundation material at a reasonable expense. At locations where soil conditions would normally permit the use of spread footings, but the potential for erosion or earthquake-induced liquefaction exists, piles may be used as a protection against scour or as a method of avoiding loss of bearing support and settlement associated with liquefaction. Piles may also be required to develop uplift resistance for loading cases involving large overturning moments being applied to the pile foundation.

load. The potential consequences of reduction in vertical loads through inertial response should also be considered. This effect could temporarily decrease lateral resistance and moment capacity

C10.6.4.3.3

Liquefaction below a spread footing foundation located in SDR 4 and above could be significant because of the combination of higher ground accelerations and larger earthquake magnitudes. As the potential for liquefaction increases, the potential for damage or failure of a bridge from loss in bearing support, lateral flow or lateral spreading of the soil, or settlements of the soil as porewater pressures in the liquefied layers dissipate also increases.

Additional discussion of the consequences of liquefaction are provided in Article 10.6.4.2.2 and Appendix 3B to these Specifications. A flow chart showing the methodology for addressing the moving soil case is given in Figure 3B.4.2-1.

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10.7.1.2 PILE PENETRATION

Required pile penetration shall be determined based on the resistance to vertical and lateral loads and the displacement of both the pile and the subsurface materials. In general, unless refusal is encountered, the design penetration for any pile under compressive loading shall be not less than 3000 mm into hard cohesive or dense granular material and not less than 6000 mm into soft cohesive or loose granular material. If loose granular materials exist, the pile penetration shall be below the depth of predicted liquefaction.

Unless refusal is encountered, piles for trestle or pile bents shall penetrate a distance equal to at least one-third the unsupported length of the pile.

Piling used to penetrate a soft or loose upper stratum overlying a hard or firm stratum, shall penetrate the firm stratum by a distance sufficient to limit movement of the piles and attain sufficient bearing capacities. If the piles must resist lateral loads from ship impact or seismic loading, the piles shall penetrate the soil a distance sufficient to achieve pile fixity. Typically, this depth will be 5 to 10 pile diameters below the top of the ground surface.

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C10.7.1.2

To meet uplift loading requirements during a seismic event or during ship impact, the depth of penetration may have to be greater than minimum requirements for compressive loading to mobilize sufficient uplift resistance. This uplift requirement can impose difficult installation conditions at locations where very hard bearing layers occur close to the ground surface. In these locations the 3000 mm requirement may not be sufficient and longer piles may be required. Ground anchors, insert piles, and H-pile stingers can be used to provide extra uplift resistance in these situations. Article 10.7.3.7 provides additional information on procedures used to evaluate uplift capacity of a pile.

The depth of penetration (or embedment) for lateral loading will be determined by the soil or rock stiffness occurring in the upper 5 to 10 pile diameters. The depth at which "fixity" occurs (i.e., where the deformations of the pile are essentially zero under the imposed lateral load) will depend on the stiffness of the pile and the strength of the soil. Article 10.7.3.8 provides additional discussion of the methods used to evaluate the lateral response of a pile.

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10.7.1.3 RESISTANCE

Piles shall be designed to have adequate bearing and structural resistances, tolerable settlements, and tolerable lateral displacements.

The supporting resistance of piles under static and seismic loading shall be determined by a suitable combination of subsurface investigations, laboratory and/or in-situ tests, analytical methods, pile load tests (e.g., the use of the pile driving analyzer [PDA] in combination with stress-wave interpretation techniques such as the Case Method and CAPWAP [CAse Pile Wave Analysis Procedure]), and reference to the history of past performance. Consideration shall also be given to:

- The difference between the resistance of a single pile and that of a group of piles;
- The capacity of the underlying strata to support the load of the pile group;
- The potential effects of predrilling, jetting, and plug development on the installation and the capacity of the pile;
- The effects of driving the piles on adjacent structures;
- The possibility of scour and its effect;
- The transmission of forces, such as negative skin friction or downdrag forces, from consolidating soil; and
- The effects of seismic loads, as discussed in Article 10.7.4.

Resistance factors for pile capacities shall be as specified in Table 10.5.5-2, with the exception that in SDR 3 and above for Extreme Event Load Case I (Seismic Loading), no resistance factors shall be used. Procedures for dealing with uncertainties associated with seismic loading are addressed in Article 10.7.4.

10.7.1.4 EFFECT OF SETTLING GROUND AND DOWNDRAW LOADS

Possible development of downdrag loads on piles shall be considered where:

- Sites are underlain by compressible clays, silts, or peats;
- Fill has recently been placed on the earlier

C10.7.1.4

Where a soil deposit in which or through which piles have been installed is subject to consolidation and settlement in relation to the piles, downdrag forces are induced on the piles. The induced downdrag loads tend to reduce the usable pile capacity.

As explained in Section 3.11.8, downdrag is a load, and side friction is a resistance. Downdrag is not

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surface;

- The groundwater is substantially lowered; and
- Liquefaction is predicted to occur.

Downdrag loads shall be considered as a load when the bearing resistance and settlement of pile foundations are investigated. Downdrag is a structural capacity issue only in the case of a true end-bearing pile on very dense or hard soil or rock where the pile capacity is generally controlled by the structural strength of the pile and where settlements of the pile are negligible. In all other cases of piles bearing in compressible soils where the pile capacity is controlled by tip resistance and shaft adhesion or friction, downdrag may be regarded as a settlement issue.

The downdrag loads may be determined as specified in Article 10.7.3.3, with the direction of the side friction forces reversed. The factored drag loads shall be added to the factored vertical dead load applied to the deep foundation in the assessment of bearing capacity at the Strength Limit State.

The downdrag loads shall be added to the vertical dead load applied to the deep foundation in the assessment of settlement at service limit state and for Extreme Event Load Case I (Seismic Loads). For Extreme Event Load Case I, unfactored load and resistance factors ($\gamma = 1.0$; $\phi = 1.0$) shall be used, unless required otherwise by the Owner.

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combined with transient loads. Downdrag loads are not to be combined with transient loads because transient loads cause downward movement of the pile or pier relative to the ground, causing temporary reduction or elimination of downdrag loads.

The downdrag loads can occur during seismic loading if cohesionless soil liquefies. As porewater pressures dissipate, densification of the granular soil occurs, resulting in settlement and associated downdrag. Article 10.7.4 provides additional discussion on the seismic load case.

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10.7.1.5 PILE SPACING, CLEARANCES, AND EMBEDMENT

Center-to-center pile spacings shall not be less than the greater of 750 mm or 2.5 pile diameters of widths. The distance from the side of any pile to the nearest edge of the footing shall be greater than 225 mm.

The tops of piles shall project at least 300 mm into footings after all damaged pile material has been removed, unless the pile is attached to the footing by embedded bars or strands, in which case the pile should extend no less than 150 mm into the footing. Where a reinforced concrete beam is cast-in-place and used as a bent cap supported by piles, the concrete cover at the sides of the piles shall be greater than 150 mm, plus an allowance for permissible pile misalignment, and the piles shall project at least 150 mm into the cap. Where pile reinforcement is anchored in the cap satisfying the requirements of Article 5.14.4.1, the projection may be less than 150 mm. In all cases minimum structural design provisions required for seismic loading in Sections 5 and 6 shall be satisfied.

10.7.1.6 BATTER PILES

Batter piles shall not be used where downdrag loads are expected and in SDR 3 and above, unless special studies are performed. Batter piles may be used in the foundation for nonseismic loading applications and in SDR 1 and 2 where the lateral resistance of vertical piles is inadequate to counteract the horizontal forces transmitted to the foundation or when increased rigidity of the entire structure is required.

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C10.7.1.5

Multiple piles located at close spacing will not have the same axial and lateral capacity as the same piles at wide spacings. These effects of pile spacing, referred to as group effects, must be accounted for during design. Guidelines are provided in Article 10.7.3.10 and 10.7.3.11 regarding these methods.

Significant installation problems involving heave, lateral displacement, and hard driving can also occur when the center-to-center spacing of piles is small.

C10.7.1.6

If batter piles are used in SDR 3 and above, consideration must be given to (1) downdrag forces caused by dissipation of porewater pressures following liquefaction, (2) the potential for lateral displacement of the soil from liquefaction-induced flow or lateral spreading, (3) the ductility at the connection of the pile to the pile cap, and (4) the buckling of the pile under combined horizontal and vertical loading. These studies will have to be more detailed than those described elsewhere within Section 10. As such, use of batter piles should be handled on a case-by-case basis. Close interaction between the geotechnical engineer and the structural engineer will be essential when modeling the response of the batter pile for seismic loading.

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10.7.1.7 GROUNDWATER TABLE AND BUOYANCY

Bearing capacity shall be determined using the groundwater level consistent with that used to calculate load effects. The effect of hydrostatic pressure shall be considered in the design.

For seismic loading the groundwater table location shall be the average groundwater location, unless the Owner approves otherwise.

10.7.1.10 ESTIMATED LENGTHS

Estimated pile lengths for each substructure shall be shown on the plans and shall be based upon careful evaluation of available subsurface information, static and lateral capacity calculations, potential seismic loading effects, and/or past experience.

10.7.1.11 ESTIMATE AND MINIMUM TIP ELEVATION

Estimated and minimum pile tip elevations for each substructure shall be shown on the contract plans. Estimated pile tip elevations shall reflect the elevation where the required ultimate pile capacity can be obtained. Minimum pile tip elevations shall reflect the penetration required to support lateral pile loads, including scour and seismic loading considerations where appropriate, and/or penetration of overlying unsuitable soil strata.

10.7.4 Seismic Design for Extreme Limit States

Seismic design and detailing requirements for driven pile foundations shall be determined in accordance with Section 3. Article 3.10.3 identifies minimum Seismic Design and Analysis Procedures (SDAP's) and minimum Seismic Detailing Requirements (SDR's) for all bridges based on the characteristics of the site and the structure, and the performance objectives for the bridge.

Procedures for seismic design of driven pile foundations are outlined in the following articles. These procedures are dependent on both the SDAP

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C10.7.1.7

Unless the pile is bearing on rock, the tip resistance is primarily dependent on the effective surcharge that is directly influenced by the groundwater level. For drained loading conditions, the vertical effective stress, s'_v , is related to the groundwater level and thus affects pile capacity.

Seismic design loads will have a very low probability of occurrence. For most cases the likelihood of the seismic load and the highest groundwater elevation (e.g., extreme tidal height or river flood elevation) occurring at the same time is very low. This low probability normally justifies not using the highest groundwater level during seismic design. Different Owner's have different design philosophies regarding the assumptions on groundwater level; therefore, the design requirement for groundwater should be discussed with the Owner early in the design process.

C10.7.4

During a seismic event, the inertial response of the bridge deck results in a transient horizontal force. This inertial force is resisted by (1) the abutments, (2) the interior piers, or (3) some combination of the two. Forces imposed on the interior columns or piers result in both horizontal shear force and overturning moments being imposed on the pile foundation. The pile foundation responds to this load by combined horizontal deflection and rotation. The amount of horizontal deflection and rotation depends on the magnitude of imposed load, the size and type of the

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foundation system, and the characteristics of the soil.

For seismic design of driven pile foundations, the response of the foundation system to shear forces and moment is normally treated independently; i.e., the problem is de-coupled. If the driven pile is part of a group of piles, as normally occurs, the overturning component of the column load results in an increase in vertical loading on the piles in the direction of loading and a reduction in load in the other direction. Since the response to moment occurs as a rotation, load increase is highest at the most distant pile. This load can temporarily exceed the bearing capacity of the soil. As the overturning moment continues to increase, soil yields at the leading edge of the pile group and the pile begins to plunge. At the trailing edge, uplift loads occur, possibly, resulting in separation between the pile tip and the soil. This uplift is temporary. As the inertial forces from the earthquake change direction, loads at the opposite side increase and, if moments are large enough, uplift occurs at the opposite end. Plunging failure of the pile group occurs only when the force induced by the moment exceeds the total reactive force that the soil can develop for the entire group of piles. Soil is inherently ductile, and therefore, yielding of the forward pile and uplift at the trailing pile are acceptable phenomena, as long as global stability is preserved.

The shear component of column load is resisted by the passive pressure at the face of each pile. Normally, this resistance is mobilized in the upper 5 to 10 pile diameters. If the foundation system includes a pile cap, the reaction to the shear load results from the resistance of the piles and the resistance of the pile cap. The cap develops resistance from (1) the interface friction between the soil and the cap along the side of the cap and (2) the passive resistance at the face of the cap. These resistances are mobilized at different deformations. Generally, it takes more displacement to mobilize the passive pressure. However, once mobilized, it can provide the primary resistance of the foundation system.

For some sites the potential occurrence of scour around the pile is possible. If scour occurs the effective length of the pile could change, which could in turn affect the seismic response of the bridge-foundation system. If a potential for scour around the piles exists during the design life of the bridge, the seismic analysis should be made considering the likely, but not necessarily maximum, depth of scour. In this situation, the maximum depth of scour may not be required because of the low probability of both cases occurring at the same time. If the assumptions on scour depth have (or could have) a significant effect on seismic response, the Designer should meet with the Owner and establish a strategy for addressing this issue. This strategy could involve conducting a series of

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10.7.4.1 SDR 1 AND SDR 2

Driven pile foundations subject to SDR 1 and SDR 2 shall be designed in accordance with Strength Limit State requirements given in Article 10.7.3 for non seismic loads. Special design for seismic loads is not required.

Reinforced concrete piles and pile bents subject to SDR 2 shall be reinforced according to the implicit method of Article 5.10.11.4.1c

This simplified approach to design for SDR 1 and SDR 2 shall be used for regular bridges located at sites where normal performance requirements are acceptable. If the geotechnical and structural engineer for the project determine that special conditions exist, then minimum requirements described for SDR 3 and above may be applicable.

10.7.4.2 SDR 3

Driven pile foundations subject to SDR 3 shall be designed for column moments and shears developed in accordance with the principles of capacity design (Article 3.10.3.8) or the elastic design forces, whichever is smaller. The Strength Limit State requirements given in Article 10.7.3 shall apply for design.

With the exception of pile bents, it will not normally be necessary to define spring constants for displacement evaluations or moment-rotation and horizontal force-displacement analyses for SDR 3 (Article 4.8.4.4). For pile bents, the estimated depth of fixity shall be used in evaluating response.

If liquefaction is predicted at the site, the potential effects of liquefaction on the capacity of the driven pile foundation system shall be evaluated in accordance with procedures given in Article 10.7.4.2.2.

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parametric studies to bracket the range of possible responses.

C10.7.4.1

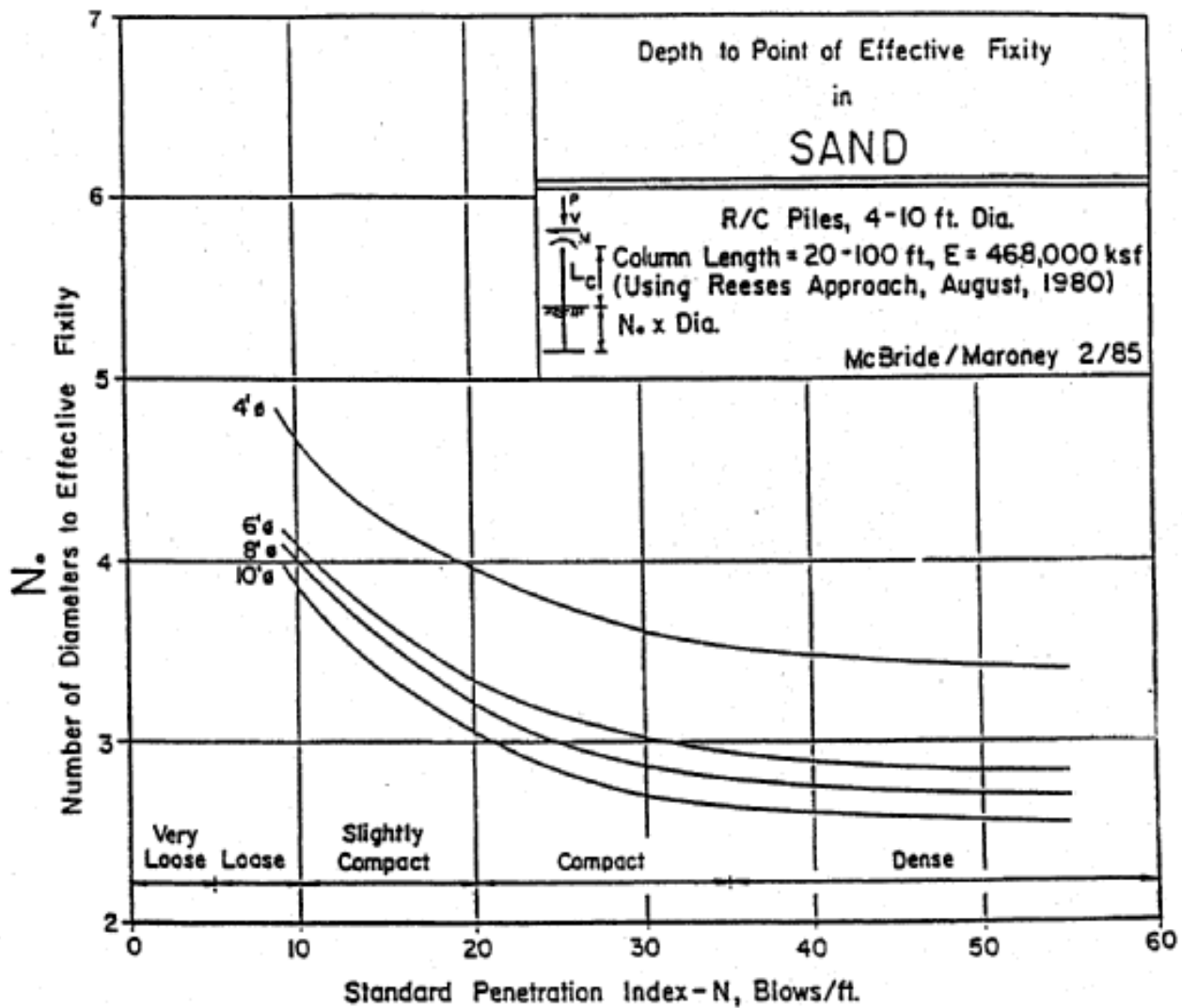
Seismic shear forces and overturning moments developing within this design category will normally be small. Except in special circumstances, the load and resistance factors associated with Strength Limit State for non-seismic loads will control the number and size of the pile foundation system.

The potential for liquefaction and liquefaction-induced flow failures is also small and can normally be disregarded in these SDRs. The small potential for liquefaction and flow failure results from the low peak ground accelerations and small earthquake magnitudes normally occurring for these categories. Article 3.10.5.1 and Appendix 3B provides further discussion about liquefaction and the potential for flow failures for sites with low seismicity (SDR 1 and SDR 2) versus sites that have higher levels of seismicity (SDR 3 and above).

C10.7.4.2

Shear forces and overturning moments developing within this design category will normally be small. Except in special circumstances, the load and resistance factors associated with Strength Limit State will control the number and size of the pile foundation system. A capacity check under overturning moment is, however, required to confirm that the specific features of the bridge design and soil conditions do not result in instability or excessive uplift of the foundation system. Checks should also be made to confirm that unacceptable displacements from flow slides or loss of bearing support from liquefaction do not occur.

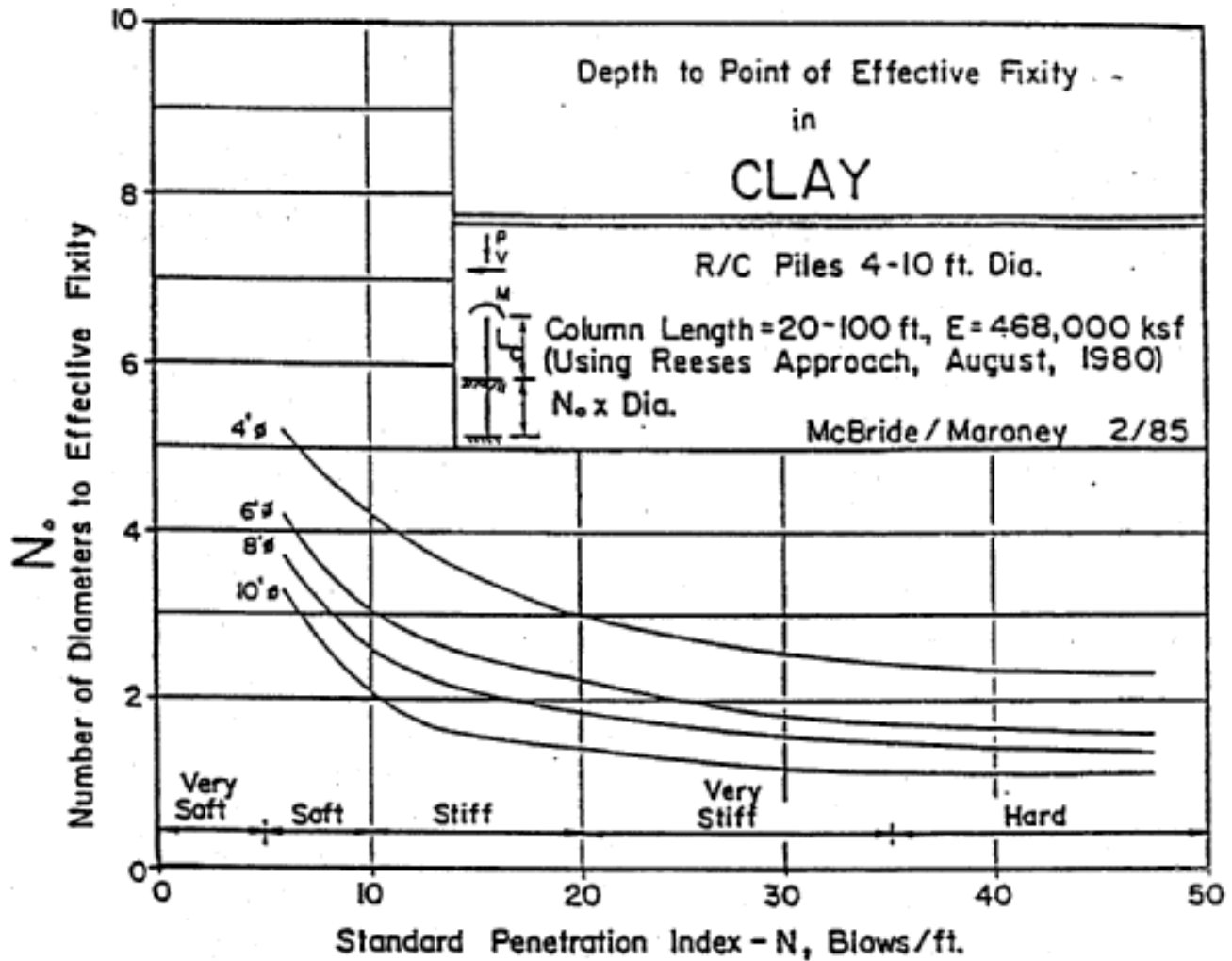
The flexibility of pile bents is included because it is relatively easy to include and it is generally more significant than that of spread and piled foundations. For pile bents the estimated depth of fixity can be determined in one of the following ways: (1) using the simplified relationships shown in Figure 10.7.4-1 and Figure 10.7.4-2, (2) using relationships given in FHWA (1997) and DM7 (1982), or (3) conducting lateral pile analyses using a beam-column approach.



0-28	28-30	30-36	36-41	Approximate ϕ in degrees

1 kip = 4.45 kN
1 in = 25.4 mm
1 ft = 0.305 m

Figure 10.7.4-1. Simplified procedure for estimating depth of pile fixity in sand (Lam et al., 1998)



0-.5	.5-1	1-2	2-4	4-6 Shear Strength, ksf

1 kip = 4.45 kN

1 in = 25.4 mm

1 ft = 0.305 m

Figure 10.7.4-1. Simplified procedure for estimating depth of pile fixity in clay (Lam et al., 1998)

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10.7.4.2.1 Moment and Shear Design

The capacity of the geotechnical elements of driven pile foundations shall be designed using 1.0 times the nominal moment capacity of the column or the elastic design force within the column (Article 3.10.3.8), whichever is smaller. Unfactored resistance ($\phi = 1.0$) shall be used in performing the geotechnical capacity check. The loads on the leading pile row during overturning shall not exceed the plunging capacity of the piles. Separation between the pile tip and the soil (i.e. gapping) shall be allowed only in the most distant row of piles in the direction of loading. Forces on all other rows of piles shall either be compressive or not exceed the nominal tension capacity of the piles.

If the plunging capacity of the leading pile is exceeded or if uplift of other than the trailing rows of piles occurs, special studies shall be conducted to show that performance of the pile system is acceptable. These studies shall be performed only with the prior consent of the Owner and SDAP E is required.

Structural elements of pile foundations shall be designed using the overstrength moment capacity of the column or the elastic design force within the column (Article 3.10.3.8), whichever is smaller.

The maximum shear force on the pile(s) shall be less than the structural shear capacity of the piles.

10.7.4.2.2 Liquefaction Check for SDR 3

An evaluation of the potential for liquefaction shall be made in accordance with requirements given in Article 3.10.5.1.3 and Appendix 3B of these Specifications. If liquefaction is predicted to occur for the design earthquake, the following additional requirements shall be satisfied:

Liquefaction without Lateral Flow or Spreading

- The pile shall penetrate beyond the bottom of the liquefied layer by at least 3 pile diameters or to a depth that axial and lateral pile capacity are not affected by liquefaction of the overlying layer, whichever is deeper.
- The shear reinforcement in a concrete or pre-stressed concrete pile shall meet the requirements of Sec 5.10.11.4.1c from the pile or bent cap to a depth of 3 diameters below the lowest liquefiable layer.
- Effects of downdrag on the pile settlements

COMMENTARY

C10.7.4.2.1

Unfactored resistance and uplift are permitted for the foundation design for two reasons: (1) the design seismic load is likely to be small, and (2) the peak load will occur for only a short duration. By allowing uplift in only the most distant row of piles, the remaining piles will be in compression. Normally piles designed for the Strength Limit State will have a capacity reserve of 2.0 or more, resulting in adequate capacity for vertical loads. The moment capacity check determines whether adequate capacity exists in rotation. If rotational capacities are not satisfied, longer piles or additional piles may be required to meet seismic requirements.

C10.7.4.2.2

The design of a pile foundation for a liquefied soil condition involves careful consideration on the part of the Designer. Two general cases occur: liquefaction with and without lateral flow and spreading.

Liquefaction without Lateral Flow or Spreading

Pile foundations should be designed to extend below the maximum depth of liquefaction by at least 3 pile diameters or to a depth that axial and lateral capacity are not affected by liquefaction of the overlying layer. Porewater pressures in a liquefied zone can result in increases in porewater within layers below the liquefied zone. Porewater pressures increases can also occur in a zone where the factor of safety for liquefaction is greater than 1.0, as discussed in Appendix 3B. These increases in porewater pressures will temporarily reduce the strength of the material from its pre-earthquake (static) strength. The potential for this decrease should be evaluated, and the capacity of the foundation evaluated for the lower strength. Alternatively, the toe of the pile should be founded at a

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shall be determined in accordance with procedures given in Appendix 3B.

Liquefaction with Lateral Flow or Lateral Spreading

- Design the piles to resist the forces generated by the lateral spreading.
- If the forces cannot be resisted, assess whether the structure is able to tolerate the anticipated movements and meet the geometric and structural constraints of Table 3.10.1-2. The maximum plastic rotation of the piles shall be as defined in Article 5.16 and Article 6.15.6.
- If the structure cannot meet the performance requirements of Table 3.10.1-1, assess the costs and benefits of various mitigation measures to reduce the movements to a tolerable level to meet the desired performance objective. If a higher performance is desired so that the piles will not have to be replaced, the allowable plastic rotations of Article 5.16.3 shall be met.

COMMENTARY

depth where the effects of porewater pressure changes are small. Normally, the static design of the pile will include a resistance factor of 0.6 or less. This reserve capacity allows an increase in porewater pressures by 20 percent without significant downward movement of the pile.

As porewater pressures dissipate following liquefaction, drag loads will develop on the side of the pile. The drag loads occur between the pile cap and the bottom of the liquefied layer. The side friction used to compute drag loads will increase with dissipation in porewater pressure from the residual strength of the liquefied sand to a value approaching the static strength of the sand. The maximum drag occurs when the porewater pressures are close to being dissipated. Simultaneously relative movement between the pile and the soil decrease as the porewater pressure decreases, resulting in the drag load evaluation being a relatively complex soil-pile interaction problem. For simplicity, it can be conservatively assumed that the drag load used in the settlement estimate is determined by the pre-liquefied side resistance along the side of the pile between the bottom of the pile cap and the bottom of the liquefied zone.

Liquefaction with Lateral Flow or Spreading

Lateral flow and spreading have been common occurrences during liquefaction at bridge sites involving an approach fill or at a river or stream crossing. The amount of movement can range from a few millimeters to over a meter. This amount of movement is generally sufficient to develop full passive pressures on pile or pile cap surfaces exposed to the moving soil. If the pile-pile cap system is not strong enough to resist these movements, the pile cap system will displace horizontally under the imposed load.

Procedures for estimating either the forces and displacements of the pile from the moving ground are discussed in Appendix 3B. If these forces or displacements are large, some type of ground remediation might be used to reduce these displacements. These ground remediation methods can include vibro densification, stone columns, pressure grouting, or in-place soil mixing. Costs of these improvements can range from \$10/m³ to in excess of \$40/m³ (in 2000 dollars). Depending on the specific conditions and design requirements for a site, the use of ground improvement could increase construction costs by 10 percent or more. In view of these costs, the Owner needs to be made aware of the potential risks and the costs of remediation methods as soon as these conditions are identified.

Appendix 3B provides a more detailed discussion of the process to follow when designing for lateral flow or spreading ground.

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10.7.4.3 SDR 4, 5, & 6

The design of driven pile foundations subject to SDR 4, 5, & 6 shall be based column loads determined by capacity design principles (Article 3.10.3.8) or elastic seismic forces, whichever is smaller. Both the structural and geotechnical elements of the foundation shall be designed for the capacity design forces of Article 3.10.3.8.

Foundation flexibility (Article 4.8.4.4) shall be incorporated into design for Soil Profile Types C, D, and E, if the effects of foundation flexibility contribute more than 20 percent to the displacement of the system. Foundation flexibility will not normally be a consideration for Soil Profile Types A and B, due to the stiffness of the soil or rock. For SDAP E foundations flexibility shall be included in the push-over analysis whenever it is included in the dynamic analysis.

Liquefaction shall be considered during the development of spring constants and capacity values for these seismic design and analysis procedures.

COMMENTARY

C10.7.4.3

Similar to the discussion in Article C10.6.4.3, various approaches are available to evaluate the response of the bridge-foundation system during the design event. In most cases the Designer will use equivalent linear springs to represent the soil-foundation system. Guidance provided in these Specifications focuses on these simple procedures. Comments provided in Article C10.6.4.3 regarding more rigorous modeling methods are equally valid for pile foundation systems.

Most recent research on seismic response of pile-supported foundations has focused on lateral pile loading. Lam et al. (1998) report that many pile-supported foundations are more sensitive to variations in axial pile stiffness, and therefore, the axial pile-load stiffness problem warrants more consideration. Moment demand on a pile group also generally should govern foundation design, which is determined by axial response of the group, rather than lateral loading for most soil conditions.

Characterization of the stiffness of an individual pile or pile group involves an evaluation of the pile load-displacement behavior for both axial and lateral loading conditions. The overall pile-soil stiffness can be estimated in a number of ways, and the method used should reflect the soil characteristics (e.g., type, strength, and nonlinearity) and the structural properties of the pile or pile group (e.g., type, axial and bending stiffness, diameter, length, and structural constraints). If a stiffness matrix is used, it is critical that it be positive-definite and symmetric for it to be suitable for implementation in a global response analyses. This will require p-y curves to be linearized prior to assembly of the stiffness components of the matrix. Such a procedure has been adopted in the charts shown in Article 10.7.4.3.2. If the stiffness matrix is used in a computer program to determine foundation loading demands, then programs such as LPILE or GROUP should be used to determine bending moments and shear forces for design, with nonlinear p-y curves used as appropriate.

The seismic displacement capacity verification step described in Article 4.8.5.4 requires development of moment-rotation and lateral load-displacement relationships. These relationships are normally assumed to be uncoupled because the lateral loads are mobilized in the upper portion of the pile while the axial load is mobilized at relatively deep elevations. For most push-over analyses a secant stiffness can be used to represent soil springs. If design uplift or plunging limits are exceeded, nonlinear springs should be used. In most cases a bi-linear spring will be an acceptable model of the nonlinear behavior of the soil.

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10.7.4.3.1 Axial and Rocking Stiffness for Driven Pile/Pile Cap Foundations (Nonliquefiable Sites)

The axial stiffness of the driven pile foundations shall be determined for design cases in which foundation flexibility is included. For many applications, the axial stiffness of a group of piles can be estimated within sufficient accuracy using the following equation:

:

$$K_{sv} = \Sigma 1.25AE/L$$

where A = cross-sectional area of the pile

E = modulus of elasticity of the piles

L = length of the piles

N = number of piles in group and is represented by the summation symbol in the above equations.

The rocking spring stiffness values about each horizontal pile cap axis can be computed assuming each axial pile spring acts as a discrete Winkler spring. The rotational spring constant (i.e., moment per unit rotation) is then given by

$$K_{srv} = \Sigma k_{vn} S_n^2$$

where k_{vn} = axial stiffness of the nth pile

S_n = distance between the nth pile and the axis of rotation

The effects of group action on the determination of stiffness shall be considered if the center-to-center spacing of piles for the group in the direction of loading is closer than 3 pile diameters.

COMMENTARY

C10.7.4.3.1

Axially loaded piles transfer loads through a combination of end bearing and side resistance along the perimeter of the pile. Their true axial stiffness is a complex nonlinear interaction of the structural properties of the pile and the load-displacement behavior of the soil for friction and end bearing (Lam et al., 1998). Both simplified and more rigorous computer methods are used for evaluating axial stiffness. In most cases simplified methods are sufficient for estimating the axial stiffness of piles. However, at sites where the soil profile changes appreciably with depth or where the effects of group action occur, computer models will often provide a better representation of soil-pile interaction.

Use of Simplified Methods

The axial stiffness value in the simplified equation, $K_{sv} = \Sigma 1.25AE/L$, represents an average value that accounts for uncertainties in the determination of soil properties, the mechanism for developing resistance (i.e., side resistance versus end bearing), and the simplified computational method being used. The basis of this equation is summarized as follows:

- If the pile develops reaction from purely end bearing, the tip bearing stiffness must be relatively large compared with the side resistance stiffness of the soil and the axial stiffness properties of the pile. If the tip displacement is assumed to be zero, the resulting axial stiffness is

$$K_{sv} = \Sigma AE/L$$

- At the other extreme, a purely friction pile implies that the force at the tip is zero. For zero tip force and a uniform total transfer to the soil by side resistance along the pile, the axial stiffness of the pile approximates:

$$K_{sv} = \Sigma 2AE/L$$

- Allowing for some tip displacements and recognizing the inherent complexity of the problem, a reasonable range is from $0.5AE/L$ to $2AE/L$.

Other methods of estimating the axial stiffness of the pile are also available. Lam et al. (1998) present a simplified graphical procedure that uses the average between a rigid and flexible pile solution.

Nonlinear Computer Methods

In the above discussion simplified methods are used to define the axial stiffness of a single pile. More rigorous computer methods that accommodate the nonlinear behavior of the soil and structure are also available. These more rigorous methods involve more effort on the part of the Designer. In many cases the increased accuracy of the more rigorous method is limited by the uncertainty associated with selection of input parameters for the analyses.

A number of computer programs are available for conducting more rigorous determination of the axial stiffness of the soil-pile system (e.g., Lam and Law, 1994). These programs are analogous to the program used to estimate the lateral load-displacement response of piles. Rather than "p-y" curves, they use "t-z" curves and "q-z" curves to represent the side resistance and end bearing load-displacement relationships, respectively.

These same procedures can be used to determine uplift stiffness values. For these determinations the end bearing component of the load-displacement relationship is deleted, and the resistance in uplift is assumed to be the same as that in compression.

Computer programs such as APILE Plus (Reese et al., 1998) provide recommendations for load-transfer relationships in end bearing and side resistance for driven piles. Typical amounts of displacement to mobilize side resistance are on the order of a few millimeters in sands and up to 2 percent of the pile diameter in clay. According to Reese et al. (1998), up to 10 percent of the pile diameter can be required to mobilize the full end bearing of a pile, whether it is in clay or sand. Actual determination of the deformations to mobilize either end bearing or side resistance involves considerable judgment. While the computer programs often make the material property selection and the analysis procedure easy, the uncertainty of the analysis can still be very large. For this reason it is important to involve a person knowledgeable in soil properties and pile loading in the selection of the soil parameters used to model the load-displacement relationship.

The effects of group action for axial loading can be modeled in some computer programs by modification of "t-z" and "q-z" curves. The modifications to these curves will depend on the soil type, with cohesionless soils showing increasing stiffness as the spacing decreases and cohesive soils softening with decreasing spacing. In contrast to lateral loading, explicit relationships for modifying the "t-z" and "q-z" curves are not provided. However, in the limit the adjusted curves should result in an ultimate capacity similar to ultimate capacity of a group determined by static methods (i.e., $Q_g = nQ_s \cdot \eta$ where Q_g is the capacity of the group, n is the number of piles in the group, Q_s is the capacity of

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10.7.4.3.2 Lateral Stiffness Parameters for Driven Pile/Pile Cap Foundations (Nonliquefiable Sites)

The lateral stiffness parameters of driven pile foundations shall be estimated for design cases in which foundation flexibility is included. Lateral response of a pile foundation system depends on the stiffness of the piles and, very often, the stiffness of the pile cap. Procedures for defining the stiffness of the pile component of the foundation system are covered in this article. Methods for introducing the pile cap stiffness are addressed in Article 10.7.4.3.5.

For preliminary analyses involving an estimate of the elastic displacements of the bridge, pile stiffness values can be obtained by using a series of charts prepared by Lam and Martin (1986). These charts are reproduced in Figures 10.7.4.3-1 through 10.7.4.3-6. The charts are applicable for mildly nonlinear response, where the elastic response of the pile dominates the nonlinear soil stiffness.

For push-over analyses the lateral load displacement relationship must be extended into the nonlinear range of response. It is usually necessary to use computer methods to develop the load-displacement relationship in this range, as both the nonlinearity of the pile and the soil must be considered. Programs such as LPILE (Reese et al., 1997), COM 624 (Wang and Reese, 1991), and FLPIER (Hoit and McVay, 1996) are used for this purpose. These programs use nonlinear "p-y" curves to represent the load-displacement response of the soil; they also can accommodate different types of pile-head fixity. Procedures for determining the "p-y" curves are discussed by Lam and Martin (1986) and more recently by Reese et al. (1997).

The effects of group action on lateral stiffness shall be considered if the center-to-center spacing of the piles is closer than 3 pile diameters.

COMMENTARY

an isolated pile, and η is an efficiency factor that will vary with pile spacing and soil type. In the user's manual for GROUP (Reese and Wang, 1996), the authors indicate that the efficiency of pile groups in sands is greater than 1 and by implication the stiffness of a closely spaced group will be greater. They also show that the efficiency of pile groups in clays is less than 1, with the implication that the stiffness of a closely spaced group will be lower.

C10.7.4.3.2

As with axial stiffness, a variety of methods are available for determining the lateral stiffness of a pile or group of piles. Generally, these methods involve the use of simplified charts or the use of more rigorous computer models. The simplified methods normally provide a convenient method for initial design of a pile-supported bridge and may be sufficient for final design if earthquake loads are small. Computer models allow the user to explicitly account for variations in soil stiffness along the embedded depth of the pile, and to account for the effects of group action and changes in the flexural stiffness of the pile during loading. For these reasons, the computer models are often used in final design, particularly where significant changes in soil profile occur with depth or where earthquake loads are large.

Use of Simplified Linear Charts

The charts developed by Lam and Martin (1986) and presented as Figures 10.7.4.3-1 through 10.7.4.3-6 require that an "f" value be defined for the soil. Lam et al. (1998) recommend that the "f" value be selected at a depth of approximately 5 pile diameters. The charts assume no pile top embedment, but yield reasonable stiffnesses for shallow embedment of no more than 1.5 m. Lateral pile stiffness increases quickly with depth for most piles, and therefore, if greater embedment occurs, nonlinear computer methods should be used, as the charts will potentially result in a considerable underestimation of stiffness.

These charts are applicable for pile-head deflections between 5 and 50 mm. The charts also assume that the piles are sufficiently long to achieve full fixity.

Use of Computer Methods

Procedures for conducting nonlinear lateral pile analyses are described by Lam and Martin (1986). Lam and Martin's discussion includes procedures for developing "p-y" curves in both sands and clays. Reese et al. (1997), as well as a number of other technical

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papers, also discuss the development of "p-y" curves.

A number of these methods identify a factor for cyclic loading. Generally, this factor is not applicable to seismic loading conditions. It was developed for problems involving wave loading to offshore structures, where thousands of cycles of load were being applied. For earthquake problems, the non-cyclic "p-y" curves are most applicable.

Group interaction should usually be considered in the evaluation of lateral response of closely spaced piles. Interaction results when the lateral stress developed during loading of one pile interacts with the adjacent pile. Group reduction curves are usually used to represent this interaction. Early studies suggested significant reduction in stiffness for pile spacings of 8 diameters or less. More recent studies indicate that the group effects are not normally as significant as once thought. A reduction factor of 50 percent is recommended by Lam et al. (1998) as being appropriate for most seismic loading situations. According to Lam et al. (1998), this reduction accounts for the effects of gapping, local porewater pressure effects, and the interaction of the stress field from individual piles. Alternatively, p-multiplier methods suggested by Brown et al. (1988) provide a systematic method of introducing group effects for various pile group configurations.

Another consideration in the use of computer programs is whether a cracked or uncracked section modulus should be used in the representation of concrete piles. This modulus will have a significant influence on the resulting load-deformation response calculation, and therefore requires careful consideration by the person performing the analyses. Programs such as FLPIER and LPILE can explicitly account for the transition from uncracked to cracked section modulus during the loading sequence.

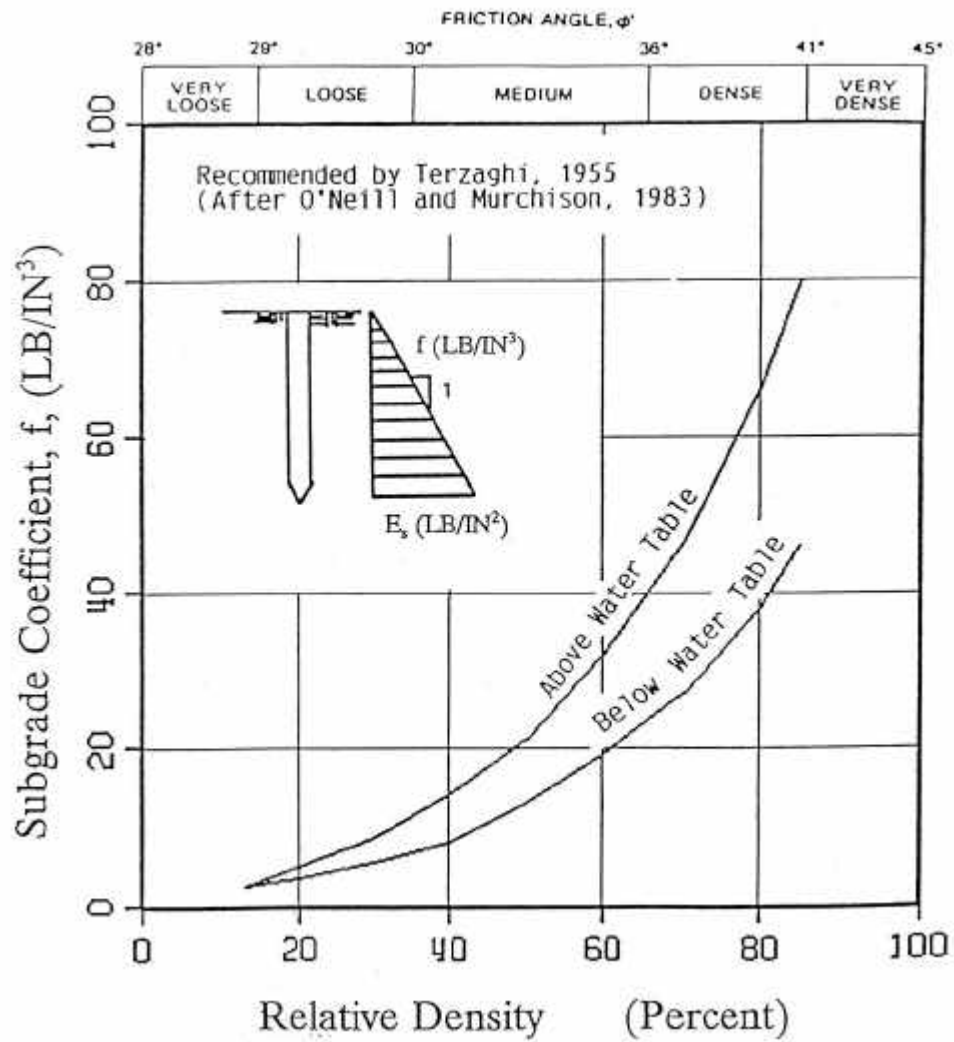


Figure 10.7.4.3-1. Recommendations for Coefficient of Variation in Subgrade Modulus with Depth for Sand (ATC, 1996)

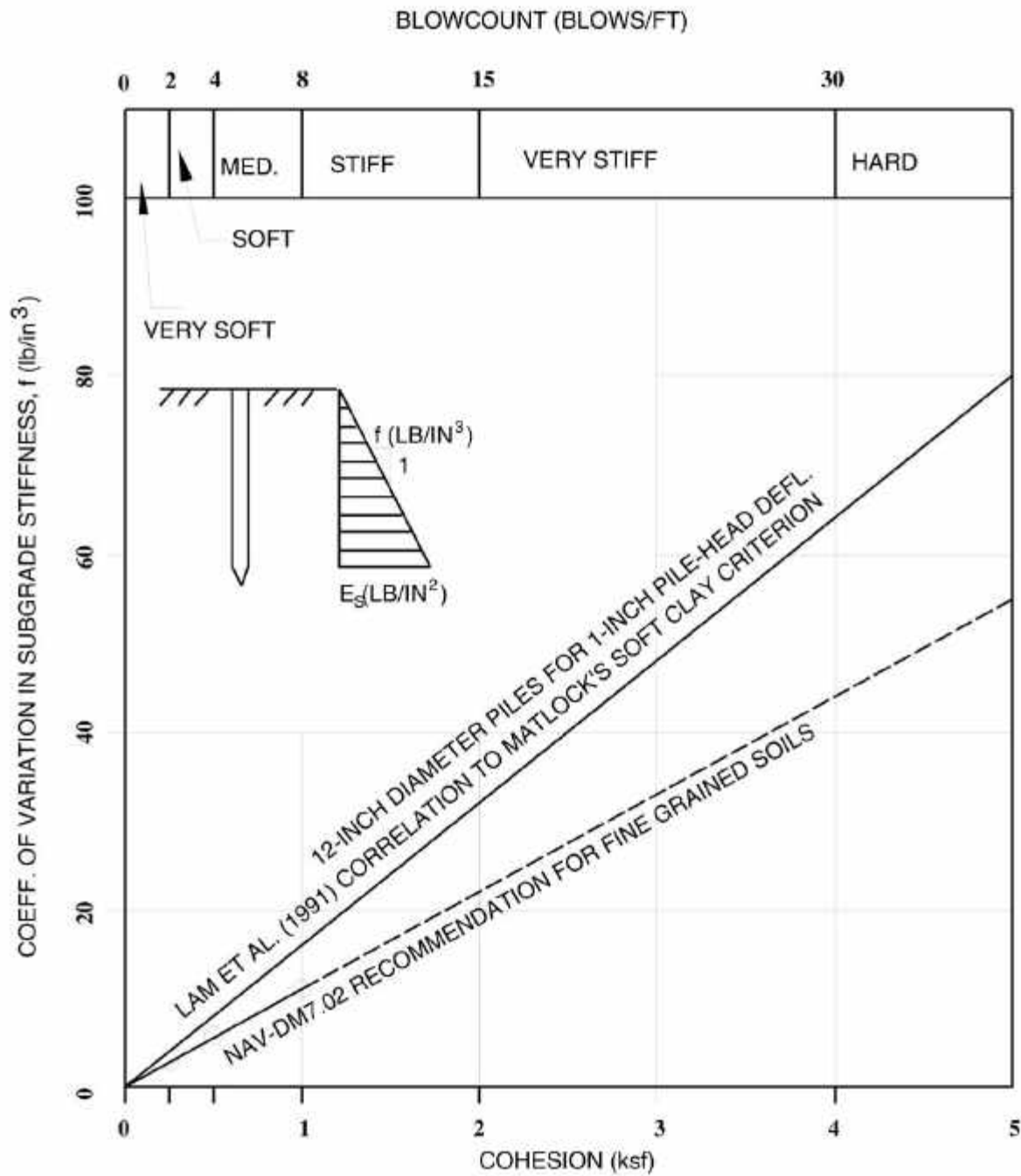


Figure 10.7.4.3-2. Recommendations for Coefficient of Variation in Subgrade Modulus with Depth for Clay (ATC, 1996)

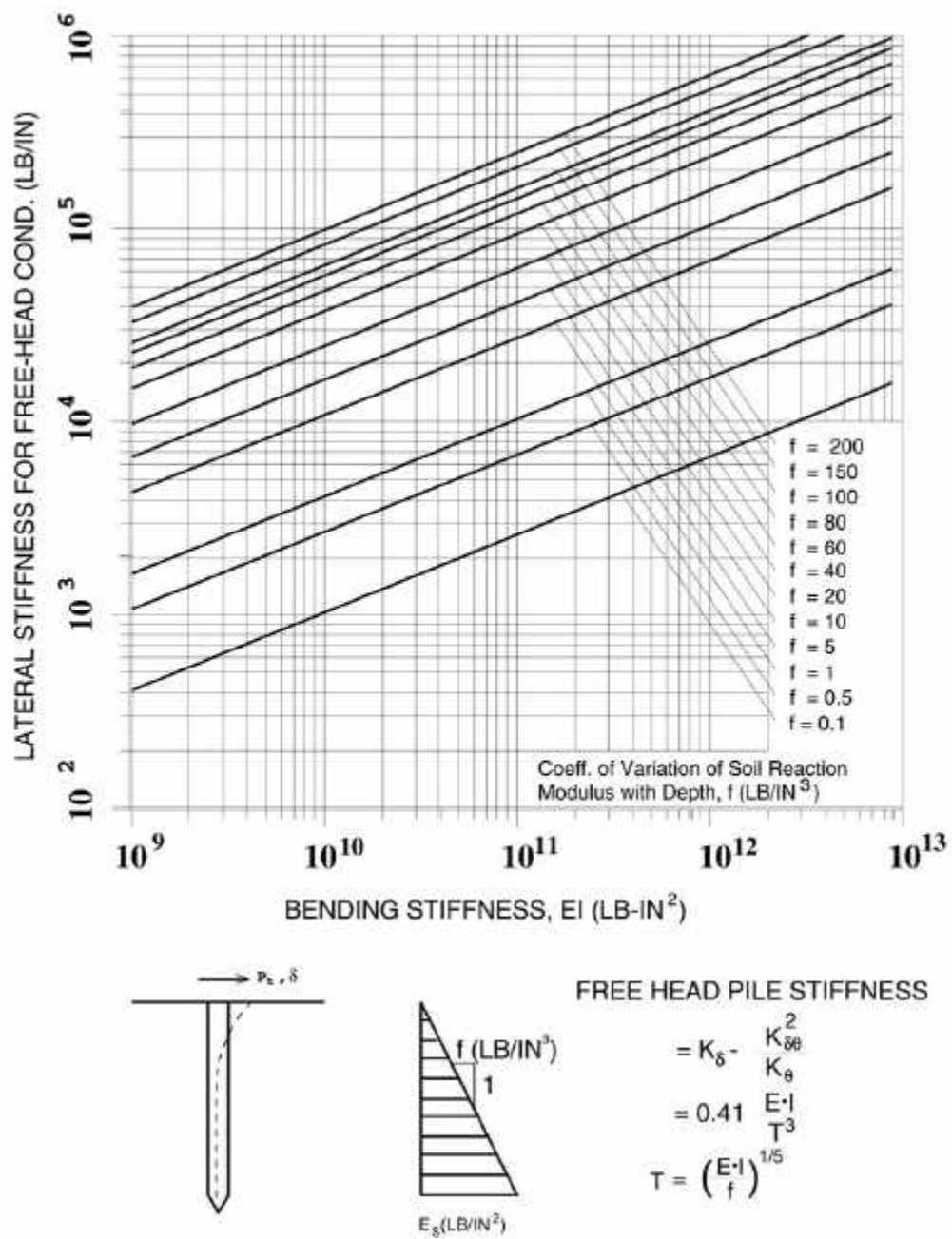


Figure 10.7.4.3-3. Coefficient of Lateral Pile Head Stiffness for Free-Head Pile Lateral Stiffness (ATC, 1996)

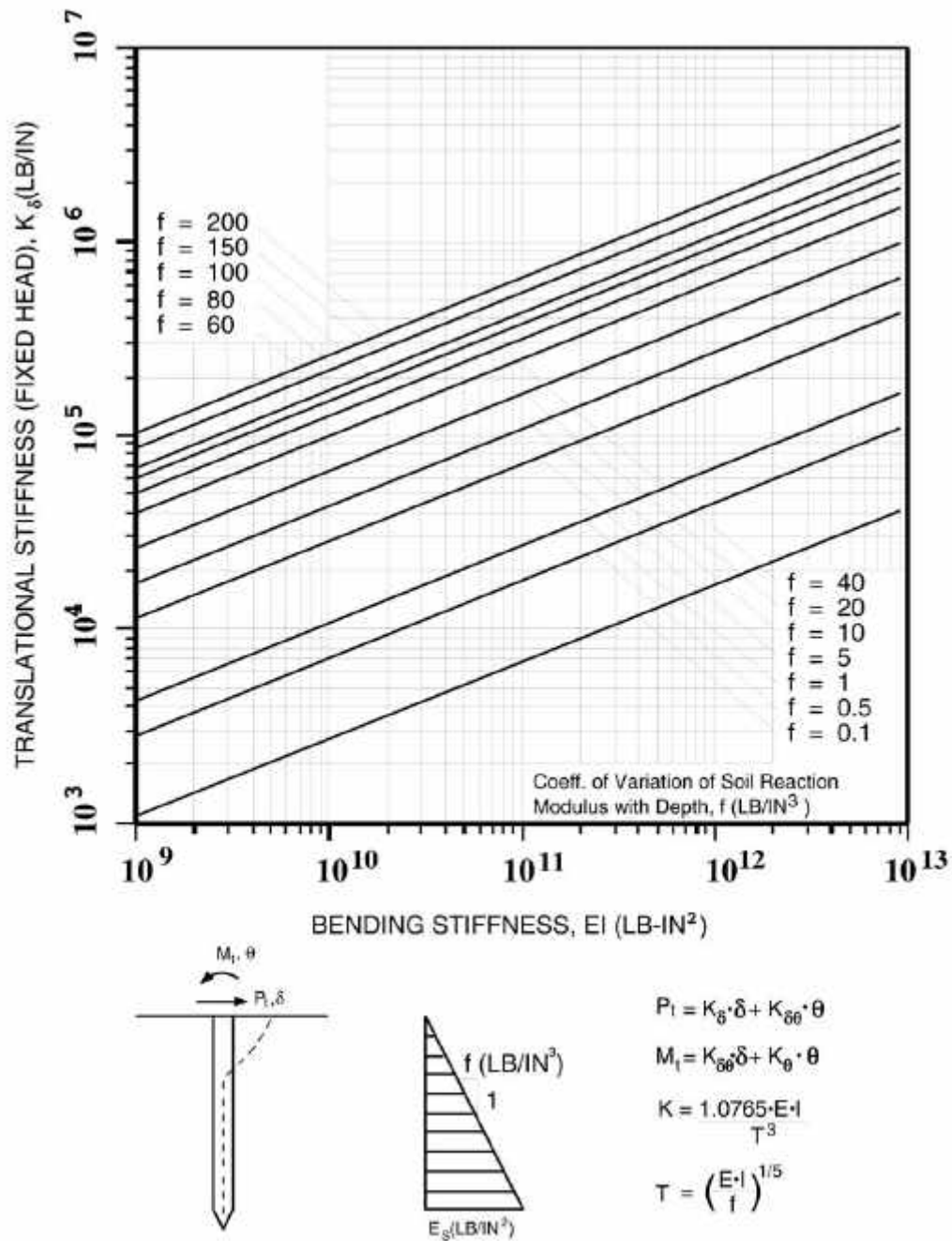


Figure 10.7.4.3-4

Coefficient for Lateral Pile-Head Stiffness for Fixed-Head Pile Lateral Stiffness (ATC, 1996)

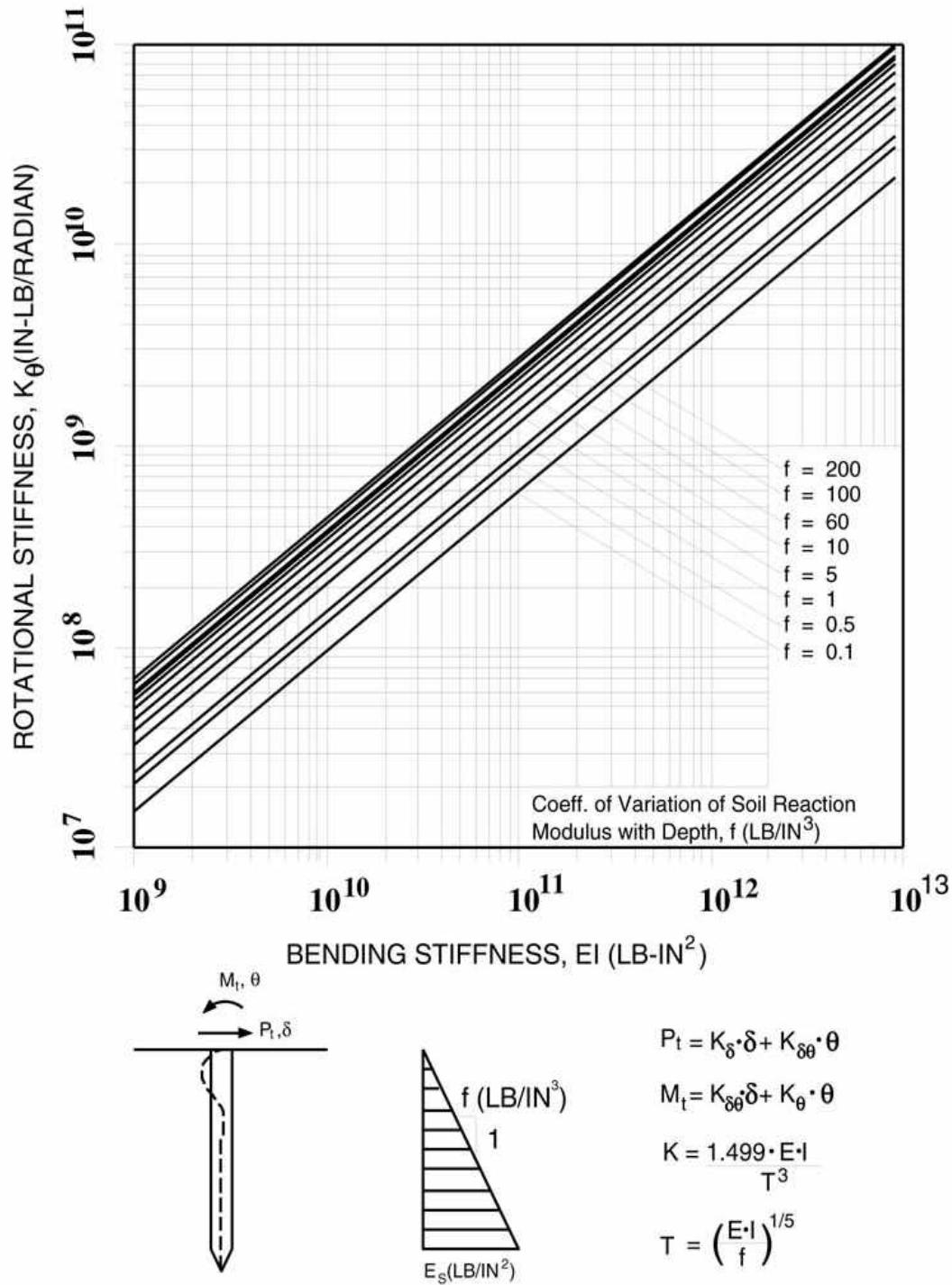


Figure 10.7.4.3-5. Coefficient for Pile Head Rotation (ATC, 1996)

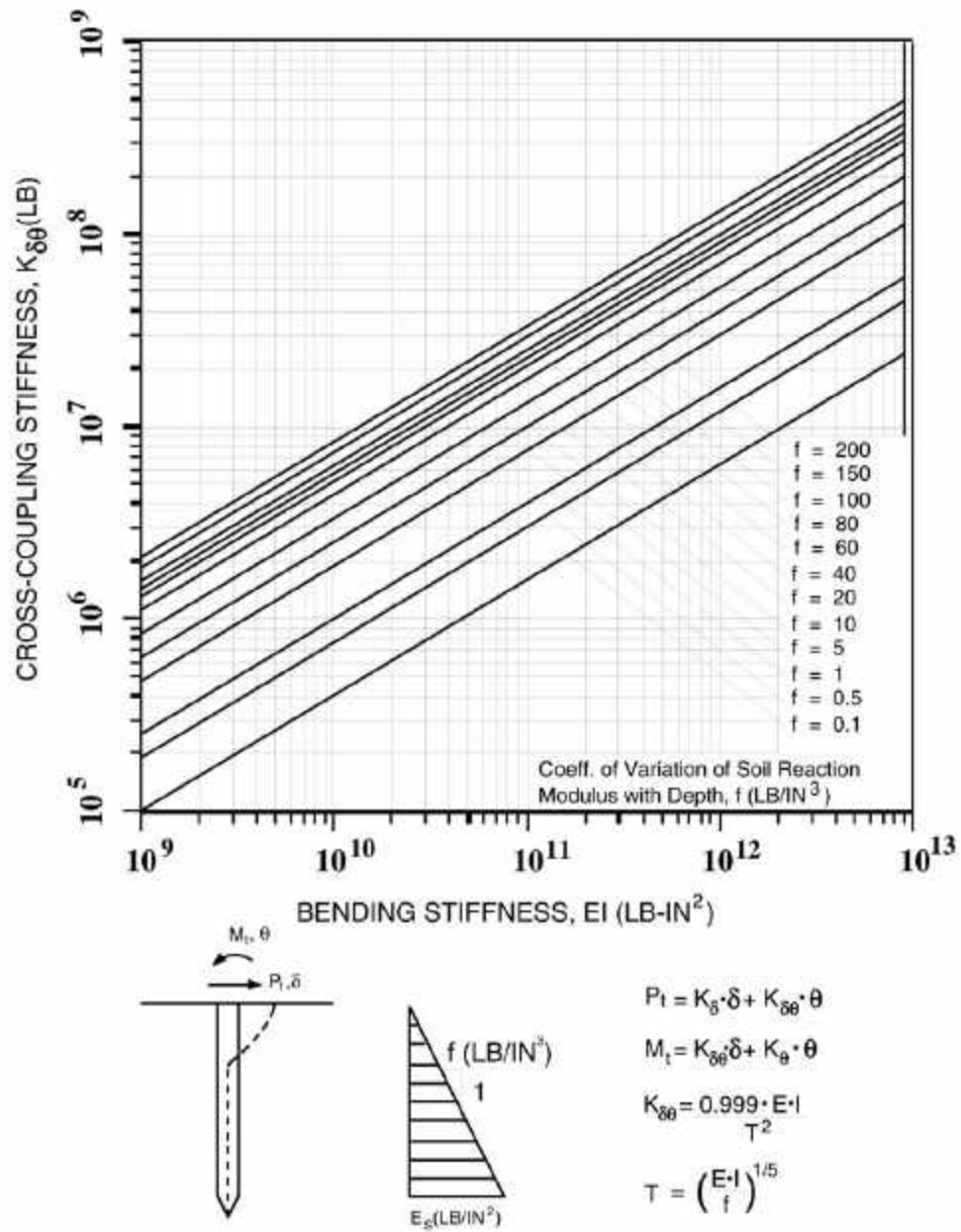


Figure 10.7.4.3-6. Coefficient for Cross-Coupling Stiffness Term (ATC, 1996)

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10.7.4.3.3 Pile Cap Stiffness and Capacity

The stiffness and capacity of the pile cap shall be considered in the design of the pile foundation subject to SDR 4, 5, & 6. The pile cap provides horizontal resistance to the shear loading in the column. Procedures for evaluating the stiffness and the capacity of the footing in shear shall follow procedures given in Article 10.6.4 for spread footings, except that the base shear resistance of the cap shall be neglected.

When considering a system comprised of a pile and pile cap, the stiffness of each shall be considered as two springs in parallel. The composite spring shall be developed by adding the reaction for each spring at equal displacements.

10.7.4.3.4 Moment and Shear Design (Nonliquefiable Sites)

The capacity of the structural elements of driven pile foundations shall be designed to resist the capacity design forces of Article 3.10.3.8 or the elastic design force within the column, whichever is smaller. Unfactored resistance ($\phi = 1.0$) shall be used in performing the geotechnical capacity check. The leading row piles during overturning shall not exceed the plunging capacity of the piles. Separation between the pile tip and the soil (i.e. gapping) shall be allowed only in the most distant row of trailing piles. Forces on all other rows of piles shall either be compressive or not exceed the nominal tension capacity of the piles. The maximum shear force on the pile(s) shall be less than the structural shear capacity of the piles.

If the plunging capacity is exceeded or gapping of other than the trailing row of piles occurs, special studies shall be conducted to show that performance of the pile system is acceptable. Special studies shall be performed only with the prior consent of the Owner and require SDAP E.

COMMENTARY

C10.7.4.3.3

The response of the pile-supported footing differs in one important respect from a spread footing foundation: resistance at the base of the footing is not included in the response evaluation. The base resistance is neglected to account for likely separation between the base of the foundation and soil, as soil settlement occurs.

As noted in Article 10.6.4.3.2, the pile cap will have to deform by as much as 2 percent of the pile cap thickness to mobilize the passive pressure of the cap. If this displacement is significantly greater than the design displacement, it may be possible to neglect the contribution of the pile cap without significant effects on the total stiffness calculation. At these low displacements, the stiffness of the pile will govern response.

C10.7.4.3.4

The stiffness of the pile in axial loading is limited by the plunging capacity of the pile. Side resistance and end bearing soil springs should be limited by the unfactored axial capacity at large deformations. Similarly, moment capacity checks are normally made with the unfactored axial capacity of the pile.

Resistance factors are not applied to enable the Designer to obtain a better understanding of pile performance under seismic loading. By using unfactored capacities, a best-estimate of the displacement for a given force in the bridge structure can be obtained. If factored capacities are used, the deformation could be greater than under best-estimate conditions, resulting in design decisions that may not be appropriate.

It is recognized that uncertainty exists even with the best-estimate capacity. Although it may not be economical to evaluate these uncertainties in all bridges, uncertainty should be considered during evaluations of stiffness and capacity and should be evaluated for more important bridges. To account for uncertainty, upper and lower bound capacities and stiffnesses can be determined, allowing the Designer to assess the potential effects to design if higher or lower capacities occur for the site.

The range for the upper and lower bound evaluation will depend on the characteristics of the site, the type of analysis used to estimate capacity, and whether or not a field load test is conducted (e.g., PDA, static load test with head measurements only, or fully instrumented pile-load test). Common practice is to use an upper bound that is 100 percent greater than the unfactored stiffness and capacity and a lower bound that is 50 percent of the unfactored stiffness and capacity.

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10.7.4.3.5 Liquefaction and Dynamic Settlement Evaluations

If liquefaction is predicted to occur at the site, effects of liquefaction on the bridge foundation shall be evaluated. This evaluation shall consider the potential for loss in lateral bearing support, flow and lateral spreading of the soil, settlement below the toe of the pile, and settlement from drag loads on the pile as excess porewater pressures in liquefied soil dissipate. Procedures given in Appendix 3B shall be followed when making these evaluations.

If liquefaction causes unacceptable bridge performance, consideration should be given to the use of ground improvement methods to meet design requirements. In light of the potential costs of ground improvement, the Owner shall be consulted before proceeding with a design for ground improvement to review the risks associated with liquefaction relative to the costs for remediating the liquefaction potential.

10.7.5 Structural Design

The connection of the outer row of piles shall be designed for a tension capacity equal to 1.5 times the nominal uplift capacity of the pile unless it can be shown through capacity design principles (Article 3.10.3.8) that the tension force in the pile is less, in which case the lower force may be used. Other rows of piles and their connections shall be designed to resist their calculated tension and compressive forces.

COMMENTARY

The range of uncertainty is normally higher than the uncertainty implied by the resistance factor used for static design for several reasons: (1) there is greater uncertainty in the seismic resistance of the pile in seismic loading than static loading, (2) there is a greater potential for cyclic degradation of resistance properties during seismic loading, and (3) there are rate of loading effects.

The Designer can reduce the range of uncertainty by conducting more detailed site explorations to fully characterize the soil, by performing more rigorous analyses that treat the full load-deformation process, and by conducting pile-load test to quantify the load-displacement response of the pile. Even with a full-scale field load test, some uncertainty exists as discussed in the previous paragraph. For this reason, a range of values to represent upper and lower bound response may be warranted even under the best circumstances.

C10.7.4.3.5

Liquefaction design for SDR 4 and above is similar to that described previously for SDR 3 with one exception. For SDR 4 and above, the change in lateral stiffness of the pile resulting from liquefaction is also determined. This change in stiffness is usually accomplished by defining the liquefied zone as a cohesive soil layer with the ultimate strength in the "p-y curve" being equal to the residual strength of the liquefied soil. Appendix 3B identifies procedures for making these adjustments.

C10.7.5

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10.8 DRILLED SHAFTS

10.8.4 Seismic Design for Extreme Event State

The design of drilled shafts for Extreme Event State - Case I (Seismic Loading) generally follows procedures given Article 10.7.4 for driven piles. Only differences in design approach are noted in this article. As with driven pile foundations, the seismic design requirements for shaft foundations will depend on which seismic category occurs at the site. Generally, there are three classes of analyses: (1) SDR 1 and 2, (2) SDR 3, and (3) SDR 4, 5, and 6. The potential liquefaction in each of these classes must be considered in the same way and for the same reasons as discussed for driven piles.

10.8.4.1 SDR 1 AND 2

Procedures identified in Article 10.7.4.1 apply.

10.8.4.2 SDR 3

Procedures identified in Article 10.7.4.2, including those for liquefaction and dynamic settlement, shall be applied with the exception that the ultimate capacity in compression or uplift loading for single shaft foundations in SDR 3 shall not be exceeded during maximum seismic loading without special design studies and the Owner's approval. The flexibility of the drilled shaft shall also be represented in the design using either the estimated depth of fixity or soil springs in a lateral pile analysis.

Diameter adjustments shall be considered during lateral load analyses of shafts with a diameter greater than 600 mm if the shaft is free to rotate, as in the case of a column extension (i.e., no pile cap). Contributions from base shear shall also be considered.

C10.8.4

Lam et al. (1998) provide a detailed discussion of the seismic response and design of drilled shaft foundations. Their discussion includes a summary of procedures to determine the stiffness matrix required to represent the shaft foundation in most dynamic analyses.

Drilled shaft foundations will often involve a single shaft, rather than a group of shafts, as in the case of driven piles. In this configuration the relative importance of axial and lateral response change. Without the pile cap, lateral-load displacement of the shaft becomes more critical than the axial-load displacement relationships discussed for driven piles.

C10.8.4.2

Many drilled shaft foundation systems consist of a single shaft supporting a column. Compressive and uplift loads on these shafts during seismic loading will normally be within limits of load factors used for gravity loading. However, checks should be performed to confirm that any changes in axial load don't exceed ultimate capacities in uplift or compression. In contrast to driven piles in a group, no reserve capacity exists for a single shaft; i.e., if ultimate capacity is exceeded, large deformations can occur.

Special design studies can be performed to demonstrate that deformations are within acceptable limits if axial loads approach or exceed the ultimate uplift or compressive capacities if the drilled shaft is part of a group. These studies can be conducted using computer programs, such as APILE Plus (Reese, et al., 1997). Such studies generally will require rigorous soil-structure interaction modeling.

Various studies (Lam et al., 1998) have found that conventional p-y stiffnesses derived for driven piles are too soft for drilled shafts. This softer response is attributed to a combination of (1) higher unit side friction, (2) base shear at the bottom of the shaft, and (3) the rotation of the shaft. The rotation effect is often implicitly included in the interpretation of lateral load tests, as most lateral load tests are conducted in a free-head condition. A scaling factor equal to the ratio of shaft diameter to 600 mm is generally applicable, according to Lam et al. (1998). The scaling factor is applied to either the linear subgrade modulus or the resistance value in the p-y curves. This adjustment is thought to be somewhat dependent on the construction method.

Base shear can also provide significant resistance

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10.8.4.3 SDR 4, 5, & 6

Procedures identified in Article 10.7.4.3, including liquefaction and dynamic settlement, generally apply with the exceptions that, as in SDR 3, (1) the ultimate capacity of single shaft foundations in compression and uplift shall not be exceeded under maximum seismic loads and (2) the flexibility of the drilled shaft shall be represented using either the estimated depth of fixity or soil springs in a lateral pile analysis.

Checks shall be conducted to confirm that minimum shaft lengths occur. The stable length can be determined by conducting nonlinear computer modeling or by using a length $(L) > \pi\lambda$ where $\lambda = [EI_p/E_s]^{0.25}$ and EI_p and E_s are the bending stiffness of the pile and the subgrade modulus of the soil, respectively.

The nonlinear properties of the shaft shall be considered in evaluating the lateral response of the pile to lateral loads during a seismic event. Diameter adjustments shall be considered during lateral analyses of shafts with a diameter greater than 600 mm if the shaft is free to rotate, as in the case of a column extension (i.e., no pile cap). Contributions from base shear shall also be considered

10.8.4.4 OTHER DESIGN AND CONSTRUCTION PRACTICES TO IMPROVE SEISMIC PERFORMANCE.

In view of the uncertainties associated with seismic response and the dependence of response on the method of construction, the geotechnical engineer and structural engineer shall meet with the Owner at the start of design to discuss design and construction issues and to determine whether the state of understanding, the type of soils, or the method of construction warrant methods different than identified in Article 10.8.4 and discussed in Article C10.8.4.

to lateral loading for large diameter shafts. The amount of resistance developed in shear will be determined by conditions at the base of the shaft during construction. For dry conditions where the native soil is relatively undisturbed, the contributions for base shear can be significant. However, in many cases the base conditions result in low interface strengths. For this reason the amount of base shear to incorporate in a lateral analyses will vary from case-to-case.

C10.8.4.3

Typically it is necessary to embed shafts to between 2 diameters in rock to 3 and 5 shaft diameters in soil to achieve stable conditions. The depth for stable conditions will depend on the stiffness of the rock or soil. Lower stable lengths are acceptable if the embedment length and the strength of the drilled shaft provide sufficient lateral stiffness with adequate allowances for uncertainties in soil stiffness. Generally, it will be necessary to conduct a lateral load analysis using a program such as COM624 or LPILE to demonstrate that lower stable lengths are acceptable.

Section properties of the drilled shaft should be consistent with the deformation caused by the seismic loading. In many cases it is necessary to use the cracked section modulus in the evaluation of lateral load-displacement relationships. In the absence of detailed information regarding reinforcing steel and applied load, an equivalent cracked section can be estimated by reducing the stiffness of the uncracked section by half. In general the cracked section is a function of the reinforcement ratio (i.e., volume of steel reinforcement versus that of concrete), but is often adequate to assume as one-half of the uncracked section.

C10.8.4.4

The behavior of a drilled shaft is very dependent on the construction methods used by the contractor. In recognition of this dependence, most Owner's have detailed requirements for construction and construction inspection. The Designer should review these requirements and determine which if any could affect assumptions regarding shaft frictional capacity and end resistance, and particularly the deformation required to mobilize these capacities.

The depth of maximum moment for a drilled shaft will often be located 2 to 3 diameters below the ground surface for loose and dense soils, respectively (Lam et al., 1998) if the shaft diameter and stiffness are the same as the column diameter. For a cracked section with flexural rigidity equal to one-half that of the uncracked section, the depth of maximum moment is shallower, being approximately 1.5 to 2 diameters

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below the ground surface, respectively.

Since damage following an earthquake cannot be easily evaluated if the hinge forms below the ground surface, it is generally good practice to assure that the hinge occurs above the ground surface. This can be accomplished by sizing the shaft diameter or stiffness to be greater than the column diameter or by creating a fuse at the ground surface. It is critical that the structural engineer gives specific consideration to this requirement.

Design should also consider the shear capacity of the shaft. Generally the smallest length that results in the highest possible design shear forces should be used. Particular attention should be placed on the possibility of near-ground restraint from the presence of pavement, road barriers, or a stiff desiccated soil crust. Instances of failure have occurred from assuming overly deep plastic hinge locations when constraints at the surface were ignored.

10.8.5 Structural Design

The connection of the drilled shaft to the deck shall be designed for the capacity design forces of Article 3.10.3.8, including tension if it occurs. If the tensile demand due to overturning exceeds the nominal tension capacity provided by the soil, then the tension connection shall be designed for 1.5 times the nominal tensile soil capacity of the shaft unless an analysis that takes the overstrength of the pier into account and assumes no relief in tensile capacity due to geotechnical failure of the piles, shows that lower pile connection forces are assured.

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11.1 SCOPE

This section provides requirements for design of abutments and walls. Conventional retaining walls, non-gravity cantilevered walls, anchored walls, mechanically stabilized earth (MSE) walls and prefabricated modular walls are considered.

11.2 DEFINITIONS

Abutment - A structure that supports the end of a bridge span, and provides lateral support for fill material on which the roadway rests immediately adjacent to the bridge.

Anchored Wall - An earth retaining system typically composed of the same elements as non-gravity cantilevered walls, and which derive additional lateral resistance from one or more tiers of anchors.

Mechanically Stabilized Earth Wall - A soil retaining system, employing either strip or grid-type, metallic or polymeric tensile reinforcements in the soil mass, and a facing element which is either vertical or nearly vertical.

Non-Gravity Cantilever Wall - A soil retaining system which derives lateral resistance through embedment of vertical wall elements and support retained soil with facing elements. Vertical wall elements may consist of discrete elements, e.g., piles, drilled shafts or auger-cast piles spanned by a structural facing, e.g., lagging, panels or shotcrete. Alternatively, the vertical wall elements and facing may be continuous, e.g., sheet piles, diaphragm wall panels, tangent piles or tangent drilled shafts.

Pier - That part of a bridge structure between the superstructure and the connection with the foundation.

Prefabricated Modular Wall - A soil retaining system employing interlocking soil-filled timber, reinforced concrete or steel modules or bins to resist earth pressures by acting as gravity retaining walls.

Rigid Gravity and Semi-Gravity (Conventional) Retaining Wall - A structure that provides lateral support for a mass of soil and that owes its stability primarily to its own weight and to the weight of any soil located directly above its base.

In practice, different types of rigid gravity and semi-gravity retaining walls may be used. These include:

- A **gravity** wall depends entirely on the weight of the stone or concrete masonry and of any soil resting on the masonry for its stability. Only a nominal amount of steel is placed near the exposed faces to prevent surface cracking due to temperature changes.
- A **semi-gravity** wall is somewhat more slender than a gravity wall and requires reinforcement consisting of vertical bars along the inner face and dowels continuing into the footing. It is provided with temperature steel near the exposed face.
- A **cantilever** wall consists of a concrete stem and a concrete base slab, both of which are relatively thin and fully reinforced to resist the moments and shears to which they are subjected.
- A **counterfort** wall consists of a thin concrete face slab, usually vertical, supported at intervals on the inner side by vertical slabs or counterforts that meet the face slab at right angles. Both the face slab and the counterforts are connected to a base slab, and the space above the base slab and between the counterforts is backfilled with soil. All the slabs are fully reinforced.

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11.3 NOTATION

A	=	maximum earthquake acceleration (dim) (C11.8.6)
A _c	=	cross-sectional area of reinforcement unit (mm ²) (11.10.6.4)
A _m	=	maximum wall acceleration coefficient at the centroid (11.10.7.1)
B	=	wall base width (mm) (11.10.2)
b	=	unit width of reinforcement; width of bin module (mm) (11.10.6.4) (11.11.5.1)
b _f	=	width of applied footing load (mm) (11.10.11.1)
C	=	overall reinforcement surface area geometry factor (dim) (11.10.6.3.2)
CR _s	=	reduction factor to account for reduced strength due to connection pullout (dim) (11.10.6.4.3b)
CR _u	=	reduction factor to account for reduced ultimate strength resulting from connection (dim) (11.10.6.4.3b)
C _u	=	coefficient of uniformity (dim) (11.10.6.3.2)
C _o	=	uniaxial compressive strength of rock (MPa) (11.5.6)
D	=	diameter of bar or wire (mm) (11.10.6.3.2)
D*	=	diameter of bar or wire corrected for corrosion loss (mm) (11.10.6.4)
D _o	=	embedment for which net passive pressure is sufficient to provide moment equilibrium (mm) (C11.8.4.2)
D _{eff}	=	effective gap width at abutment after passive soil resistance is mobilized (m) (Fig 11.6.5.2)
D _g	=	gap width at abutment (m) (11.6.5.1.1b)
D ₆₀ /D ₁₀	=	uniformity coefficient of soil defined as ratio of the particle size of soil which is 60% finer in size to the particle size of soil which is 10% finer in size (dim) (11.10.6.3.2)
d	=	diameter of anchor drill hole; fill above wall (mm) (11.9.4.2)(11.10.8)
E _c	=	thickness of metal reinforcement at end of service life (mm) (11.10.6.4)
E _n	=	nominal thickness of steel reinforcement at construction (mm) (11.10.6.4.1a)
E _s	=	sacrificial thickness of metal expected to be lost by uniform corrosion during service life (mm) (11.10.6.4.1a)
E _{AE}	=	total active static and seismic force (N/mm) (A11.1.1.1)
E _{PE}	=	total passive static and seismic force (N/mm) (A11.1.1.1)
e	=	eccentricity of load from centerline of foundation (mm) (11.10.8)
F _y	=	minimum yield strength of steel (MPa) (11.10.6.4.2a)
F*	=	reinforcement pullout friction factor (dim) (11.10.6.3.2)
G _u	=	distance from center of gravity of a horizontal segmental facing block unit, including aggregate fill, measured from the front of the unit (mm) (11.10.6.4.3b)
H	=	height of wall (mm) (11.9.1), height of abutment wall (m) (11.6.5.1.1b)
H _h	=	hinge height for segmental facing (mm) (11.10.6.4.3b)
H _u	=	segmental facing block unit height (mm) (11.10.6.4.3b)
H ₁	=	equivalent wall height (mm) (11.10.6.3.1)
h _i	=	height of reinforced soil zone contributing horizontal load to reinforcement at level i (mm) (11.10.6.2.1)
I _s	=	point load strength index (MPa) (11.5.6)
k _a	=	active earth pressure coefficient (dim) (11.8.4.2)
K _{af}	=	active earth pressure coefficient of backfill (dim) (11.10.5.2)
K _{eff1}	=	effective initial stiffness of abutment backwall and soil including the initial gap (kN/m) (Fig 11.6.5.2)
K _{eff2}	=	secant stiffness of abutment backwall and soil at maximum EQ displacement (kN/m) (Fig 11.6.5.2)k _h = horizontal seismic acceleration coefficient (dim) (11.8.6)
K _i	=	initial stiffness of abutment backfill based on soil resistance alone (kN/m) (Fig 11.6.5.2)
k _v	=	vertical seismic acceleration coefficient (dim) (A11.1.1.1)
k _{AE}	=	seismic active pressure coefficient (dim) (A11.1.1.1)
k _{PE}	=	seismic passive pressure coefficient (dim) (A11.1.1.1)
k _r	=	horizontal earth pressure coefficient of reinforced fill (dim) (11.10.5.2.1)
L	=	spacing between vertical elements or facing supports (mm); length of retaining wall foundation (mm) (11.8.5.2) (11.10.2)
L _a	=	length of reinforcement in active zone (mm) (11.10.2)
L _b	=	anchor bond length (mm) (11.9.4.2)
L _e	=	length of reinforcement in resistance zone (mm) (11.10.2)
L _{ei}	=	effective reinforcement length for layer i (mm) (11.10.7.2)
l _b	=	slope of facing base downward into backfill (deg) (11.10.6.4.3b)
MARV	=	minimum average roll value (11.10.6.4.2b)

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$M_{\max}$	=	maximum bending moment in vertical wall element or facing (N·mm or N·mm/mm) (11.8.5.2)
N	=	normal component of resultant on base of foundation (N/mm) (11.6.3.2)
P_{AE}	=	dynamic horizontal thrust (N/mm) (11.10.7.1)
P_b	=	pressure inside bin module (MPa) (11.10.5.1)
P_H	=	lateral force due to superstructure or other concentrated loads (N/mm) (11.10.11.1)
P_i	=	factored horizontal force per mm of wall transferred to soil reinforcement at level i ; internal inertial force, due to the weight of the backfill within the active zone (N/mm) (11.10.6.2.1) (11.10.7.2)
P_{IR}	=	horizontal inertial force (N/mm) (11.10.7.1)
P_{ir}	=	horizontal inertial force caused by acceleration of reinforced backfill (N/mm) (11.10.7.1)
P_{is}	=	internal inertial force caused by acceleration of sloping surcharge (N/mm) (11.10.7.1)
P_v	=	load on strip footing (N/mm) (11.10.11.1)
p_p	=	passive pressure acting against the abutment backwall under EQ loading (MPa) (11.6.5.1.1b)
P_p	=	passive force acting against abutment backwall under EQ loading kN (11.6.5.1.1b)
$P_{?v}$	=	load on isolated rectangular footing or point load (N) (11.10.11.1)
p	=	average lateral pressure, including earth, surcharge and water pressure, acting on the section of wall element being considered (MPa) (11.9.5.2)
Q_a	=	nominal (ultimate) anchor resistance (N) (11.9.4.2)
q	=	surcharge pressure (MPa) (11.10.5.2)
$q_{\max}$	=	maximum unit soil pressure on base of foundation (MPa) (11.6.3.2)
R_c	=	reinforcement coverage ratio (dim) (11.10.6.3.2)
R_n	=	nominal resistance (N or N/mm) (11.5.4)
R_R	=	factored resistance (N or N/mm) (11.5.4)
RF	=	combined strength reduction factor to account for potential long-term degradation due to installation damage, creep and chemical/biological aging of geosynthetic reinforcements (dim) (11.10.6.4.1b)
RF_c	=	combined strength reduction factor for long-term degradation of geosynthetic reinforcement facing connection (dim) (11.10.6.4.3b)
RF_{CR}	=	strength reduction factor to prevent long-term creep rupture of reinforcement (dim) (11.10.6.4.2b)
RF_D	=	strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation (dim) (11.10.6.4.2b)
RF_{ID}	=	strength reduction factor to account for installation damage to reinforcement (dim) (11.10.6.4.2b)
S_h	=	horizontal reinforcement spacing (mm) (11.10.6.4)
S_t	=	spacing between transverse grid elements (mm) (11.10.6.3.2)
S_u	=	undrained shear strength (MPa) (11.9.5.2)
S_v	=	vertical spacing of reinforcements (mm) (11.10.6.2.3)
S_{rs}	=	ultimate reinforcement tensile resistance required to resist static load component (N/mm) (11.10.7.2)
S_{rt}	=	ultimate reinforcement tensile resistance required to resist transient load component (N/mm) (11.10.7.2)
T_{al}	=	nominal long-term reinforcement design strength (N/mm) (11.10.6.4)
T_c	=	nominal long-term reinforcement/facing connection design strength (N/mm) (11.10.6.4)
T_{lot}	=	ultimate wide width tensile strength for the reinforcement material lot used for the connection strength testing (N/mm) (11.10.6.4.3b)
T_{md}	=	factored incremental dynamic inertia force (N/mm) (11.10.7.2)
T_{sc}	=	peak load per unit of reinforcement width in the connection test at a specified confining pressure where pullout is known to be the mode of failure (N/mm) (11.10.6.4.3b)
T_{ultc}	=	peak load per unit reinforcement width in the connection test at a specified confining pressure where rupture of the reinforcement is known to be the mode of failure (N/mm) (11.10.6.4.3b)
T_{ult}	=	ultimate tensile strength of reinforcement (N/mm) (11.10.6.4.2b)
$T_{\max}$	=	applied load to reinforcement (N/mm) (11.10.6.2.1)
T_o	=	factored tensile load at reinforcement/facing connection (N/mm) (11.10.6.2.2)
t	=	thickness of transverse elements (mm) (11.10.6.3.2)
T_{total}	=	total load on reinforcement layer (static & dynamic) per unit width of wall (N/mm) (11.10.7.2)
W_u	=	unit width of segmental facing (mm) (11.10.2.3.2)
x	=	spacing between vertical element supports (mm) (11.9.5.2)
Z	=	depth below effective top of wall or to reinforcement (mm) (11.10.6.2.1)
Z_p	=	depth of soil at reinforcement layer at beginning of resistance zone for pullout calculation (mm) (11.10.6.2.1)
a	=	scale effect correction factor (dim) (11.10.6.3.2)
β	=	inclination of ground slope behind face of wall (DEG) (11.5.5)

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γ_{EQ}	=	load factor for earthquake loading in Section 3.4.1 (dim) (11.6.5)
γ_P	=	load factor for earth pressure in Section 3.4.1 (dim) (11.10.6.2.1)
γ_s	=	soil density (kg/m^3)
γ'_s	=	effective soil density (kg/m^3) (C11.8.4.2)
γ_r	=	density of reinforced fill (kg/m^3) (11.10.5.2)
γ_f	=	density of backfill (kg/m^3) (11.10.5.2)
δ	=	wall-backfill interface friction angle (DEG) (11.5.5)
σ_{Hh}	=	horizontal stress on reinforcement from concentrated horizontal surcharge (MPa) (11.10.6.2.1)
σ_{sh}	=	traffic barrier impact stress applied over reinforcement tributary area (N/mm) (11.10.11.2)
σ_{sv}	=	vertical stress due to footing load (MPa) (11.10.8)
d_{max}	=	maximum displacement (mm) (11.10.4.2)
d_r	=	relative displacement coefficient (11.10.4.2)
β	=	wall batter from horizontal (DEG) (11.10.6.2.1)
α	=	soil-reinforcement angle of friction (DEG) (11.10.5.3)
f	=	resistance factor (11.5.4)
f_f	=	internal friction angle of foundation or backfill soil (DEG) (11.10.2)
f_r	=	internal friction angle of reinforced fill (DEG) (11.10.5.2)
f'_f	=	effective internal friction angle of soil (DEG) (11.8.4.2)
σ_H	=	factored horizontal stress at reinforcement level (MPa) (11.10.6.2.1)
σ_{Hmax}	=	maximum stress in soil reinforcement in abutment zones (11.10.8)
σ_v	=	vertical stress in soil (MPa) (11.10.6.2.1)
σ_{v1}	=	vertical soil stress (MPa) (11.10.8)
t_a	=	nominal anchor bond stress (MPa) (11.9.4.2)
β	=	wall batter due to setback of segmental facing units (deg) (11.10.6.4.3b)

11.5 LIMIT STATES AND RESISTANCE FACTORS

11.5.1 General

Design of abutments, piers and walls shall satisfy the criteria for the service limit state specified in Article 11.5.2, and for the strength limit state specified in Article 11.5.3.

Abutments, piers and retaining walls shall be designed to withstand lateral earth and water pressures, including any live and dead load surcharge, the self weight of the wall, temperature and shrinkage effects, and earthquake loads in accordance with the general principles specified in this section.

Earth retaining structures shall be designed for a service life based on consideration of the potential long-term effects of material deterioration, seepage, stray currents and other potentially deleterious environmental factors on each of the material components comprising the structure. For most applications, permanent retaining walls should be designed for a minimum service life of 75 years. Retaining walls for temporary applications are typically designed for a service life of 36 months or less.

A greater level of safety and/or longer service life (i.e., 100 years) may be appropriate for walls which support bridge abutments, buildings, critical utilities, or other facilities for which the consequences of poor performance or failure would be severe.

The quality of in-service performance is an important consideration in the design of permanent earth retaining structures. Permanent structures shall be designed to retain

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an aesthetically pleasing appearance, and be essentially maintenance free throughout their design service life.

11.5.6 Resistance Factors

Resistance factors for geotechnical design of foundations are specified in Tables 10.5.4-1 through 10.5.4-3 and Table 1.

If methods other than those prescribed in these Specifications are used to estimate resistance, the resistance factors chosen shall provide the same reliability as those given in Tables 10.5.4-1, 10.5.4-3 and Table 1.

Vertical elements, such as soldier piles, tangent piles and slurry trench concrete walls shall be treated as either shallow or deep foundations, as appropriate, for purposes of estimating bearing resistance, using procedures described in Articles 10.6, 10.7 and 10.8.

Some increase in the prescribed resistance factors may be appropriate for design of temporary walls consistent with increased allowable stresses for temporary structures in allowable stress design.

Table 11.5.6-1 - Resistance Factors for Permanent Retaining Walls

WALL-TYPE AND CONDITION		RESISTANCE FACTOR
Non-Gravity Cantilevered and Anchored Walls		
Bearing resistance of vertical elements		Article 10.5 applies
Passive resistance of vertical elements		1.00
Pullout resistance of anchors	• Cohesionless (granular) soils • Cohesive soils • Rock	0.65 ⁽¹⁾ 0.70 ⁽¹⁾ 0.50 ⁽¹⁾
Tensile resistance of anchor		0.90
Flexural capacity of vertical elements		0.90
Mechanically Stabilized Earth Walls		
Bearing resistance		Article 10.5 applies
Sliding		Article 10.5 applies
Tensile resistance of metallic reinforcement	Strip reinforcements ⁽²⁾	
	• Static loading • Combined static/earthquake loading	0.85 1.10
	Grid reinforcements ^{(2) (3)}	
	• Static loading • Combined static/earthquake loading	0.75 1.00

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	Connectors ⁽²⁾ • Static loading • Combined static/earthquake loading	0.75 1.00
Tensile resistance of geosynthetic reinforcement	Reinforcements • Static loading • Combined static/earthquake Connectors • Static loading • Combined static/earthquake loading	0.90 1.20 0.90 1.20
Pullout resistance of tensile reinforcement	• Static loading • Combined static/earthquake loading	0.90 1.20
Prefabricated Modular Walls		
Bearing		Article 10.5 Applies
Sliding		Article 10.5 Applies
Passive Pressure		Article 10.5 Applies

⁽¹⁾ Apply to presumptive ultimate unit bond stresses in Article C11.9.4.2.

⁽²⁾ Apply to gross cross-section less sacrificial area. For sections with holes, reduce gross area in accordance with Article 6.8.3 and apply to net section less sacrificial area.

⁽³⁾ Applies to grid reinforcements connected to a rigid facing element (e.g., a concrete panel or block). For grid reinforcements connected to a flexible facing mat or which are continuous with the facing mat, use the resistance factor for strip reinforcements.

11.6.1.2 LOADING

Abutments and retaining walls shall be investigated for:

- lateral earth and water pressures, including any live and dead load surcharge,
- the self weight of the abutment/wall,
- loads applied to the bridge superstructure,
- temperature and shrinkage deformation effects, and
- earthquake loads, as specified herein, in Section 3 and elsewhere in these Specifications.

The provisions of Articles 3.11.5 and 11.5.5 shall apply. For stability computations, the earth loads shall be multiplied by the maximum and/or minimum load factors given in Table 3.4.1-2, as appropriate.

The design shall be investigated for any combination of forces which may produce the most severe condition of

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loading. The design of abutments on Mechanically Stabilized Earth and Prefabricated Modular Walls shall be in accordance with Articles 11.10.8 and 11.11.6.

For computing load effects in abutments, the weight of filling material directly over an inclined or stepped rear face, or over the base of a reinforced concrete spread footing may be considered as part of the effective weight of the abutment.

Where spread footings are used, the rear projection shall be designed as a cantilever supported at the abutment stem and loaded with the full weight of the superimposed material, unless a more exact method is used.

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11.6.5 Seismic Design

The effect of earthquakes shall be investigated using the extreme event limit state of Table 3.4.1-1 with resistance factors $f = 1.0$. Requirements for static design should first be met, as detailed in Articles 11.6.1 through 11.6.4. Selection of abutment types prior to static design shall recognize type selection criteria for seismic conditions, as described in Articles 2.5.6, 2.5.6.1, and 3.10.3, Table 2.5.6-2 and Figure C2.5.6-4. Seismic design provisions for abutment and wingwalls are described in Article 11.6.5.1, and those for free-standing conventional retaining walls (not supporting a bridge structure) are detailed in Article 11.6.5.2.

11.6.5.1 Abutments and Wingwalls

The participation of abutment walls and wingwalls in the overall dynamic response of bridge systems to earthquake loading and in providing resistance to seismically induced inertial loads shall be considered in the seismic design of bridges, as outlined in these provisions. Damage to walls that is allowed to occur during earthquakes shall be consistent with the performance criteria. Abutment participation in the overall dynamic response of the bridge systems shall reflect the structural configuration, the load-transfer mechanism from the bridge to the abutment system, the effective stiffness and force capacity of the wall-soil system, and the level of expected abutment damage. The capacity of the abutments to resist the bridge inertial load shall be compatible with the structural design of the abutment wall (i.e., whether part of the wall will be damaged by the design earthquake), as well as the soil resistance that can be reliably mobilized. The lateral load capacity of walls shall be evaluated based on an applicable passive earth-pressure theory.

11.6.5.1.1 Abutment Design: Longitudinal Direction

Under earthquake loading, the earth pressure action on abutment walls changes from a static condition to one of generally two possible conditions, depending on the magnitude of seismically induced movement of the abutment walls, the bridge superstructure, and the bridge/abutment configuration. For seat-type abutments where the expansion joint is sufficiently large to accommodate both the cyclic movement between the abutment wall and the bridge superstructure (i.e., superstructure does not push against abutment wall), the seismically induced earth pressure on the abutment wall would be the dynamic active pressure condition. However, when the gap at the expansion joint is not sufficient to accommodate the cyclic wall/bridge movements, a transfer of forces will occur from the superstructure to the abutment

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C11.6.5

C11.6.5.1

One of the most frequent observations of damage during past earthquakes has been damage to the abutment wall. This damage has been due to two primary causes: (1) the approach fill has moved outward, carrying the abutment with it, and (2) large reactive forces have been imposed on the abutment as the bridge deck has forced it into the approach fill. This latter cause of damage has often resulted from a design philosophy that assumed that the abutment wall had to survive only active seismic earth pressures, and that gaps between the bridge deck and abutment wall would not close. In many cases the gap was not sufficient to remain open, and very large loads were imposed by the deck. The passive reaction from the soil was as much as 30 times the forces used for active pressure design, resulting in overloading to and damage of the wall.

These seismic provisions have been prepared to specifically acknowledge the potential for this higher load to the abutment wall. If designed properly, the reactive capacity of the approach fill can provide significant benefit to the bridge-foundation system.

C11.6.5.1.1.

Common practice is to use the Mononobe-Okabe equations to estimate the magnitude of seismic earth pressures, for both active and passive pressure conditions. Previous editions of the AASHTO specifications have specifically discussed these methods and presented equations for making these estimates. These equations have, however, been found to have significant limitations.

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wall. As a result, the active earth pressure condition will not be valid and the earth pressure approaches a passive pressure condition behind the backwall.

For stub or integral abutments, the abutment stiffness and capacity under passive pressure loading, are primary design concerns, as discussed in paragraphs 11.6.5.1.1a and 11.6.5.1.1b. However, for partial depth or full depth seat abutment walls, earthquake-induced active earth pressures will continue to act below the backwall following separation of a knock-off backwall. These active pressures need to be considered in evaluating wall stability.

COMMENTARY

For the case of seismic active earth pressures, the Mononobe-Okabe equations are based on the Coulomb failure wedge assumption and a cohesionless backfill. For high accelerations or for backslopes, the equations lead to excessively high pressures that asymptote to infinity at critical acceleration levels or backslope angles. For the latter conditions, no real solutions to the equations exist implying equilibrium is not possible (Das, 1999). For horizontal backfills for example, for a friction angle for sand of 40 degrees, a wall friction angle of 20 degrees and a peak acceleration of 0.4g, the failure surface angle is 20 degrees to the horizontal. For a peak acceleration of 0.84g, the active pressure becomes infinite, implying a horizontal failure surface.

Clearly, for practical situations, cohesionless soil is unlikely to be present for a great distance behind an abutment wall and encompass the entire failure wedge under seismic conditions. In some cases, free-draining cohesionless soil may only be placed in the static active wedge (say at a 60 degree angle) with the remainder of the soil being cohesive embankment fill (c, ϕ soil) or even rock. Under these circumstances, the maximum earthquake-induced active pressure should be determined using trial wedges (Figure C11.6.5.1.1), with the strength on the failure planes determined from the strength parameters for the soils through which the failure plane passes. This approach will provide more realistic estimates of active pressure.

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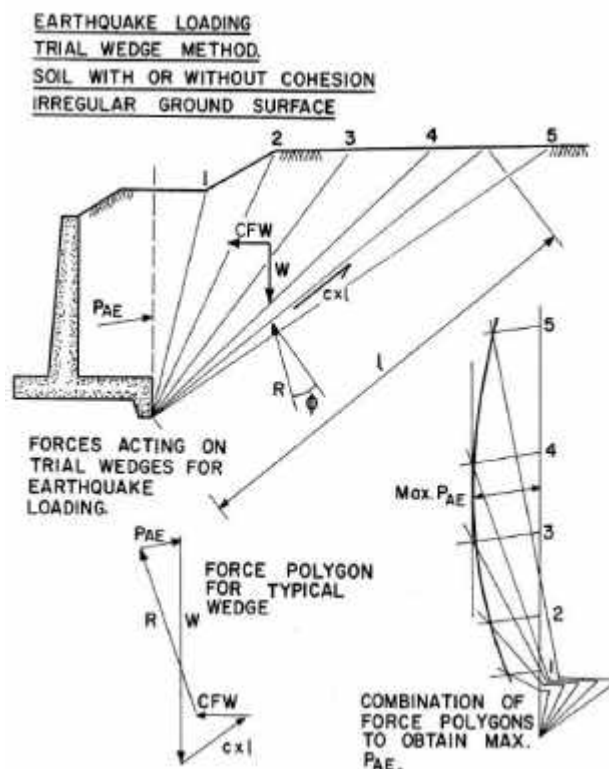


Figure C11.6.5.1 – Trial Wedge Method For Determining Critical Earthquake-Induced Active Forces

11.6.5.1.1a SDAP A1, A2, B and C

Abutments designed for service load conditions in these categories should resist earthquake loads with minimal damage with the exception of bridges in Seismic Hazard Level IV using SDAP C. For seat-type abutments, minimal abutment movement could be expected under dynamic active pressure conditions. However, bridge superstructure displacement demands could be 100 mm or more and potentially impact the abutment backwall. Where expected displacement demands are greater than a normal expansion gap of 25 to 50 mm, a knock-off backwall detail is recommended to minimize foundation damage, or alternatively, a cantilever deck slab to extend the seat gap should be provided, with a knock-off backwall tip.

In the case of integral abutments, sufficient reinforcing should be provided in the diaphragm to accommodate higher lateral pressures. For spread footing foundations, knock-off tabs or other fuse elements should be provided to minimize foundation damage. For pile-supported foundations, fuse elements should be used or connection detailing should ensure increased moment ductility in the piles.

C11.6.5.1.1a

No seismic provisions are required for bridges covered by these SDAPs because increased earth pressures from the approach fill and bridge displacements will normally be within tolerable levels. In the case of seismically induced active earth pressures, the static design of the wall will usually result in the controlling load case, if normal load and resistance factors are used. In the case of integral abutments, the designs based on static at-rest pressures will also be sufficiently conservative to meet seismic demand. For cases where the abutment is engaged and high passive forces could develop, the preferred approach is to design a fuse into the system to protect against damage. Alternatively, the Owner could decide to accept some level of damage, given the low likelihood of occurrence of the design earthquake.

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11.6.5.1.1b SDAP D, and E

For these design categories passive pressure resistance in soils behind integral abutment walls and knock-off walls for seat abutments will usually be mobilized due to the large longitudinal superstructure displacements associated with the inertial loads. For design purposes static passive pressures may be used without potential reductions associated with inertial loading in abutment backfill. Inclusion of abutment stiffness and capacity in bridge response analyses will reduce ductility demands on bridge columns as discussed in Article C2.5.6.

Case 1: To ensure that the columns are always able to resist the lateral loads, designers may choose to assume zero stiffness and capacity of abutments. In this case designers should check abutment damage potential and performance due to abutment displacement demand. Knock-off backwall details for seat abutments should be utilized to protect abutment foundations and increased reinforcing used in diaphragms or integral abutments to accommodate passive pressures.

Case 2: Where abutment stiffness and capacity is included in the design, it should be recognized that the passive pressure zone mobilized by abutment displacement extends beyond the active pressure zone normally adapted for static service load design, as illustrated schematically in Figure 11.6.5.1. Whether presumptive or computed passive pressures are used for design as described in the commentary paragraphs, backfill in this zone should be controlled by specifications unless the passive pressure that is used is less than 70% of the presumptive value.

Abutment stiffness and passive pressure capacity for either (1) SDAP D or (2) SDAP E two-step analysis methods should be characterized by a bi-linear relationship as shown in Figure 11.6.5.2. For seat type abutments, knock-off backwall details should be utilized with superstructure diaphragms designed to accommodate passive pressures, as illustrated in Figure C2.5.6-4. For integral abutments the end diaphragm should be designed for passive pressures, and utilize a stub pile footing or normal footing for support, with a sliding seat. Passive pressures may be assumed uniformly distributed over the height (H) of the backwall or diaphragm. Thus the total passive force is:

$$P_p = p_p * H \quad (11.6.5.1.1b-1)$$

where:

H = wall height in meters

p_p = passive pressure behind backwall

Calculation of Best-Estimate Passive Force P_p

If the strength characteristics of compacted or natural soils in the "passive pressure zone" (total stress strength

COMMENTARY

C10.6.5.1.1b

The determination of stiffness and capacity is a key step during the design of many bridges by these SDAPs. Procedures for calculating passive force, P_p , and abutment stiffness are described below. These procedures should use best-estimate soil properties. The approach is based upon using a uniform distribution of passive soil pressure against the abutment backwall. The uniform pressure approach is a simplification of more complex distribution patterns, which are functions of wall friction and deformation patterns (ie translation or tilting).

The two cases are discussed in more detail in Article C2.5.6.1.

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parameters c and ϕ) are known, then the passive force for a given height H may be computed using accepted analysis procedures. These procedures should account for the interface friction between the wall and the soil. The properties used shall be those indicative of the entire "passive pressure zone" as indicated in Figure 11.6.5.1. Therefore the properties of backfill that is only placed adjacent to the wall in the active pressure zone may not be appropriate.

If presumptive passive pressures are to be used for design, then the following criteria should apply:

- (1) Soil in the "passive pressure zone" should be compacted to a dry density greater than 95 percent of the maximum per ASTM Standard Method D1557 or equivalent.
- (2) For cohesionless, non-plastic backfill (fines content less than 30 percent), the passive pressure p_p may be assumed equal to $H/10$ MPa per meter of length of wall ($2H/3$ ksf per foot length of wall).
- (3) For cohesive backfill (clay fraction > 15 percent), the passive pressure p_p may be assumed equal to 0.25 MPa (5 ksf) provided the estimated unconfined compressive strength is greater than 0.20 MPa (4 ksf).

The presumptive values given above apply for use in the " *Permissible with Owner's Approval*" category, as defined in Section 2.5.6.1. If the design is based upon presumptive resistances that are no larger than 70 percent of the values listed above, then the structure may be classified in the "*Permissible*" category.

In all cases granular drainage material must be placed behind the abutment wall to ensure adequate mobilization of wall friction.

Calculation of Stiffness

For SDAP D one-step analyses and for the demand calculation of SDAP E analyses, an equivalent linear secant stiffness, K_{eff1} , is required for analyses. For integral or diaphragm abutments, an initial secant stiffness (Figure 11.6.5.2) may be calculated as follows:

$$K_{eff1} = P_p / 0.02H \quad (11.6.5.1.1b-2)$$

If computed abutment forces exceed the capacity, the stiffness should be softened iteratively (K_{eff2} to K_{effn}) until abutment displacements are consistent (within 30 percent) with the assumed stiffness. For seat abutments the expansion gap should be included in the initial estimate of the secant stiffness. Thus:

$$K_{eff1} = P_p / (0.02H + D_g) \quad (11.6.5.1.1b-3)$$

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where:

D_g = gap width

For SDAP E two-step analyses, where push-over analyses are conducted, values of P_p and the initial estimate of K_{eff1} should be used to define a bilinear load-displacement behavior of the abutment for the capacity assessment.

For partial depth or full-depth seat abutment walls, where knock-off backwalls are activated, the remaining lower wall design and stability check under the action of continuing earthquake-induced active earth pressures should be evaluated. For a no-collapse performance criteria, and assuming conventional cantilever retaining wall construction, horizontal wall translation under dynamic active pressure loading is acceptable. However, rotational instability may lead to collapse and thus must be prevented.

The design approach is similar to that of a free-standing retaining wall, except that lateral force from the bridge superstructure needs to be included in equilibrium evaluations, as the superstructure moves outwards from the wall. Earthquake-induced active earth pressures should be computed using horizontal accelerations at least equal to 50 percent of the site peak ground acceleration (i.e., $F_a S_s / 5.0$). Using less than the expected site acceleration implies that limited sliding of the wall may occur during the earthquake. A limiting equilibrium condition should be checked in the horizontal direction. To ensure safety against potential overturning about the toe, a restoring moment of at least 50 percent more than the driving overturning moment should exist. If necessary, wall design (initially based on a static loading condition) should be modified to meet the above condition.

11.6.5.1.2 Abutment Design -- Transverse Direction

In general, abutments shall be designed to resist earthquake forces in the transverse direction elastically for the 50% in 75-year earthquake. For the 3% in 75-year event, the abutment may either be designed to resist transverse forces elastically or a fuse shall be provided to limit the transverse force transfer at the abutment. If a fuse is used, then the effects of internal force redistribution resulting from fusing shall be taken into account in the design of the bridge. Limitations on the use of fusing for the various Seismic Design and Analysis Procedures are listed below.

In the context of these provisions, elastic resistance includes the use of elastomeric, sliding, or isolation bearings designed to accommodate the design displacements, soil frictional resistance acting against the base of a spread footing-supported abutment, pile resistance provided by piles acting in their elastic range, or

C11.6.5.1.2 Abutment Design: Transverse Direction

To meet the performance criteria, abutments shall experience essentially no damage in the 50% in 75-year earthquake, and this may be achieved if the abutments are designed to resist the elastic forces for the 50% in 75-year event. For the larger 3% in 75-year event, the elastic forces may be large enough that they cannot be resisted without some abutment damage. In general, the design of the abutment should attempt to restrict damage to locations that are inspectable and which can be reasonably accessed for repair.

Two preferred strategies may be considered. One is to use isolation, elastomeric or other bearings that accommodate the full seismic movement at the abutment and thereby significantly reduce the likelihood of damage to the abutment itself. The second strategy is to use fuse elements (isolation bearings with a high yield level or shear keys) that are intended to yield or breakaway thereby

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passive resistance of soil acting at displacements less than 2 percent of the wall height.

Likewise, fusing includes: breakaway elements, such as isolation bearings with a relatively high yield force; shear keys; yielding elements, such as wingwalls yielding at their junction with the abutment backwall; elastomeric bearings whose connections have failed and upon which the superstructure is sliding; spread footings that are proportioned to slide in the rare earthquake; or piles that develop a complete plastic mechanism. Section 2.5.6.1 outlines those mechanisms that are permissible with the Owner's approval.

The stiffness of abutments under transverse loading may be calculated based on the procedures given in Chapter 10 for foundation stiffnesses. Where fusing elements are used, allowance shall be made for the reduced stiffness of the abutment after fusing occurs.

11.6.5.1.2a SDAP A1, A2, B and C

Connection design forces also apply to shear restraint elements such as shear keys.

11.6.5.1.2b SDAP D, and E

For structures in these categories, either elastic resistance or fusing shall be used to accommodate transverse abutment loading. The elastic forces used for transverse abutment design shall be determined from an elastic demand analysis of the structure.

For short, continuous superstructure bridges (Length/Width < 4) with low skew angles (<20 degrees), low plan curvature (subtended angle < 30 degrees), and which also are designed for sustained soil mobilization in the transverse direction, the elastic forces and displacements for the transverse earthquake design may be reduced by 1.4 to account for increased damping provided by the soil at the abutments. Herein transverse earthquake is defined as acting perpendicular to a chord extending between the two abutments. Sustained soil mobilization requires resistance to be present throughout the range of cyclic motion. Where

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limiting the forces transferred to the abutment. It should be noted that it is difficult to predict the capacity of a concrete shear key and hence this is a less reliable concept when compared to isolation elements with a high yield force. Such fuse elements should be designed to restrict damage to inspectable locations. In situations where neither of these strategies is practical, then damage may be incurred in the foundation of the abutment, but such a design approach shall only be undertaken with the approval of the Owner.

The calculation of stiffness may require the estimation of effective secant stiffnesses based on ultimate strength and estimates of yield displacements. The approach will be similar to that used in calculating longitudinal abutment stiffness. Alternately, bounding analyses may be used wherein a resisting element is completely released. Where a complete loss of resistance may occur, for example breakaway shear keys or blocks, a small nominal spring resistance may be necessary to obtain reasonable and stable results from a multimode dynamic analysis.

C11.6.5.1.2a

For abutments of bridges in the lower seismic design categories, the abutment as typically designed for service loads should be adequate for resisting the seismic effects. Where lateral restraint is provided at the abutment, with for example shear keys, minimum design forces are specified to provide a reasonable amount of strength to resist the forces that are likely to develop in an earthquake.

Abutments designed for non-seismic loads and for the connection forces outlined in Section 3.10.3.2 for SDAP A1 and A2 or in Section 3.10.3.3 for SDAP B should resist earthquakes with minimal damage. Bridges designed using SDAP C are proportioned such that the abutments are not required to resist inertial forces. Therefore some damage may occur in abutments of such bridges, particularly in the higher Seismic Hazard Levels.

C11.6.5.1.2b

For SDAP D, and E, seismic design and analysis is required and the actual restraint conditions at the abutments will determine the amount of force that is attracted to the abutments. These forces shall either be resisted elastically or fuse elements may be used.

Short bridges that have abutments, which can continuously provide soil resistance under cyclic deformations, will exhibit damping that likely exceeds the normal 5 percent value. Therefore for shorter bridges that have small skew and horizontal curves, a 1.4 reduction value is allowed for all the elastic forces and displacements resulting from a transverse earthquake. This provision only applies to shorter bridges where the effects of the transverse abutment response extend throughout the entire bridge. To rely on this reduction, the soil must be able to continuously provide resistance under cyclic loading.

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combined mechanisms provide resistance, at least 50 percent of the total resistance must be provided by a sustained mechanism for the system to qualify for the 1.4 reduction.

The design of concrete shear keys should consider the unequal forces that may develop in a skewed abutment, particularly if the intermediate piers are also skewed. (This effect is amplified if intermediate piers also have unequal stiffness, such as wall piers.) The shear key design should also consider unequal loading if multiple shear keys are used. The use of recessed or hidden shear keys should be avoided if possible, since these are difficult to inspect and repair.

COMMENTARY

Friction against the base of foundations not supported on piles or shafts may be considered sustained resistance, as may be friction against vertical surfaces not subject to gapping as described below. The force reduction is not permitted for other types of abutment resistance, for instance, passive mobilization of backfill where a gap may form between the soil and the backwall. These provisions have been adapted from the "short bridge" provisions outlined by Caltrans in their Seismic Design Criteria and Memo 20-4.

Wingwalls, in general, should not be relied upon to resist significant transverse forces. Typical configurations of wingwalls are normally inadequate to resist large forces corresponding to the passive resistance of the soil retained by the wingwalls. The wingwalls' yield resistance may, however, be counted in the resistance, even though this value will likely not contribute significantly to the lateral resistance.

In cases where the backfill may be displaced passively, whether intended to be part of the ERS or not, the possibility of a gap opening in the backfill should be considered when calculating the transverse lateral capacity of an abutment. If a gap could open between the backfill soil and the abutment, the transverse resistance provided by the wingwalls may be compromised. Specifically, cohesion in the backfill may produce such a situation. If this occurs, reduction of the transverse resistance may be necessary.

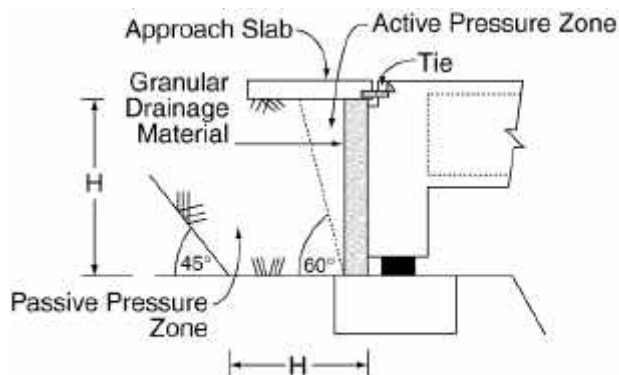


Figure 11.6.5.1 Design Passive Pressure Zone

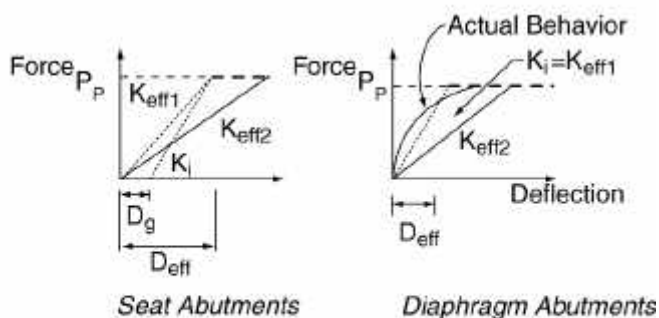


Figure 11.6.5.2 Characterization of Abutment Capacity and Stiffness

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11.6.5.2 Conventional Retaining Walls

11.8.6 Seismic Design

The effect of earthquake loading shall be investigated using the extreme event limit state of Table 3.4.1-1 with resistance factors $f = 1.0$ and an accepted methodology.

11.9.5.2 VERTICAL WALL ELEMENTS

Vertical wall elements shall be designed to resist all horizontal earth pressure, surcharge, water pressure, anchor and seismic loadings, as well as the vertical component of the anchor loads and any other vertical loads. Horizontal supports may be assumed at each anchor location and at the bottom of the excavation if the vertical element is sufficiently embedded below the bottom of the excavation.

11.9.6 Seismic Design

The provisions of Article 11.6.5 shall apply.

11.10.7 Seismic Design

11.10.7.1 EXTERNAL STABILITY

Stability determinations shall be made by applying static forces, the horizontal inertial force, P_{IR} , and 50% of the dynamic horizontal thrust, P_{AE} . The dynamic horizontal thrust, P_{AE} , shall be evaluated using the pseudo-static Mononobe-Okabe method and shall be applied to the back surface of the reinforced fill at the height of $0.6H$ from the base and the horizontal inertial force at the mid-height of the structure. Values of P_{AE} and P_{IR} for structures with horizontal backfill may be determined using the following:

$$A_m = (1.45 - A)A \quad (11.10.7.1-1)$$

$$P_{AE} = 0.375A_m g_s H^2 \times 10^{-9} \quad (11.10.7.1-2)$$

C11.8.6

In general, the pseudo-static approach developed by Mononobe and Okabe may be used to estimate the equivalent static forces provided the maximum lateral earth pressure be computed using a seismic coefficient $k_h = 1.5A$. Forces resulting from wall inertia effects may be ignored in estimating the seismic lateral earth pressure. Refer to Appendix A.

C11.9.6

In general, the pseudo-static approach developed by Mononobe and Okabe may be used to estimate the equivalent static forces provided the maximum lateral earth pressure be computed using a seismic coefficient $k_h = 1.5A$. Forces resulting from wall inertia effects may be ignored in estimating the seismic lateral earth pressure. Refer to Appendix A, a reproduction of portions of "Standard Specifications for Seismic Design of Highway Bridges", 1983, relating to the Mononobe-Okabe method.

Note: Retaining walls are outside the scope of NCHRP 12-49, but future projects should consider modifying this Article similar to Article C11.6.5.1.1.

The equation for P_{AE} was developed assuming a friction angle of 30° . P_{AE} may be adjusted for other soil friction angles using the Mononobe-Okabe method, with the horizontal acceleration k_h equal to A_m and k_v equal to zero.

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$$P_{IR} = 0.5 A_m g \rho_s H^2 \times 10^{-9} \quad (11.10.7.1-3)$$

where:

A = maximum earthquake acceleration coefficient (Article 3.10.2) (dim)

A_m = maximum wall acceleration coefficient at the centroid of the wall mass (dim)

g = acceleration of gravity (m/s^2)

ρ_s = soil density (kg/m^3)

H = height of wall (mm)

For structures with sloping backfills, the inertial force, P_{IR} , shall be based on an effective mass having a height H_2 and a base width equal to $0.5 H_2$ determined as follows:

$$H_2 = H + \frac{0.5H \tan(\beta)}{(1 - 0.5 \tan(\beta))} \quad (11.10.7.1-4)$$

where:

β = slope of backfill (DEG)

The inertia force shall be taken to act simultaneously with one-half the dynamic horizontal thrust, P_{AE} , computed using the pseudo-static Mononobe-Okabe method, and applied at $0.6 H_2$ above the base on the back surface of the effective mass. P_{IR} for sloping backfills shall be calculated as follows:

$$P_{IR} = P_{ir} + P_{is} \quad (11.10.7.1-5)$$

where:

$$P_{ir} = 0.5 A_m g \rho_s H_2 H \times 10^{-9} \quad (11.10.7.1-6)$$

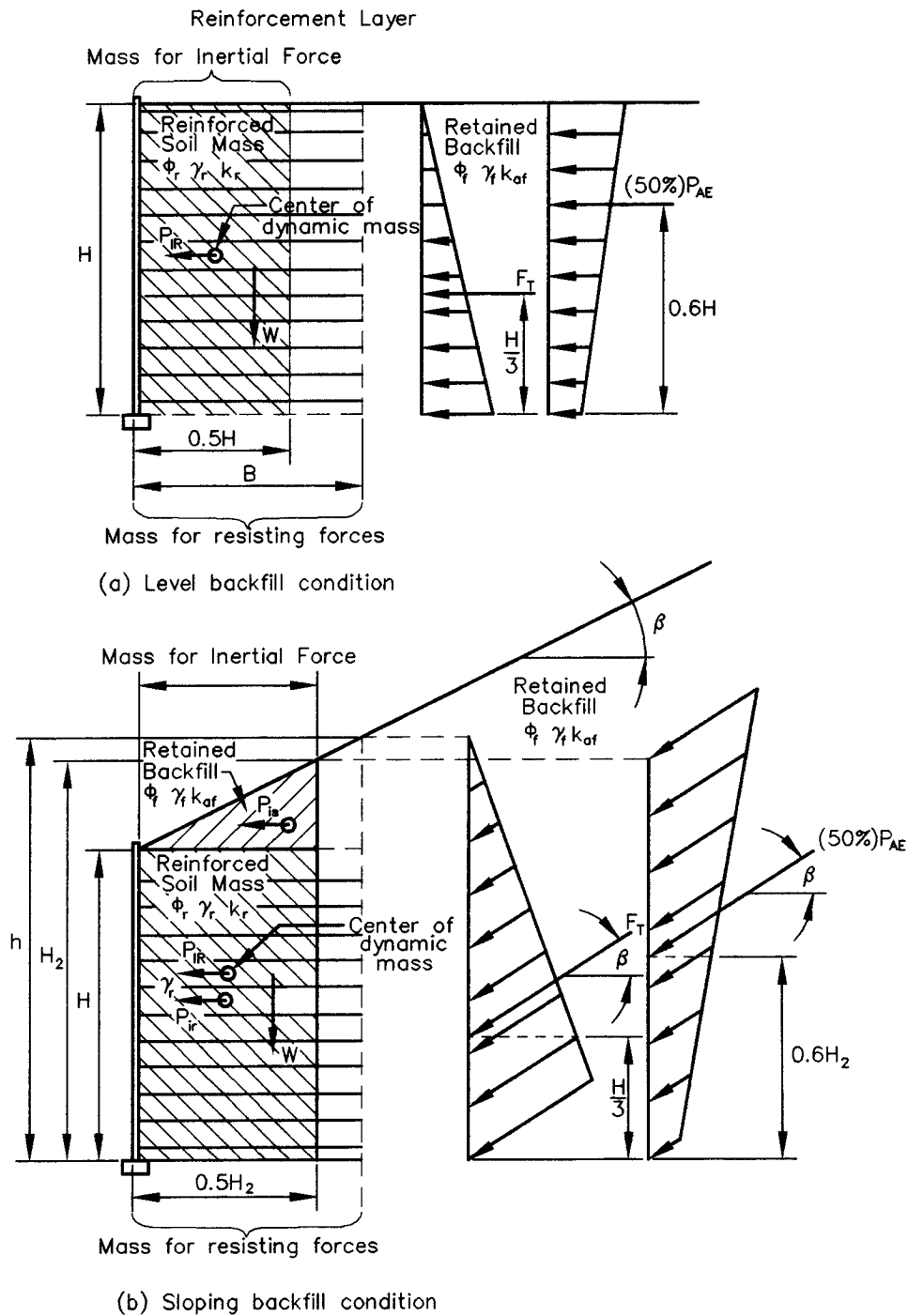
$$P_{is} = 0.125 A_m g \rho_s (H_2)^2 \tan \beta \times 10^{-9} \quad (11.10.7.1-7)$$

where, P_{ir} is the inertial force caused by acceleration of the reinforced backfill and P_{is} is the inertial force caused by acceleration of the sloping soil surcharge above the reinforced backfill, with the width of mass contributing to P_{IR} equal to $0.5 H_2$. P_{IR} acts at the combined centroid of P_{ir} and P_{is} . The locations of P_{AE} and P_{IR} are illustrated in Figure 1.

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11.10.7.2 INTERNAL STABILITY

Reinforcements shall be designed to withstand horizontal forces generated by the internal inertia force, P_i , and the static forces. The total inertia force, P_i , per unit length of structure shall be considered equal to the mass of the active zone times the maximum wall acceleration coefficient A_m . This inertia force shall be

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distributed to the reinforcements proportionally to their resistant areas on a load per unit width of wall basis as follows:

$$T_{md} = g P_i \frac{L_{ei}}{\sum_{i=1}^m (L_{ei})} \quad (11.10.7.2-1)$$

where:

T_{md} = factored incremental dynamic inertia force at Layer i (N/mm of structure)

g = load factor for EQ loads from Table 3.4.1-1 (dim)

P_i = internal inertia force (N/mm)

L_{ei} = effective reinforcement length for layer i (mm)

As shown in Figure 1, the total factored load applied to the reinforcement on a load per unit of wall width basis is as follows:

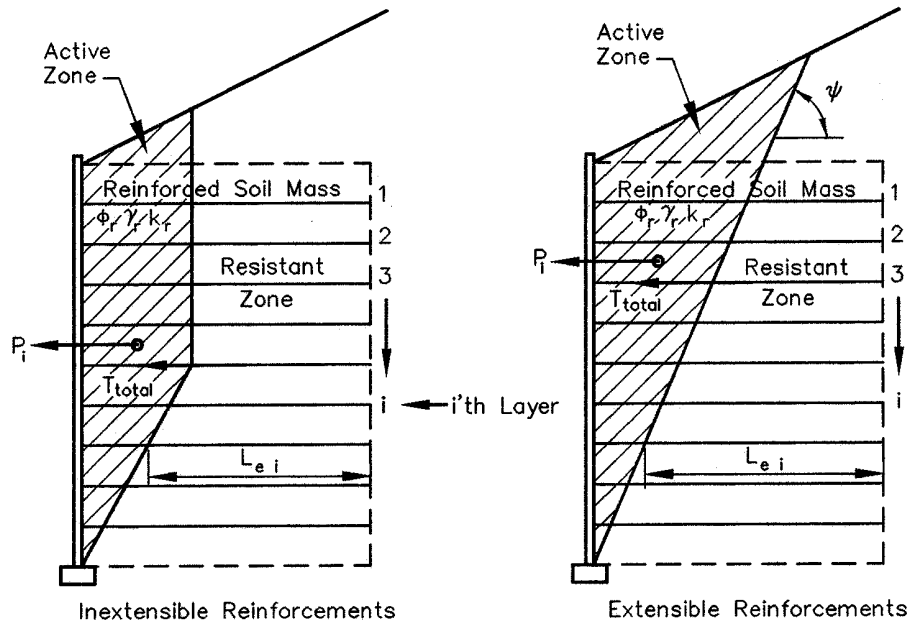
$$T_{total} = T_{max} + T_{md} \quad (11.10.7.2-2)$$

where, T_{max} is determined using Equation 11.10.6.2.1-3.

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P_i = Internal inertial force due to the weight of the backfill within the active zone.

L_{ei} = The length of reinforcement in the resistant zone of the i'th layer.

T_{max} = The load per unit wall width applied to each reinforcement due to static forces.

T_{md} = The load per unit wall width applied to each reinforcement layer due to dynamic forces.

The total load per unit wall width applied to each layer, $T_{total} = T_{max} + T_{md}$

For geosynthetic reinforcement rupture, the reinforcement must be designed to resist the static and dynamic components of the load as follows:

For the static component:

$$T_{max} \leq \frac{f S_{rs} R_c}{RF} \quad (11.10.7.2-3)$$

For the dynamic component:

$$T_{md} \leq \frac{f S_{rt} R_c}{RF_{ID} RF_D} \quad (11.10.7.2-4)$$

where:

The reinforcement must be designed to resist the dynamic component of the load at any time during its design life. Design for static loads requires the strength of the reinforcement at the end of the design life to be reduced to account for creep and other degradation mechanisms. Strength loss in polymeric materials due to creep requires long term, sustained loading. The dynamic component of load for seismic design is a transient load and does not cause strength loss due to creep. The resistance of the reinforcement to the static component of load, T_{max} , must, therefore, be handled separately from the dynamic component of load, T_{md} . The strength required to resist T_{max} must include the effects of creep, but the strength required to resist T_{md} should not include the effects of creep.

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S_{rs} = Ultimate reinforcement tensile resistance required to resist static load component (N/mm)
 S_{rt} = Ultimate reinforcement tensile resistance required to resist dynamic load component (N/mm)

R_c = reinforcement coverage ratio from Article 11.10.6.4 (dim)

RF = combined strength reduction factor to account for potential long-term degradation due to installation damage, creep and chemical aging from Article 11.10.6.4.2b (dim)

RF_{ID} = strength reduction factor to account for installation damage to reinforcement from Article 11.10.6.4.2b (dim)

RF_D = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation from Article 11.10.6.4.2b (dim)

Therefore, the required ultimate tensile resistance of the geosynthetic reinforcement is:

$$T_{ult} = S_{rs} + S_{rt} \quad (11.10.7.2-5)$$

For pullout of steel or geosynthetic reinforcement:

$$L_e \geq \frac{T_{total}}{f (0.8F^* a s_v C R_c)} \quad (11.10.7.2-6)$$

where:

L_e = length of reinforcement in resisting zone (mm)

T_{total} = maximum factored reinforcement tension from Eq. 11.10.7.2-2 (N/mm)

F = resistance factor for reinforcement pullout from Table 11.5.6-1 (dim)

F^* = pullout friction factor (dim)

a = scale effect correction factor (dim)

s_v = unfactored vertical stress at the reinforcement level in the resistant zone (MPa)

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C = overall reinforcement surface area
geometry factor (dim)

R_c = reinforcement coverage ratio from Article
11.10.6.4 (dim)

For seismic loading conditions, the value of F^* ,
the pullout resistance factor, shall be reduced to 80
percent of the value used for static design.

11.10.7.3 FACING REINFORCEMENT CONNECTIONS

Facing elements shall be designed to resist the
seismic loads determined in accordance with Article
11.10.7.2 (i.e., T_{total}). Facing elements shall be designed
in accordance with applicable provisions of Sections 5, 6
and 8 for reinforced concrete, steel and timber,
respectively.

For segmental concrete block faced walls, the
blocks located above the uppermost backfill
reinforcement layer shall be designed to resist toppling
failure during seismic loading.

For geosynthetic connections, the factored long
term connection strength ($f T_c$) must be greater than T_{max}
+ T_{md} . Where the long-term connection strength is
partially or fully dependent on friction between the facing
blocks and the reinforcement, and connection pullout is
the controlling failure mode, the long-term connection
strength to resist seismic loads shall be reduced to 80
percent of its static value.

For the static component:

$$T_{max} \leq \frac{f S_{rs} CR_u}{RF_c} \leq 0.8 f S_{rs} CR_s$$

(11.10.7.3-1)

For the dynamic component:

$$T_{md} \leq \frac{f S_{rt} CR_u}{RF_D} \leq 0.8 f S_{rt} CR_s$$

(11.10.7.3-2)

where:

S_{rs} = ultimate reinforcement tensile resistance
required to resist static load component
(N/mm)

S_{rt} = Ultimate reinforcement tensile resistance
required to resist dynamic load component
(N/mm)

CR_u = reduction factor to account for reduced
ultimate strength resulting from connection

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from Article 11.10.6.4.3b (dim)

CR_s = reduction factor to account for reduced strength due to connection pullout from Article 11.10.6.4.3b (dim)

RF_c = reduction factor to account for long-term degradation of reinforcement at wall face connection due to environmental factors from Article 11.10.6.4.3b (dim) RF_D = reduction factor to prevent rupture of reinforcement due to chemical and biological degradation from Article 11.10.6.4.3b (dim)

The required ultimate tensile resistance of the geosynthetic reinforcement at the connection is:

$$T_{ult} = S_{rs} + S_{rt} \quad (11.10.7.3-3)$$

For structures in seismic performance zones 3 or 4, facing connections in segmental block faced walls shall not be fully dependent on frictional resistance between the backfill reinforcement and facing blocks. Shear resisting devices between the facing blocks and backfill reinforcement such as shear keys, pins, etc. shall be used.

For steel reinforcement connections, resistance factors for combined static and seismic loads may be increased by 33 percent of factors used for static loading.

Based on these resistance factors, the available factored connection strength must be greater than T_{total} .

11.10.11.2 TRAFFIC LOADS AND BARRIERS

C11.10.11.2

Traffic loads shall be treated as uniform surcharge loads in accordance with the criteria outlined in Article 3.11.6.2. The live load surcharge pressure shall be equal to not less than 600 mm of earth. Parapets and traffic barriers, constructed over or in line with the front face of the wall shall be designed to resist overturning moments by their own mass. Base slabs shall not have any transverse joints except construction joints, and adjacent slabs shall be joined by shear dowels. The upper row(s) of soil reinforcements shall have sufficient tensile capacity to resist a concentrated horizontal load of P_H where $P_H = 45000$ N distributed over a barrier length of 1500 mm. This force distribution accommodates the local peaking of force in the soil reinforcements in the vicinity of the concentrated load. This distributed force would be equal to P_{H1} where $P_{H1} = 30$ N/mm and is applied as shown in Figure 11.10.11.1-2. P_{H1} would be distributed to the reinforcements assuming b_f equal to the width of the base slab. Adequate space shall be provided laterally between the back of the facing panels and the traffic barrier/slab to allow the traffic barrier and

The force distribution for pullout computations is different than that used for tensile computations because the entire base slab must move laterally to initiate a pullout failure due to the relatively large deformation required.

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slab to resist the impact load in sliding and overturning without directly transmitting load to the top facing units.

For checking pullout safety of the reinforcements, the lateral traffic impact load shall be distributed to the upper soil reinforcement and facing units using Figure 11.10.11.1-2, assuming b_f equal to the width of the base slab. The full length of reinforcements shall be considered effective in resisting pullout due to the impact load. The upper row(s) of soil reinforcement shall have sufficient pullout capacity to resist a horizontal load of $\frac{1}{2}P_{H1}$ where $P_{H1} = 45000$ N distributed over the full 6000 mm base slab length. This distributed force would be equal to $\frac{1}{2}P_{H1}$ where P_{H1} is applied as shown in Figure 11.10.11.1-2.

Due to the transient nature of traffic barrier impact loads, when designing for reinforcement rupture, the geosynthetic reinforcement must be designed to resist the static and transient (impact) components of the load as follows:

Refer to C11.10.7.2 which applies to transient loads, such as impact loads on traffic barriers, as well as earthquake loads.

For the static component, see equation 11.10.7.2-3.

For the transient components,

$$\Delta S_H S_V \leq \frac{f S_{rt} R_c}{RF_{ID} RF_D}$$

(11.10.11.2-1)

where:

$\frac{1}{2} s_h$ = traffic barrier impact stress applied over reinforcement tributary area per Article 11.10.11.1 (N/mm)

S_v = vertical spacing of reinforcement (mm)

S_{rt} = ultimate reinforcement tensile resistance required to resist dynamic load component (N/mm)

R_c = reinforcement coverage ratio from Article 11.10.6.4 (dim)

RF_{ID} = strength reduction factor to account for installation damage to reinforcement from Article 11.10.6.4.2b (dim)

RF_D = strength reduction factor to prevent rupture of reinforcement due to chemical and biological degradation from Article 11.10.6.4.2b (dim)

The reinforcement strength required for the static load component must be added to the reinforcement strength required for the transient load component to determine the required total ultimate strength using Eq. 11.10.7.3-3.

Parapets and traffic barriers shall satisfy crash testing requirements as specified in Section 13. The anchoring slab shall be strong enough to resist the ultimate

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strength of the standard parapet.

Flexible post and beam barriers, when used, shall be placed at a minimum distance of 1000 mm from the wall face, driven 1500 mm below grade, and spaced to miss the reinforcements where possible. If the reinforcements cannot be missed, the wall shall be designed accounting for the presence of an obstruction as described in Article 11.10.11.4. The upper two rows of reinforcement shall be designed for an additional horizontal load γP_{H1} , where $P_{H1} = 4400$ N per linear mm of wall.

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SECTION 15 – SEISMIC ISOLATION

SPECIFICATIONS

15.1 SCOPE

Criteria provided herein for bearings used in implementing seismic isolation design are supplemental to Section 14. These provisions are necessary to provide a rational design procedure for isolation systems incorporating the displacements resulting from the seismic response. If a conflict arises between the provisions of Sections 14 and 15, the provisions contained in Section 15 govern.

These specifications are intended for systems that isolate in the horizontal plane only – that is, the system is assumed to be essentially rigid in the vertical direction. In addition, the criteria are currently intended for passive isolation systems only.

COMMENTARY

C15.1 SCOPE

Isolating structures from the damaging effects of earthquakes is not a new idea. The first patents for base isolation schemes were obtained in the 1870s, but until the past two decades, few structures were built using these ideas. Early concerns were focused on the displacements at the isolation interface. These have been largely overcome with the successful development of mechanical energy dissipators. When used in combination with a flexible device such as an elastomeric bearing, an energy dissipator can control the response of an isolated structure by limiting both the displacements and the forces. Interest in seismic isolation, as an effective means of protecting bridges from earthquakes, was revived in the 1970s. To date there are several hundred bridges in New Zealand, Japan, Italy, and the United States using seismic isolation principles and technology for their seismic design.

Seismically isolated buildings such as the University of Southern California Hospital in Los Angeles, and the West Japan Postal Savings Computer Center in Kobe, Japan, performed as expected in the 1994 Northridge and 1995 Kobe earthquakes. Records from these isolated structures show good correlation between the analytical prediction and the recorded performance.

The basic intent of seismic isolation is to increase the fundamental period of vibration such that the structure is subjected to lower earthquake forces. However, the reduction in force is accompanied by an increase in displacement demand that must be accommodated within the isolation system. Furthermore, flexible bridges can be lively under service loads.

The three basic elements in seismic isolation systems that have been used to date are

- (a) a vertical-load carrying device that provides lateral flexibility so that the period of vibration of the total system is lengthened sufficiently to reduce the force response,
- (b) a damper or energy dissipator so that the relative deflections across the flexible mounting can be limited to a practical design level, and
- (c) a means of providing rigidity under low (service) load levels, such as wind and braking forces.

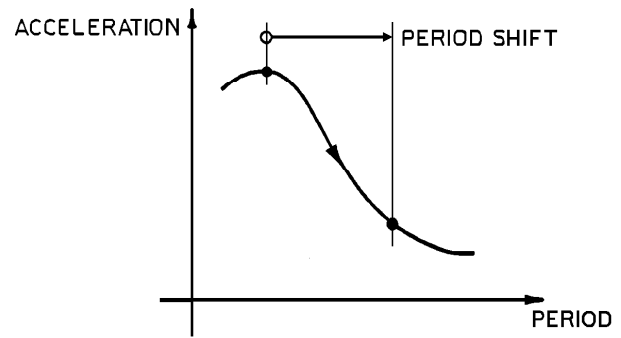
Flexibility – Elastomeric and sliding bearings are two

SECTION 15 – SEISMIC ISOLATION

SPECIFICATIONS

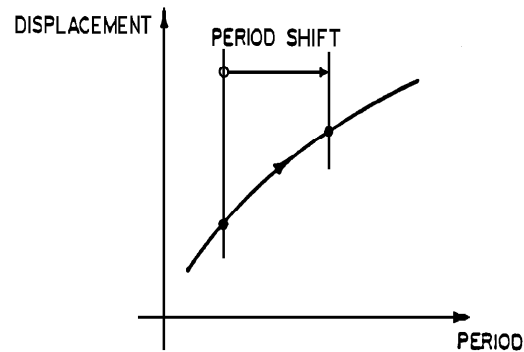
COMMENTARY

ways of introducing flexibility into a structure. The typical force response with increasing period (flexibility) is shown schematically in the typical acceleration response curve in Figure C15.1-1. Reductions in base shear occur as the period of vibration of the structure is lengthened. The extent to which these forces are reduced primarily depends on the nature of the earthquake ground motion and the period of the fixed-base structure. However, as noted above, the additional flexibility needed to lengthen the period of the structure will give rise to relative displacements across the flexible mount. Figure C15.1-2 shows a typical displacement response curve from which displacements are seen to increase with increasing period (flexibility).



ACCELERATION RESPONSE SPECTRUM

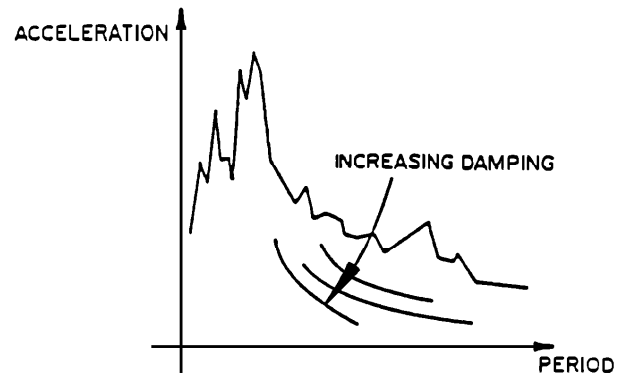
Figure C15.1-1 Typical Acceleration Response Curve



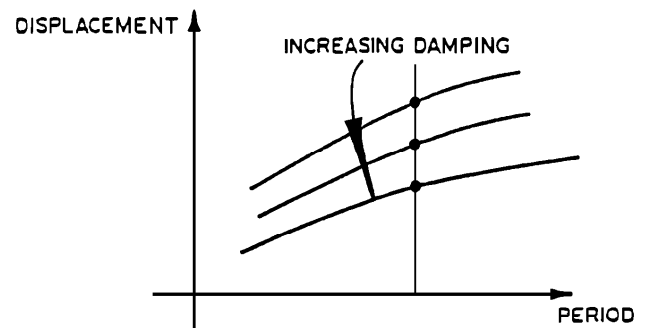
DISPLACEMENT RESPONSE SPECTRUM

Figure C15.1-2 Typical Displacement Response Curve

Energy Dissipation – Relative displacements can be controlled if additional damping is introduced into the structure at the isolation level. This is shown schematically in figure C15.1-3.



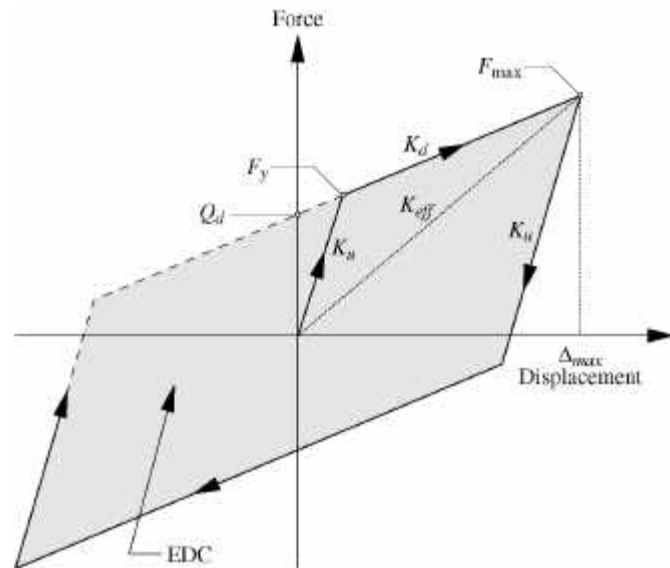
ACCELERATION RESPONSE SPECTRUM



DISPLACEMENT RESPONSE SPECTRUM

Figure C15.1-3 Response Curves for Increasing Damping

Two effective means of providing damping are hysteretic energy dissipation and viscous energy dissipation. The term *viscous* refers to energy dissipation that is dependent on the magnitude of the velocity. The term *hysteretic* refers to the offset between the loading and unloading curves under cyclic loading. Figure C15.1-4 shows an idealized force-displacement hysteresis loop where the enclosed area is a measure of the energy dissipated during one cycle (EDC) of motion.



Q_d = Characteristic strength

F_y = Yield force

F_{max} = Maximum force

K_d = Post-elastic stiffness

K_u = Elastic (unloading) stiffness

K_{eff} = Effective stiffness

D_{max} = Maximum bearing displacement

EDC = Energy dissipated per cycle = Area of hysteresis loop (shaded)

Figure C15.1-4 Characteristics of Bilinear Isolation Bearings

Rigidity Under Low Lateral Loads – While lateral flexibility is very desirable for high seismic loads, it is clearly undesirable to have a bridge that will vibrate perceptibly under frequently occurring loads, such as wind or braking. External energy dissipators and modified elastomers may be used to provide rigidity at these service loads by virtue of their high initial elastic stiffness (K_u in Figure C15.1-4). As an alternative, friction in sliding isolation bearings may be used to provide the required rigidity.

Example – The principles for seismic isolation are illustrated by figure C15.1-5. The dashed line is the elastic ground response spectrum as specified in Article 3.10.2. The solid line represents the composite response spectrum for an isolated bridge. The period shift provided by the flexibility of the isolation system reduces the spectral acceleration from A_1 to A_2 . The increased damping provided by the isolation system further reduces the spectral acceleration from A_2 to A_3 . Note that spectral acceleration A_1 and A_3 are used to determine forces for the design of conventional and

SECTION 15 – SEISMIC ISOLATION

SPECIFICATIONS

COMMENTARY

isolated bridges, respectively.

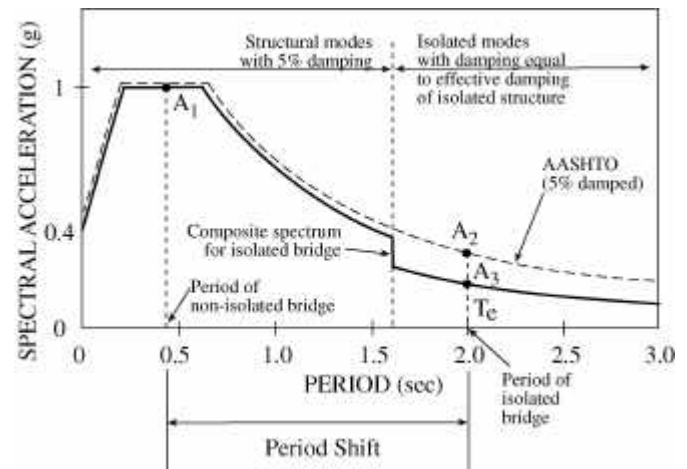


Figure C15.1-5 Response Spectrum for Isolated Bridge

15.2 DEFINITIONS

- **DESIGN DISPLACEMENT** is the lateral seismic displacement at the center of rigidity, required for design of the isolation system.
- **EFFECTIVE DAMPING** is the value of equivalent viscous damping corresponding to the energy dissipated during cyclic response at the design displacement of the isolated structure.
- **EFFECTIVE STIFFNESS** is the value of the maximum lateral force at instance of maximum lateral displacement in the isolation system, or an element thereof, divided by the maximum lateral displacement.
- **ELASTIC RESTRAINT SYSTEM** is the collection of structural elements that provide restraint of the seismically isolated structure for nonseismic lateral loads. The elastic restraint system may be either an integral part of the isolation system or may be a separate device.
- **ISOLATION SYSTEM** is the collection of all the elements that provide vertical stiffness, lateral flexibility, and damping to the system at the isolation interface. It includes the isolator units and the elastic restraint system, if one is used.
- **ISOLATOR UNIT** is a horizontally flexible and vertically stiff bearing of the isolation system, which permits large lateral deformation under seismic load.

C15.2 DEFINITIONS

ISOLATION SYSTEM

The isolation system does not include the substructure and deck.

SECTION 15 – SEISMIC ISOLATION

SPECIFICATIONS

The isolator unit may or may not provide energy dissipation.

- **OFFSET DISPLACEMENT** is the lateral displacement of an isolator unit resulting from creep, shrinkage, and 50 percent of the thermal displacement.
- **TOTAL DESIGN DISPLACEMENT** is the maximum lateral seismic displacement of an isolator unit resulting from the analysis and required for design of the isolation system, including both translational displacement at the center of rigidity, D_i , and the component of torsional displacement in the direction under consideration.

15.3 NOTATION

- A = Acceleration coefficient from section 3.
- A_b = Bonded area of elastomer.
- A_r = Overlap area between the top-bonded and bottom-bonded elastomer areas of displaced bearing (figure C15.3-1).
- B = Numerical coefficient related to the effective damping of the isolation system as set forth in Table 15.4.1-1.
- B_d = Bonded plan dimension or bonded diameter in loaded direction of rectangular bearing or diameter of circular bearing (Figure C15.3-1).
- C_s = Elastic seismic response coefficient.
- DL = Dead load.
- E = Young's modulus of elastomer.
- EDC = Energy dissipated per cycle (area of hysteresis loop).
- F = Statically equivalent seismic force.
- F_A = Design force for connections for bridges in Seismic Design and Analysis Procedure (SDAP A).
- F_i = Force in the isolator unit at displacement D_i .
- F_n = Maximum negative force in an isolator unit during a single cycle of prototype testing.
- $F_{n,max}$ = Maximum negative force in an isolator unit for all cycles of prototype testing at a common displacement amplitude.
- $F_{n,min}$ = Minimum negative force in an isolator unit

COMMENTARY

OFFSET DISPLACEMENT

The offset displacement is used for prototype testing and designing the isolator units.

C15.3 NOTATION

A_r is defined as the overlap area between the top-bonded and bottom-bonded elastomer areas of a displaced bearing, as shown in figure C15.3-1.

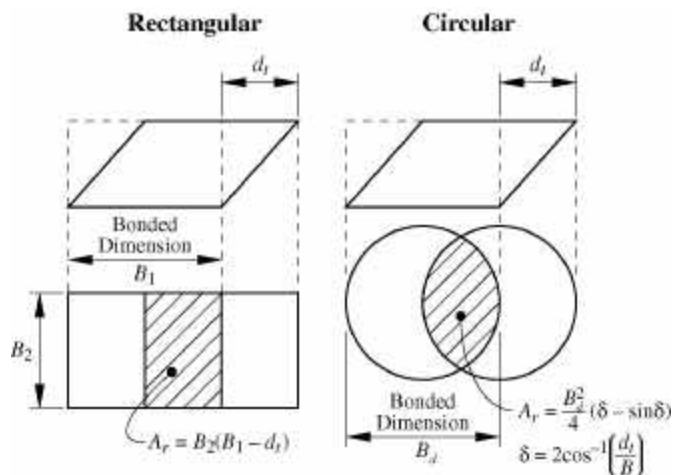


Figure C15.3-1

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for all cycles of prototype testing at a common displacement amplitude.

F_p = Maximum positive force in an isolator unit during a single cycle of prototype testing.

$F_{p,max}$ = Maximum positive force in an isolator unit for all cycles of prototype testing at a common displacement amplitude.

$F_{p,min}$ = Minimum positive force in an isolator unit for all cycles of prototype testing at a common displacement amplitude.

F_v = Site soil coefficient given in Article 3.10.2.3.3.

G = Shear modulus of elastomer.

g = Acceleration due to gravity.

$\bar{k}$ = Elastomer material constant.

k_{iso} = Effective stiffness of an isolator unit determined by prototype testing.

k_{max} = Maximum effective stiffness of the isolator unit at the design displacement in the horizontal direction under consideration.

k_{min} = Minimum effective stiffness of the isolator unit at the design displacement in the horizontal direction under consideration.

k_{sub} = Stiffness of the substructure protected by the isolation unit(s)

K = Bulk modulus of the elastomer (Article 15.11).

K_d = The second slope stiffness of the bilinear hysteresis curve.

K_{eff} = The sum of the effective linear stiffnesses of all bearings and substructures supporting the superstructure segment as calculated at displacement D_i for the bearings and displacement D_{Sub} for the substructure.

LL = Live load.

LL_s = Seismic live load.

OT = Additional vertical load on bearing resulting from overturning moment effect of horizontal loads.

P = Maximum vertical load resulting from the combination of dead load plus live load (including seismic live load, if applicable) using a g factor of 1.

$\bar{k}$ = Material constant related to hardness. (Refer to Roeder, Stanton, and Taylor 1987 for values.)

LL_s , the seismic live load, shall be determined by the engineer as a percentage of the total live load considered applicable for the design. Typically live load is not considered. However, since isolated structures are generally much more flexible, additional mass from the live load may need to be considered.

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- Q_d = Characteristic strength of the isolator unit. It is the ordinate of the hysteresis loop at zero bearing displacement. Refer to Figure C15.1-4.
- S_1 = The one-second period spectral acceleration given in Article 3.10.2.1.
- S = Shape factor (Article 15.11).
- S_A = Spectral acceleration.
- S_D = Spectral displacement.
- T_{eff} = Period of seismically isolated structure, in seconds, in the direction under consideration.
- T_r = Total elastomer thickness.
- t_i = Thickness of elastomer layer number i , which is equivalent to the term h_{ri} in Article 14.7.5.1.
- W = The total vertical load for design of the isolation system (DL + LL_s).
- D = Total deck displacement relative to ground ($D_i + D_{sub}$).
- D_i = Design displacement at the center of rigidity of the isolation system in the direction under consideration.
- D_{os} = Offset displacement of the isolator unit, including creep, shrinkage, and 50 percent of the thermal displacement.
- D_{sub} = Substructure displacement.
- D_t = Total design displacement.
- D_n = Maximum negative displacement of an isolator unit during each cycle of prototype testing.
- D_p = Maximum positive displacement of an isolator unit during each cycle of prototype testing.
- D_s = Shear deformation of bearing from non-seismic displacement of the superstructure (including temperature, shrinkage, and creep).
- β = Equivalent viscous damping ratio for the isolation system.
- β_i = Equivalent viscous damping ratio for isolator.
- g_c = Shear strain due to vertical loads.

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$g_{s,eq}$ = Shear strain due to D_t , the total seismic design displacement.

$g_{s,s}$ = Shear strain due to maximum horizontal displacement resulting from creep, post-tensioning, shrinkage, and thermal effects computed between the installation temperature and the least favorable extreme temperature.

g_r = Shear strain due to imposed rotation.

q = Rotation imposed on bearing.

l_{max}, l_{min} = System property modification factors to account for effects of temperature, aging, scragging, velocity, and variability of materials (Article 15.5.2).

15.4 ANALYSIS PROCEDURES

Article 4.8.5 shall be used to define the analysis procedures.

The analysis of the bridge shall be performed using the design properties of the isolation system. To simplify the nonlinear behavior of the isolator unit, a bilinear simplification may be used. The analysis shall be repeated using upper-bound properties ($Q_{d,max}$, $K_{d,max}$) in one analysis and lower-bound properties ($Q_{d,min}$, $K_{d,min}$) in another, where the maximum and minimum values are defined in Article 15.5.1.2. The purpose of this upper- and lower-bound analysis is to determine the maximum forces on the substructure elements and the maximum displacements of the isolation system.

An upper- and lower-bound analysis is not required if the displacements, using Equation 15-3, and the statically equivalent seismic force, using Equations 15-1 and 15-2a, do not vary from the design values by more than ± 15 percent when the maximum and minimum values of the isolator units properties are used. For these simplified calculations, B values corresponding to more than 30-percent damping can be used to establish the ± 15 -percent limits.

A nonlinear time-history analysis is required for structures with effective periods greater than 3 seconds.

For isolation systems where the effective damping expressed as a percentage of critical damping exceeds 30 percent of critical, a three-dimensional nonlinear time-history analysis shall be performed utilizing the hysteresis curves of the isolation system.

COMMENTARY

C15.4 ANALYSIS PROCEDURES

The basic premise for the analysis (consistent with those for buildings and hospitals) is twofold. First, the energy dissipation of the isolation system can be expressed in terms of equivalent viscous damping; and second, the stiffness of the isolation system can be expressed as an effective linear stiffness. These two basic assumptions permit both the single and multimodal methods of analysis to be used for seismic isolation design.

The force deflection characteristics of a bilinear isolation system (Figure C15.1-4) have two important variables, some of which are influenced by environmental and temperature effects. The key variables are K_d , the stiffness of the second slope of the bilinear curve, and Q_d , the characteristic strength. The area of the hysteresis loop, EDC, and hence the damping coefficient, are affected primarily by Q_d . The effective stiffness K_{eff} is influenced by Q_d and K_d .

The two important design variables of an isolation system are K_{eff} and B , the damping coefficient, since they affect the period (Equation 15-4), the displacement (Equation 15-3), and the base shear forces (Equation 15-2). Since K_{eff} and B , the damping coefficient, are affected differently by K_d and Q_d , the impact variations in K_d and Q_d have on the key design variables needs to be assessed (Figure C15.4-1). Article 15.5 provides a method to determine l_{min} and l_{max} values for both K_d and Q_d .

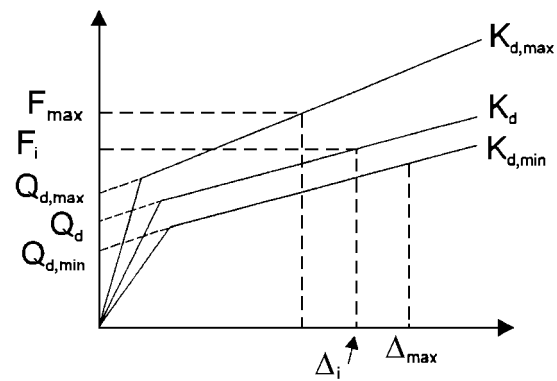


Figure C15.4-1 Impact Variations on Key Design Variables

The design forces on the columns and abutments generally will be at their maximum value when both K_d and Q_d are their maximum values. Therefore, an analysis is required using $Q_{d,max}$ and $K_{d,max}$ to determine the maximum forces that will occur on the substructures. The design displacements will be at their maximum value when both Q_d and K_d are at their minimum values. Therefore, an analysis is required using $Q_{d,min}$ and $K_{d,min}$ to determine the maximum displacements that will occur across the isolator units.

Using the design properties of the isolator units, Q_d and K_d (Figures C15.1-4 and C15.4-1), the design forces F_i and displacements D_i are first calculated with Equations 15.4.1-1, 15.4.1-2a, and 15.4.1-3. The design properties K_d and Q_d are then multiplied by $I_{max,Kd}$, $I_{max,Qd}$, $I_{min,Kd}$, and $I_{min,Qd}$ as prescribed in Article 15.5.1.2 to obtain upper- and lower-bound values of K_d and Q_d . The analyses are then repeated using the upper-bound values, $K_{d,max}$ and $Q_{d,max}$ to determine F_{max} , and the lower-bound values $K_{d,min}$ and $Q_{d,min}$ to determine D_{max} . These upper- and lower-bound values account for all anticipated variations in the design properties of the isolation system resulting from temperature, aging, scragging, velocity, wear or travel, and contamination. The exception is that only one analysis is required using the design properties, provided that the maximum and minimum values of the forces and displacements are within ± 15 percent of the design values.

The I_{max} and I_{min} factors for each of the six variables are to be determined by the system characterization tests prescribed in Article 15.10.1, or the default values given in appendix 15A.

The prototype tests of Article 15.10.2 are required to

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validate the design properties of the isolation system. Prototype tests do not include any of the variables from the characterization tests that affect the design properties of the isolation system, because they are incorporated in the design process through the use of system property modification factors.

In order to provide guidance on some of the available systems, potential variations in the key parameters are as follows:

- Lead-Rubber Isolator Unit – The value of Q_d is influenced primarily by the lead core. In cold temperatures, natural rubber will cause the most significant increase in Q_d . The value of K_d depends on the properties of the rubber. Rubber properties are affected by aging, frequency of testing, strain, and temperature.
- High-Damping Rubber Isolator Unit – The value of Q_d is a function of the additives to the rubber. The value of K_d is also a function of the additives to the rubber. High-damping rubber properties are affected by aging, frequency of testing, strain, temperature, and scragging.
- Friction Pendulum System® – The value of Q_d is a function primarily of the dynamic coefficient of friction. The value of K_d is a function of the curvature of the sliding surface. The dynamic coefficient of friction is affected by aging, temperature, velocity of testing, contamination, and length of travel or wear.
- Eradiquake® – The value of Q_d is a function of the dynamic coefficient of the disc bearing and the preload friction force, when it is used. The value of K_d is a function of whatever springs are incorporated in the device. The dynamic coefficient of friction is affected by aging, temperature, velocity of testing, contamination, and length of travel or wear. The variations in spring properties depend on the materials used.
- Viscous Damping Devices – These can be used in conjunction with either elastomeric bearings or sliders. The value of Q_d is a function of both the viscous damper and the bearing element. The value of K_d is primarily a function of the bearing element.

15.4.1 Capacity Spectrum Method

This method of analysis can be used when the regularity requirements of Article 4.8.5.3.1 are met.

C15.4.1 Capacity Spectrum Method

The capacity spectrum method of Article 3.10.3.4 and Article 4.8.5.1 is based on the same principles

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regularity requirements of Article 4.8.5.3.1 are met.

The statically equivalent seismic force is given by

$$F = C_s W \quad (15.4.1-1)$$

The elastic seismic response coefficient, C_s , used to determine the equivalent force, is given by the dimensionless relationship

$$C_s = \frac{K_{eff} \times \Delta}{W} \quad (15.4.1-2)$$

$$C_s = \frac{F_v S_l}{T_{eff} B} \quad (15.4.1-2a)$$

The displacement Δ is given by

$$\Delta = \frac{0.25 F_v S_l T_{eff}}{B} \quad (m) \quad (15.4.1-3a)$$

$$\Delta = \frac{10 F_v S_l T_{eff}}{B} \quad (inches) \quad (15.4.1-3b)$$

$$T_{eff} = 2p \sqrt{\frac{W}{K_{eff} g}} \quad (15.4.1-4)$$

Note: This method of analysis shall not be used if Type E and F soils are present. For systems that include a viscous damper, the maximum force in the system may not correspond to the point of maximum displacement (Equation 15.4.1-1). The procedure described in the commentary shall be used.

Table 15.4.1-1 Damping coefficient B_L

	Damping (Percentage of Critical)*						
	≤2	5	10	20	30	40	50
B	0.8	1.0	1.2	1.5	1.7	1.9	2.0

*The percentage of critical damping depends on the energy dissipated and stored by the isolation system, which shall be determined by test of the isolation system's characteristics, and by the substructure. The damping coefficient shall be based on linear interpolation for damping levels other than those given. Note that for isolation systems where the effective damping exceeds 30 percent, a nonlinear time-history analysis shall be performed utilizing the hysteresis curves of the system.

COMMENTARY

used in the original derivation of the simplified seismic isolation design approach. The only difference is the sequence in which it is applied. For non-isolated bridges, it is recommended that a designer sum the strength of the columns to obtain C_s and then determine if the displacement capacity of the columns is adequate using Equation 4.8.5.1-1. If not, the columns must be strengthened. In an isolation design the bridge achieves its single degree of freedom response characteristics by virtue of using flexible isolation bearings rather than having columns of very similar stiffness characteristics. The design procedure uses the stiffness characteristics of the isolation bearings sized to resist the non-seismic loads to determine the design displacement (Equation 15.4.1-3). The lateral force that the substructure must resist is then calculated using Equation 15.4.1-2 where K_{eff} is the sum of the effective linear stiffnesses of all bearings and substructures supporting the superstructure; and C_s is the lateral force coefficient. The derivation of the isolation design equations follows.

For the design of conventional bridges, the form of the elastic seismic coefficient in the longer period segment of the spectra is

$$C_s = \frac{F_v S_l}{T}$$

For seismic isolation design, the elastic seismic coefficient is directly related to the elastic ground-response spectra and damping of the isolation system.

$$C_s = \frac{F_v S_l}{T_{eff} B}$$

where B is the damping coefficient given in Table 15.4.1-1. Note that for 5 percent damping, $B = 1.0$.

The quantity C_s is a dimensionless design coefficient, which when multiplied by g produces the spectral acceleration. This spectral acceleration (S_A) is related to the spectral displacement (S_D) by the relationship

$$S_A = w^2 S_D$$

where w is the circular natural frequency and is given by $2\pi/T_{eff}$. Therefore, since $S_A = C_s \cdot g$

$$S_A = \frac{F_v S_l}{T_{eff} B} g$$

and

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curves of the system.

COMMENTARY

$$\begin{aligned}
 S_D &= \frac{1}{w^2} \frac{F_v S_l}{T_{eff} B} g \\
 &= \frac{T_{eff}^2 F_v S_l}{(2p)^2 T_{eff} B} (9.81) \frac{\text{m}}{\text{sec}^2}; \frac{T_{eff}^2 F_v S_l}{(2p)^2 T_{eff} B} (386.4) \frac{\text{inches}}{\text{sec}^2} \\
 &= \frac{0.249 F_v S_l T_{eff}}{B} \text{m}; \frac{9.79 F_v S_l T_{eff}}{B} \text{inches}
 \end{aligned}$$

Denoting S_D as d (Article 15.4), which is the deck displacement relative to the ground, the above is approximated by

$$\Delta = \frac{0.25 F_v S_l T_{eff}}{B} \text{m}; \frac{10 F_v S_l T_{eff}}{B} \text{inches}$$

An alternate form for C_s is possible. The quantity C_s is defined by the relationship

$$F = C_s W$$

where F is the earthquake design force and W is the weight of the structure. Therefore,

$$C_s = \frac{F}{W} = \frac{K_{eff} \times \Delta}{W}$$

where K_{eff} is the sum of the effective linear springs of all bearings supporting the superstructure segment. The equivalence of this form to the previous form is evident by observing that $K_{eff} = w^2 W/g$, from which

$$\begin{aligned}
 C_s &= \frac{w^2 W}{g} \times \frac{d}{W} = \frac{(2p)^2}{T_{eff}^2} \times \frac{1}{9.81} \times \frac{0.25 F_v S_l T_{eff}}{B} = \frac{F_v S_l}{B T_{eff}}; \\
 C_s &= \frac{w^2 W}{g} \times \frac{d}{W} = \frac{(2p)^2}{T_{eff}^2} \times \frac{1}{386.4} \times \frac{9.79 F_v S_l T_{eff}}{B}
 \end{aligned}$$

In calculating the effective stiffness, the configuration, flexibility, and individual stiffnesses of the isolator units (k_{iso}) and substructure (k_{sub}) shall be taken into account.

$$K_{eff} = \sum_j \left(\frac{k_{sub} k_{iso}}{k_{sub} + k_{iso}} \right) = \sum_j K_{eff,j}$$

where the sum Σ extends over all substructures.

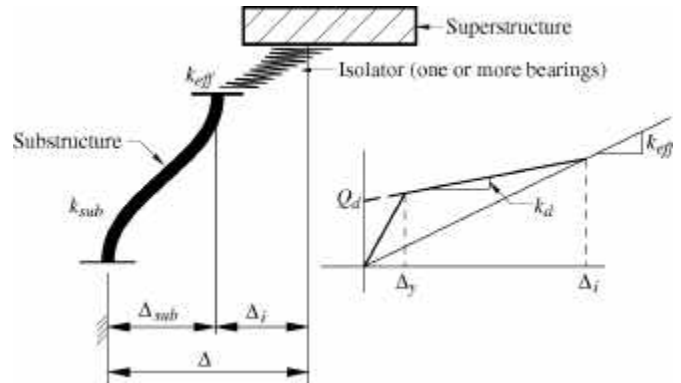


Figure C15.4.1-1 (figure shows only one isolator and one substructure)

The corresponding equivalent viscous damping may be calculated as follows:

$$b = \frac{\text{Energy Dissipated}}{2p K_{eff} \Delta^2} = \frac{\text{Total Dissipated Energy}}{2p \sum_j (K_{eff,j} \Delta^2)}$$

$$b = \frac{2Q_d(\Delta_i - \Delta_y)}{p(\Delta_i + \Delta_{sub})^2 K_{eff}} = \frac{2 \sum_j [Q_d(\Delta_i - \Delta_y)]}{p \sum_j [K_{eff,j}(\Delta_i + \Delta_{sub})^2]}$$

$$\text{Hysteretic Energy Dissipated at Isolator} = 4Q_d(D_i - D_y)$$

Note: These equations exclude contribution to damping from the substructure.

Damping coefficients were adopted from the 1994 Uniform Building Code.

If damping is truly linear viscous, then damping coefficient in Table 15.4.1-1 may be extended to 50 percent ($B=2$).

If damping exceeds 30 percent, and a B of 1.7 is used, then a time-history analysis is not required.

Equations 15.4.1-1 and 15.4.1-2 are strictly applicable to hysteretic systems, that is, systems without added damping of truly viscous nature such as viscous dampers.

For systems with added viscous damping, as in the case of elastomeric or sliding systems with viscous dampers, Equations 15.4.1-3a and 15.4.1-3b are valid, provided that the damping coefficient B is based on the energy dissipated by all elements of the isolation system, including the viscous dampers. Equivalent damping shall be determined by Equation 15.10.3-2. The seismic force shall be determined in three distinct stages as follows:

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15.4.2 Uniform Load Method

The statically equivalent force determined according to Article 15.4.1, which is associated with the displacement across the isolation bearings, shall be applied using the uniform load method of analysis described in Article 4.8.5.3.3 independently along two perpendicular axes and combined as specified in Article 3.10.2.4. The effective stiffness of the isolators used in the analysis shall be calculated at the design displacement.

15.4.3 Multimode Spectral Method

An equivalent linear response spectrum shall be performed using the requirements of Article 4.8.5.3.4

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1. At the stage of maximum bearing displacement. The seismic force shall be determined by Equation 15.4.1-1. Note that at this stage, the viscous damping forces are zero.
2. At the stage of maximum velocity and zero bearing displacement. The seismic force shall be determined as the combination of characteristic strength of the isolation bearings and the peak viscous damper force. The latter shall be determined at a velocity equal to $2\pi d_d/T_{eff}$, where d_d is the peak damper displacement. (Note that displacement d_d is related to bearing displacement D).
3. At the stage of maximum total inertia force (that is, superstructure acceleration). The seismic force shall be determined by

$$F = (f_1 + 2 b_d f_2) C_s W$$

where C_s is determined by Equation 15.4.1-2; K_{eff} is determined from the contribution of all elements of the isolation system other than viscous dampers; b_d is the portion of the effective damping ratio of the isolated bridge contributed by the viscous dampers and

$$f_1 = \cos [\tan^{-1} (2\beta_d)]$$

$$f_2 = \sin [\tan^{-1} (2\beta_d)]$$

The modified equation provides an estimate of the maximum total inertia force on the bridge superstructure. The distribution of this force to elements of the substructure shall be based on bearing displacements equal to $f_1 D_i$, and substructure displacements equal to $f_1 D_{sub}$, and damper velocities equal to $f_2 (2\pi d_d/T_{eff})$ where d_d is the peak damper displacement.

C15.4.2 Uniform Load Method

The uniform load method of analysis given in Article 4.8.5.3.3 is appropriate for seismic isolation design.

C15.4.3 Multimode Spectral Method

The guidelines given in Article 4.8.5.3.4 are appropriate for the response spectrum analysis of an

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when required by the regularity limitations of Article 4.8.5.3.1. The 5% damped spectra may be scaled by the damping coefficient (B), as defined in Article 15.4.1, to represent the actual seismic hazard and the effective damping of the isolation system for the isolated modes. Scaling by the damping coefficient B shall apply only for periods greater than $0.8 T_{eff}$. The 5-percent ground-motion response spectra shall be used for all other modes. The effective linear stiffness of the isolators shall correspond to the design displacement.

The combination of orthogonal seismic forces shall be as specified in Article 3.10.2.4.

15.4.4 Time-History Method

For isolation systems requiring a time-history analysis, the following requirements and Article 4.8.5.5 shall apply:

- (a) The isolation system shall be modeled using the nonlinear deformational characteristics of the isolators determined and verified by test in accordance with the requirements of Article 15.10.
- (b) Pairs of horizontal ground-motion time-history components shall be selected from no fewer than three earthquakes as required by Article 3.10.2.5.
- (c) Time-history analysis shall be performed with at least three appropriate pairs of horizontal time-history components.

Each pair of time histories shall be applied simultaneously to the model. The maximum displacement of the isolation system shall be calculated from the vectorial sum of the orthogonal displacements at each time step.

The parameter of interest shall be calculated for each time-history analysis. If three time-history analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history

COMMENTARY

isolated structure with the following modifications:

- (a) The isolation bearings are modeled by use of their effective stiffness properties determined at the design displacement D_i (Figure C15.1-4).
- (b) The ground response spectrum is modified to incorporate the effective damping of the isolated structure (Figure C15.1-5).

The response spectrum required for the analysis needs to be modified to incorporate the higher damping value of the isolation system. This modified portion of the response spectrum should only be used for the isolated modes of the bridge and will then have the form shown in figure C15.1-5.

The effective damping of the structure system shall be used in the multimode spectral analysis method. Structure system damping shall include all structural elements and be obtained by rational method as discussed in C15.4.1.

C15.4.4 Time-History Method

When a time-history analysis is required, the ground-motion time histories may be frequency scaled so they closely match the appropriate ground-response spectra for the site.

A two-dimensional nonlinear analysis may be used on normal structures without skews or curves.

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analyses are performed, then the average value of the response parameter of interest may be used for design.

15.5. DESIGN PROPERTIES OF THE ISOLATION SYSTEM

15.5.1 Nominal Design Properties

The minimum and maximum effective stiffness of the isolation system (K_{min} and K_{max}) shall be determined from the minimum and maximum values of K_d and Q_d .

The minimum and maximum values of K_d and Q_d shall be determined as follows:

$$K_{d,max} = K_d \cdot I_{max,K_d} \quad (15.5.1-1)$$

$$K_{d,min} = K_d \cdot I_{min,K_d} \quad (15.5.1-2)$$

$$Q_{d,max} = Q_d \cdot I_{max,Q_d} \quad (15.5.1-3)$$

$$Q_{d,min} = Q_d \cdot I_{min,Q_d} \quad (15.5.1-4)$$

System property modification factors (λ) (defined in Article 15.5.2) used for design shall be established by system characterization tests and approved by the engineer. In lieu of the test values, the λ values given in Appendix 15A may be used.

For an explanation of the system property modification factors concept, see Constantinou et al. (1999).

15.5.2 System Property Modification Factors (I)

The mechanical properties of the isolator units are affected by temperature, aging, scragging, velocity, travel, and contamination.

15.5.2.1 MINIMUM AND MAXIMUM SYSTEM PROPERTY MODIFICATION FACTORS

$$I_{min,K_d} = I_{min,t,K_d} \cdot I_{min,a,K_d} \cdot I_{min,v,K_d} \cdot I_{min,tr,K_d} \cdot I_{min,c,K_d} \cdot I_{min,scrag,K_d} \quad (15.5.2-1)$$

$$I_{max,K_d} = I_{max,t,K_d} \cdot I_{max,a,K_d} \cdot I_{max,v,K_d} \cdot I_{max,tr,K_d} \cdot I_{max,c,K_d} \cdot I_{max,scrag,K_d} \quad (15.5.2-2)$$

$$I_{min,Q_d} = I_{min,t,Q_d} \cdot I_{min,a,Q_d} \cdot I_{min,v,Q_d} \cdot I_{min,tr,Q_d} \cdot I_{min,c,Q_d} \cdot I_{min,scrag,Q_d} \quad (15.5.2-3)$$

$$I_{max,Q_d} = I_{max,t,Q_d} \cdot I_{max,a,Q_d} \cdot I_{max,v,Q_d} \cdot I_{max,tr,Q_d} \cdot I_{max,c,Q_d} \cdot I_{max,scrag,Q_d} \quad (15.5.2-4)$$

C15.5.2.1

All I_{min} values are unity at this time. The Task Group that developed these provisions determined that available test data for I_{min} values would produce forces and displacements that are within 15 percent of the design values. If the engineer believes a particular system may produce displacements outside of the ± 15 -percent range, then a I_{min} analysis should be performed.

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where:

- l_t = Factors to account for effects of temperature
- l_a = Factors to account for effects of aging (including corrosion)
- l_v = Factors to account for effects of velocity (including frequency for elastomeric systems)
- $l_v = \frac{\text{Property value at relevant velocity}}{\text{Property value at velocity of testing}}$
- l_{tr} = Factors to account for effects of travel (wear)
- l_c = Factors to account for effects of contamination (in sliding systems)
- l_{scrag} = Factors to account for effects of scragging a bearing (in elastomeric systems)

15.5.2.2 SYSTEM PROPERTY ADJUSTMENT FACTORS

Adjustment factors are applied to individual λ factors to account for the probability of occurrence. The following adjustment factors shall apply to all λ factors except λ_v :

1.0 for operational bridges

0.67 for all other bridges

The adjustment factors shall apply to the portion of a λ that deviates from unity.

15.6 CLEARANCES

The clearances in the two orthogonal directions shall be the maximum displacement determined in each direction from the analysis. The clearance shall not be less than

$$\frac{0.20F_v S_l T_{eff}}{B} (\text{m}) \quad (15.6-1a)$$

$$\frac{8F_v S_l T_{eff}}{B} (\text{inches}) \quad (15.6-1b)$$

or 1 inch (25 mm), whichever is greater.

COMMENTARY

C15.5.2.2 SYSTEM PROPERTY ADJUSTMENT FACTORS

It is the opinion of the Task Group that developed these provisions that only critical bridges need to consider all maximum λ factors at the same time. The reduction factors for essential and other bridges are based on engineering judgment.

Example:

$$\lambda_{\max,c} = 1.2 \text{ without adjustment factor}$$

$$\lambda_{\max,c} = 1 + (1.2 - 1) 0.67 = 1.13 \text{ for adjustment factor of 0.67}$$

C15.6 CLEARANCES

Adequate clearance shall be provided for the displacements resulting from the seismic isolation analysis in either of the two orthogonal directions. As a design alternate in the longitudinal direction, a knock-off abutment detail (Figure C2.5.6) may be provided for the seismic displacements between the abutment and deck slab. Adequate clearance for the seismic displacement must be provided between the girders and the abutment. In addition, the design rotation capacity of the bearing shall exceed the maximum seismic rotation.

The purpose of the minimum clearance default value is to guard against analysis procedures that produce excessively low clearances.

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Displacements in the isolators resulting from load combinations involving LF, W, WL, CF, and T shall be calculated and adequate clearance provided.

The minimum design forces shall be consistent with the clearances calculated with Equation 15.6-1.

15.7 DESIGN FORCES FOR SDAP A1 AND A2

The seismic design force for the connection between superstructure and substructure at each bearing is given by

$$F_A = k_{eff}D \quad (15.7-1)$$

where

D shall be based on a minimum value of $F_v S_i$, not less than 0.25.

15.8 DESIGN FORCES FOR SDAP C, D, AND E

The seismic design force for columns and piers shall not be less than the forces resulting from the yield level of a softening system, the friction level of a sliding system, or the ultimate capacity of a sacrificial service restraint system. In all cases the larger of static or dynamic conditions shall apply.

If the elastic foundation forces are less than the forces resulting from column hinging, they may be used for the foundation design. The foundation shall be designed using an R value equal to 1.0.

The seismic design force for the connection between the superstructure and substructure at each bearing is given by

$$F_a = k_{eff} D_t \quad (15.8-1)$$

Where D_t is the total design displacement and includes D_i the center of mass displacement plus any displacement resulting from torsional effects.

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excessively low clearances.

Displacements in the isolators resulting from longitudinal forces, wind loads, centrifugal forces, and thermal effects will be a function of the force-deflection characteristics of the isolators. Adequate clearance at all expansion joints must be provided for these movements.

C15.7 DESIGN FORCES FOR SDAP A1 AND A2

This section permits utilization of the real elastic force reduction provided by seismic isolation. It should be noted, however, that $F_v S_i$ has a maximum value of 0.25 for SDAP A bridges and is specified to have a minimum value of 0.25 if seismic isolation is used.

C15.8 DESIGN FORCES FOR SDAP C, D, AND E

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15.9 OTHER REQUIREMENTS

15.9.1 Non-Seismic Lateral Forces

The isolation system must resist all non-seismic lateral load combinations applied above the isolation interphase. Such load combinations are those involving W, WL, LF, CF, and T.

15.9.1.1 SERVICE FORCE RESISTANCE

Resistance to forces such as wind, centrifugal, and braking, and forces induced by restraint of thermal displacements, shall be established by testing in accordance with Article 15.10.2.

15.9.1.2 COLD WEATHER REQUIREMENTS

Cold weather performance shall be considered in the design of all types of isolation systems. Low-temperature zones shall conform with Figure 14.7.5.2-1 in the absence of more site-specific data.

15.9.2 Lateral Restoring Force

The isolation system shall be configured to produce a lateral restoring force such that the period corresponding to its tangent stiffness based on the restoring force alone at any displacement, Δ , up to its design displacement shall be less than 6 seconds (figure C15.9.2-1). Also the restoring force at D_i shall be greater than the restoring force at $0.5 D_i$ by not less than $W/80$. Isolation systems with constant restoring force need not satisfy the requirements above. In these cases, the combined constant restoring force of the isolation system shall be at least equal to 1.05 times the characteristic strength of the isolation system under service conditions.

Forces that are not dependent on displacements, such as viscous forces, may not be used to meet the minimum restoring force or tangent stiffness requirements.

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C15.9 OTHER REQUIREMENTS

C15.9.1 Non-Seismic Lateral Forces

Since an element of flexibility is an essential part of an isolation system, it is also important that the isolation system provide sufficient rigidity to resist frequently occurring wind and other service loads. The displacements resulting from non-seismic loads need to be checked.

C15.9.1.2 COLD WEATHER REQUIREMENTS

Low temperatures increase the coefficient of friction on sliding systems and the shear modulus and characteristic strength of elastomeric systems. These changes increase the effective stiffness of the isolation system.

The test temperatures used to determine low-temperature performance in Article 15.10.1 represent 75 percent of the difference between the base temperature and the extreme temperature in Table 14.7.5.2-2.

C15.9.2 Lateral Restoring Force

The basic premise of these seismic isolation design provisions is that the energy dissipation of the system can be expressed in terms of equivalent viscous damping and the stiffness by an effective linear stiffness. The requirement of this section provides the basis for which this criteria is met.

The purpose for the lateral restoring force requirement is to prevent cumulative displacements and to accommodate isolator installation imperfections, such as out of level.

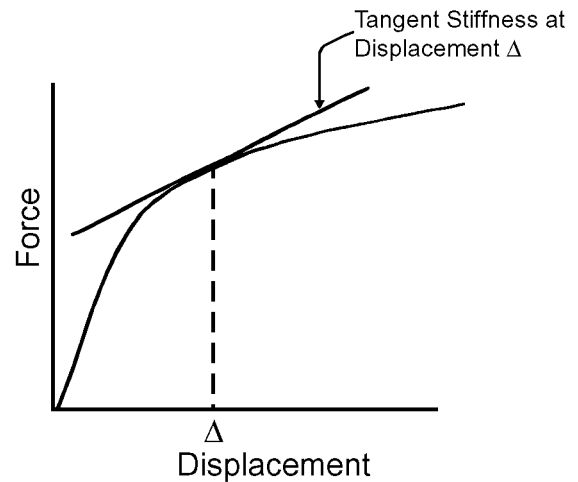


Figure C15.9.2-1 Tangent Stiffness of Isolation System

The lateral restoring force requirements are applicable to systems with restoring force that is dependent on displacement, that is, spring-like restoring force. However, it is possible to provide constant restoring force that is independent of displacement. There are two known means for providing constant restoring force: (a) using compressible fluid springs with preload and (b) using sliding bearings with a conical surface. Figure C15.9.2-2 illustrates a typical force-displacement relation of these devices.

The requirement for lateral restoring force in these cases is that the combined constant lateral restoring force of the isolation system is at least equal to 1.05 times the combined characteristic strength of the isolation system under service conditions. For example, when constant restoring force devices are combined with frictional elements (e.g., sliding bearings), the restoring force must be at least equal to 1.05 times the static friction force. This requirement ensures that the restoring force is sufficiently large to overcome the characteristic strength and, thus, provide re-centering capability.

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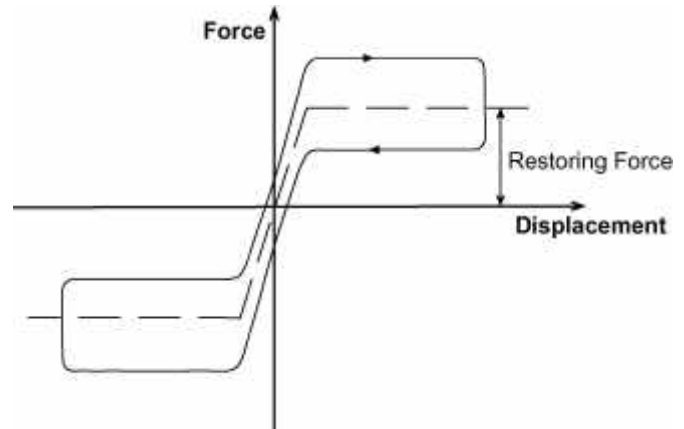


Figure C15.9.2-2 Force-Displacement Relation of Systems with Constant Restoring Force

15.9.3 Vertical Load Stability

The isolation system shall provide a factor of safety of at least three (3) for vertical loads (dead load plus live load) in its laterally undeformed state. It shall also be designed to be stable under 1.2 times the dead load plus any vertical load resulting from seismic live load, plus overturning at a horizontal displacement equal to the offset displacement plus 1.1 times the total design displacement.

C15.9.3 Vertical Load Stability

This section provides minimum requirements for the design of the isolation system. The detailed design requirements of the system will be dependent on the type of system. The 1.2 factor accounts for vertical acceleration effects and uncertainty in the dead load.

15.9.4 Rotational Capacity

The design rotation capacity of the isolation unit shall include the effects of dead load, live load, and construction misalignments. In no case shall the design rotation for the construction misalignment be less than 0.005 radians.

C15.9.4 Rotational Capacity

Larger construction rotations may be allowed, provided that they do not damage the isolator unit.

15.10 REQUIRED TESTS OF ISOLATION SYSTEMS

All isolation systems shall have their seismic performance verified by testing. In general, there are three types of tests to be performed on isolation systems: (1) system characterization tests, described in Article 15.10.1; (2) prototype tests, described in Article 15.10.2; and (3) quality control tests, described in Articles 15.12, 15.14 and 15.15.

C15.10 REQUIRED TESTS OF ISOLATION SYSTEMS

The code requirements are predicated on the fact that the isolation system design is based on tested properties of isolator units. This section provides a comprehensive set of prototype tests to confirm the adequacy of the isolator properties used in the design. Systems that have been previously tested with this specific set of tests on similar type and size of isolator units do not need to have these tests repeated. Design properties must therefore be based on manufacturers' preapproved or certified test data. Extrapolation of design properties from tests of similar type and size of isolator units is permissible.

Isolator units used for the system characterization

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15.10.1 System Characterization Tests

The fundamental properties of the isolation system shall be evaluated by testing prior to its use. The purpose of system characterization tests is to substantiate the properties of individual isolator units as well as the behavior of an isolation system. Therefore, these tests include both component tests of individual isolator units and shake table tests of complete isolation systems.

At a minimum, these tests shall consist of

- Tests of individual isolator units in accordance with the National Institute of Standards and Technology (NIST) guidelines or the Highway Innovative Technology Evaluation Center (HITEC) guidelines.
- Shaking table tests at a scale no less than 1/4 full scale. Scale factors must be well-established and approved by the engineer.

15.10.1.1 LOW-TEMPERATURE TEST

If the isolators are for low-temperature areas, then the test specified in section 15.10.2(b)(6) shall be performed at temperatures of 20, 5, –5, or –15 degrees F (–7, –15, –21, or –26 degrees C) for temperature zones A, B, C, and D, respectively.

The specimen shall be cooled for a duration not less than the maximum number of consecutive days below freezing specified in Table 14.7.5.2-2.

15.10.1.2 WEAR AND FATIGUE TESTS

Wear or travel and fatigue tests are required to account for movements resulting both from imposed thermal displacements and live load rotations. Thermal displacements and live load rotations shall correspond to at least 30 years of expected movement. Tests shall be performed at the design contact pressure at 68 degrees F $\pm$ 15 degrees (20 degrees C $\pm$ 8 degrees). The rate of application shall not be less than 2.5 inches/minute (63.5 mm/minute). As a minimum, the following displacements shall be used for the test:

- Bearings: 1 mile (1.6 km)
- Dampers (attached to the web at the neutral axis): 1 mile (1.6 km)

COMMENTARY

tests (except shaking table), prototype tests, and quality control tests shall have been manufactured by the same manufacturer with the same materials.

C15.10.1 System Characterization Tests

These tests are usually not project specific. They are conducted to establish the fundamental properties of individual isolator units as well as the behavior of an isolation system. They are normally conducted when a new isolation system or isolator unit is being developed or a substantially different version of an existing isolation system or isolator unit is being evaluated.

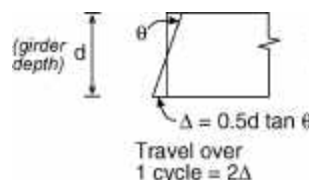
Several guidelines for these tests have been developed. The NIST Guidelines are currently being further developed into the ASCE Standard for Testing Seismic Isolation Systems, Units, and Components. This new standard currently exists in draft form. Testing guidelines have also been developed for the HITEC evaluation of seismic isolation and energy dissipation devices.

C15.10.1.1 LOW-TEMPERATURE TEST

The test temperatures represent 75 percent of the difference between the base temperature and the extreme temperature in Table 14.7.5.2-2. Prior to testing, the core temperature of the isolator unit shall reach the specified temperature.

C15.10.1.2 WEAR AND FATIGUE TESTS

The movement that is expected from live load rotations is dependent on structure type, span length and configuration, girder depth, and average daily traffic. The total movement resulting from live load rotations can be calculated as follows:



Travel over 1 cycle = 2Δ

$\Delta = 0.5d \tan \theta$

Total Travel = $2 (0.5d \tan \theta) \times N \times 24 \times 365 \times 30$

↑ ↑ ↑ ↑
travel per live load per per total for
live load per hour day year 30 years

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mile (1.6 km)

- Dampers (attached to the girder bottom):
2 miles (3.2 km)

Additional wear or travel and fatigue will occur in long structures with greater thermal movements, high traffic counts, and lively spans.

If the isolator units are for low-temperature areas, then 10 percent of the test shall be performed at temperatures of 20, 5, –5, or –15 degrees F (–7, –15, –21, –26 degrees C) for temperature zones A, B, C, and D, respectively.

In lieu of the low-temperature test criteria, the components may be tested for a cumulative travel of twice the calculated service displacements or twice the values above when approved by the engineer.

15.10.2 Prototype Tests

The deformation characteristics and damping values of the isolation system used in the design and analysis shall be verified by prototype tests. Tests on similarly sized isolator units may be used to satisfy the requirements of this section. Such tests must validate design properties that can be extrapolated to the actual sizes used in the design.

- Prototype tests shall be performed on a minimum of two full-size specimens of each type and size similar to that used in the design. The test specimens shall include the elastic restraint system if such a system is used in the design. Prototype test specimens may be used in construction, if they have the specified stiffness and damping properties and they satisfy the project quality control tests after having successfully completed all prototype tests. All sacrificial elements shall be replaced prior to use.

Reduced-scale prototype specimens will only be allowed when full-scale specimens exceed the capacity of existing testing facilities and approval is granted by the engineer of record.

If reduced-scale prototype specimens are used to quantify properties of isolator units, specimens shall be geometrically similar and of the same type and material. The specimens shall also be manufactured with the same processes and quality as full-scale prototypes, and shall be tested at a frequency that

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represents full-scale prototypes.

- The following sequence of tests shall be performed for the prescribed number of cycles at a vertical load similar to the typical or average dead load on the isolator units of a common type and size. The design displacement for these tests is defined in Article 15.4.

Test 1, Thermal – Three fully reversed cycles of loads at a lateral displacement corresponding to the maximum thermal displacement. The test velocity shall not be less than 0.003 inches per minute.

Test 2, Wind and Braking – Twenty fully reversed cycles between limits of plus and minus the maximum load for a total duration not less than 40 seconds. After the cyclic testing, the maximum load shall be held for 1 minute.

Test 3, Seismic – Three fully reversed cycles of loading at each of the following multiples of the total design displacement: 1.0, 0.25, 0.50, 0.75, 1.0, and 1.25, in the sequence shown.

Test 4, Seismic – 20 cycles of loading at 1.0 times the design displacement. The test shall be started from a displacement equal to the offset displacement.

Test 5, Wind and Braking – Three fully reversed cycles between limits of plus and minus the maximum load for a total duration not less than 40 seconds. After the cyclic testing, the maximum load shall be held for 1 minute.

Test 6, Seismic Performance Verification – Three fully reversed cycles of loading at the total design displacement.

Test 7, Stability Verification – The vertical load-carrying elements of the isolation system shall be demonstrated to be stable under one fully reversed cycle at the displacements given in Article 15.4. In these

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This test verifies the lateral force exerted by the isolation system at maximum thermal displacement.

This test verifies the resistance of the isolation system under service load conditions.

This test verifies the dynamic response of the isolation system for various displacements.

The sequence of fully reversed cycles is important to developing hysteresis loops at varying displacements. By starting with a multiple of 1.0 times the total design displacement, the performance of the unscragged and scragged bearing may be directly compared.

This verifies the survivability of the isolator after a major earthquake. The test is started from a displaced position to reflect the uncertainty of the starting position when an earthquake occurs. The seismic displacements shall be superimposed on the offset load displacement so that the peak displacements will be asymmetric.

This test verifies service load performance after a seismic event.

The seismic performance verification test verifies the performance of the bearing after the sequence of tests has been completed.

Stability is demonstrated if the isolator shows a positive incremental force carrying capacity satisfying the requirements of Article 15-4.

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tests, the combined vertical load of

$$1.2 D + LL_s + OT \quad (15.10.2-1)$$

shall be taken as the maximum downward force, and the combined vertical load of

$$0.8 D - OT \quad (15.10.2-2)$$

shall be taken as the minimum downward force.

- If a sacrificial elastic restraint system is utilized, then its ultimate capacity shall be established by test.
- The prototype and quality control tests shall include all components that comprise the isolation system.
- For systems that are not restrained to perform unidirectionally, Test 6 shall be performed in the direction of loading orthogonal to the original direction of loading. For systems that include unidirectional devices, or those that are sensitive to orthogonal effects, Test 6 shall be repeated at 45 degrees to the primary axis of the unidirectional device.
- The force-deflection properties of an isolator unit shall be considered to be dependent on the rate of loading if there is greater than a plus or minus 15-percent difference in either K_d or Q_d for the test at the design displacement when dynamically tested at any frequency in the range of 0.5 to 1.5 times the inverse of the effective period of the isolated structure.

If the force-deflection properties of the isolator units are dependent on the rate of loading, then each set of tests specified in Article 15.10.2 shall be performed dynamically at a frequency equal to the inverse of the effective period of the isolated structure. If the test can not be performed dynamically, then a λ factor must be established that relates properties K_d or Q_d determined at the actual speed of testing with the dynamic velocities in accordance with Article 15.5.2.1.

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15.10.3 Determination of System Characteristics

- The force-deflection characteristics of the isolation system shall be based on the cyclic load test results for each fully reversed cycle of loading.
- The effective stiffness of an isolator unit shall be calculated for each cycle of loading as follows:

$$k_{eff} = \frac{F_p - F_n}{\Delta_p - \Delta_n} \quad (15.10.3-1)$$

where D_p and D_n are the maximum positive and maximum negative test displacements, respectively, and F_p and F_n are the maximum positive and maximum negative forces at instance of displacements D_p and D_n , respectively.

- Equivalent Damping. The equivalent viscous damping ratio (β) of the isolation system shall be calculated as

$$b = \frac{1}{2p} \times \frac{\text{Total EDC Area}}{\sum (k_{eff} \Delta_i^2)} \quad (15.10.3-2)$$

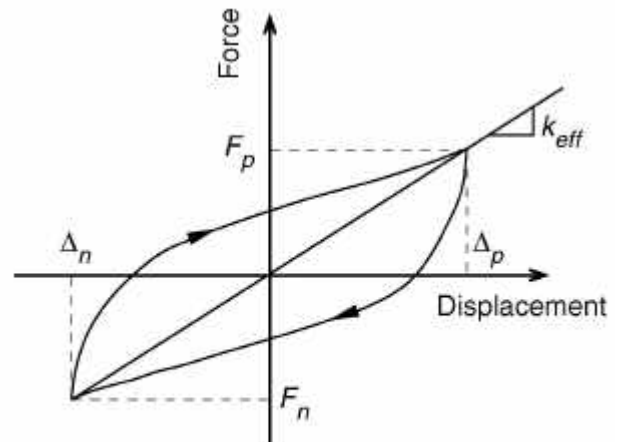
The total EDC area shall be taken as the sum of the areas of the hysteresis loops of all isolator units. The hysteresis loop area of each isolator unit shall be taken as the minimum area of the three hysteresis loops established by the cyclic tests of section 13.2(b)(3) at a displacement amplitude equal to the design displacement.

15.10.3.1 SYSTEM ADEQUACY

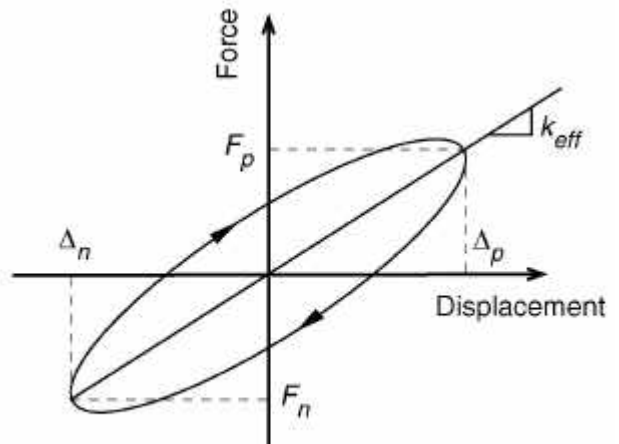
The performance of the test specimens shall be assessed as adequate if the following conditions are satisfied:

- The force-deflection plots, excluding any viscous damping component, of all tests specified in Article 15.10.2 show a positive incremental force-carrying capacity consistent with the requirements of Article 15.9.2.

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Hysteretic behavior



Viscoelastic behavior

Figure C15.10.3-1 Definition of Effective Stiffness

An isolation system needs a positive incremental force-carrying capability to satisfy the requirements of Article 15.9.2. The purpose of this requirement is to ensure that the hysteretic elements of the system are stable. A viscous damper will have a negative incremental force-carrying capacity toward the point of maximum displacement. Since this is acceptable performance, it needs to be deleted from the other components prior to their stability evaluation.

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- For Test1, the maximum measured force shall be less than the design value.
- For Tests 2 and 5, the maximum measured displacement shall be less than the design value.
- The average effective stiffness measured in the last three cycles to the total design displacement specified in Test 3 shall lie within 10 percent of the value used in design.
- For each test displacement level specified for Test 3, the minimum effective stiffness measured during the three cycles shall not be less than 80 percent of the maximum effective stiffness.
- For Test 4, the minimum effective stiffness measured during the specified number of cycles shall not be less than 80 percent of the maximum effective stiffness. At the discretion of the engineer, a larger variation may be accepted, provided that both the minimum and maximum values of effective stiffness are used in the design.
- For Test 4, the minimum EDC measured during the specified number of cycles shall not be less than 70 percent of the maximum EDC. At the discretion of the engineer, a larger variation may be accepted, provided that both the minimum and maximum values of EDC are used in the design.
- All vertical load-carrying elements of the isolation system shall remain stable (positive incremental stiffness) at the displacements specified in Article 15.9.3 for static loads as prescribed for Test 7.
- Test specimens shall be visually inspected for evidence of significant deterioration. If any deterioration exists, then the adequacy of the test specimen shall be determined by the engineer.

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If the change in effective stiffness is greater than 20 percent, the minimum effective stiffness value should be used to calculate the system displacements, and the maximum effective stiffness values should be used to calculate the structure and isolation system forces.

A decrease in stiffness during cyclic testing may occur in some systems and is considered acceptable if the degradation is recoverable within a time frame acceptable to the engineer. That is, the bearing will return to its original stiffness after a waiting period.

A decrease in EDC during cyclic testing may occur in some systems and is considered acceptable if the degradation is recoverable within a time frame acceptable to the engineer.

At the conclusion of testing, the test specimens shall be externally inspected or, if applicable, disassembled and inspected for the following faults, which shall be cause for rejection:

- (1) Lack of rubber-to-steel bond.
- (2) Laminate placement fault.
- (3) Surface cracks on rubber that are wider or deeper than 2/3 of the rubber cover thickness.
- (4) Material peeling.

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15.11 ELASTOMERIC BEARINGS

15.11.1 General

The following shall be considered supplemental to Section 14.

Elastomeric bearings utilized in implementing seismic isolation design shall be designed by the procedures and specifications given in the following subsections. Additional test requirements for seismic isolation bearings are given in Article 15.12. The design procedures are based on service loads excluding impact. The elastomeric bearings must be reinforced using steel reinforcement. Fabric reinforcement is not permitted.

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- (5) Lack of polytetrafluorethylene (PTFE)-to-metal bond.
- (6) Scoring of stainless steel plate.
- (7) Permanent deformation.
- (8) Leakage.

C15.11 ELASTOMERIC BEARINGS

Elastomeric bearings used for seismic isolation will be subjected to earthquake-induced displacements (D_i) and must therefore be designed to safely carry the vertical loads at these displacements. Since earthquakes are infrequently occurring events, the factors of safety required under these circumstances will be different from those required for more frequently occurring loads.

Since the primary design parameter for earthquake loading is the displacement (D_i) of the bearing, the design procedures must be capable of incorporating this displacement in a logical, consistent manner. The requirements of Article 14.7.5.3 limit vertical loads by use of a limiting compressive stress, and therefore do not have a mechanism for including the simultaneous effects of seismic displacements. The shear displacement is also limited to half of the elastomer thickness. The British specifications BE 1/76 and BS 5400 recognize that shear strains are induced in reinforced bearings by compression, rotation, and shear deformations. In BE 1/76, the sum of these shear strains is limited to a proportion of the elongation-at-break of the rubber. The proportion (1/2 or 1/3 for service load combinations and 3/4 for seismic load combinations) is a function of the loading type. In BS 5400 and the 1995 draft Eurocode EN 1337, the limit is a constant 5.0.

Since the approach used in BE 1/76 and BS 5400 incorporates shear deformation as part of the design criteria, it can be readily modified for seismic isolation bearings. The design requirements given are based on the appropriate modifications to BE 1/76 and BS 5400.

In the extensive testing conducted for NCHRP Report No. 298 (Roeder, Stanton, and Taylor 1987), no correlation was found between the elongation-at-break and the ability of the elastomers to resist shearing strain without debonding from the steel reinforcement. Furthermore, the French code UIC772R and the BS 5400 also imply no dependence on ϵ_u , but rather use a single limit of 5.0 for the sum of the strains, regardless of the elastomer type.

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15.11.2 Shear Strain Components for Isolation Design

The various components of shear strain in the bearing shall be computed as follows:

- Shear strain (γ_c) due to compression by vertical loads is given by

$$g_c = \frac{3SP}{2A_r G(1 + 2kS^2)} \quad (15.11.2-1)$$

if $S \leq 15$,

or

$$g_c = \frac{3P(1 + 8G\bar{k}S^2 / K)}{4G\bar{k}SA_r} \quad (15.11.2-2)$$

if $S > 15$,

where K is the bulk modulus of the elastomer. In absence of measured data, K may be taken as 300,000 psi (2,000 MPa). The shape factor S shall be taken as the plan area of the elastomer layer divided by the area of perimeter free to bulge.

- Shear strain ($\gamma_{s,s}$) due to imposed non-seismic lateral displacement is given by

$$g_{s,s} = \frac{\Delta_s}{T_r} \quad (15.11.2-3)$$

- Shear strain ($\gamma_{s,eq}$) due to earthquake-imposed lateral displacement is given by

COMMENTARY

The allowable vertical load on an elastomeric bearing is not specified explicitly. The limits on vertical load are governed indirectly by limitations on the equivalent shear strain in the rubber due to different load combinations and to stability requirements.

The effects of creep of the elastomer shall be added to the instantaneous compressive deflection, when considering long-term deflections. They are not to be included in the calculation of Article 15.11.3. Long-term deflections shall be computed from information relevant to the elastomer compound used, if it is available. If not, the values given in Article 14.7.5.3.3.

For incompressible isotropic material $E = 3G$, however, this is not true for rubber. For rubber, $E = (3.8 \text{ to } 4.4)G$ depending on its hardness, which indicates anisotropy in rubber. Accordingly, Equation 15.11.2-1 is based on Equation 8 of the 1991 AASHTO Guide Specifications with E replaced by $4G$. It should be noted that the quantity $4G(1 + 2kS^2)$ is the compression modulus of the bearing, as calculated on the assumption of incompressible rubber. For bearings with large shape factors, the assumption of incompressible rubber leads to significant overestimation of the compression modulus and, thus, underestimation of the shear strain due to compression. Equation 15.11.2-2 is introduced to account for the effects of rubber compressibility. It is based on the empirical relation that the compression modulus is given by $[1/(8G\bar{k}S^2) + 1/K]^{-1}$.

The shear modulus (G) shall be determined from the secant modulus between 25- and 75-percent shear strain in accordance with ASTM D 4014, published by the American Society of Testing and Materials.

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$$g_{s,eq} = \frac{\Delta_t}{T_r} \quad (15.11.2-4)$$

- Shear strain (g_r) due to rotation is given by

$$g_r = \frac{B_d^2 q}{2t_r T_r} \quad (15.11.2-5)$$

The design rotation (θ) shall include the rotational effects of DL, LL, and construction.

15.11.3 Load Combinations

Elastomeric bearings shall satisfy

$$g_c \leq 2.5 \quad (15.11.3-1)$$

$$g_c + g_{s,s} + g_r \leq 5.0 \quad (15.11.3-2)$$

$$g_c + g_{s,eq} + 0.5 g_r \leq 5.5 \quad (15.11.3-3)$$

The design rotation is the maximum rotation of the top surface of the bearing relative to the bottom surface. Any negative rotation due to camber will counteract the DL and LL rotation and should be included in the calculation

C15.11.3

Tests for NCHRP at the University of Washington, Seattle, have shown that static rotation is significantly less damaging than dynamic rotation.

15.12 ELASTOMERIC BEARINGS – CONSTRUCTION

15.12.1 General Requirements

The following shall be considered supplemental to article 18.2 of the AASHTO Standard Specifications (Division II). The provision of Article 15.12.2 replaces those in articles 18.2.7.6, 18.2.7.7, and 18.2.7.8 of the AASHTO Standard Specifications (Division II).

The layers of elastomeric bearings used in seismic isolation shall be integrally bonded during vulcanization. Cold bonding is not allowed.

15.12.2 Quality Control Tests

The following quality control tests shall also be performed on elastomeric bearings.

15.12.2.1 COMPRESSION CAPACITY

A 5-minute sustained proof load test shall be conducted on each bearing. The compressive load for the test shall be 1.5 times the maximum (dead load plus live load). If bulging suggests poor laminate bond, the bearing shall be rejected.

15.12.2.2 COMBINED COMPRESSION AND SHEAR

All bearings shall be tested in combined compression and shear. The bearings may be tested in

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pairs. The compressive load shall be the average dead load of all bearings of that type, and the bearings shall be subjected to five fully reversed cycles of loading at the larger of the total design displacement or 50 percent of the elastomer thickness.

For each bearing, the effective stiffness and EDC shall be averaged over the five cycles of the test. For each group of similar bearings of the same type and size, the effective stiffness and EDC shall be averaged. The results shall not differ from the design values by more than the limits given in Table 15.12.2.2-1.

Table 15.12.2.2-1

	K_{eff}	EDC
Individual Bearings	±20%	–25%
Average of Group	±10%	–15%

15.12.2.3 ACCEPTANCE CRITERIA

After quality control testing, all bearings shall be visually inspected for defects. The following faults shall be cause for rejection:

- Lack of rubber-to-steel bond.
- Laminate placement fault.
- Surface cracks on the rubber that are wider or deeper than 2/3 of the rubber cover thickness.
- Permanent deformation.

15.13 SLIDING BEARINGS – DESIGN

15.13.1 General

Sliding bearings used in isolation systems may use flat or curved surfaces.

COMMENTARY

C15.13.1 General

The sliding bearing is typically made from two dissimilar materials that slide against each other. Low friction is achieved when a softer material, usually PTFE and herein called the bearing liner, slides against a hard, smooth surface that is usually stainless steel and is herein called the mating surface. Lubrication may be used.

The restoring force may be provided either by gravity acting through a curved sliding surface or by a separate

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15.13.2 Materials

15.13.2.1 PTFE BEARING LINERS

All PTFE surfaces, other than guides, shall satisfy the requirements specified herein. The PTFE bearing liner shall be made from virgin PTFE resin satisfying the requirements of ASTM D1457. It may be fabricated as unfilled sheet, filled sheet, or fabric woven from PTFE and other fibers.

Unfilled sheets shall be made from PTFE resin alone. Filled sheets shall be made from PTFE resin uniformly blended with glass fibers, carbon fibers, or other chemically inert reinforcing fibers.

Sheet PTFE may contain dimples to act as reservoirs for lubricant. Their diameter shall not exceed 0.32 inch (8 mm) at the surface of the PTFE and their depth shall be not less than 0.08 inch (2 mm) and not more than half the thickness of the PTFE. The reservoirs should cover more than 20 percent, but less than 30 percent of the contact surface. Dimples should not be placed to intersect the edge of the contact area. Lubricant shall be silicone grease, effective to $-30\frac{1}{2}$ F (-34° C). Silicone grease shall conform to Military specification MIL-S-8660.

15.13.2.2 OTHER BEARING LINER MATERIALS

Other materials may be used for the bearing liner if test results demonstrate a stable long-term coefficient of friction, chemical stability, and wear resistance in

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device such as a spring.

C15.13.2 Materials

Certain combinations of materials have been found to promote severe corrosion and are strongly discouraged (British Standards Institution 1979; 1983). Examples are

- structural steel and brass,
- structural steel and bronze,
- structural steel and copper,
- structural steel and aluminum, and
- chromium on structural steel (chrome plating of steel).

Chrome is porous, so structural steel is exposed to oxygen.

Other combinations of materials known to promote additional but not severe corrosion are

- stainless steel and brass,
- stainless steel and bronze, and
- stainless steel and copper.

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accordance with Article 15.10.1.2, and are approved by the engineer.

15.13.2.3 MATING SURFACE

Mating surfaces shall be stainless steel (welded overlay, solid, or sheet metal). Stainless steel shall have a corrosion resistance and strength equal to or exceeding type 304, conforming to ASTM A167/A264. The average surface roughness shall not exceed 32 micro inches (0.8 micro meters) R_a (arithmetic average) as determined by procedures described in ANSI/ASME B46.1-1985 (ASME, 1985).

15.13.3 Geometry

15.13.3.1 MINIMUM THICKNESS

15.13.3.1.1 PTFE Bearing Liner

The minimum thickness for PTFE shall be at least 0.0625 inch (1.6 mm) after compression. Recessed sheet PTFE shall be at least 0.1875 inch (4.8 mm) thick when the maximum dimension of the PTFE is less than or equal to 24.0 inches (610 mm), and 0.25 inch (6.4 mm) when the maximum dimension of the PTFE is greater than 24.0 inches (610 mm). Woven fabric PTFE shall have, after compression, a minimum thickness of 0.0625 inch (1.6 mm) and a maximum thickness of 0.125 inch (3.2 mm).

15.13.3.1.2 Other Bearing Liner Materials

The minimum thickness for all other bearing liners shall be determined by conducting wear tests in accordance with Article 15.10.1.2.

15.13.3.2 Mating Surface

The thickness of the stainless steel mating surface sheet shall be at least 16 gauge when the maximum dimension of the surface is less than or equal to 12.0 inches (305 mm), and at least 13 gauge when the maximum dimension is larger than 12.0 inches (305 mm) and less than or equal to 36.0 inches (915 mm). When the maximum dimension is larger than 36.0 inches (915 mm), the thickness of the stainless steel mating surface shall be verified by performance of suitable system characterization tests.

The minimum thickness of stainless steel weld overlays shall be 3/32 inch (2.4 mm) thick after welding, grinding, and polishing.

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C15.13.2.3 MATING SURFACE

Higher grades of stainless steel such as type 316, conforming to ASTM A 240, should be considered for applications in severe corrosive environments.

Measurements of surface roughness need to be reported together with information on profilometer stylus tip radius, traversing length and instrument cutoff length. It is recommended that the stylus tip radius not be more than 200 micro inches (5 micro meters) and the cutoff length be 0.03 inches (0.8 mm).

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15.13.3.3 DISPLACEMENT CAPACITY

The mating surface dimensions shall be large enough to ensure that the sliding surface does not come into contact with the edge of the mating surface at the total design displacement plus the offset displacement.

15.13.4 Loads and Stresses

15.13.4.1 CONTACT PRESSURE

Contact stresses for bearing liners shall be established by testing. Test pressures shall be at least 110 percent of the value used in design and must satisfy the wear requirements in Article 15.10.1.2. As a minimum, 50 percent of the usable bearing liner thickness must remain after completion of the wear test. Allowable contact stresses for PTFE liners tabulated in Table 15.13.4.1-1 may be used without completing the wear test, provided that the stainless steel mating surface has a surface roughness less than 20 micro inches (0.5 micro meter) Ra.

Table 15.13.4.1-1
Allowable Average Contact Stress for PTFE

Material	Allowable Contact Stress					
	Service Loads				Seismic Loads	
	Average Stress		Edge Stress		Average Stress	
	ksi	MPa	ksi	MPa	ksi	MPa
Unfilled sheets (recessed)	3.5	24	5.0	34	6.0	41
Filled sheets (recessed)	3.5	24	5.0	34	6.0	41
Woven PTFE fiber over a metallic substrate	3.5	24	10.0	69	6.0	41

The rotation-induced edge stresses must be calculated by a rational method that accounts for the rotational stiffness and rotational demand of the bearing.

15.13.4.2 COEFFICIENT OF FRICTION

15.13.4.2.1 Service Coefficient of Friction

The service limit state coefficient of friction of the PTFE sliding surface shall be taken as specified in Table 15.13.4.2.1-1. Intermediate values may be

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determined by interpolation. The coefficient of friction shall be determined by using the stress level associated with the service load combination specified in Table 3.4.1-1. Different values may be used if verified by tests and adjusted by the appropriate λ values in accordance with Article 15.5.

Table 15.13.4.2.1-1 Service Coefficients of Friction

Type of Surface	Temp.		Average Bearing Stress				
			0.5	1.0	2.0	³ 3.0	ksi
	°F	°C	3.5	6.9	13.8	20.7	MPa
Dimpled lubricated PTFE sheets	68	20	0.04	0.03	0.025		0.02
	-13	-25	0.06	0.045	0.04		0.03
	-49	-45	0.10	0.075	0.06		0.05
Unfilled PTFE sheets	68	20	0.08	0.07	0.05		0.03
	-13	-25	0.20	0.18	0.13		0.10
	-49	-45	0.20	0.18	0.13		0.10
Filled PTFE sheets	68	20	0.24	0.17	0.09		0.06
	-13	-25	0.44	0.32	0.25		0.20
	-49	-45	0.65	0.55	0.45		0.35
Woven PTFE fiber	68	20	0.08	0.07	0.06		0.045
	-13	-25	0.20	0.18	0.13		0.10
	-49	-45	0.20	0.18	0.13		0.10

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Service coefficients of friction for various types of PTFE were determined at a test speed of 2.5 inches/min (63.5 mm/min) on a mirror finish (no. 8) stainless steel mating surface with scaled samples (Stanton, Roeder, and Campbell 1993).

Service coefficients of friction for other surface finishes, stresses, and bearing liners shall be established by testing. The testing procedures and results shall be subject to the approval of the engineer.

15.13.4.2.2 Seismic Coefficient of Friction

The seismic coefficient of friction may be determined from the area under the force displacement loops of three cycles divided by the total travel distance and vertical load (Q_d /vertical load).

15.13.5 Other Details

15.13.5.1 BEARING LINER ATTACHMENT

All sheet PTFE shall be recessed for one-half of its thickness and bonded into a metal backing plate.

All bearing liners shall be attached to resist a shear force of 0.15 times the applied compressive force or 2 times Q_d , whichever is greater.

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15.13.5.2 MATING SURFACE ATTACHMENT

The mating surface for the bearing liner shall be attached to a backing plate by welding or other suitable means in such a way that it remains free of undulations and in full contact with its backing plate throughout its service life. The attachment shall include an effective moisture seal around the entire perimeter of the mating surface to prevent interface corrosion. The attachment shall be capable of resisting the maximum friction force that can be developed by the bearing under service limit state and seismic load combinations. The welds used for the attachment shall be clear of the contact and sliding area of the bearing liner.

15.13.6 Materials for Guides

Bearing guides may be made from materials not described in Article 15.13.2. The materials used shall have sufficient strength, stiffness, and resistance to creep and decay to ensure the proper functioning of the guide throughout its design life.

15.14 SLIDING BEARINGS – CONSTRUCTION

15.14.1 General Requirements

Isolator units that use sliding bearings shall be constructed in accordance with the applicable provisions of articles 18.4 and 18.8.2 of the AASHTO Standard Specifications (Division II).

15.14.2 Quality Control Tests

The following quality control tests shall also be performed on sliding isolation bearings.

15.14.2.1 COMPRESSION CAPACITY

A 5-minute sustained proof load test shall be conducted on each bearing. The compressive load for the test shall be 1.5 times the maximum (dead load plus live load). If flow of the bearing liner suggests inadequate bonding, or it leaves a permanent deformation in the mating surface, the bearing shall be rejected.

15.14.2.2 COMBINED COMPRESSION AND SHEAR

All bearings shall be tested in combined compression and shear. The bearings may be tested in pairs. The compressive load shall be the average dead load of all bearings of that type, and the bearings shall be subjected to five fully reversed cycles of loading at the total design displacement.

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For each bearing, the effective stiffness and EDC shall be averaged over the five cycles of the test. For each group of similar bearings of the same type and size, the effective stiffness and EDC shall be averaged. The results shall not differ from the design values by more than the limits given in table 15.12.2.2-1.

15.14.2.3 ACCEPTANCE CRITERIA

After quality control testing, all bearings shall be visually inspected and, if applicable, disassembled and inspected for defects. The following faults shall be cause for rejection:

- (1) Lack of bearing-liner-to-metal bond.
- (2) Scoring of stainless steel plate.
- (3) Permanent deformation.
- (4) Leakage.

15.15 OTHER ISOLATION SYSTEMS

15.15.1 Scope

All isolation units or systems that contain a flexible element, restoring force capacity, and energy dissipation capacity, and that are not covered in Articles 15.11 to 15.14 of this specification, shall be subject to the requirements of this section and approved by the engineer.

Isolation bearings that depend on a metal roller element for lateral displacement shall satisfy the requirements of Article 14.7.

Acceptance of the system shall be based on satisfying the requirements of Articles 15.15.2 through 15.15.6.

Materials used for contact surfaces, such as sliding or rolling elements, shall be selected so as to provide the least possible change in those properties over time.

15.15.2 System Characterization Tests

The characteristics of the isolation system that are used in design shall be verified by tests and approved by the engineer. At a minimum, the following tests shall be conducted:

- Lateral load tests to determine properties and capacities in accordance with tests prescribed in the

COMMENTARY

C15.15.1 Scope

This chapter is intended to cover new isolation systems that are not addressed in the preceding chapters.

C15.15.2 System Characterization Tests

The purpose of these tests is to demonstrate that the principles on which the system is intended to function are realized in practice. The number and details of the test must be approved by the engineer.

The phenomena to be investigated for development of I_{min} and I_{max} values shall be agreed upon with the

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NIST report (National Institute of Standards and Technology 1996; ASCE Standards Committee on Testing of Base Isolation Systems 1996) or HITEC report (Highway Innovation Technology Center 1996).

- Shaking table tests at a scale no less than 1/4 full scale. Scale factors must be well-established and approved by the engineer.
- Tests to investigate the variations in system properties and their effects on response. At a minimum, the effects on temperature, rate-dependency, prior loading (including wear), and environmental effects shall be investigated. Values for I_{min} and I_{max} , similar to those defined in Article 15.5, shall be developed from these tests.

In addition to the foregoing test data, information from previous field experience in other applications may be used to demonstrate the system characteristics.

For all tests, no adjustments to the system may be made except those that are explicitly included in the maintenance plan, which must be given to the engineer prior to the start of prototype testing.

15.15.3 Design Procedure

A complete, rational design procedure for the isolation system shall be provided to the engineer prior to the start of the prototype testing defined in section 18.5. This procedure shall include

- the basis for the selection of the limiting material stresses, deformations, or other critical response quantities;
- the method for predicting the cyclic load deformation relationship of the system; and
- the method for predicting the stability limit of the system.

At least one design example shall be submitted with the design procedure, including the calculations for obtaining the maximum force response and maximum displacement response.

15.15.4 Fabrication, Installation, Inspection, and Maintenance Requirements

All special requirements for fabrication, installation, inspection, and maintenance shall be submitted, in

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engineer prior to the start of testing.

C15.15.4 Fabrication, Installation, Inspection, and Maintenance Requirements

The maintenance requirements must be known at the time of submission of the design procedure in order

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writing, to the engineer prior to the start of prototype testing. At a minimum, these shall include

- materials to be used and the specifications they must satisfy,
- any special material testing requirements,
- fabrication sequence and procedures,
- fabrication tolerances and surface finish requirements,
- any special handling requirements,
- installation procedures and tolerances, and
- maintenance requirements, including a schedule for replacement of any components, for the lifetime of the system.

15.15.5 Prototype Tests

Prototype testing shall be conducted for each job in order to demonstrate that the design achieves the performance requirements set out in the job specifications. Insofar as possible, the tests shall conform to those defined in Article 15.10.2. The engineer may, at his or her discretion, require additional tests to verify particular characteristics of the system.

Prior to the start of testing, design values for critical response quantities shall be submitted to the engineer, and the engineer shall establish criteria for accepting the system on the basis of the prototype tests. At a minimum, those criteria shall include permissible variations from the design values of the resistance and energy dissipation at critical displacements, velocities, or accelerations.

15.15.6 Quality Control Tests

Quality control testing shall be conducted on every bearing. Test requirements and acceptance requirements shall be established by the engineer.

15.15.6.1 COMPRESSION CAPACITY

A 5-minute sustained proof load test shall be conducted on each bearing. The compressive load for the test shall be 1.5 times the maximum (dead load plus live load).

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that the engineer may assess their impact on the reliability and life-cycle costs of the system.

C15.15.5 Prototype Tests

The purpose of the prototype testing is to verify that the as-built bearing system satisfies the design requirements for the particular size and configuration used in the job in question.

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15.15.6.2 COMBINED COMPRESSION AND SHEAR

All bearings shall be tested in combined compression and shear. The bearings may be tested in pairs. The compressive load shall be the average dead load of all bearings of that type, and the bearings shall be subjected to five fully reversed cycles of loading at the total design displacement.

15.15.6.3 ACCEPTANCE CRITERIA

Acceptance criteria for requirements specified in this section shall be determined by the engineer.

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APPENDIX 15A

15A.1 SLIDING ISOLATION SYSTEMS

C15A.1 SLIDING ISOLATION SYSTEMS

The I factors on sliding systems are applied to Q_d .

Woven PTFE shall be treated as unlubricated PTFE.

15A.1.1 Factors for Establishing I_{min}

$$I_{min} = 1.0$$

15A.1.2 Factors for Establishing I_{max}

15A.1.2.1 $\lambda_{MAX,A}$

C15A.1.2.1 $I_{MAX,A}$

	Unlubricated PTFE		Lubricated PTFE		Bimetallic Interfaces	
Condition Environment	Sealed	Unsealed	Sealed	Unsealed	Sealed	Unsealed
Normal	1.1	1.2	1.3	1.4	2.0	2.2
Severe	1.2	1.5	1.4	1.8	2.2	2.5

Notes:

- Values are for 30-year exposure of stainless steel. For chrome-plated carbon steel, multiply values by 3.0.
- Unsealed conditions assumed to allow exposure to water and salt, thus promoting further corrosion.
- Severe environments include marine and industrial environments.
- Values for bimetallic interfaces apply for stainless steel and bronze interfaces.

The aging factor is based on friction data for rough stainless steel plates with PTFE or other materials. It is assumed that the plate has uniform corrosion, which creates a rougher sliding surface.

For bimetallic interfaces, the factor is based on data for stainless steel and leaded bronze interfaces (Lee 1993). Increases in friction due to stress effects have been observed in the absence of corrosion.

15A.1.2.2 $\lambda_{MAX,V}$

Established by test.

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15A.1.2.3 $\lambda_{MAX,C}$

	Unlubricated PTFE	Lubricated PTFE	Bimetallic Interfaces
Sealed with stainless steel surface facing down	1.0	1.0	1.0
Sealed with stainless steel surface facing up*	1.1	1.1	1.1
Unsealed with stainless steel surface facing down	1.1	3.0	1.1
Unsealed with stainless steel surface facing up	Not Allowed	Not Allowed	Not Allowed

* Use factor of 1.0 if bearing is galvanized or painted for 30-year lifetime.

15A.1.2.4 $\lambda_{MAX,TR}$

Cumulative Travel		Unlubricated PTFE*	Lubricated PTFE	Bimetallic Interfaces
ft	m			
<330 0	1005	1.0	1.0	To be established by test
<660 0	2010	1.2	1.0	To be established by test
>660 0	2010	To be established by test	To be established by test	To be established by test

* Test data based on 1/8-inch sheet, recessed by 1/16 inch and bonded.

15A.1.2.5 $\lambda_{MAX,T}$

Minimum Temp for Design		Unlubricated PTFE	Lubricated PTFE	Bimetallic Interfaces
°F	°C			
70	21	1.0	1.0	To be established by test
32	0	1.1	1.3	
14	-10	1.2	1.5	
-22	-30	1.5	3.0	

15A.2 ELASTOMERIC BEARINGS

The λ factors on elastomeric systems are applied to K_d and Q_d .

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C15A.1.2.3 $I_{MAX,C}$

Values shown in the table assume that the sliding interface will not be separated.

Sealed bearings shall have a protective barrier to prevent contamination of the sliding interface. The protective barrier shall remain effective at all service load displacements.

C15A.2 ELASTOMERIC BEARINGS

Elastomeric bearings are produced in a variety of compounds (particularly high-damping rubber bearings), so that a vast number of experiments are

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15A.2.1 Factors for Establishing l_{min}

$$l_{min} = 1.0$$

15A.2.2 Factors for Establishing l_{max}

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needed to establish the relevant λ factors.

Moreover, available data on the behavior of rubber bearings are limited to a small range of parameters, usually established for a particular application. Even in the case of lead-rubber bearings (which found wide application in bridges), data on the effect of temperature are scarce and include one bearing tested in New Zealand at temperatures of -31, 5, 64, and 113° F (-35, -15, 18, and 45°C); one tested in the United States (Kim et al. 1996) at temperatures of -18 and 68°F (-28 and 20°C); and one in Japan tested at -4 and 68°F (-20 and 20°C).

The factors listed herein are based on the available limited data. In some cases the factors could not be established and need to be determined by test.

It is assumed that elastomeric bearings are tested when unscragged at temperature of 70°F ±10°F (21°C ±5°C) to establish the relevant properties. Testing is performed at the design displacement and a frequency less than the inverse of period T_{eff} . The first cycle loop is used to establish the maximum value of effective stiffness (k_{max}) and area under loop (A_{max}). The minimum values (as a result of scragging) are established as the average of three cycles to be k_{min} and A_{min} .

It is also assumed here that scragging is a reversible phenomenon – that is, rubber recovers after some time its initial, unscragged properties. High-damping rubber bearings may exhibit significant difference between unscragged and scragged properties, although this difference depends entirely on the rubber compound.

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15A.2.2.1 $\lambda_{MAX,A}$

The aging factor depends significantly on the rubber compound. As a general rule, it is expected that this factor is close to unity for low-damping natural rubber and to be more for high-damping rubber.

C15A.2.2.1 $\lambda_{MAX,A}$

The relationship between aging and scragging was assumed in the table. However, such a relationship has not been verified by testing.

	K_d	Q_d
Low-Damping natural rubber	1.1	1.1
High-Damping rubber with small difference between scragged and unscragged properties	1.2	1.2
High-Damping rubber with large difference between scragged and unscragged properties	1.3	1.3
Lead	–	1.0
Neoprene	3.0	3.0

Notes:

- A large difference is one in which the unscragged properties are at least 25 percent more than the scragged ones.

15A.2.2.2 $\lambda_{MAX,V}$

Established by test.

15A.2.2.3 $\lambda_{MAX,C}$

$$\lambda_{max,c} = 1$$

15A.2.2.4 $\lambda_{MAX,TR}$

Established by test.

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15A.2.2.5 $\lambda_{MAX,T}$

C15A.2.2.5

Values for lead-rubber bearings are based on grade 3 natural rubber.

Minimum Temp for Design		Q_d			K_d		
°F	°C	HDRB†	HDRB‡	LDRB‡	HDRB†	HDRB‡	LDRB‡
70	21	1.0	1.0	1.0	1.0	1.0	1.0
32	0	1.3	1.3	1.3	1.2	1.1	1.1
14	-10	1.4	1.4	1.4	1.4	1.2	1.1
-22	-30	2.5	2.0	1.5	2.0	1.4	1.3

HDRB = High-Damping Rubber Bearing

LDRB = Low-Damping Rubber Bearing

† Large difference between scragged and unscragged properties.

‡ Small difference between scragged and unscragged properties.

Notes:

- A large difference is one in which the unscragged properties are at least 25 percent more than the scragged ones.

15A.2.2.6 $\lambda_{MAX,SCRAG}$

Q_d			K_d		
LDRB	HDRB with $\beta_{eff} - 0.15$	HDRB with $\beta_{eff} > 0.15$	LDRB	HDRB with $\beta_{eff} - 0.15$	HDRB with $\beta_{eff} > 0.15$
1.0	1.2	1.5	1.0	1.2	1.8