LIQUEFACTION HAZARD EVALUATION OF INTERSTATE, FEDERAL, AND STATE HIGHWAY BRIDGE SITES IN UTAH

EXECUTIVE SUMMARY

A Report Presented to

Utah Department of Transportation

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liquefaction hazard at little additional cost as part of foundation investigations for the new bridges, a more complete assessment of liquefaction hazard should be made as part of the I-15 reconstruction effort.

Profiles showing depths and thicknesses of possibly liquefiable layers were made for each bridge site in Utah where regional investigations indicated potential for liquefiable sediments and where adequate geotechnical information was available to estimate liquefaction resistance. These profiles are used as figures in the project report. A summary listing the bridge locations and the cumulative thickness of liquefiable sediment in the upper 15 m of the soil profile is given in table 6 for all of the Priority I sites. As noted above, all of the Priority I sites are at river or creek crossings. Twenty-five bridges are listed as Priority I sites. Because of the general greater importance of interstate highway bridges, the Priority 1 sites were divided into two subcategories: Priority I(1) sites for bridges in the Interstate highway system (13 bridges) and Priority I(2) sites for bridges in the Federal and State Highway system (12 bridges).

Evaluation of Consequences of Liquefaction

Liquefaction of subsurface sediment layers may or may not be harmful to bridge structures depending on whether the liquefied condition induces damaging ground displacements or loss of foundation bearing strength. If predicted ground displacements or loss of bearing strength is likely to damage the bridge, mitigative measures should be implemented. Typical measures include either ground modification to increase liquefaction resistance of the soil or strengthening of the structure to resist ground displacement.

For those sites where predicted displacements are too small to be damaging to the bridge and the foundation is capable of transferring loads to competent layers, liquefaction would be harmless to the bridge and of little engineering concern. In those instances, the sites were classed as Priority IV sites even though liquefaction might occur. The following analyses are required to evaluate potential for damaging ground displacement and loss of foundation bearing strength.

Embankment Stability Analysis

The first step in evaluating ground displacement hazard is to assess the stability of bridge approach embankments. If massive embankment instability were to occur, consequent ground displacements could disrupt bridge foundation elements as well as the embankment. Embankment stability can be evaluated using a standard limit equilibrium analyses where all subsurface liquefiable layers are assigned post liquefaction residual strengths. Computer programs such as *SlopeW* or *UTEXAS2* may be used for such stability analyses. If the static factor of safety against instability is greater than 1.1, the embankment can be considered safe against mass instability as a consequence of liquefaction, and the analysis proceeds to the next step. If the factor of safety is 1.1 or less, the site should be classed as potentially hazardous and further site-specific investigations initiated to fully assess the hazard and recommend remedial measures.

The available geotechnical data were insufficient to fully assess embankment stability for any of the bridge sites evaluated, including the I-15 sites in Salt Lake County. The following

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analysis, using the best available data and conservative assumptions, demonstrates the procedure. The site chosen for the analysis is the off-ramp from southbound I-15 to eastbound 600 South in Salt Lake City. Site information and geotechnical data from nearby boreholes were used along with slope data estimated from topographic maps. Conservative assumptions were made where data were unavailable.

The embankment is approximately 9 m high. The boreholes from which the soil information was estimated, holes DH-15 and DH-15A, are located approximately 50 m from the off ramp embankment. Gerber (1995) compiled extensive borehole information for the site including static strength values for various sediment layers. These strengths were used in the analysis, except for the liquefiable layers. Application of the simplified procedure indicates that liquefiable layers lie between depths of 1.2 m to 4.0 m and 17.1 m to 17.7 m. The deeper layer was ignored because it is too deep to adversely affect slope stability. A residual strength of 28 kPa was estimated for the upper liquefiable layer based on a corrected blow count of 16 and charts published by Seed and Harder (1990) (figure 11). The limit-equilibrium analysis was

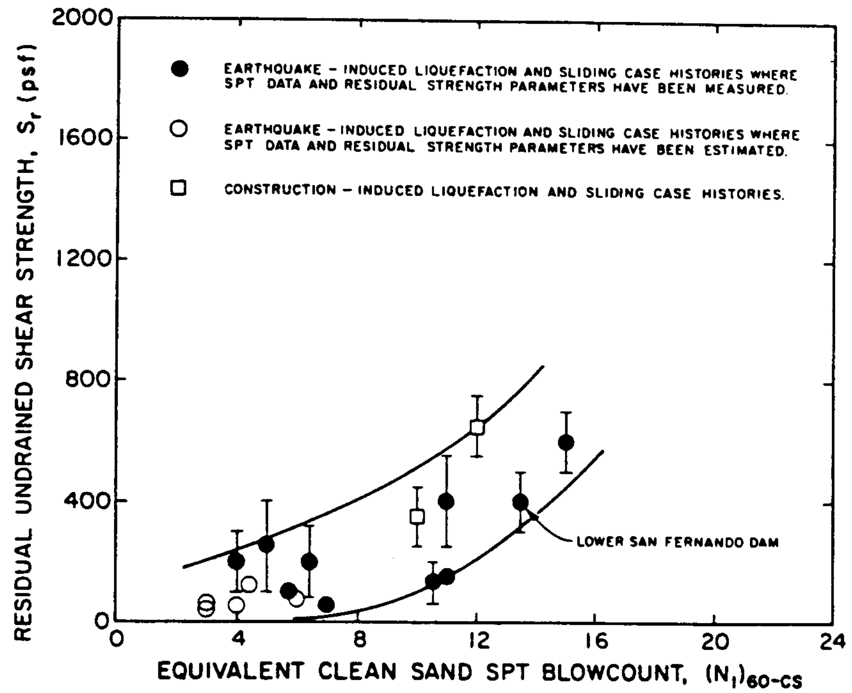

Figure 11: Empirical relationship between residual shear strength and $(N_1)_{60cs}$ (After Seed and Harder, 1990)

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applied with the aid of the computer program *UTEXAS2*. The result was a factor of safety, FS, of 1.5 against static failure for the embankment with the underlying liquefied layer. A cross section of the embankment along with the critical failure surface determined from the analysis is plotted on figure 12.

With a factor of safety of 1.5, this embankment was classed as safe against static failure, even if liquefaction were to occur. Deformation of the embankment due to strong ground shaking, however, still could cause damage to the bridge. Thus, the analysis proceeded to the next step as indicated in the flow chart reproduced in figure 2.

Embankment Deformation Analysis

Damaging ground deformations may occur within or beneath embankments or slopes as a consequence of liquefaction, even though the site may be stable against flow failure. Such deformations occur as a consequence of soil softening and yielding due to liquefaction and the inertial forces generated by the earthquake. In these instances, cyclic mobility and limited strains within liquefied layers may lead to ground deformations and displacements that could damage the bridge foundation. Analyses of embankment or slope deformation at liquefiable sites is complicated because of the complex nature of constitutive relations for liquefied soils. In particular, stress-strain relations are very complex for moderately dense or dilative soils that may deform under either undrained or partially drained conditions. Embankment deformation can be estimated using mechanistic (Newmark sliding block), finite element, or other numeric analyses. These analyses usually neglect the restraining of influence the bridge structure, which is difficult to quantify.

Youd (1998) suggests the following simplified screening criteria for dynamic displacements based on the mechanistic analyses. A key parameter in the mechanistic analysis is the yield acceleration, or the pseudostatic horizontal acceleration required to reduce the calculated static factor of safety to 1.0. For this analysis, all liquefiable layers are assigned the appropriate residual strength as in the static stability analysis noted above. During earthquake shaking, only acceleration pulses with amplitudes greater than the yield acceleration generate permanent slope displacement. Displacement continues only as long as dynamic inertial forces exceed the resisting forces (factor of safety transiently less than 1.0). Past analyses show that the amount of dynamic displacement decreases markedly as the static factor of safety, or yield acceleration increases (Makdisi and Seed, 1978). These analyses indicate that permanent displacements are generally small for sites with an adequate static factor of safety. Youd (1998) suggests that acceptable displacements (less than about 100 mm) are likely with the following combinations: factor of safety greater than 1.5 for magnitude 6.5 earthquakes; greater than 2.0 for magnitude 7.5 earthquakes; and greater than 2.5 for magnitude 8.5 earthquakes.

These criteria were applied to the 600 South off ramp of I-15. The calculated factor of safety for the embankment is approximately 1.5 as noted above. A maximum earthquake magnitude of about 7.0 to 7.3 is generally estimated for the Salt Lake segment of the Wasatch fault (Arabasz et al., 1992). For this combination and the criteria above, deformations at the 600 South offramp should be less than 100 mm.

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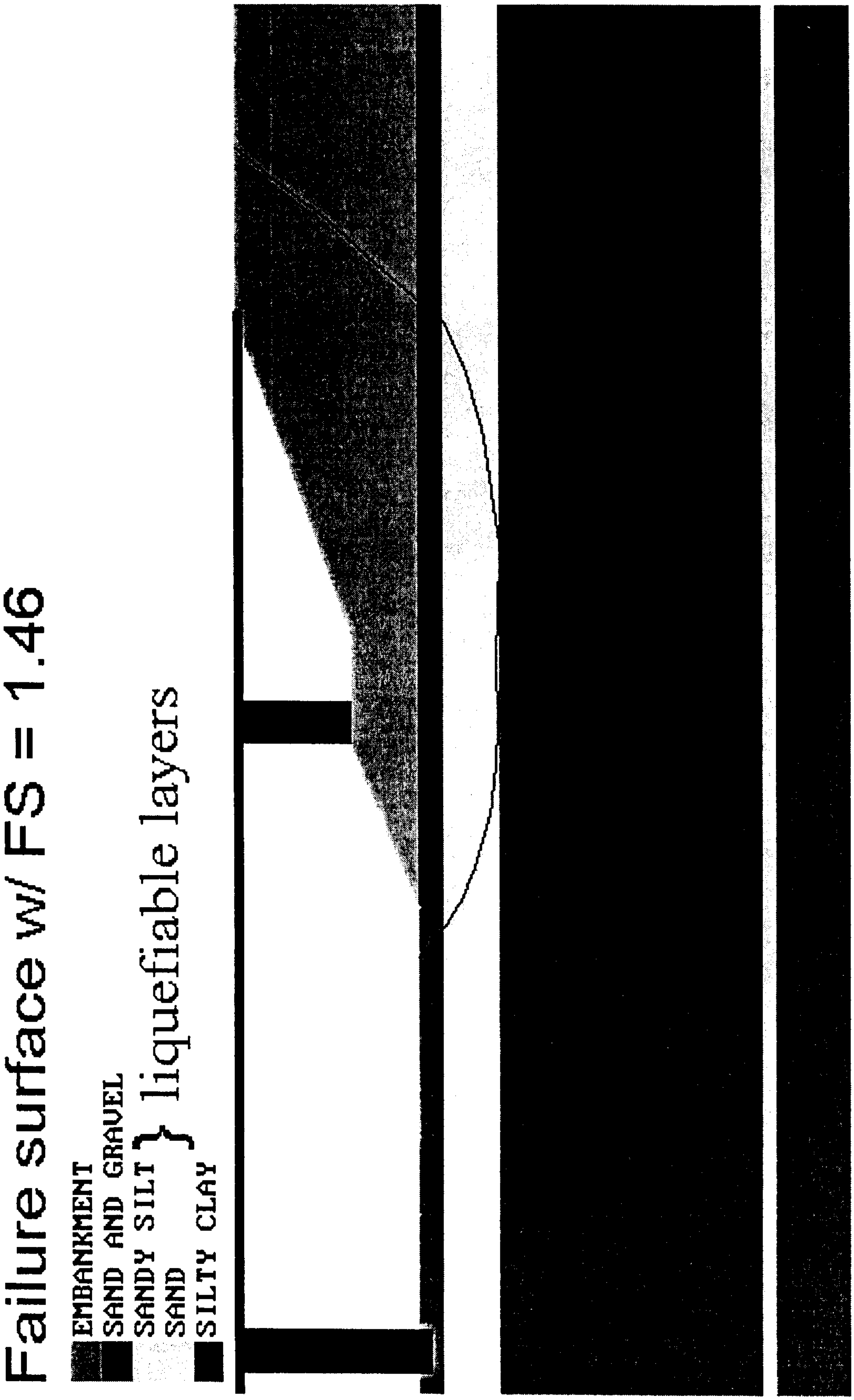

Figure 12: Profile of 600 South bridge site showing critical failure plane with factor of safety, FS, of 1.46

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To verify that the dynamic deformation at the 600 South offramp would be small, a second estimate was made using the finite element analysis program QUAD4M. Input data required for this analysis included the information used in calculating the static factor of safety, a shear-wave velocity profile, and a time history of accelerations expected during a magnitude 7.0 earthquake. Using this information, the Quad4M analysis calculated an acceleration time-history at various points in the embankment. All accelerations that exceeded the yield acceleration (approximately 0.1 g for the 600 South site) were double integrated with respect to time to estimate permanent deformation. The estimated deformation from this analysis was less than 40 mm, in agreement with the small deformation estimated from Youd's simplified magnitude-factor of safety criteria.

Most modern reinforced-concrete, steel, or heavy timber bridges should be able to withstand 100 mm to 200 mm of unrestrained embankment deformation without significant damage. Some lightweight timber bridges have been damaged by displacements smaller than 100 mm, but these types of bridges are not commonly used for highway bridges in Utah. Such small ground displacements are usually accommodated by soil compression or shear rather than structural deformation. The displacements predicted for the 600 South off ramp are smaller than the 100 mm to 200 mm of allowable displacement noted above. Thus, this structure was classed as not susceptible to damage from embankment deformation. This analysis was conducted as a demonstration. This and other structures should be reanalyzed as more geotechnical data are compiled during foundation investigations for the new bridges to be built in the I-15 corridor.

Lateral Spread Analysis

As noted in the introductory paragraph, most liquefaction-induced bridge damage has been due to lateral spread of flood plain deposits toward river channels. Lateral spread displacement is generally estimated using empirical equations such as those developed by Bartlett and Youd (1995). Data required for lateral displacement analyses include the stratigraphic and penetration data used for evaluation of liquefaction resistance, grain-size distribution data, and site topography. If predicted lateral spread displacements are tolerable (less than 100 mm or less for most highway bridges) then the site may be classed as not susceptible to damage from lateral spread and the analysis proceeds to the next step. If predicted displacements are potentially damaging (typically greater than 100 mm) then a more detailed site study should be initiated to better define lateral spread potential and consequent structural damage.

The empirical equations developed by Bartlett and Youd (1995) were used to calculate lateral spread displacements for this study. For mildly sloping ground conditions, such as those at the 600 South off ramp in Salt lake City, the following equation is recommended by Bartlett and Youd for calculation of possible lateral spread displacement.

$$\begin{aligned}\text{LOG } D_H = &-15.7870 + 1.1782\,M - 0.9275\,\text{LOG } R - 0.0133\,R + 0.4293\,\text{LOG } S + \\ &0.3483\,\text{LOG } T_{15} + 4.5270\,\text{LOG } (100 - F_{15}) - 0.9224\,D50_{15}\end{aligned} \tag{1}$$

where: D_H is the estimated lateral ground displacement in meters, M is the earthquake magnitude (moment magnitude), R is the horizontal distance from the site to seismic energy source, in kilometers, S is the ground slope, in percent, T_{15} is the cumulative thickness of saturated granular

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layers with corrected blow counts less that 15, [$(N1)_{60} < 15$], in meters, F_{15} is the average fines content (fraction of soil sample passing a No. 200 sieve) for granular layers included in T_{15}, in percent, and $D50_{15}$ is the average mean grain size, in mm, for granular layers included in $T_{15.}$

The only layer which poses a significant lateral spread hazard is the liquefiable layer between depths of 1.2 m and 2.7 m. The lower liquefiable layers are too thin and deep to pose a lateral spread hazard (depth > 15 m). The following input values for the lateral spread analysis were estimated for the 600 South offramp: M = 7.0, R = 3.0 km (under the 5.0 km limit), S = 0.25 percent, T_{15} = 1.5 m, F_{15} = 75 percent (over the 50 percent limit), and $D50_{15}$ = 0.06 mm (under the 0.1 mm limit). Entering these values into equation (1) yields a predicted displacement of 11 mm. Unfortunately, many of the required values are outside the limits specified by Bartlett and Youd (1995), which increases the uncertainty of the predicted displacement. The analysis was repeated with site variables outside the recommended limits adjusted to the nearest acceptable limiting value; that is R= 5 km rather than 3 km; F_{15} = 50 percent rather than 75 percent. and $D50_{15}$ = 0.1 mm rather than 0.06 mm. The estimated displacement from this second analysis is 140 mm. The actual displacement is likely to lie between 11 mm and 140 mm. Even if 140 mm of ground displacement were to occur, a well-built highway bridge should withstand the displacement with minimal damage. Thus probable lateral spread displacement at this site were classed as sufficiently small to pose little hazard and the investigation proceeded to the next step listed on the flow chart in figure 2.

Ground Settlement Analysis

If earthquake-generated embankment deformation and lateral spread displacements are tolerable, the next step is to estimate the expected ground settlement. Settlement occurs as a consequence of compaction of cohesionless soils when subjected to earthquake shaking. Empirical procedures developed by Tokimatsu and Seed (1987) were applied to predict ground settlement. The same input data is required for this analysis as was used to evaluate liquefaction resistance. If settlements are tolerable, then the bridge may be assessed as safe against liquefaction-induced ground settlement and the screening proceeds to the next step. Past experience indicates that well built bridges on shallow or deep foundations can withstand 100 mm or more settlement without damage (Youd, 1998).

The premise of the Tokimatsu and Seed procedure is that earthquake shaking generates cyclic shear strains that compact granular soils, causing volumetric strain. Where drainage cannot occur rapidly, the tendency to compact also generates transient pore water pressures that prevent immediate decrease in volume. However, as pore pressures dissipate, the layer consolidates, producing volumetric strain and ground settlement. The induced volumetric strains are primarily a function of the amplitude of the cyclic shear strains generated by the earthquake and the initial relative density of the sand. The cyclic shear strains are a function of the cyclic stress ratio (CSR), relative density, and earthquake magnitude. Tokimatsu and Seed corrected the cyclic stress ratio for magnitude by dividing the CSR by an appropriate magnitude scaling factor (Youd et al., 1997). Relative density was estimated directly from corrected penetration resistance, $(N_1)_{60}$. For silty sands, $(N_1)_{60}$ was corrected to $(N_1)_{60cs}$ using the correction factors specified for calculation of liquefaction resistance. Figure 13 is a synthesis diagram developed by Tokimatsu and Seed from available laboratory test data and field observations of earthquake-induced settlements in clean

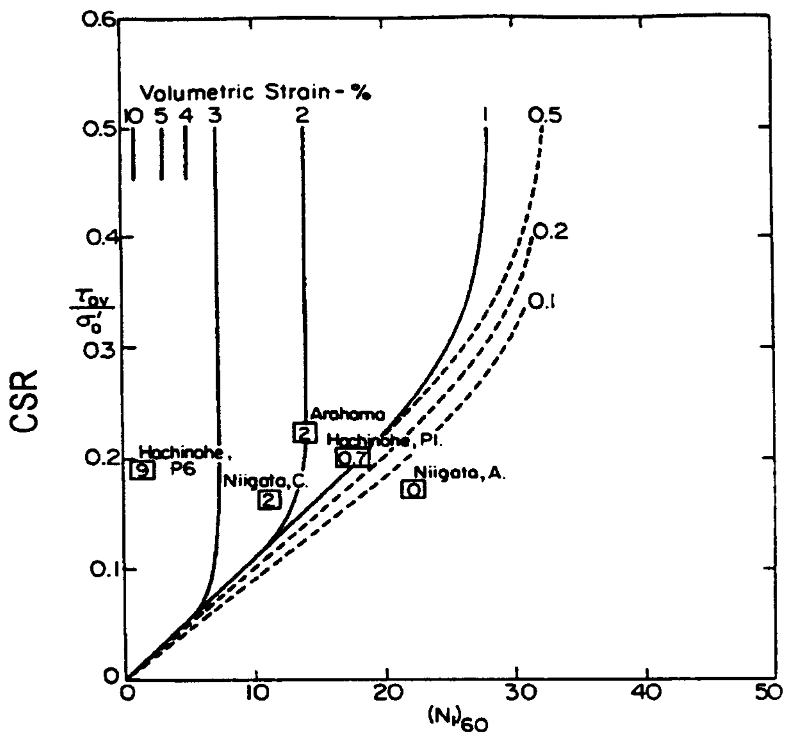

Figure 13: Curves for estimating volumetric strain at liquefiable sites (After Tokimatsu and Seed, 1987)

sands. They recommend use of this diagram to estimate volumetric strains from magnitude-corrected CSR and fines-content-corrected $(N_1)_{60}$. The volumetric strain is then multiplied by the layer thickness, assuming one-dimensional consolidation, to compute the change of layer thickness. The changes of thickness from all layers at the site are then summed to estimate ground settlement.

Table 5: Values used in settlement calculations for 600 South southbound off-ramp from I-15. Layers are at depths of 1.2 m and 2.7 m

Layer Thickness	$(N_1)_{60}$	$(N_1)_{60\text{-cor}}$	CSR	Volumetric Strain (%)	Incremental Settlement
1.5 m	11	19	0.476	1.5	23 mm
1.2 m	19	28	0.524		11 mm
Total Settlement					34 mm

Settlements were calculated for the 600 South off ramp site. Volumetric strains were calculated for the cohesionless layers between depths of 1.2 m and 2.7 m. The values of $(N_1)_{60}$ and the CSR were taken from the analysis of liquefaction resistance. Table 5 tabulates the fines-content corrections, estimated volumetric strains, and settlements. Because the bridges in the 600

South interchange are all constructed on deep foundations, a general ground settlement of 34 mm should not adversely affect these structures. Thus, this site was classified as low hazard for ground settlement.

Bearing Capacity Analysis

If liquefaction-induced ground deformations and ground settlements are tolerable, the remaining possible liquefaction-induced hazard is loss of foundation bearing strength. Loss of bearing strength could lead to penetration of shallow or deep foundations into the liquefied sediment or possibly to buckling of piles as a consequence of reduced lateral resistance in liquefied soil layers.

For shallow spread-footing foundations, a standard bearing capacity analysis is used to assess bearing capacity for shallow foundations. For liquefiable soils, however, residual strengths are assigned to the liquefiable layers. For axial load capacity of deep foundations, liquefiable layers are commonly assumed to have negligible strength. Lateral load resistance of deep foundations is usually estimated by multiplying the lateral resistance for nonliquefied conditions by a factor ranging from 0.1 to 0.3, depending on relative density.

If the load capacity analyses indicate an adequate factor of safety (say 1.5 or greater) against loss of foundation failure, the site may be classed as nonhazardous and immune to detrimental effects of liquefaction (Priority IV site) even though liquefaction of some subsurface layers may occur. At this juncture, all of the possible detrimental effects of liquefaction have been considered and determined to be nondamaging to the bridge. Conversely, if the load capacity analysis indicates a marginal factor of safety (less than 1.5), unacceptable foundation penetration may occur and the site is classified as a Priority I site and recommended for additional investigation and possible remediation.

Bridges near the 600 South offramp of I-15 are founded on deep pile foundations which likely extend to deep competent layers. Foundation plans were not available for this study, however, so an analysis of load capacity could not be made. The soils beneath the 600 South site contain liquefiable layers at depths of 1.2 m to 4.0 m, 17.1 m to17.7 m, 24.4 m to 25.6 m, 26.5 m to 27.7 m, and 28.6 m to 31.1 m. If these layers provide a substantial part of the bearing resistance, failure in the form of pile penetration might occur during an earthquake. For future structures at the site, care should be taken to assure that liquefiable layers are not relied on to provide significant bearing strength.

Prioritization of Bridge Sites for Further Investigation

The primary objective of this study was to prioritize the bridge sites in Utah for further investigation. From the various evaluations noted above, bridge sites were prioritized into the four categories listed in the Introduction. Approximately 325 bridge sites were reviewed and analyzed using the site-specific steps in the screening guide. The majority of these sites (about 279) were identified as underlain by possibly liquefiable soil layers. Twenty-five of these sites were identified as Priority I sites for further investigation (table 6). These sites have high priority

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