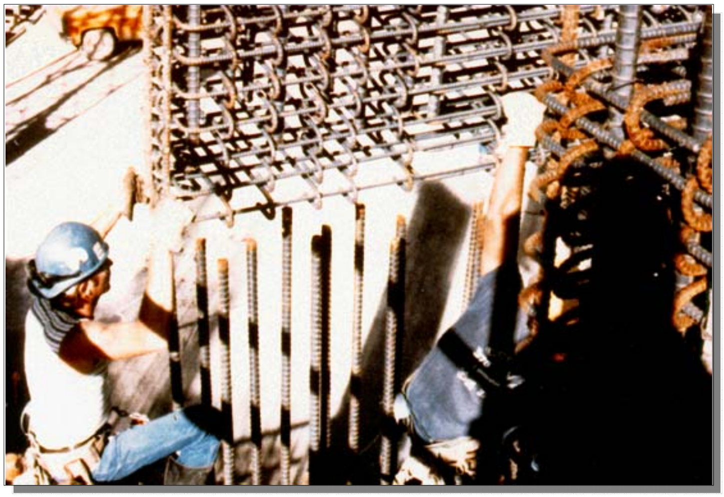
SEISMIC DESIGN OF REINFORCED CONCRETE STRUCTURES



Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 1

Topic 11 is the seismic design of reinforced concrete structures, primarily buildings. During this lesson you will learn the basics of seismic design of reinforced concrete buildings. Buildings designed using these principles will fare better in a seismic event than the building shown in this slide.

Note that some of the examples in this topic draw heavily on the examples in the FEMA 451, *NEHRP Recommended Provisions: Design Examples*. Please see Chapter 6 of that CD for additional details regarding these examples.

NEHRP Recommended Provisions Concrete Design Requirements

- Context in the *NEHRP Recommended Provisions*
- Concrete behavior
- Reference standards
- Requirements by Seismic Design Category
- Moment resisting frames
- Shear walls
- Other topics
- Summary

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This slide presents the outline of this presentation

Context in *NEHRP Recommended Provisions*

Design basis: Strength limit state

Using *NEHRP Recommended Provisions*:

Structural design criteria:	**Chap. 4**
Structural analysis procedures:	**Chap. 5**
Components and attachments:	**Chap. 6**
Design of concrete structures:	**Chap. 9 and ACI 318**

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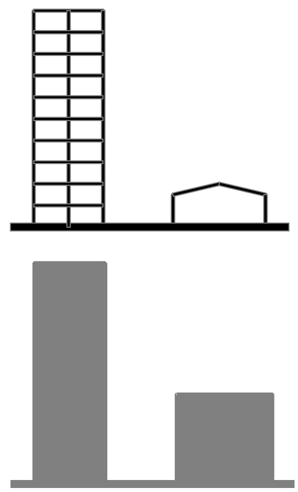
The 2003 *NEHRP Recommended Provisions* (FEMA 450) uses strength limit state for design of concrete (and all other materials). Since ACI 318 provides ultimate strength procedures, the *Provisions* design will be very familiar to most engineers. Required strength (demand) is determined from *Provisions* Chapters 4 and 5 and provided strength (capacity) is calculated using Chapter 9. The chapters of *Provisions* that affect concrete design are as follows: Chapter 4 for load combinations and R and C_d factors, Chapter 5 for seismic load determination and distribution, Chapter 6 for specific component and attachment requirements, and Chapter 9 for the design of concrete elements and connections. Chapter 9 also provides detailing provisions that are used to ensure stable inelastic behavior.

Seismic-Force-Resisting Systems
Reinforced Concrete

Unbraced frames (with rigid “moment resisting” joints):
- Three types
  - Ordinary
  - Intermediate
  - Special

R/C shear walls:
- Ordinary
- Special

Precast shear walls:
- Special
- Intermediate
- Ordinary



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Two possible seismic resisting systems using reinforced concrete are moment frames and shear walls. *Provisions* Chapter 4 presents design coefficients and system limitations for various Seismic Design Categories. Precast walls can be used, however they will not be addressed in detail in this lecture. To understand some of the detailing requirements and how they relate to the ductility of these structural systems, we will first review basic reinforced concrete behavior.

NEHRP Recommended Provisions Concrete Design

- **Context in the *Provisions***
- **Concrete behavior**

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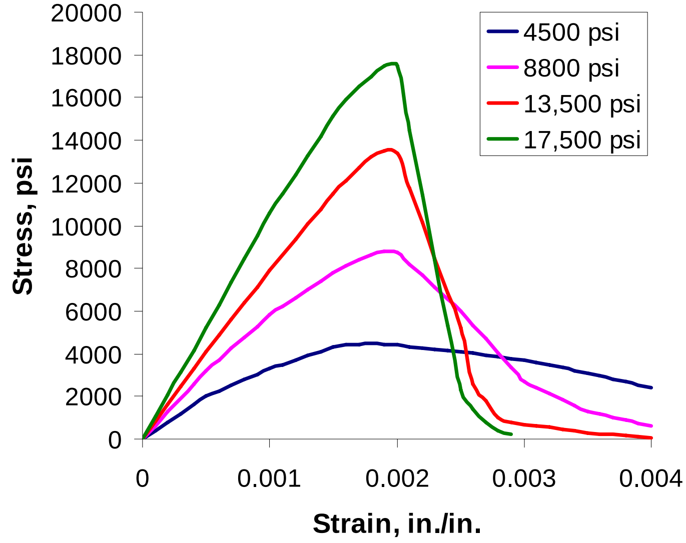
Design for Concrete Structures 11 - 5

This section will discuss mechanical properties of reinforcing steel and concrete and how the two materials work together in reinforced concrete members.

Unconfined Concrete Stress-Strain Behavior



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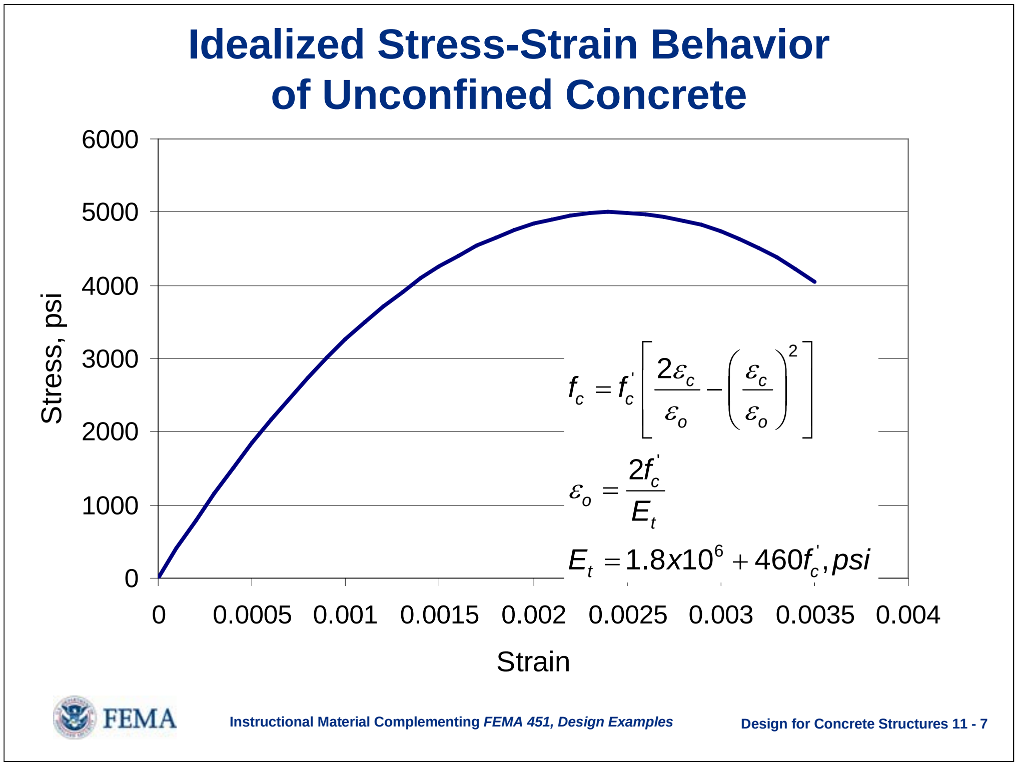
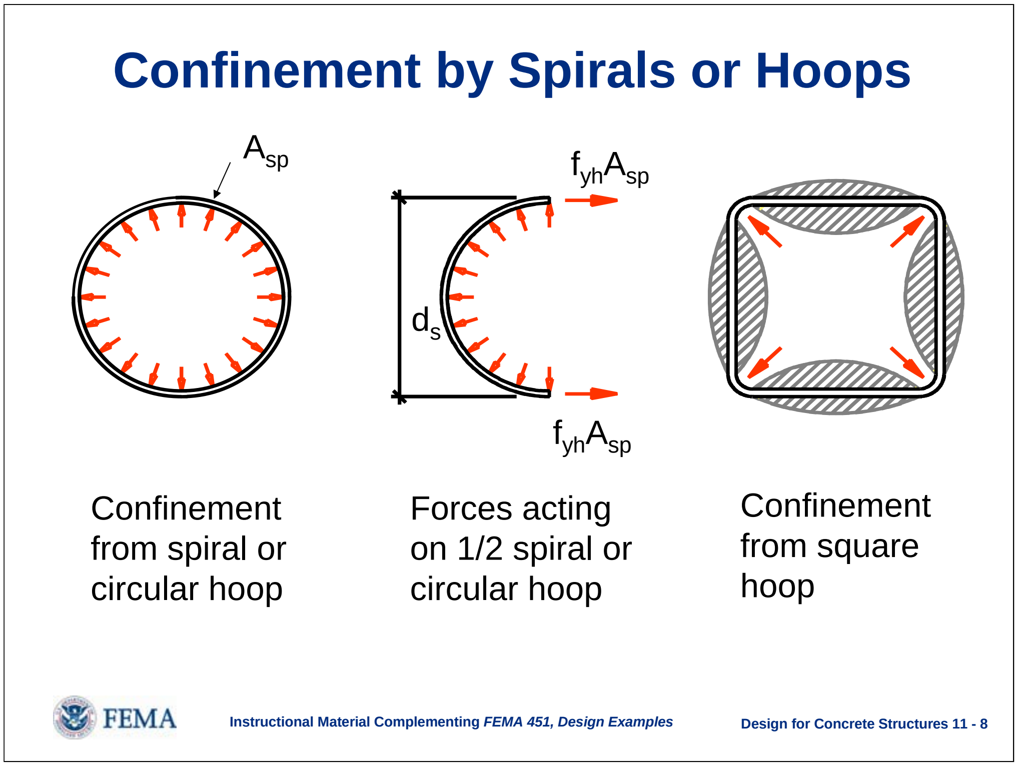
This slide presents stress-strain diagrams for unreinforced, unconfined concrete in compression. Behavior is relatively linear up to about one-half of the maximum compressive stress. Concrete exhibits no precise yield point. Strain at maximum strength is close to 0.002 regardless of maximum stress. Lower strength concrete can have strains at crushing that exceed 0.004, however a typical design value is 0.003 at crushing. Stronger concretes are more brittle.



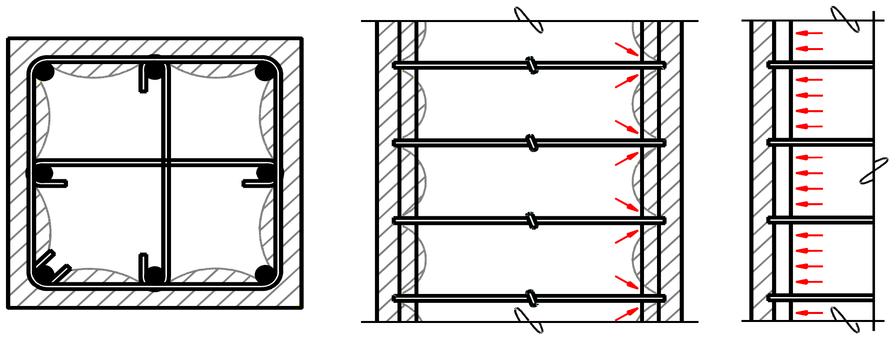
This slide shows one commonly used, relatively simple, idealized model of the stress-strain behavior of unconfined concrete (Hognestad). This type of mathematical model can be used in a strain compatibility approach to predict behavior of reinforced concrete members.



Confining reinforcing can improve concrete behavior in two ways. First it can enhance strength by restraining lateral strains. Second it can increase the usable concrete compressive strain well beyond the typical value of 0.003.

This slide shows confinement in practical structural sections. Confinement is typically provided by spirals, circular hoops, or square hoops. The hatched areas in the figures may spall. Confining steel is in tension (hoop stress effect) because, due to Poisson's effect, as the concrete is compressed in one direction, it expands in the orthogonal directions. This is shown in the center illustration. Note that hoops are not as efficient as spirals in confining concrete because the sides of the hoop can flex outward as the confined concrete expands outward.

Confinement



Rectangular hoops with cross ties

Confinement by transverse bars

Confinement by longitudinal bars

FEMA

Instructional Material Complementing *FEMA 451, Design Examples*

Design for Concrete Structures 11 - 9

This slide shows confinement for a square column, which can be provided by transverse and longitudinal bars. The hatched areas may spall.

Opened 90° hook on hoops

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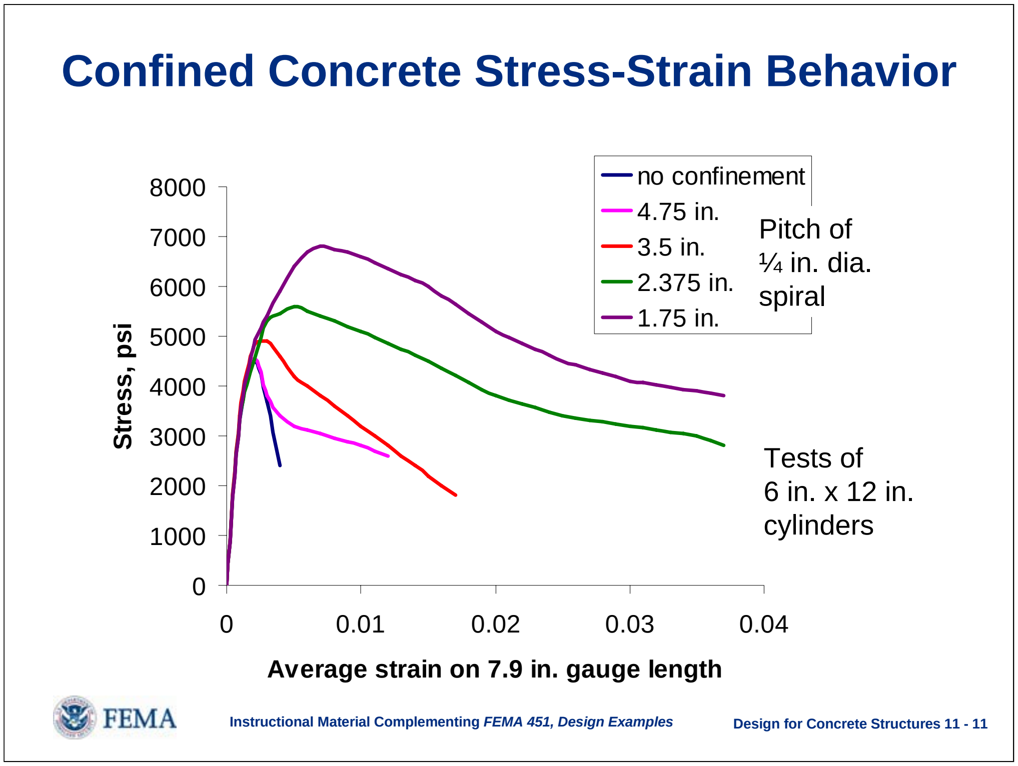
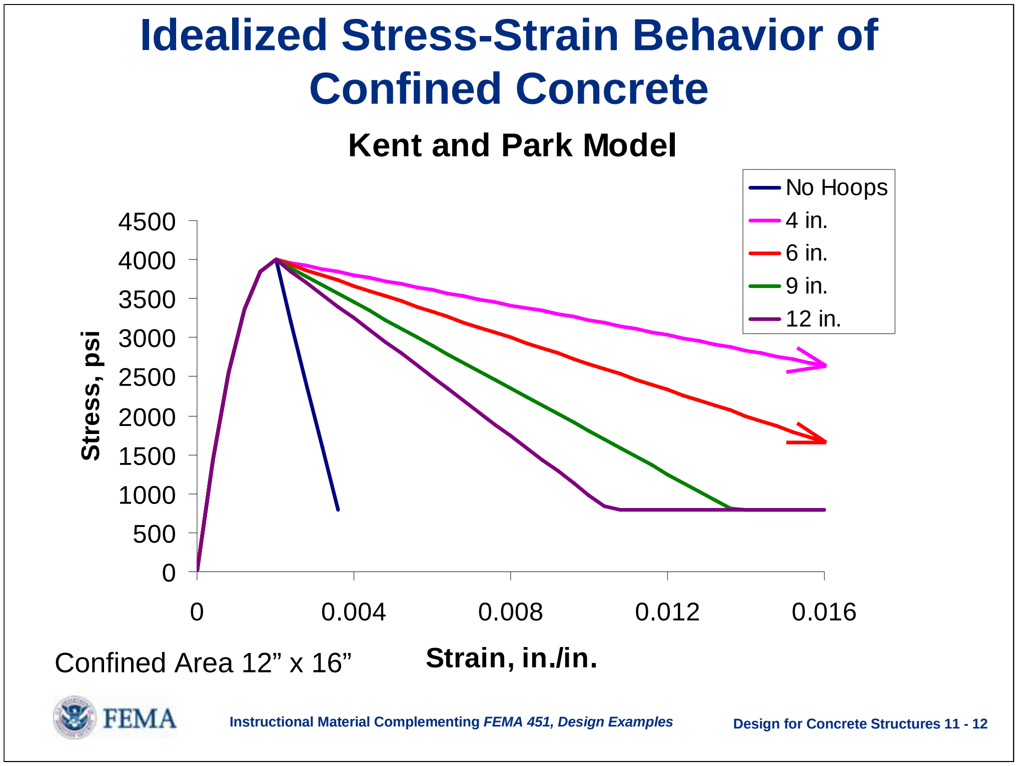
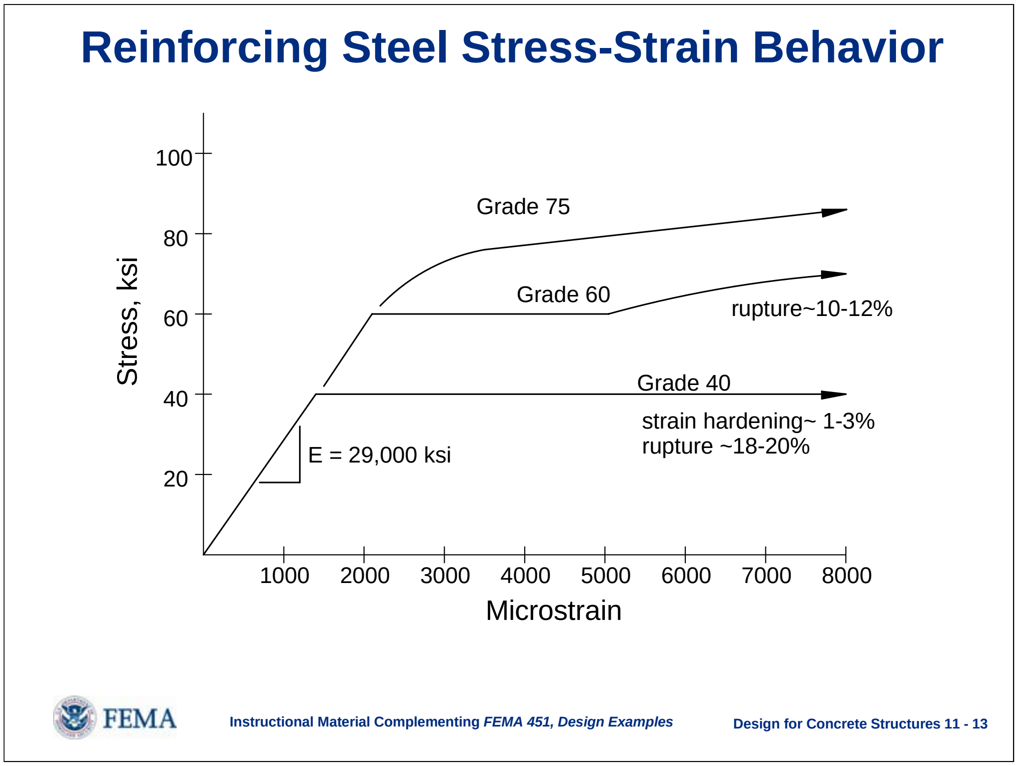
This slide shows 90 degree hooks on square hoops which have opened. As stated earlier, the confining steel is in tension. After spalling, the hooks can open up. The solution is to use 135 degree hooks. The arrow points to an open hook.



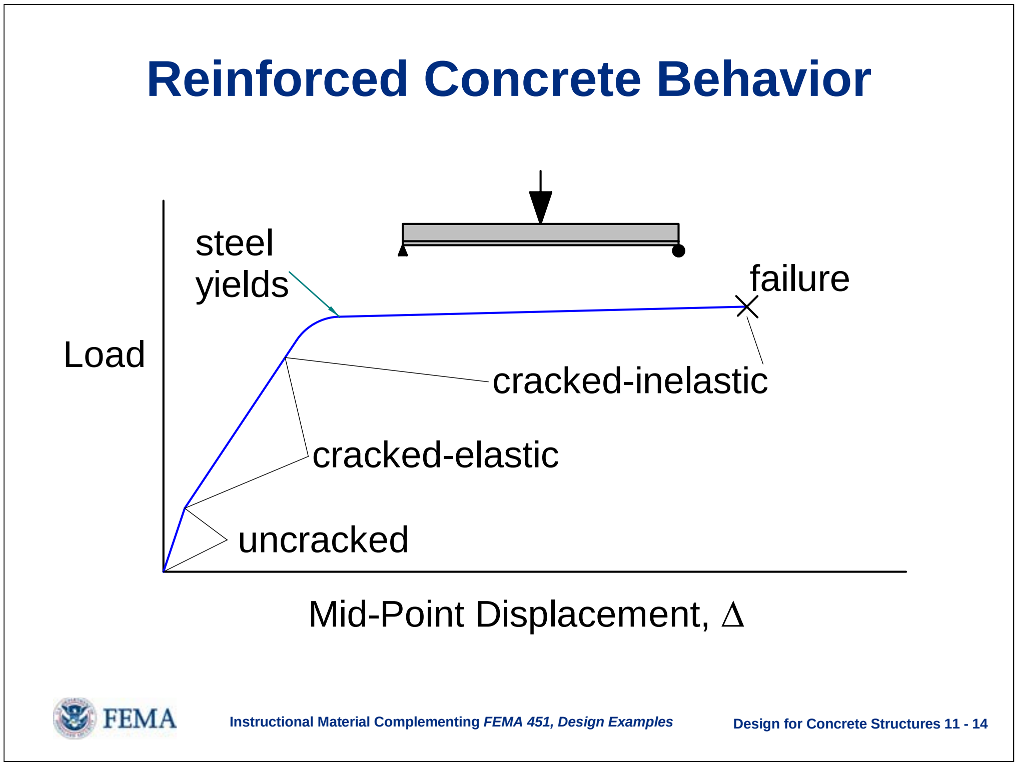
This slide shows the benefits of confinement on concrete behavior. Presented are stress-strain diagrams for confined concrete in compression. The specimens were 6 in. by 12 in. cylinders. Confinement was provided by spiral reinforcement. Reducing spiral pitch (or hoop spacing) increases maximum concrete stress and strain capacity (ductility).



This slide shows the idealized stress-strain behavior of confined concrete proposed by Kent and Park. Note that the model reflects the additional strain, but not the additional strength, provided by the confinement. Another model that reflects both strength and strain gain is Scott, Park, and Priestley. This type of model can be used with the strain compatability method to predict the behavior of confined reinforced concrete.

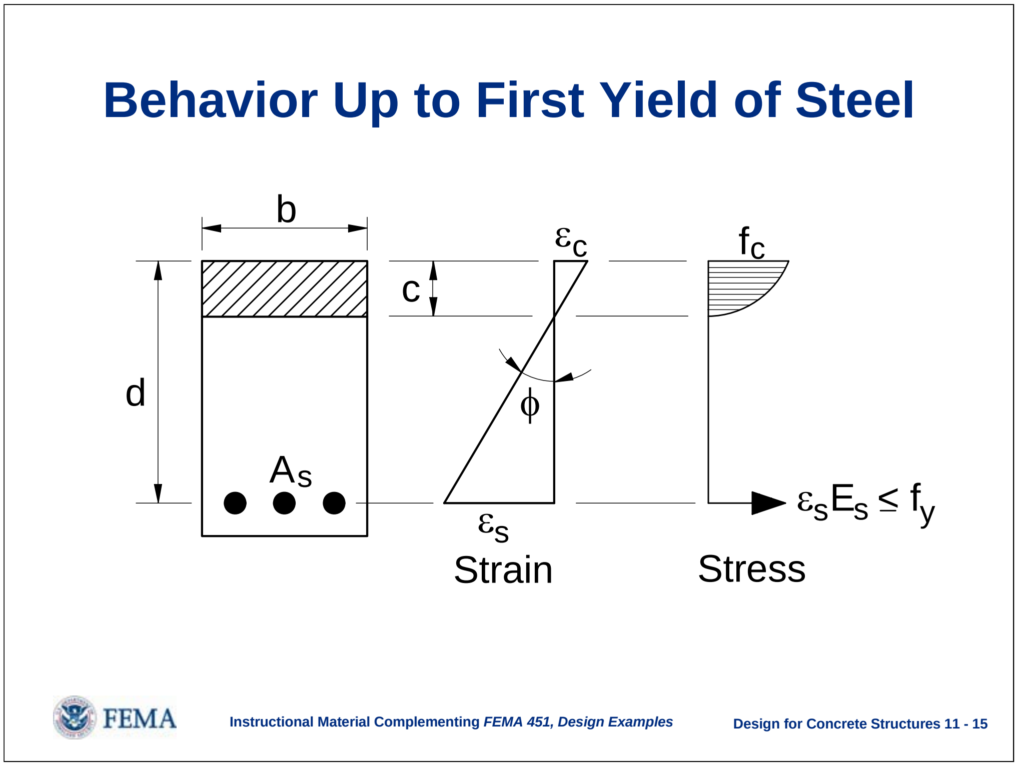


This slide shows typical stress-strain behavior of common grades of reinforcing steel. The most commonly used is Grade 60 which shows a distinct yield plateau and strain hardening at between 0.5% and 1% elongation. For common analysis of reinforced concrete behavior, strain hardening is ignored. For seismic design, it is important that the actual yield strain of the steel is not significantly higher than the value used in design.

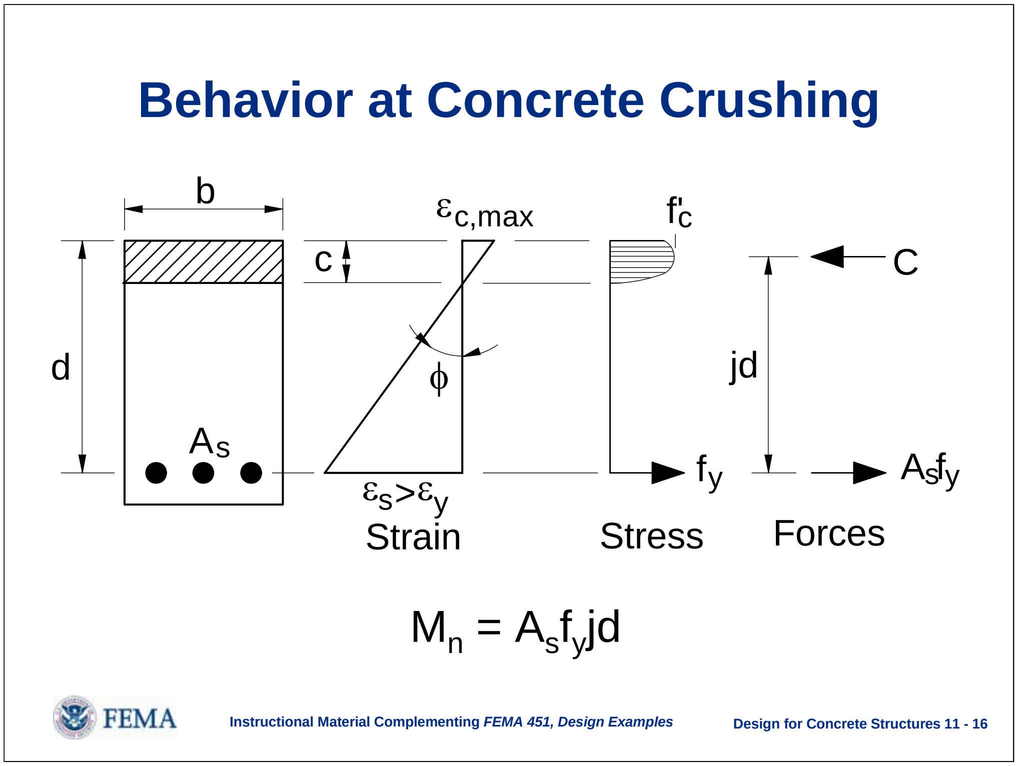


This slide shows stages of behavior of a reinforced concrete beam. At low loads the section is uncracked and an analysis using uncracked-transformed section properties can be used to predict behavior. After the concrete cracks, the concrete on the tension side of the beam is neglected, and a cracked-transformed section analysis can be used to predict behavior. However, this method is only valid as long as both the steel and the concrete stress-strain behaviors are linear. Concrete can be assumed to have a linear stress-strain behavior up to approximately 50% of maximum concrete stress (f'_c).

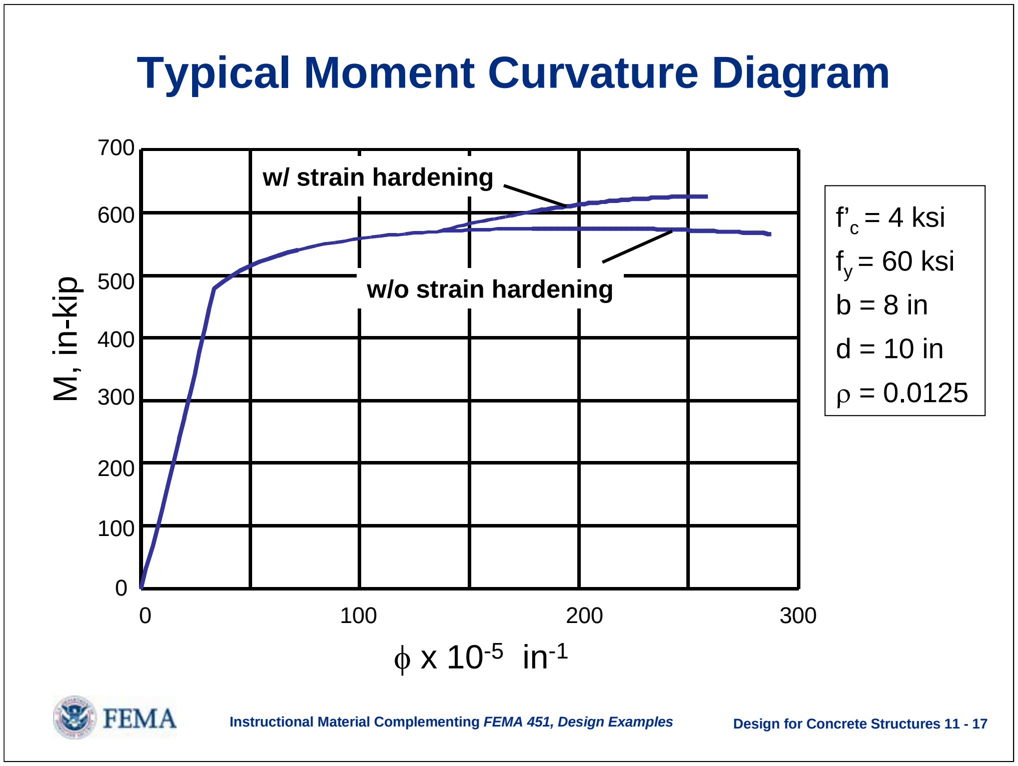
After the concrete stress exceeds about 50%f'_c, a strain compatibility approach can be used, using a realistic concrete stress-strain model such as the Hognestad model presented in Slide 7. After the steel yields, there is typically an extended plateau in which the displacement increases significantly with very little increase in applied load. A commonly used indicator of member ductility is the ratio of the displacement at ultimate to the displacement at first yield. This is known at the displacement ductility, and for seismic design in particular, bigger is better.



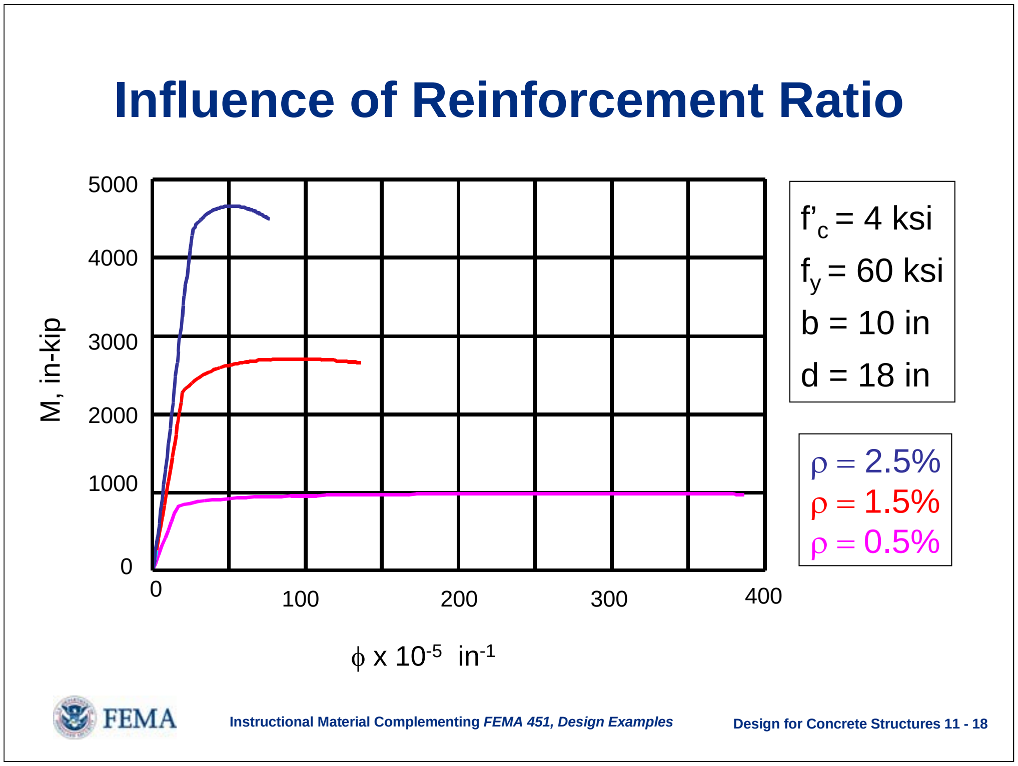
To characterize section behavior, moment-curvature (M-ϕ) diagrams are often employed. This slide shows the type of strain compatibility approach that would be used to locate points on the curve up until first yield of the steel. To locate a point, first a concrete strain is selected. Then an iterative method is used in which the depth to the neutral axis is assumed and modified until internal equilibrium is achieved. The tension force is equal to the strain (based on the strain diagram with the selected concrete strain and neutral axis depth) times the area and the modulus of elasticity of the steel. The compression force is determined by integrating under the stress-position curve from the neutral axis to the extreme compression fiber, and multiplying by the width of the beam. The value of “c” is adjusted until $C = T$. Then the curvature is calculated as the concrete strain divided by the neutral axis depth, and the moment is the force (T or C) times the distance between the forces. This can be repeated for several selected concrete strains to determine points on the M-ϕ diagram.



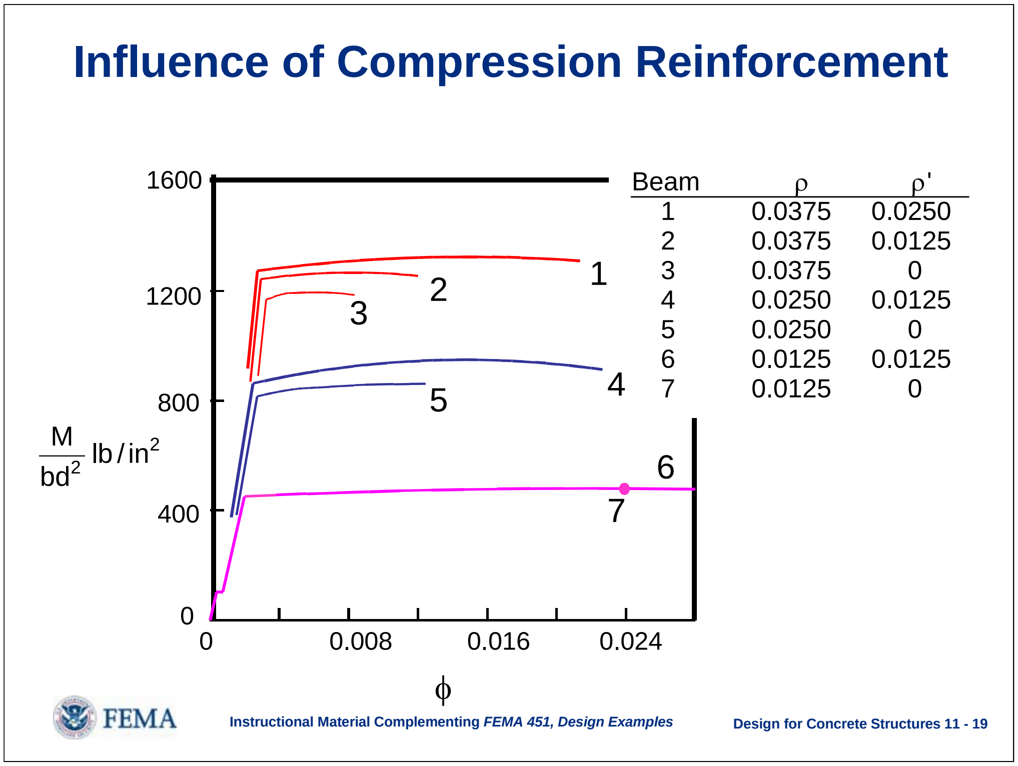
After yield but before the onset of strain hardening, the same method as presented on the previous slide can be used; however, the force in the steel will be $A_s f_y$. This method can be used for points up to the concrete crushing strain of 0.003. The Whitney stress block method is a good method to calculate the final point on the moment curvature diagram, but cannot be used for other points. Typically strain hardening is not considered in design.



This slide shows moment-curvature diagrams for a rectangular section in flexure. Strain hardening in the tension steel increases the final strength. A concrete strain of 0.003 corresponds to maximum strength.

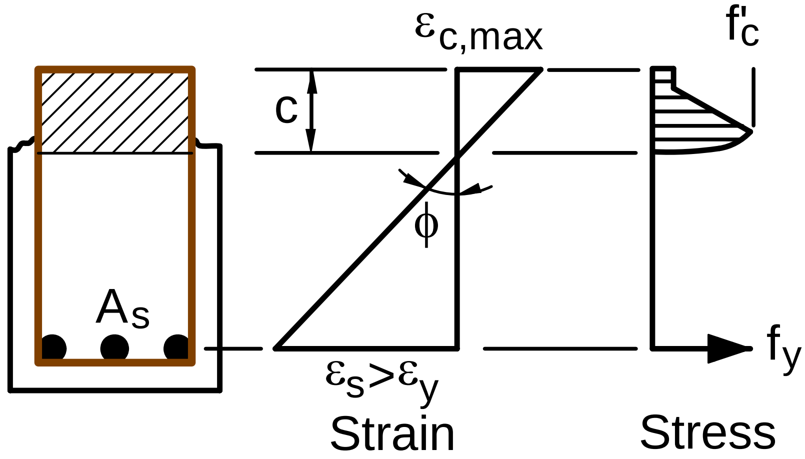


This slide shows moment-curvature diagrams for various amounts of tension reinforcement. As the steel percentage increases, the moment capacity also increases, but the curvature at ultimate moment capacity is decreased (less ductility). Ductile behavior is very desirable in seismic force resisting systems. A common measure of ductility is the ratio of curvature at first yield to curvature at ultimate. This is known as curvature ductility.



This slide shows moment-curvature diagrams for various amounts of tension and compression reinforcement. An increase in the compression reinforcement ratio only slightly increases moment capacity but significantly increases curvature at ultimate moment capacity (more ductility). This is because when the tension force does not change (ρ is constant) and neither does the compression force. With larger amounts of compression reinforcement the steel carries more of the compression, so the concrete carries less. This means the depth to the neutral axis is more shallow, so the curvature at ultimate (0.003/*c*) is larger. However, since *C* and *T* do not change and there is only a slight increase in the moment arm, the moment capacity only increases slightly. (Note: Curve 7 stops at about 0.025; Curve 6 continues off the graph.)

Moment-Curvature with Confined Concrete



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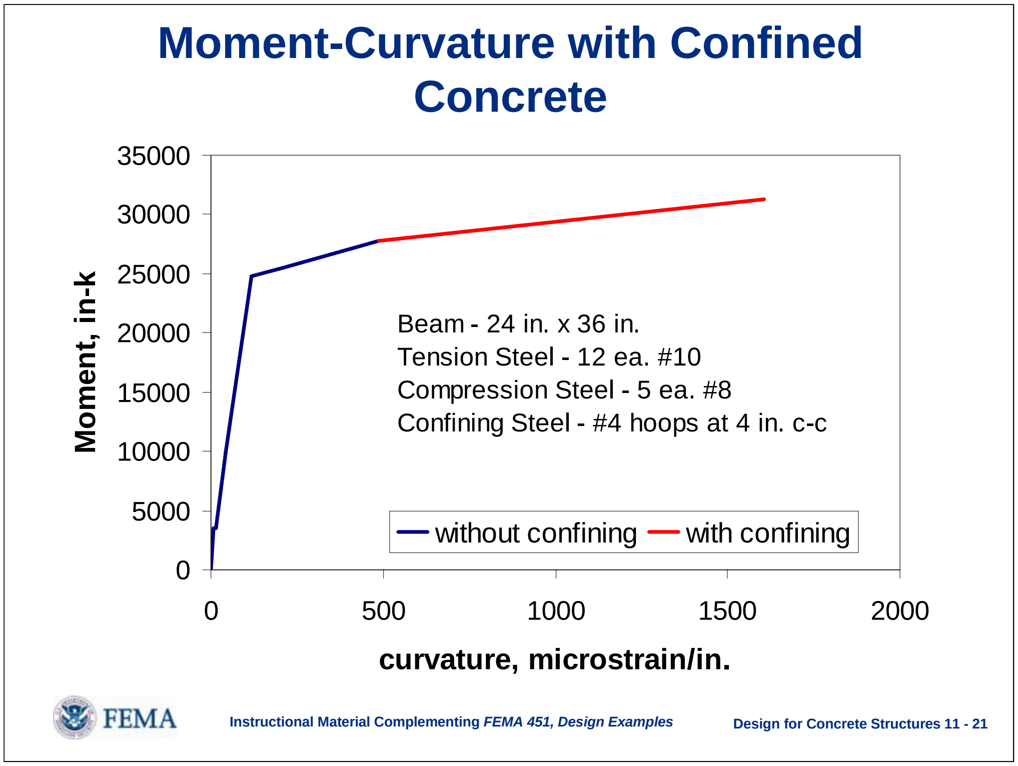
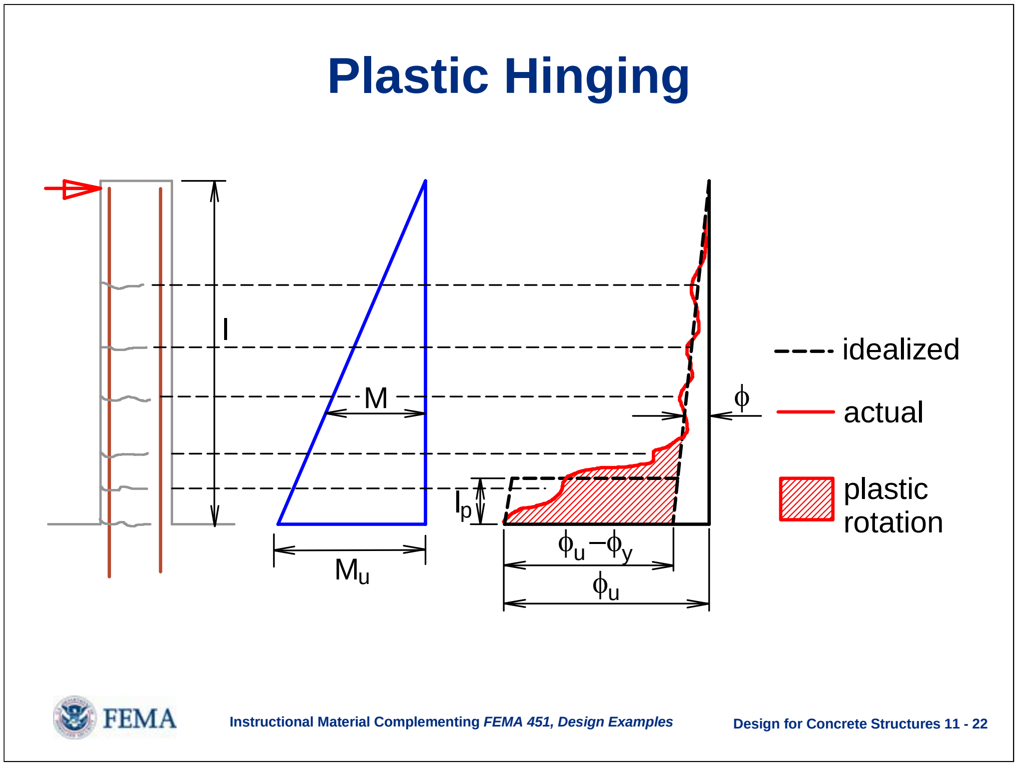
The presence of confining reinforcement can significantly increase the maximum achievable curvature. After the strain on the compression face exceeds 0.003, the cover over the confining steel will spall, however the concrete within the core will remain intact. A model such as the Kent and Park model presented earlier can be used with the strain compatibility method to calculate moments and corresponding curvatures.



This slide presents the results of the analysis of a beam, whose dimensions and reinforcing details are given on the slide. As you can see, the addition of the confining reinforcing increases the usable curvature from just under 500 microstrain per inch to just over 1600. The Scott, Park, and Preistley model was used to model the behavior of the confined concrete. This model accounts for the increase in concrete compressive strength. In addition the compression steel was able to yield, and strain hardening was considered in the tension steel. These three factors combined to result in an increase in moment capacity from the confining steel, even though the cover concrete was lost.



This slide shows how spreading plasticity can significantly increase plastic rotation and displacements. The curvature diagram shows a region of very high curvatures (beyond the yield curvature, ϕ_y) at maximum moment and elastic response in other regions. The region of curvatures past yield curvature is known as the plastic hinge region. The irregular curvature on the "actual" curve is due to cracking.

The plastic rotation and the tip displacement can be calculated from the "actual" curvature diagram, or from the idealized curvature diagram. The idealized diagram is based on a bi-linear approximation of the moment-curvature diagram and an assumed length of the plastic hinge, l_p.

Strategies to Improve Ductility

- **Use low flexural reinforcement ratio**
- **Add compression reinforcement**
- **Add confining reinforcement**

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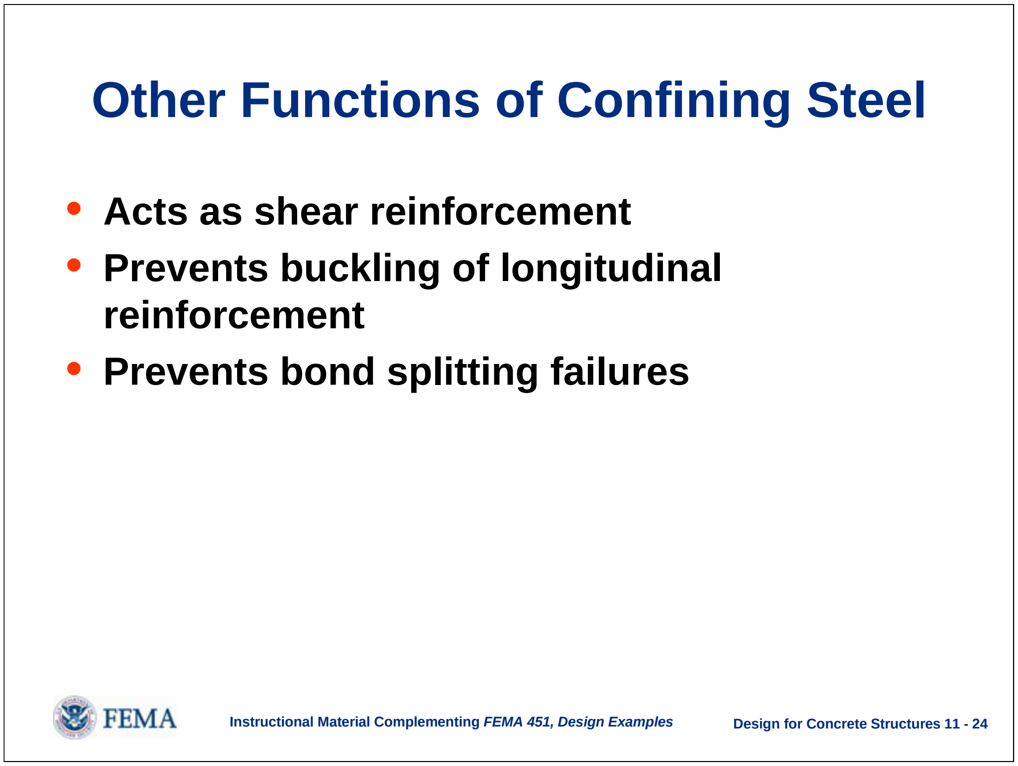
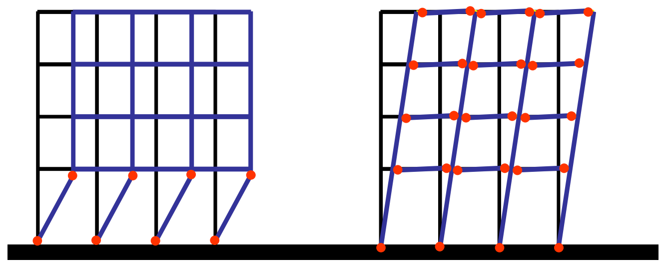
This discussion presented three strategies for improving ductility. These are presented in this slide.



Confining reinforcing also has other useful functions that are presented in this slide.

Structural Behavior
Frames



Story Mechanism   Sway Mechanism

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With an understanding of reinforced concrete member behavior, reinforced concrete systems can be designed to ensure acceptable behavior in a seismic event. We will now discuss desirable system behaviors.

The goal in design of structural frames is to size and reinforce members such that when subjected to large lateral displacements the hinges form in the beams adjacent to the columns, but the columns remain relatively undamaged. This is known as the “strong column-weak beam” approach that is illustrated in the right frame in this slide. A “weak column-strong beam” design can result in the undesirable story mechanism, also known as a soft story, that is shown in the left illustration.

Story Mechanism

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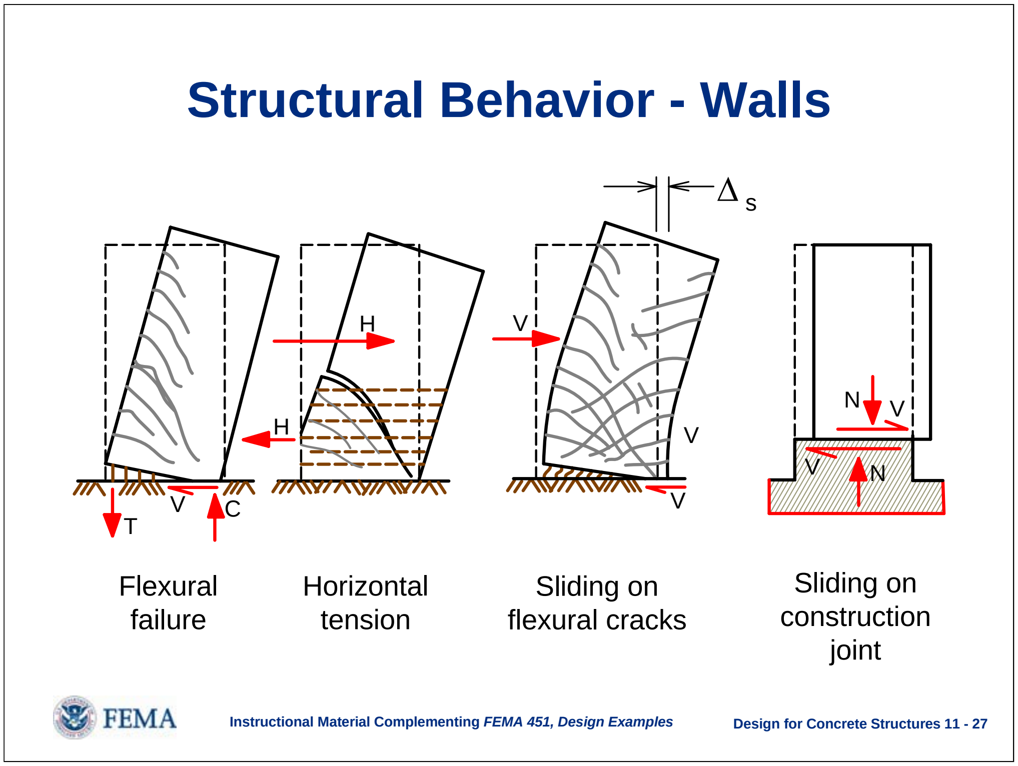
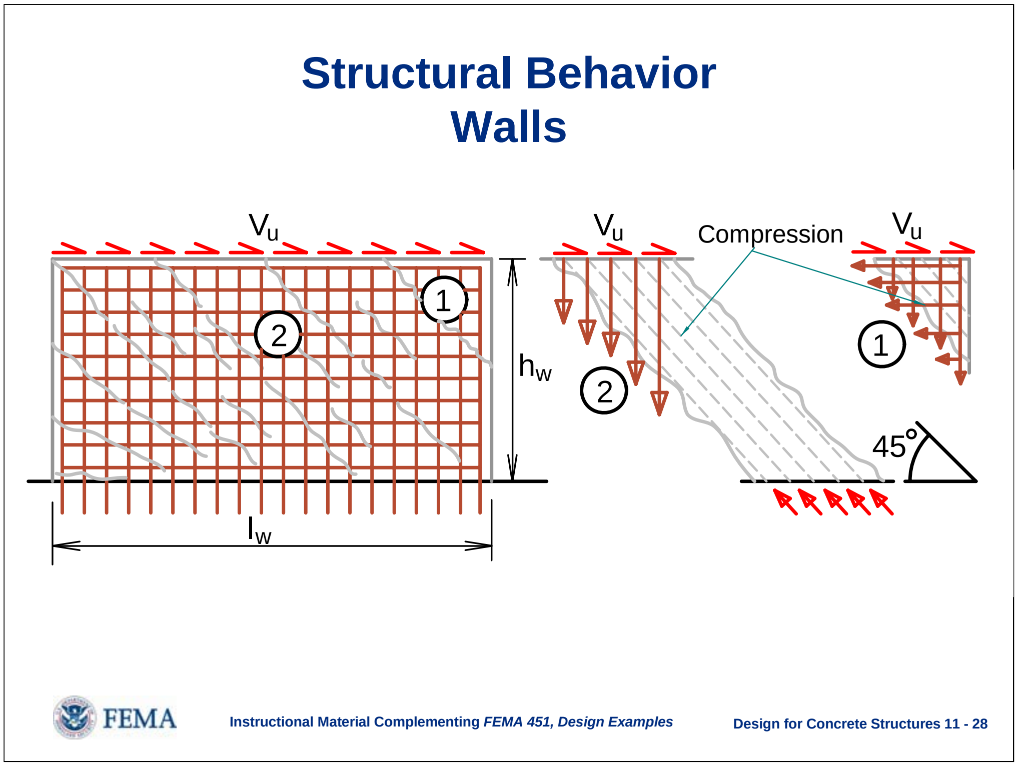
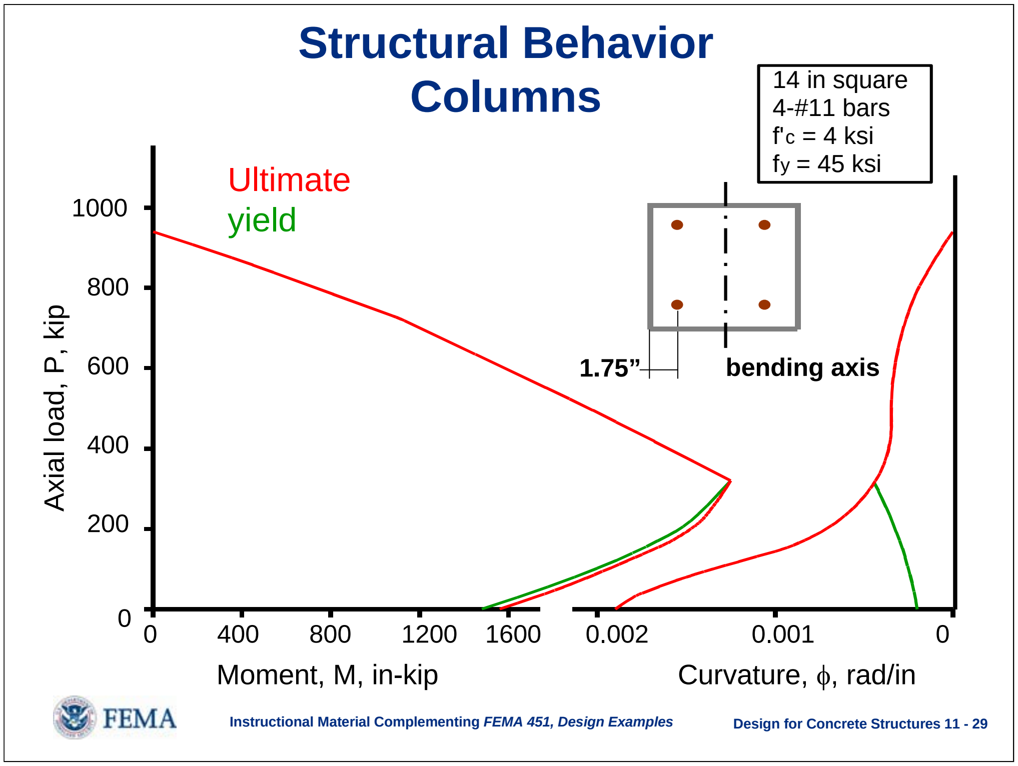
This slide illustrates a story mechanism.



This figure shows types of failures in shear walls. The left figure shows a flexural failure with a plastic hinge zone at the base of the wall. The second figure shows that severe cracking necessitates that web reinforcement carries the horizontal shear force. The last two figures show types of sliding failures: sliding along full depth flexural cracks or along construction joints. The most desirable is the flexural failure with other modes precluded. With proper detailing, the wall can exhibit good strength and ductility without excessive drift or collapse.

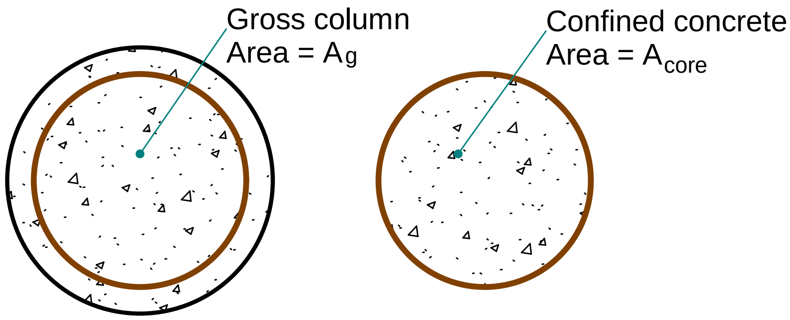


This slide shows how both horizontal and vertical reinforcement provide shear resistance for low-rise shear walls.



In strong column-weak beam design, undesirable failures in the columns must be precluded through proper design and detailing. This slide presents the *P-M* curve on the left and the *P*-curvature curve on the right. Note that the presence of large axial loads reduces the curvature at ultimate. Axial loads above the balanced point reduce ductility of beam-columns since the reinforcing steel on the tension side of the column never yields. Confinement reinforcement improves axial ductility, but this plot shows curvature ductility, which is more important in frames. The strong column-weak beam design approach ensures that failure will initiate in ductile beams rather than in brittle columns.

Influence of Hoops on Axial Strength



Before spalling-
$P = A_g f'_c$

After spalling-
$P = A_{core}(f'_c + 4\, f_{lat})$

After spalling $\geq$ Before spalling

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The strength of an unconfined concrete column is the gross area times the unconfined compressive strength. After the concrete outside the spiral, hoops or ties has spalled, the strength of the column is the core area times the enhanced compressive strength. Work done in the 1920s by Richart et al. indicated that confined concrete strength is roughly the unconfined strength plus 4 times the confining pressure, f_{lat}. The goal in designing the hoops is to ensure that the strength after cover spalling is not less than the strength before spalling.

Column with Inadequate Ties

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This photo shows a column with inadequate ties which provided almost no confinement. Olive View Hospita after the 1971 San Fernando earthquake.

Well Confined Column

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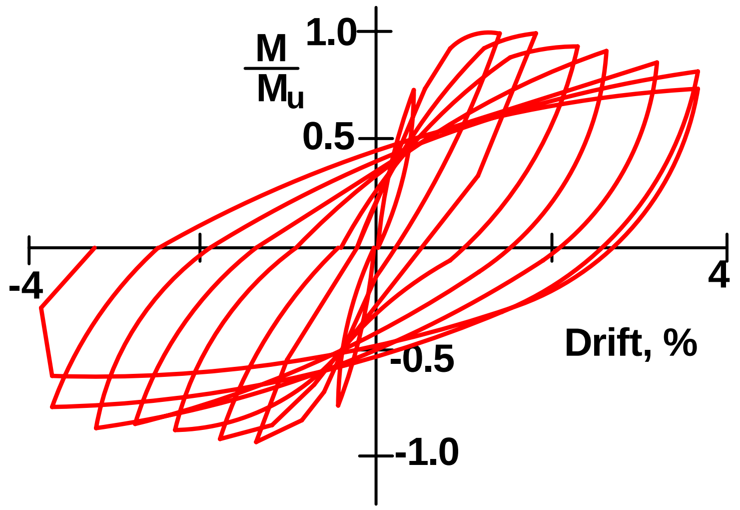
This slide shows a column with an adequate amount of spiral confinement. After the cover spalled, the well confined core remains intact and able to carry axial loads. Olive View Hospital after the 1971 San Fernando earthquake.

Hysteretic Behavior of Well Confined Column



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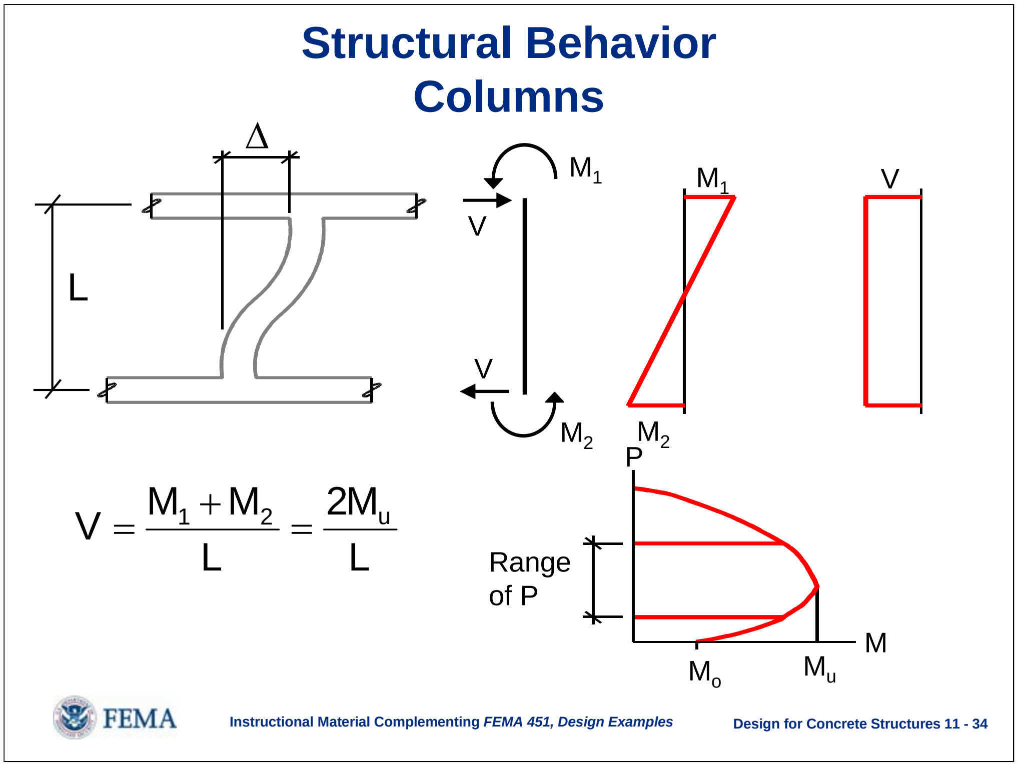
Design for Concrete Structures 11 - 33

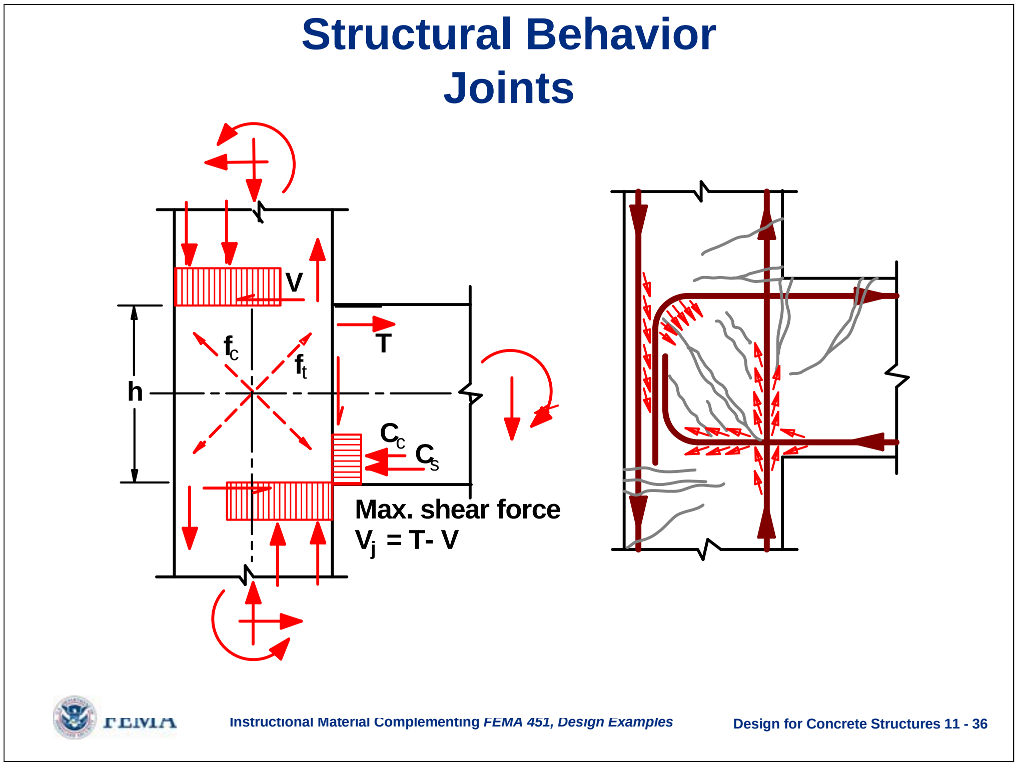
This type of hysteresis loop shows good performance of a column with generous confinement reinforcement. The preferred type of hysteresis loop shows only small degradation of moment strength with increased imposed drift. Also the loops remain “fat” which indicates good energy dissipation.



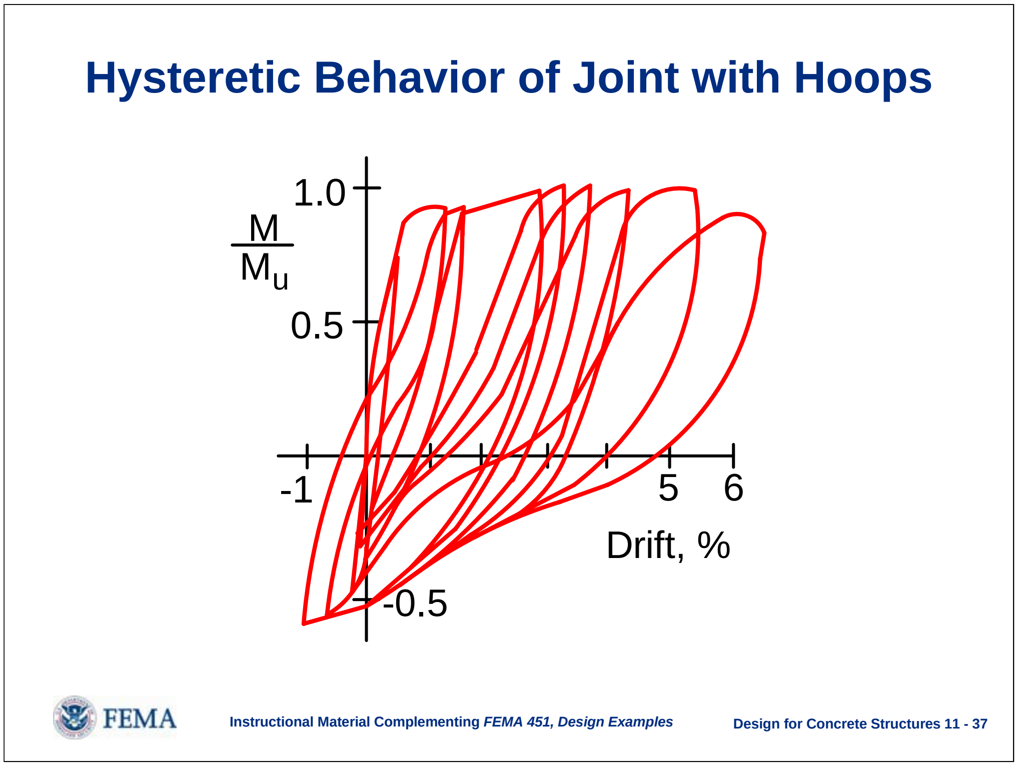
To ensure strong column-weak beam behavior, shear failures of columns must also be precluded. However, shear in a concrete column can be critical. Shear is maximum in a column when the moments at each end are at ultimate. The moment capacity of a column depends on the magnitude of the axial load. To avoid shear failures, the design should focus on the axial load that produces the largest moment capacity. The *P-M* interaction diagram shows this range of axial loads for an example column.

Column Shear Failure

This photo shows a shear failure of a bridge pier after the 1971 San Fernando earthquake.

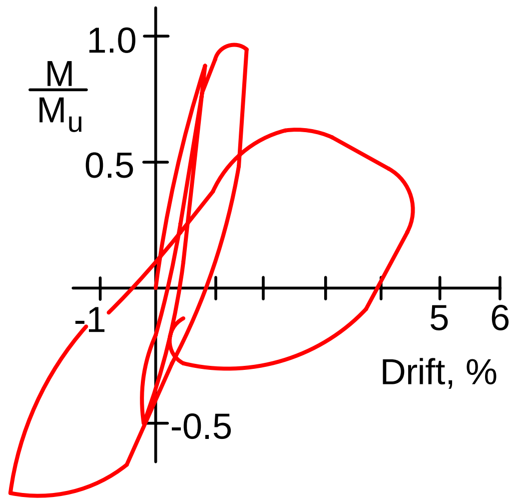


Another location in frames where premature failures must be precluded is the beam-column joints. This slide shows joint actions. The left figure shows forces (stresses) imposed on a typical exterior joint, and the right shows cracks. Upon reversal of direction, perpendicular cracks form. The anchorage of the reinforcement can be compromised. The important aspects of joint design are ensuring proper bar development and precluding shear failures in the joint. This can be accomplished through proper detailing of hoop reinforcement and bar hooks.



This slide shows a typical hysteresis loop for a joint with hoops. The joint shows good performance under repeated reversed loads.

Hysteretic Behavior of Joint with No Hoops



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This slide shows a typical hysteresis loop of a joint without confining hoops. Note the rapid deterioration of the joint.

Joint Failure – No Shear Reinforcing

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This photo is of a joint failure in shear (1971 San Fernando earthquake). Note that there is NO shear reinforcement in the joint and the joint is too small. The joint can no longer transmit moments.

Anchorage Failure in Column/Footing Joint

Another type of failure which must be prevented in order to ensure ductile frame behavior is the failure of the joint between the column and the footing. This slide shows an anchorage failure of a bridge column (1971 San Fernando earthquake).

Summary of Concrete Behavior

- **Compressive Ductility**
  - Strong in compression but brittle
  - Confinement improves ductility by
    - Maintaining concrete core integrity
    - Preventing longitudinal bar buckling
- **Flexural Ductility**
  - Longitudinal steel provides monotonic ductility at low reinforcement ratios
  - Transverse steel needed to maintain ductility through reverse cycles and at very high strains (hinge development)

FEMA Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 41

We will now review reinforced concrete behavior.

Concrete is strong in compression but brittle. Confinement improves compressive ductility by limiting transverse expansion in the concrete. As the transverse steel ties take the strain in tension, the concrete core maintains its integrity. Closely spacing the ties will limit longitudinal bar buckling and thus contribute to improved compressive ductility. Longitudinal steel provides flexural ductility at low reinforcement ratios for a single overload. Transverse steel is needed to maintain integrity of the concrete core (which carries compression and shear), and prevent longitudinal bar buckling after the cover has spalled and crossing cracks form. A relative balance of tension and compression steel aids flexural ductility. The amount of longitudinal tension steel must be limited to insure a tension-type failure mode.

Summary of Concrete Behavior

- **Damping**
  - Well cracked: moderately high damping
  - Uncracked (e.g. prestressed): low damping
- **Potential Problems**
  - Shear failures are brittle and abrupt and must be avoided
  - Degrading strength/stiffness with repeat cycles
    - Limit degradation through adequate hinge development

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The level of damping in concrete structures depends on the amount of cracking. It is important to avoid potential problems in concrete structures: shear failures in concrete are brittle and abrupt and must be avoided; repeated loadings degrade strength and stiffness as concrete cracks and steel yields. Degradation can be limited by assuring adequate hinge development.

NEHRP Recommended Provisions Concrete Design

- Context in the *Provisions*
- Concrete behavior
- Reference standards

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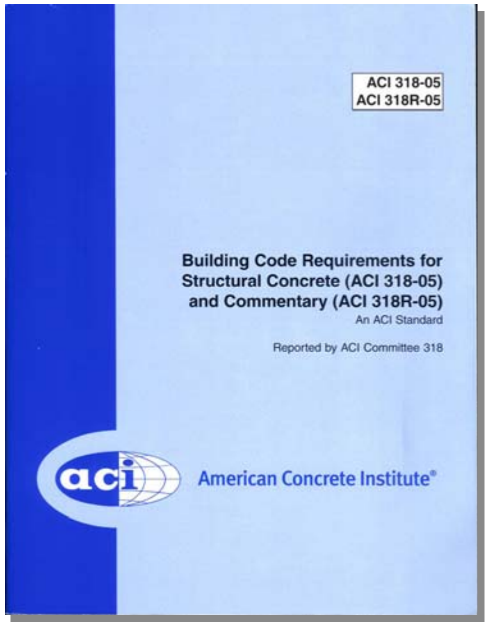
We will now discuss the standards that are referenced by the *NEHRP Recommended Provisions.*



ACI 318-05

ACI 318-05. The basis of NEHRP 2003 is actually ACI 318-02; however, changes from 2002 to 2005 were relatively minor. The examples in this presentation will be based on 2005 since it is the most recent 318 set of building code requirements and commentary.

Use of Reference Standards

- **ACI 318-05**
  - Chapter 21, Special Provisions for Seismic Design
- **NEHRP Chapter 9, Concrete Structures**
  - General design requirements
  - Modifications to ACI 318
  - Seismic Design Category requirements
  - Special precast structural walls
  - Untopped precast diaphragms (Appendix to Ch.9)

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The most important section of ACI 318-05 is Chapter 21. The *Provisions* Chapter 9 presents some modifications to ACI 318-05 Chapter 21 as well as some additional reinforced concrete structure requirements. This presentation will not cover the precast concrete provisions in any detail.

Detailed Modifications to ACI 318

- **Modified definitions and notations**
- **Scope and material properties**
- **Special moment frames**
- **Special shear walls**
- **Special and intermediate precast walls**
- **Foundations**
- **Anchoring to concrete**

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Section 9.2.2 of the *Provisions* modifies various sections of Chapter 21 of ACI 318-05. Some of the modifications are presented in this slide.

NEHRP Recommended Provisions
Concrete Design

- **Context in the *Provisions***
- **Concrete behavior**
- **Reference standards**
- **Requirements by Seismic Design Category**

This section presents some of the requirements for reinforced concrete seismic-force-resisting systems by Seismic Design Category.

Design Coefficients - Moment Resisting Frames

Seismic Force Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d
Special R/C Moment Frame	8	5.5
Intermediate R/C Moment Frame	5	4.5
Ordinary R/C Moment Frame	3	2.5

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This slide presents the design coefficients presented in the *Provisions* Table 4.3-1 and in ASCE 7-05 Table 12.2-1. These tables also present system limitations and height limits by Seismic Design Category (not shown in slides).

Design Coefficients
Shear Walls (Bearing Systems)

Seismic Force Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d
Special R/C Shear Walls	**5**	**5**
Ordinary R/C Shear Walls	**4**	**4**
Intermediate Precast Shear Walls	**4**	**4**
Ordinary Precast Walls	**3**	**3**

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This slide presents the coefficients for shear walls that are part of a bearing wall system.

Design Coefficients
Shear Walls (Frame Systems)

Seismic Force Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d
Special R/C Shear Walls	6	5
Ordinary R/C Shear Walls	5	4.5
Intermediate Precast Shear Walls	5	4.5
Ordinary Precast Walls	4	4

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This slide presents the coefficients for shear walls that are part of a building frame system.

Design Coefficients
Dual Systems with Special Frames

Seismic Force Resisting System	Response Modification Coefficient, R	Deflection Amplification Factor, C_d
Dual System w/ Special Walls	8 (7)	6.5 (5.5)
Dual System w/ Ordinary Walls	6	5

(ASCE 7-05 values where different)

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This slide presents the coefficients for dual systems that include a special moment resisting frame. Again note the differences between coefficients presented in the 2003 *NEHRP Recommended Provisions* and in ASCE 7-05.

Frames

Seismic Design Category	Minimum Frame Type	ACI 318 Requirements
A and B	Ordinary	Chapters 1 thru 18 and 22
C	Intermediate	ACI 21.2.1.3 and ACI 21.12
D, E and F	Special	ACI 21.2.1.4 and ACI 21.2, 21.3, 21.4, and 21.5

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The *Provisions* define three types of frames: ordinary, intermediate, and special. **Ordinary** moment frames are designed using the requirements of ACI 318 exclusive of the seismic provisions (Ch. 21). **Special** moment frames must meet modified requirements of ACI 318, Chapter 21, including detailing to ensure ductility, stability, and minimum degradation of strength during cyclic loading. **Intermediate** moment frames must meet requirements of ACI 318 section 21.12 (more stringent detailing than for ordinary frames but less severe than body of Ch. 21, for special frames). This extra attention to detailing creates stronger, more reliable buildings.

A review of Table 4.3-1 in the *Provisions* (excerpts shown on the previous slides) shows that the values of R and C_d reflect the expected behavior of the various types of moment frames. The Seismic Design Category (SDC) dictates what type of frame may be used. In SDCs A and B, ordinary frames may be used. Intermediate frames are required (at a minimum) in SDC C (although a special frame may be more economical because the higher R will mean lower design forces). For SDCs D, E and F, frames must be special.

There are exceptions to the limitations on type of frame, especially for nonbuilding structures of limited height.

Reinforced Concrete Shear Walls

Seismic Design Category	Minimum Wall Type	ACI 318 Requirements
A, B and C	Ordinary	Chapters 1 thru 18 and 22
D, E and F	Special	ACI 21.2.1.4 and ACI 21.2 and 21.7

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For reinforced concrete shear walls, the applicability of ACI 318 Chapter 21 varies with Seismic Design Category as shown on the slide. Note the differences between walls and frames. Plain concrete walls, designed per Chapter 22, are permitted in SDCs A and B for some circumstances.

Precast Concrete Shear Walls

Seismic Design Category	Minimum Wall Type	ACI 318 Requirements
A and B	Ordinary	Chapters 1 thru 18 and 22
C	Intermediate	ACI 21.2.1.3 and ACI 21.13
D, E and F	Special	ACI 21.2.1.4 and ACI 21.2, 21.8

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Precast shear walls are also allowed to be part of the seismic force resisting system. The intent for special precast walls is that they qualify for the same design parameters as the special cast-in-place wall

ACI 318-05 contains a section on special precast walls (Section 21.8); however, the system is not presented in the *Provisions* Table 4.3-1. There is a large section in Chapter 9 of the *Provisions*, Section 9.6, that presents acceptance criteria for special precast structural walls based on validation testing. This presentation does not include detailed information on precast walls.

Additional *Provisions* Requirements

- **Category C**
  - Discontinuous members
  - Plain concrete
    - Walls
    - Footings
    - Pedestals (not allowed)

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This slide presents additional requirements by Seismic Design Category as found in Section 9.4 of the *Provisions*.

NEHRP Recommended Provisions Concrete Design

- Context in the *Provisions*
- Concrete behavior
- Reference standards
- Requirements by Seismic Design Category
- Moment resisting frames

Instructional Material Complementing *FEMA 451, Design Examples*

Design for Concrete Structures 11 - 56

The basic requirements for moment resisting frames will be presented in this section.

Performance Objectives

- **Strong column**
  - Avoid story mechanism
- **Hinge development**
  - Confined concrete core
  - Prevent rebar buckling
  - Prevent shear failure
- **Member shear strength**
- **Joint shear strength**
- **Rebar development**

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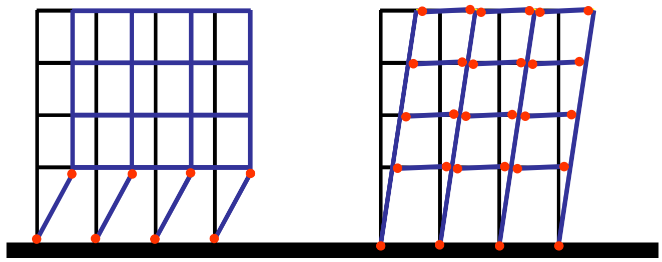
The requirements of Chapter 21 are intended to ensure the performance objectives listed on this slide. The strong column-weak beam design avoids forming a mechanism in a single story (the story mechanism presented earlier). Adequate hinge development is needed for ductility and is accomplished by the use of transverse reinforcement which confines the concrete core and prevents rebar buckling. Shear strength must be adequate to avoid abrupt failures in members and joints. Requirements for rebar anchorage and splicing (such as 135 degree hooks) are intended to maintain the integrity of the design.

Frame Mechanisms
“strong column – weak beam”



Story mechanism Sway mechanism

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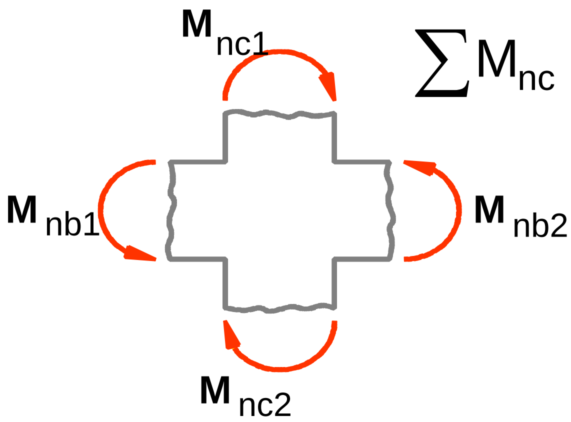
The strong column-weak beam design is required for special moment frames. This slide shows the advantages. For a system with weak columns, a mechanism is created when the columns of only one story reach their flexural capacities (less dissipation of seismic energy prior to collapse). For a system with strong columns and weak beams, a mechanism is created when ALL beams on ALL stories yield (much more seismic energy dissipated prior to collapse).

Required Column Strength



$$\sum M_{nc} \geq 12 \sum M_{nb}$$

To ensure that the beams develop plastic hinges before the columns, the sum of the flexural strengths of the columns at a joint must exceed 120% of the sum of the flexural strengths of the beams. This requirement protects against premature development of a story mechanism, but due to the realities of dynamic response, it does not assure a full building mechanism.

Hinge Development

- **Tightly Spaced Hoops**
  - Provide confinement to increase concrete strength and usable compressive strain
  - Provide lateral support to compression bars to prevent buckling
  - Act as shear reinforcement and preclude shear failures
  - Control splitting cracks from high bar bond stresses

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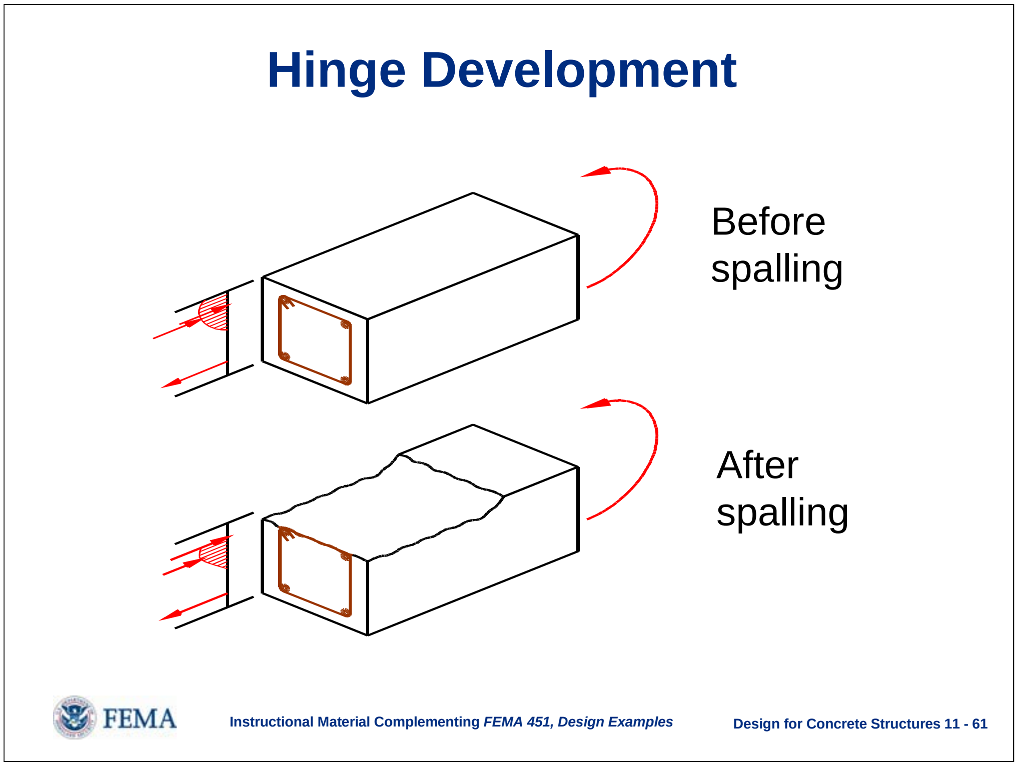
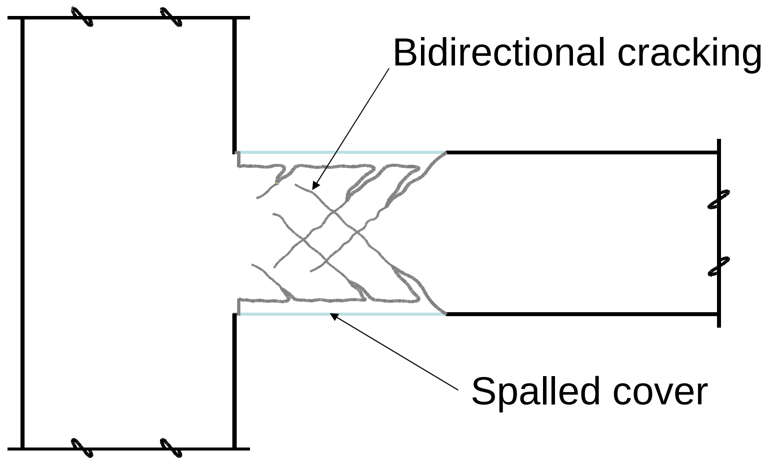
It is also important in this type of system to ensure proper hinge development. The hinges must be able to form and then undergo large rotations and load reversals without significant reduction in strength. In this way, plasticity and hinging will be able to spread throughout the frame. Tightly spaced hoops are required to ensure proper hinge development and behavior. Some of the functions of the hoops are presented in this slide.



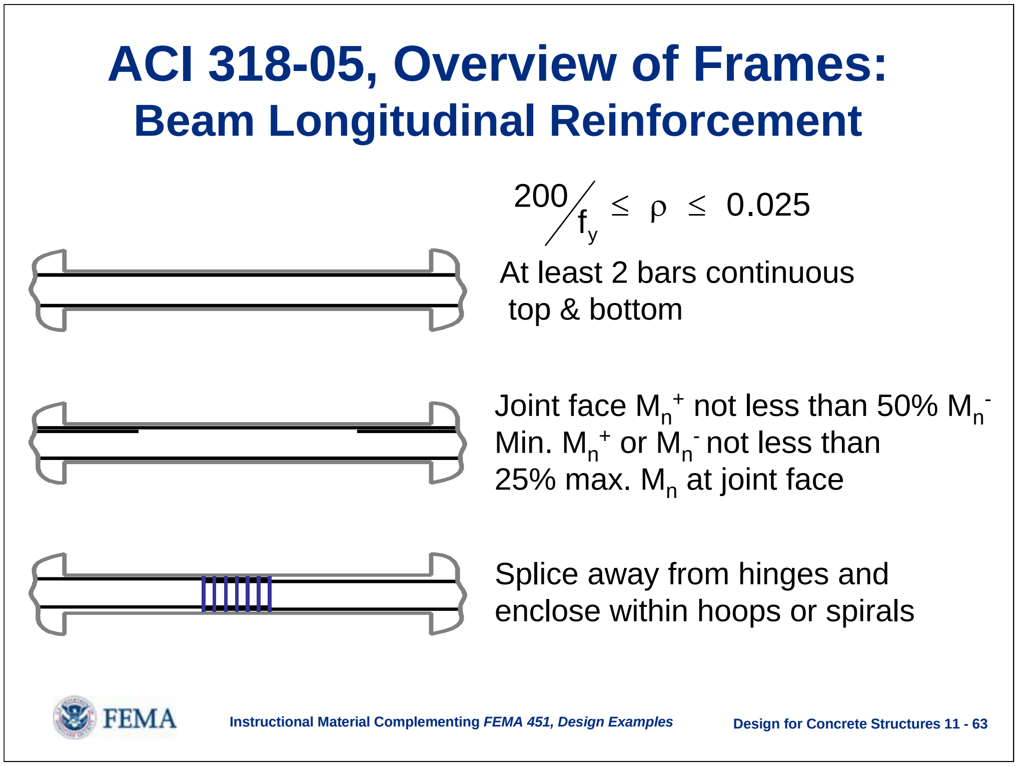
This slide presents some of the mechanics of hinge development. Prior to spalling, the familiar stress diagram is present, with tension in the bottom steel, compression in a roughly parabolic distribution in the concrete, and some compressive stress in the top steel. Upon spalling, the stress distribution changes, The compression block of the concrete moves lower in the cross section, and the stresses in the compression steel are greatly increased. To maintain section integrity, material component failures must be avoided. Concrete crushing and compression bar buckling can be prevented by transverse reinforcement. Closely spaced hoop steel limits lateral strain in the concrete and allows greater useful strains in the concrete and hence improved ductility. Proper spacing of hoops also prevents longitudinal bar buckling.

Hinge Development

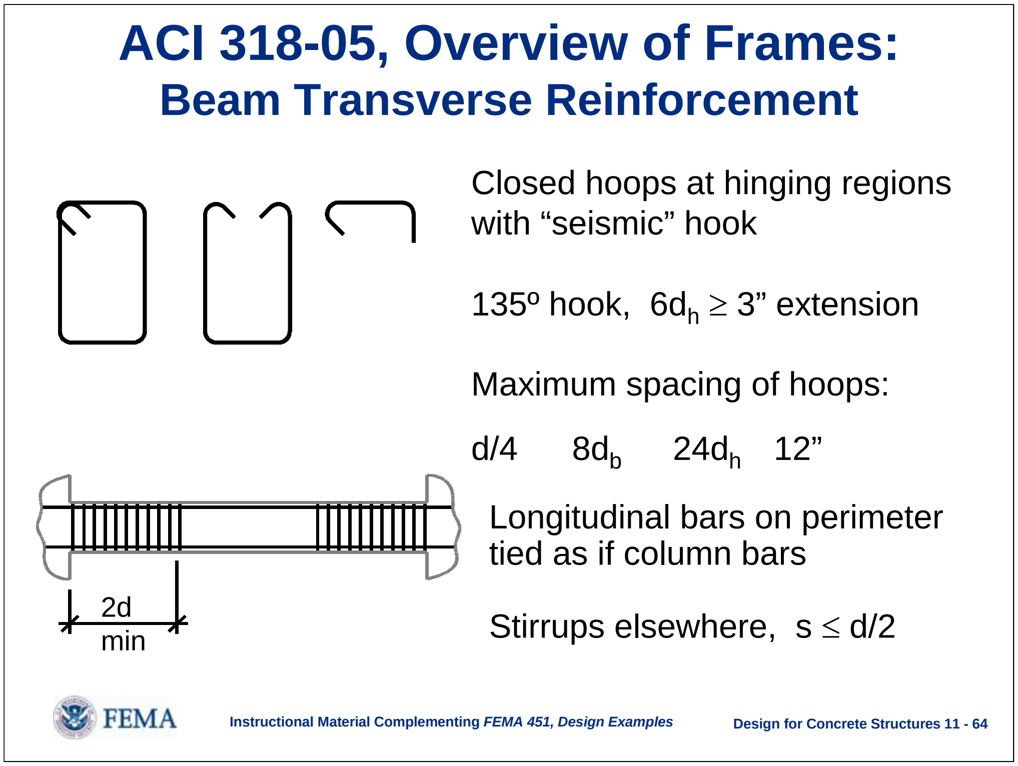


FEMA Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 62

Under reverse load applications, hinge development affects both the top and bottom faces of beams. This leads to bidirectional cracking and spalling of cover on the top and bottom of the beam.

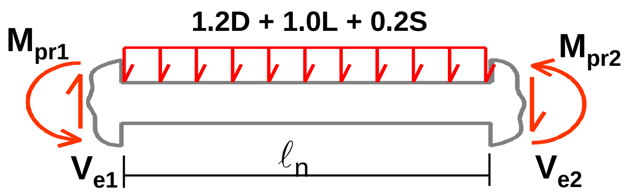


This slide presents the beam longitudinal reinforcement requirements per ACI 318-05. The reinforcement ratio limits insure a tension controlled failure mode in bending and reduce congestion of reinforcing steel. Continuous bars in the top and bottom are required due to reversal of seismic motions and variable live load. Splice locations and transverse reinforcement are specified because lap splices are unreliable and cover concrete will spall.



This slide shows additional beam longitudinal reinforcement requirements per ACI 318-05. Seismic hooks have special detailing requirements to ensure that the hoops do not open after the cover spalls. The maximum hoop spacings ensure adequate confinement of the concrete core and adequate lateral support of the compression reinforcing bars. However, maximum spacing may be dictated by shear design. To prevent longitudinal bar buckling, the requirements for tying column steel also apply to the bars in the expected plastic hinge region (2d from the face of the support).

ACI 318-05, Overview of Frames:
Beam Shear Strength



$M_{pr} = M_n$ with

$f_s = 1.25 f_y, \quad \phi = 1.0$

$$V_e = \frac{M_{pr1} + M_{pr2}}{\ell_n} \pm \frac{w_u \ell_n}{2} \quad \geq V_e \quad \text{by analysis}$$

If earthquake-induced shear force $> \frac{1}{2} V_e$

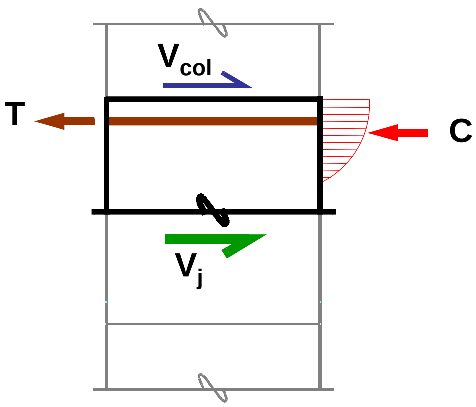
and $P_u < \frac{A_g f'_c}{20}$

then $V_c = 0$

FEMA Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 65

This slide presents the beam shear strength requirements per ACI 318-05. Shear demand is based on the maximum probable flexural strength of the beam. The probable flexural strength is based on the assumption that the flexural reinforcement will achieve a stress of 1.25 times yield. To determine the expected shear from seismic effects, the probable moment strength is applied at each end of the beam and the resulting shear is calculated. This shear demand is added to the demand from gravity loads. For beams (small axial load), concrete shear strength is neglected when the earthquake-induced shear force ((M_{pr1}+M_{pr2})/ℓ_n) represents one-half or more of the design shear strength of the beam.

ACI 318-05, Overview of Frames:
Beam-Column Joint



$$V_j = T + C - V_{col}$$

$$T = 1.25 f_y A_{s,top}$$

$$C = 1.25 f_y A_{s,bottom}$$

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Instructional Material Complementing *FEMA 451, Design Examples*

Design for Concrete Structures 11 - 66

The design shear for joints is determined from the maximum probable flexural capacities of the beams framing into the joint and the shear in the columns. The column shear is also based on the maximum probable flexural strength of the beams. In this way, the joint shear is directly related to the amount of reinforcement in the beams framing into the joints.

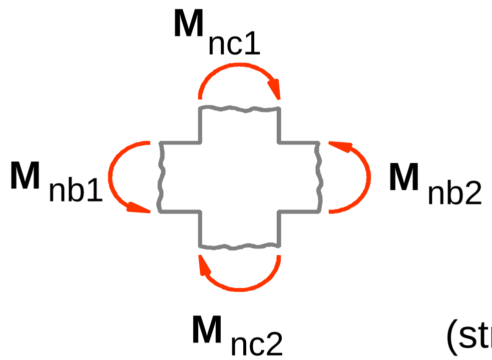
ACI 318-05, Overview of Frames:
Beam-column Joint

$$V_n = \begin{Bmatrix} 20 \\ 15 \\ 12 \end{Bmatrix} \sqrt{f'_c} A_j$$

- **V_n controls size of columns**
- **Coefficient depends on joint confinement**
- **To reduce shear demand, increase beam depth**
- **Keep column stronger than beam**

Joint shear strength is based on the area of the joint, which is usually the area of the column. Nominal joint shear stress is a function of confinement. More confinement implies higher permissible shear stress. The joint shear strength often controls the sizes of the framing members. If additional joint shear strength is required, usually the column size is increased. If beam depth is increased to reduce joint shear, care must be taken to maintain the strong column-weak beam design.

ACI 318-05: Overview of Frames:
Column Longitudinal Reinforcement



$$0.01 \le \rho \le 0.06$$

$$\sum M_{nc} \ge 1.2 \sum M_{nb}$$

At joints
(strong column-weak beam)

M_{nc} based on factored axial force, consistent with direction of lateral forces

FEMA Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 68

This slide presents the column longitudinal reinforcement requirements per ACI 318-05. The limits on reinforcement ratio provide a sizeable difference between cracking and yielding moments and prevent steel congestion. When fulfilling the strong column-weak beam rule, recognize that moment capacity varies with axial load.

ACI 318-05, Overview of Frames:
Column Transverse Reinforcement at Potential Hinging Region

Spirals

$$\rho_s = 0.45\left(\frac{A_g}{A_{ch}} - 1\right)\frac{f'_c}{f_{yt}}$$

and

$$\rho_s \geq 0.12\frac{f'_c}{f_{yt}}$$

Hoops

$$A_{sh} \geq 0.3\left(sb_c\frac{f'_c}{f_{yt}}\right)\left(\frac{A_g}{A_{ch}} - 1\right)$$

and

$$A_{sh} \geq 0.09sb_c\frac{f'_c}{f_{yt}}$$

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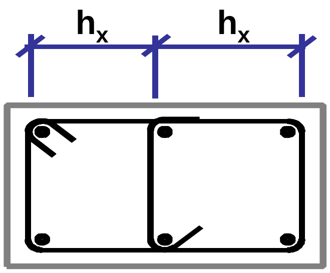
This slide presents the column transverse reinforcement requirements per ACI 318-05. The minima for the area of transverse reinforcement is based on providing adequate confinement of the core concrete to ensure that the strength of the column after the cover has spalled equals or exceeds the strength of the column prior to cover loss. The second equations for the spiral reinforcement ratio or the area of hoops typically govern for large columns.

ACI 318-05, Overview of Frames:

Column Transverse Reinforcement at Potential Hinging Region



$$s_o = 4 + \left(\frac{14 - h_x}{3} \right)$$

Spacing shall not exceed the smallest of:

b/4 or 6 d_b or s_o (4” to 6”)

Distance between legs of hoops or crossties, $h_x \leq 14”$

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Spacing of the transverse reinforcement (s_o) is limited to prevent longitudinal bar buckling. The distance between the legs of rectangular hoops (h_x) is limited because the hoops try to become circular (bend outward due to lateral expansion of confined concrete) after the concrete cover spalls.

ACI 318-05, Overview of Frames:
Potential Hinge Region

- **For columns supporting stiff members such as walls, hoops are required over full height of column if**

$$P_e > \frac{f'_c A_g}{10}$$

- **For shear strength- same rules as beams (concrete shear strength is neglected if axial load is low and earthquake shear is high)**
- **Lap splices are not allowed in potential plastic hinge regions**

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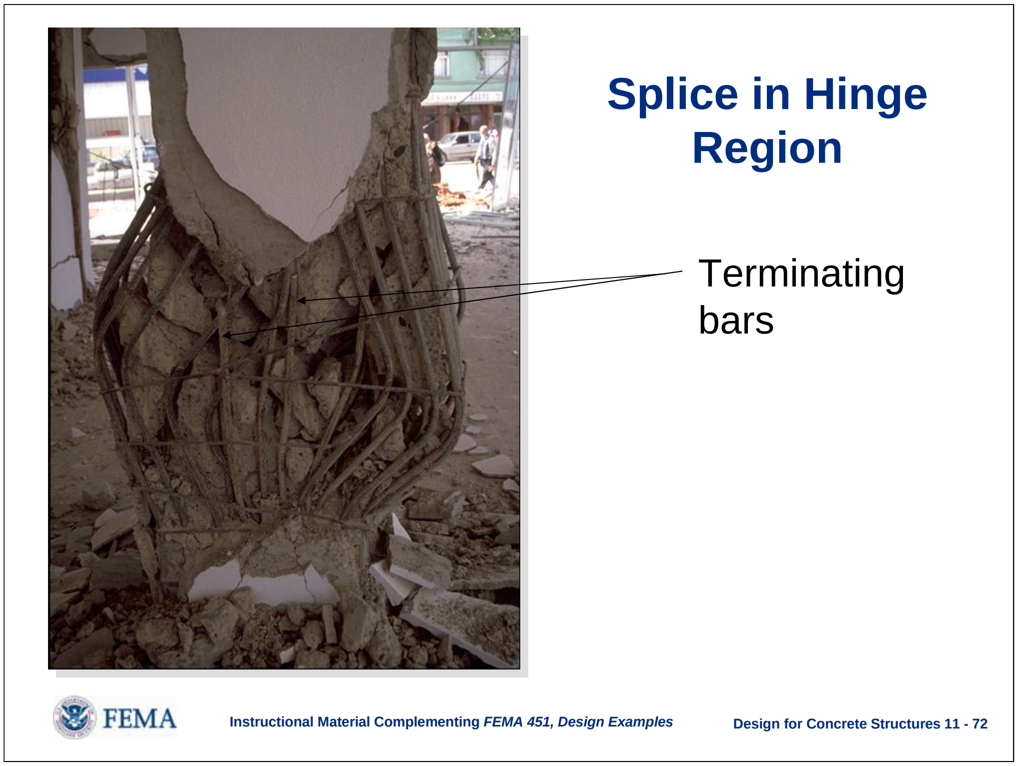
This slide presents other requirements for columns per ACI 318-05. Columns under discontinued stiff members tend to develop considerable inelastic response (thus more transverse reinforcement is required). The shear design is similar to that for beams with the demand calculated based on the maximum probable strengths of the beams framing into the columns; shear strength of concrete is neglected if axial load is low and earthquake-induced shear is more than 50% of the maximum required shear strength within the plastic hinge region.



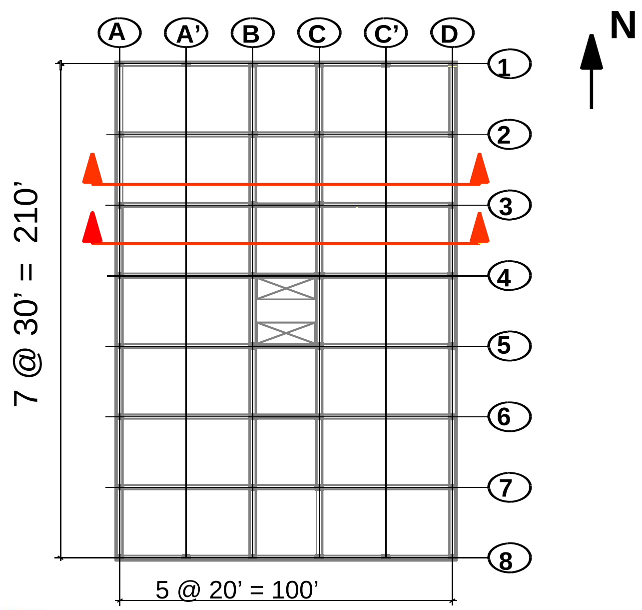
This slide shows a failure at the base of a column that had splices in the hinge region. (Building C, Adapazari, Turkey, 1999 Izmit earthquake.)

ACI 318-05, Overview of Frames:
Potential Hinge Region

$$\ell_o \geq \left\{ \begin{matrix} d \\ \\ \dfrac{clear\ height}{6} \\ \\ 18'' \end{matrix} \right\}$$

This slide presents the definition of the potential hinge region for columns per ACI 318-05. The hinge region is not to be assumed less that the largest of the three values.

Moment Frame Example



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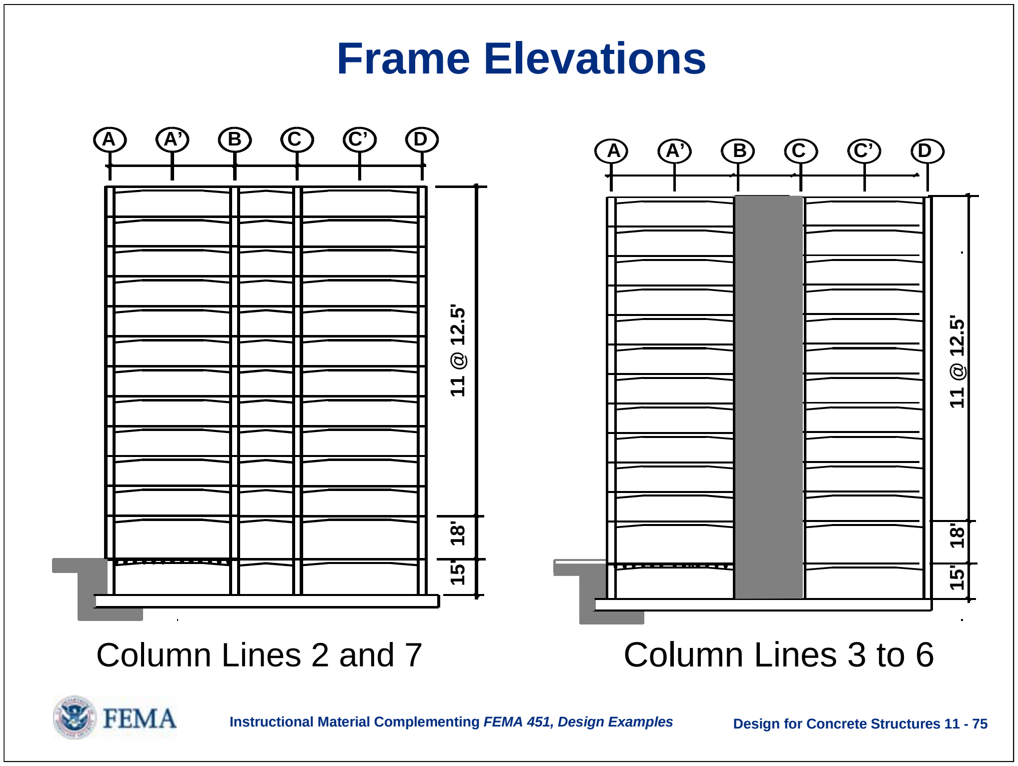
Design for Concrete Structures 11 - 74

We will now work through a design example. This moment frame example is found in Chapter 6 of the *NEHRP Recommended Provisions: Design Examples* (FEMA 451) but it has been updated to reflect changes in the reinforced concrete design requirements. In the North-South direction, the seismic force resisting system is a special moment frame. In the East-West direction, it is a dual system with moment resisting frames on Column Lines 1, 2, 7, and 8, and shear walls between Column Lines B and C along Lines 3-6. Note that Column Lines 1 and 8 have 6 columns while Column Lines 2 and 7 have only 4 columns and have haunched girders between Column Lines A and B and between Lines C and D. The example will focus on beams, columns and joints in the frame on Column Line 1.

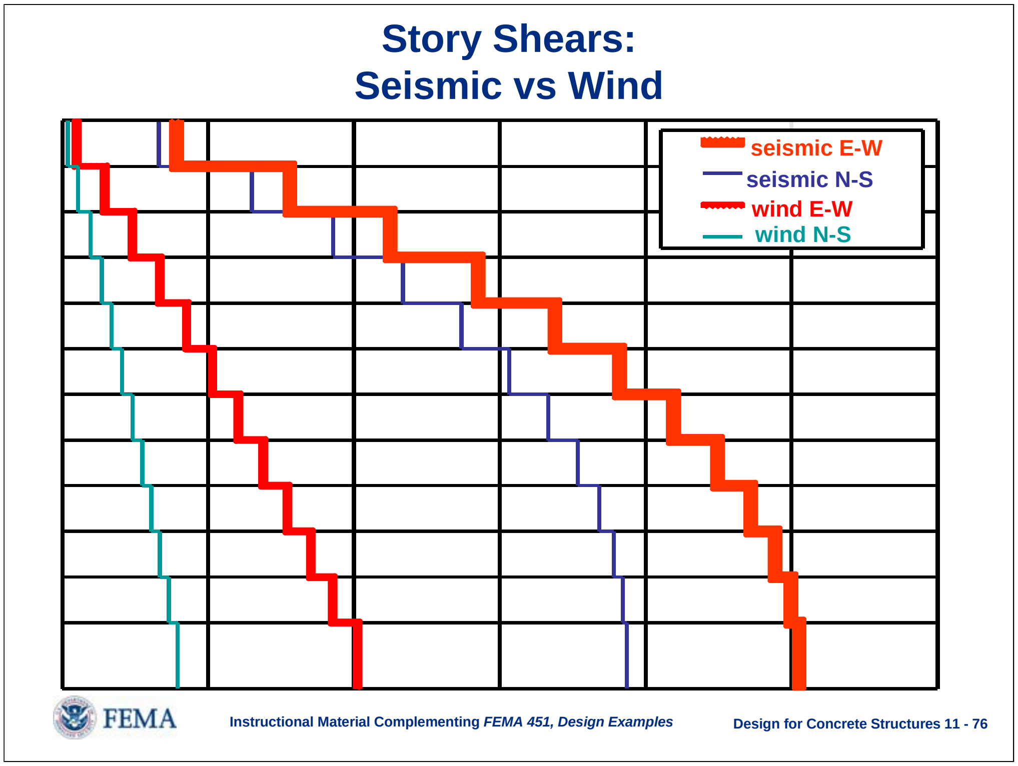
The major difference between the example as presented in FEMA 451 and this presentation is the strength reduction factor (ϕ factor) used for flexure, which was 0.8 in accordance with Appendix C of ACI 318-99 but is now 0.9 in accordance with ACI 318-05.

For this example the seismic forces and analysis of the frame are based on R = 8 and C_d = 6.5, which agrees with the 2000 and 2003 *NEHRP Recommended Provisions*. However, ASCE 7-05 requires R = 7 and C_d = 5.5 for a dual system with special moment frames and special reinforced concrete shear walls. The design is conservative with respect to the beam designs (which then dictate many other aspects of design) and would work for the ASCE requirements as well.

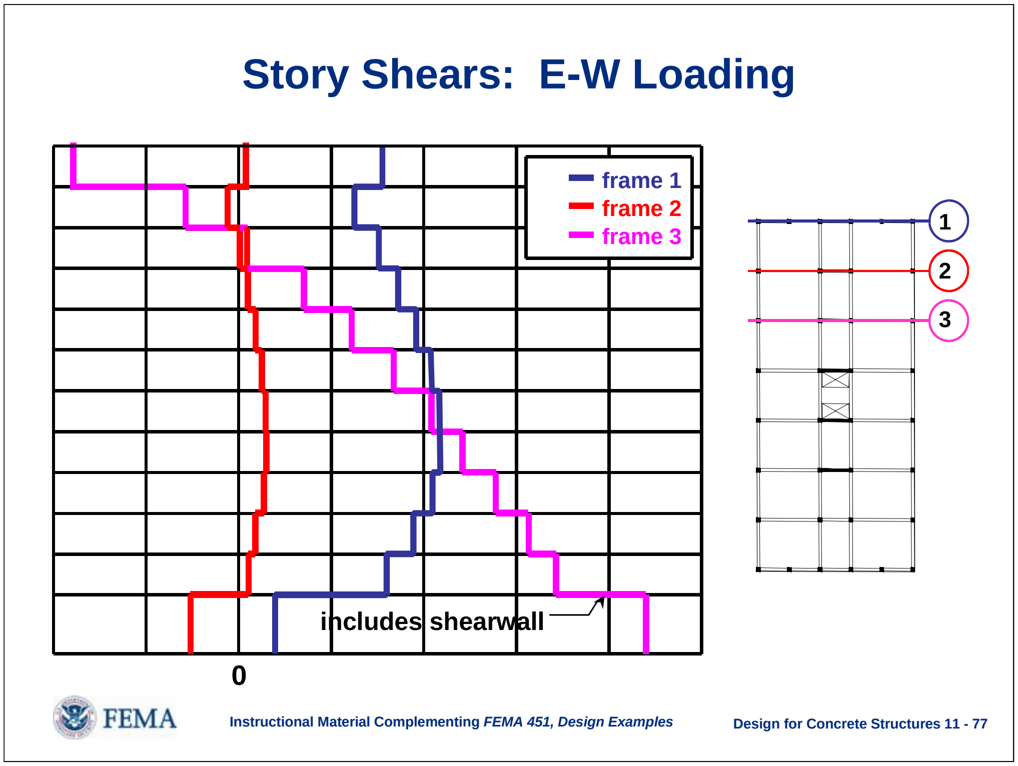


This slide shows the elevation views of the frames on Column Lines 2 and 3. Note the haunched girders on Column Lines 2 and 7 and the shear wall on Column Lines 3 to 6.

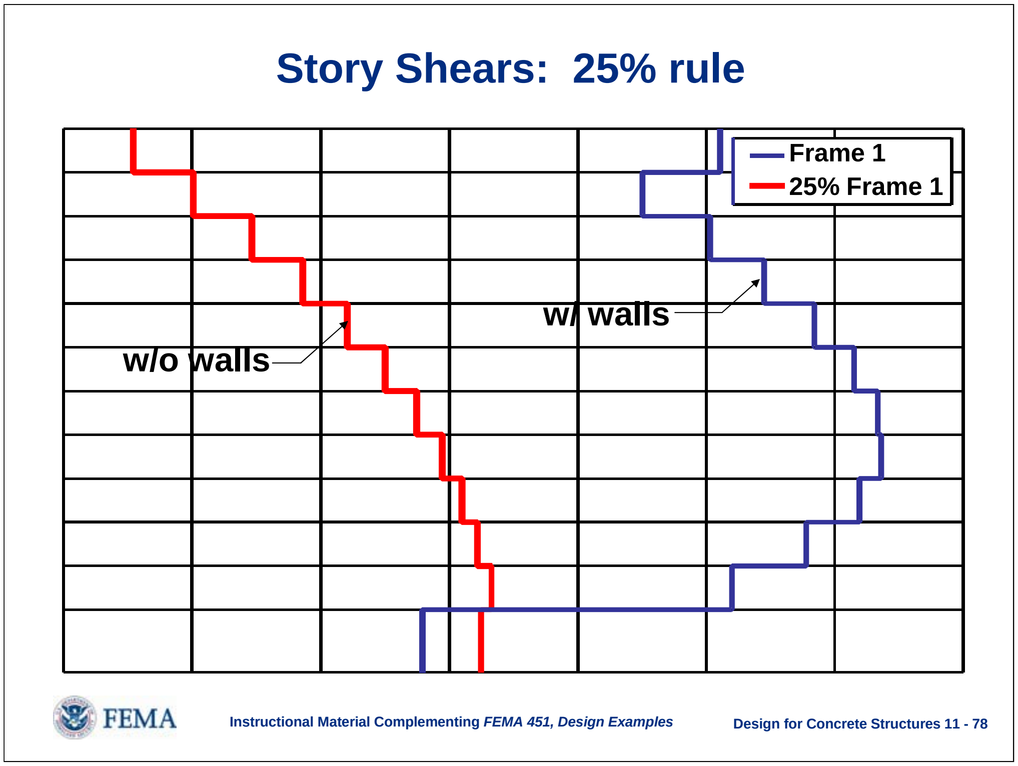
The concrete used in the majority of the building is a sand-lightweight concrete with f'_c = 4000 psi. The exception is in the lowest two levels of the shear walls which are normal weight concrete with f'_c = 6000 psi. To perform the analysis, initial member sizes were estimated then adjusted as the design process required.



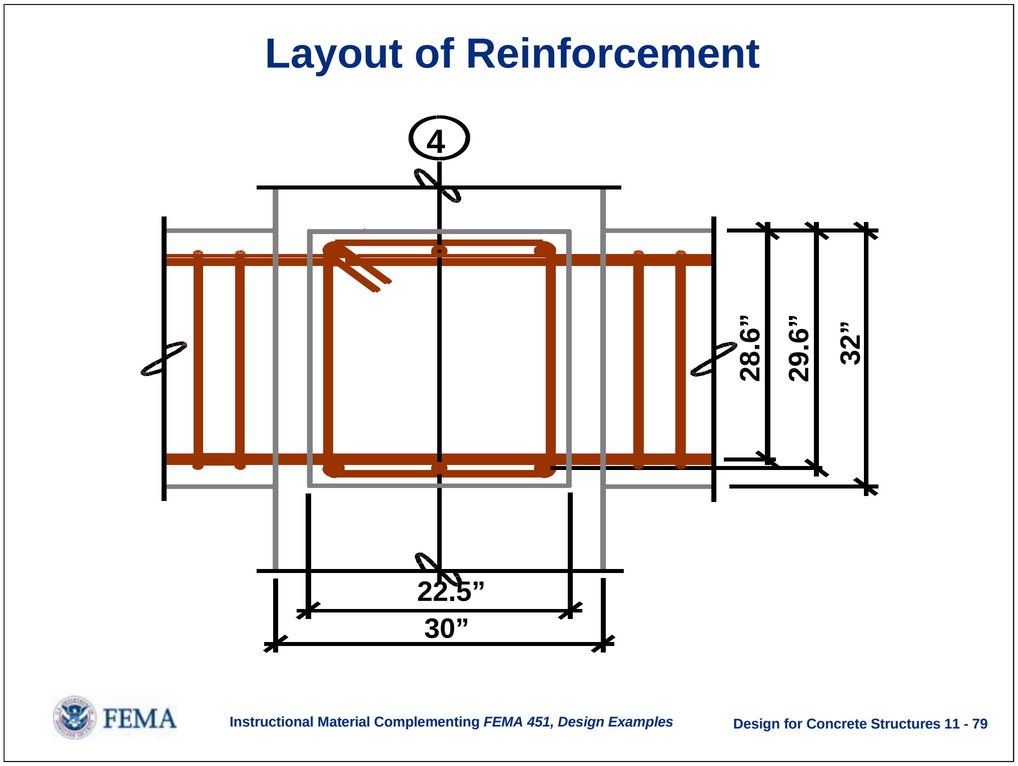
This slide shows the relative magnitudes of the story shears due to seismic and wind design loads. Note that the magnitudes of the seismic forces, even with the *R* factor of 8, are greater than wind forces. This design example is done with the *Provisions* recommended *R* factor of 8 but note that the ASCE 7-05 *R* factor for dual system with special wall is 7.



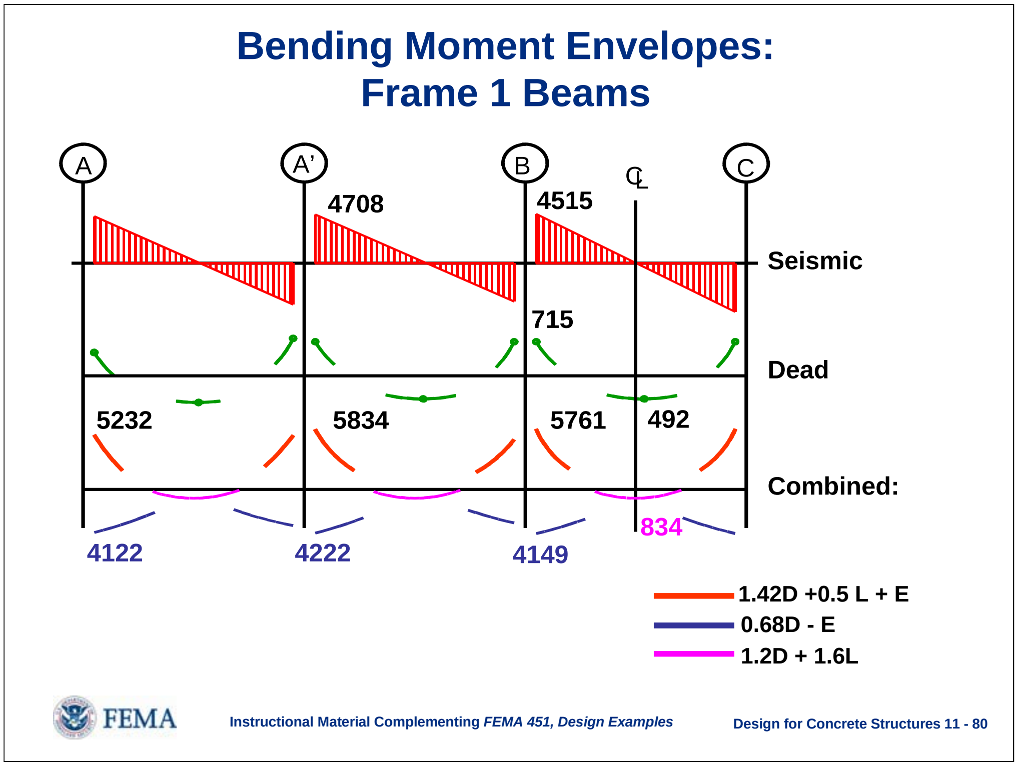
This slide presents the story shears on Frames Lines 1, 2, and 3. Note the locations where story shears are negative for Frames 2 and 3. Also note that the frame line with the shear wall attracts the greatest base shear.



Article 4.3.1.1 of the *Provisions* requires that for dual systems the moment frame without walls must be capable of resisting at least 25% of the design forces. The building was reanalyzed with the walls removed and 25% of the equivalent seismic forces applied. This slide compares story shears from the original analysis with the 25% rule. Note that this rule controls for the ground level.



This slide shows the layout of reinforcement for beams intersecting at Column Line 4 for Frame 1. Note the different values of *d* for beams in each direction.



This slide shows the bending moment envelopes for beams in Frame 1. The structure and the moment envelopes are symmetric about the centerline.

The combinations are:

$1.2D + 1.6L$

$1.2D + 1.0E + 0.5L$

$0.9D + 1.0E$

With E defined as $\rho Q_E \pm 0.2 S_{DS} D$ with $S_{DS} = 1.1$ and $\rho = 1.0$. Q_E is the effect from horizontal seismic forces.

Beam Reinforcement: Longitudinal

Max negative M_u = 5834 in-kips

b = 22.5” d = 29.6” f'_c = 4 ksi f_y = 60 ksi

$$A_{s\ req'd} = \frac{M_u / \phi}{f_y(0.875d)} = \frac{5834 / 0.9}{60 \cdot 0.875 \cdot 29.6} = 4.17\text{in}^2$$

Choose: 2 #9 and 3 #8 A_s = 4.37 in^2

ρ = 0.0066 < 0.025 <u>OK</u>

ϕM_n = 6580 in-kips <u>OK</u>

First the beam longitudinal (top) reinforcement is calculated. We will provide three (must be at least two) continuous bars top and bottom. Additional top bars are provided as necessary for moment capacity. The reinforcement ratio is checked against the maximum allowable. Note that a rough area of steel is calculated by assuming a moment arm of 0.875*d*. The actual moment capacity is then checked with the moment arm determined based on the actual depth of the compression block.

Beam Reinforcement: Longitudinal (continued)

Positive M_u at face of column = 4222 in-kips (greater than ½(5834) = 2917)

b for negative moment is the sum of the beam width (22.5 in.) plus 1/12 the span length (20 ft x 12 in./ft)/12, b = 42.5 in.

$$A_{s\,req'd} = \frac{M_u/\phi}{f_y(0.9d)} = \frac{4222/0.9}{60 \cdot 0.9 \cdot 29.6} = 2.94\text{in}^2$$

FEMA Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 82

Next the reinforcement for positive moment at the face of the column is determined. Note that the calculated positive moment demand is greater than half of the negative moment demand. The larger of these two values governs design. For calculation of positive moment capacity the effective width of the beam, since the compression block is within the slab, includes one twelfth of the span length. This is from the effective flange width provisions of ACI 318-05 Article 8.10.3. For rough calculation of required area of steel for T-beam behavior, the moment arm is assumed to be 0.9*d*.

Beam Reinforcement: Longitudinal (continued)

Choose 2 #7 and 3 #8 A_s = 3.57 in^2

ϕM_n = 5564 in-kips   OK

Run 3 #8s continuous top and bottom

ϕM_n = 3669 in-kips

This moment is greater than:

- 25% of max negative M_n = 1459 in-kips
- Max required M_u = 834 in-kips

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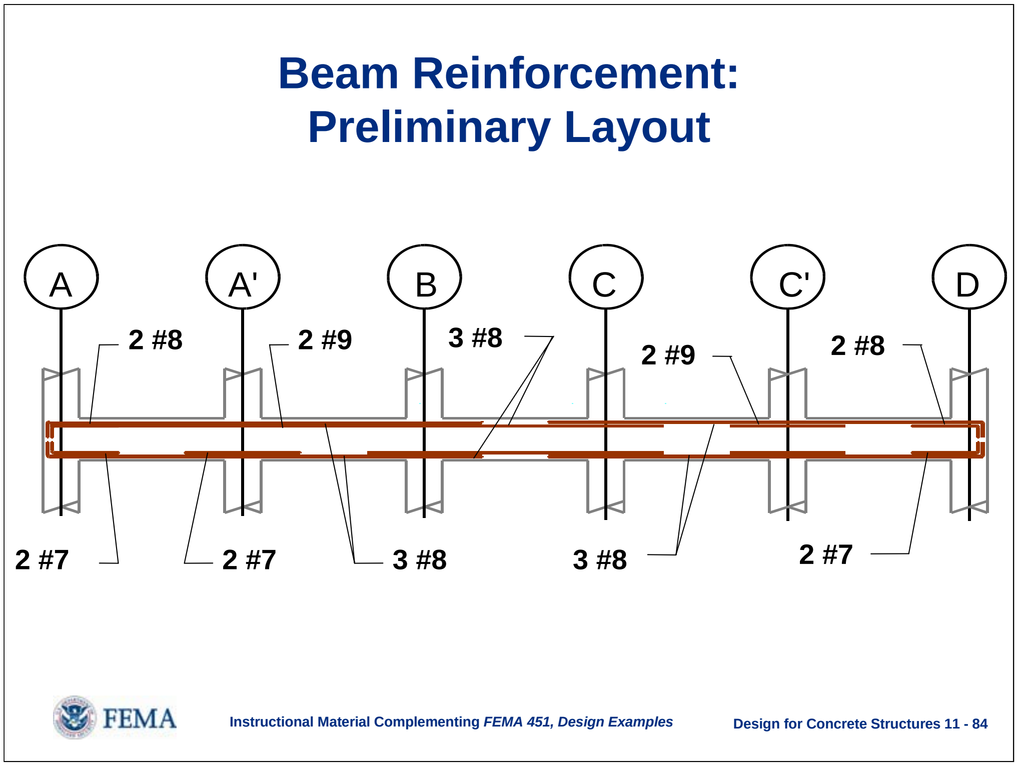
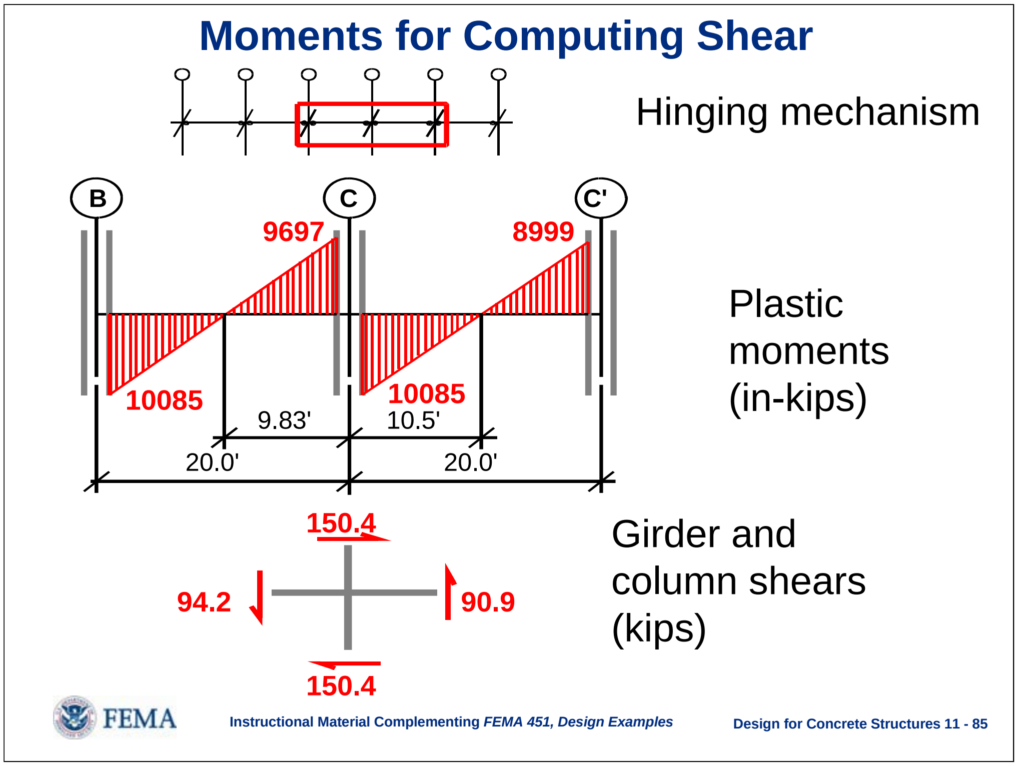
The required bottom reinforcement is calculated. Three #8 bars will run continuously along the bottom of the beam. An additional two #7 bars are placed for the positive moment demand at the face of the column. Then it is confirmed that the three continuous bars provide sufficient capacity in other circumstances.



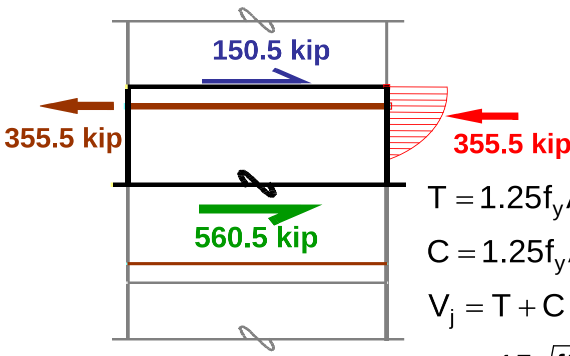
This slide presents the preliminary beam reinforcement layout for Frame 1,



Next we will determine the design shear forces for the beams, joints, and columns in Frame 1. These shears are based on the illustrated hinging mechanism in which the maximum probable negative moment is developed at one end of the beam and the maximum probable positive moment is developed at the other end.

The bottom free body shows the calculation of the shears in the columns. The free body is cut at the inflection points of the members framing into the joint. The inflection points for the columns are assumed to be at the midpoints.

Joint Shear Force



$$T = 1.25 f_y A_{s,top} = 355.5 \text{ kips}$$

$$C = 1.25 f_y A_{s,bot} = 355.5 \text{ kips}$$

$$V_j = T + C - V_{col} = 560.5 \text{ kips}$$

$$v_j = 15\sqrt{f'_c} = 949 \text{ psi}$$

$$V_n = v_j A_j = 949 \cdot 30 \cdot 30 = 854 \text{ kips}$$

$$\phi V_n = 0.85 \cdot 854 = 726 \text{ kips} > 560.5 \text{ kips}$$

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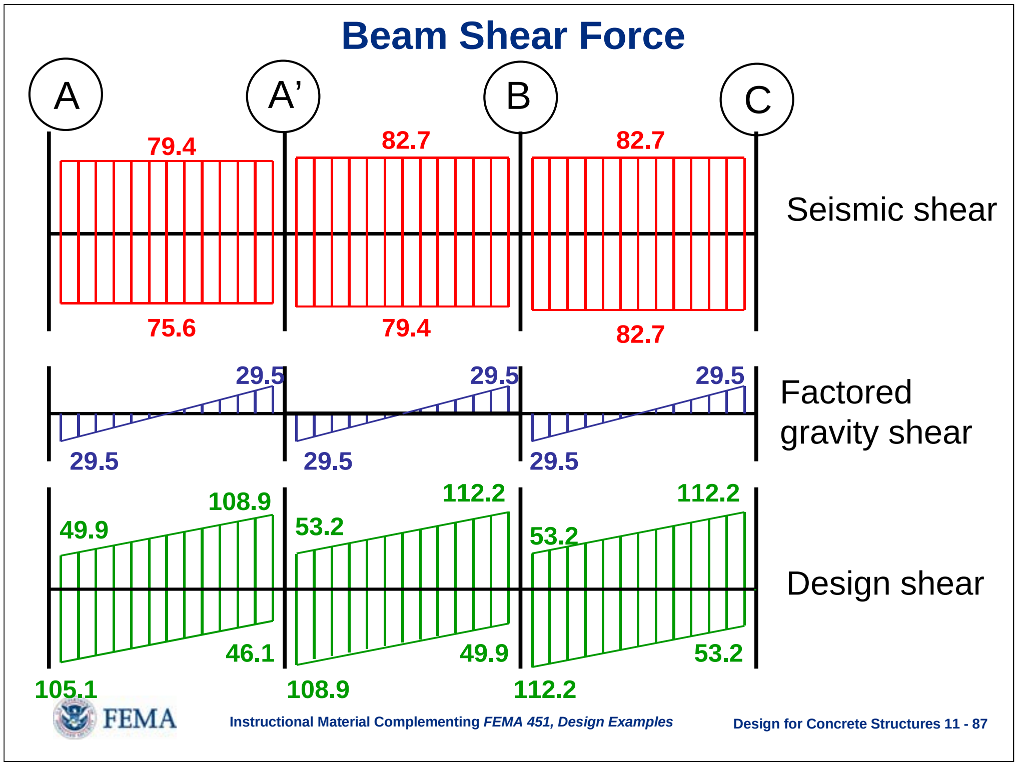
Design for Concrete Structures 11 - 86

Once the column shears are determined, the joint shear force and stress can be calculated as shown in this slide. The joint shear force is based on the same mechanism and moments shown on the previous slide. The joint area is the same as the column area (30 in. x 30 in.). The computed joint shear strength is then compared to the factored shear in the joint. In cases when the joint shear strength is less than the factored demand, the area of flexural steel, the beam depth, or the column size can be modified to remedy the problem.

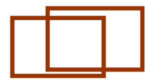


This slide shows the beam shear forces. The design shear is the gravity plus the seismic shear.

Beam Reinforcement: Transverse

$V_{seismic} > 50\% \, V_u$ therefore take $V_c = 0$

82.7 kips = 73%(112.2)

Use 4 legged #3 stirrups 

$$V_s = \frac{A_v f_y d}{s}$$

At ends of beam s = 5.5 in.

Near midspan s = 7.0 in.

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This slide presents the calculation of the beam transverse reinforcement. Note that V_c is taken as 0. #3 hoops are selected for the calculations, then the maximum spacing of hoops within the potential hinging region is determined as shown.

Beam Reinforcement: Transverse

- **Check maximum spacing of hoops within plastic hinge length (2d)**
  - d/4 = 7.4 in.
  - $8d_b$ = 7.0 in.
  - $24d_h$ = 9.0 in.

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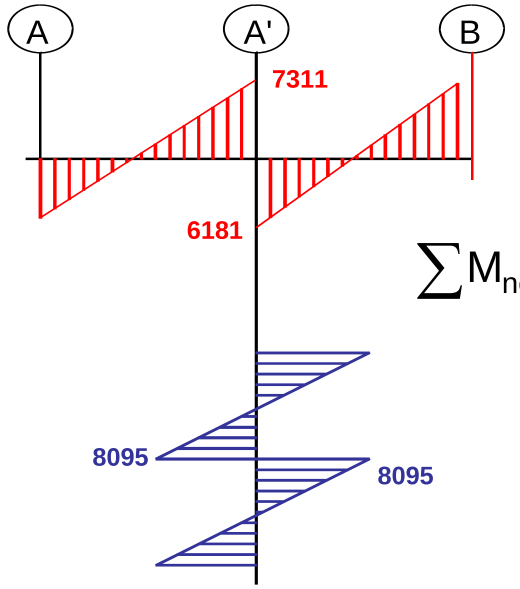
After calculating the required spacing to provide adequate shear strength, the spacing is checked against the maximum allowable spacing as shown in this slide. In this case the required spacing for shear strength governs.

Column Design Moments



Girder moments (Level 7)

$$\sum M_{nc} = 1.2(7311 + 6181)$$
$$= 16190 \text{ in} - \text{k}$$

Column moments (Level 7)

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This slide shows the calculation of the column design moments from beam flexural capacities and strong column-weak beam rule.

Column Design Moments

$$\text{if} \quad P_u > \frac{f'_c A_g}{10}$$

$$\sum M_{nc} > 1.2 \sum M_{nb}$$

Distribute relative to stiffness of columns above and below:

M_{nc} = 8095 in-kips (above)
M_{nc} = 8095 in-kips (below)

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The sum of the column capacities must exceed 120% of the sum of the beam flexural strengths at every joint. This slide shows the required nominal strengths of the columns above and below the joint

Design Strengths

Design Aspect	Strength Used
Beam rebar cutoffs	Design strength
Beam shear reinforcement	Maximum probable strength
Beam-column joint strength	Maximum probable strength
Column flexural strength	1.2 times nominal strength
Column shear strength	Maximum probable strength

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This slide reviews how strengths are calculated for various aspects of design. Maximum probable strength is used when a higher strength than the design strength causes more severe effects. Column strength is modified by the strong column-weak beam rule.

Column Transverse Reinforcement

$$A_{sh} = 0.3\left(sb_c \frac{f'_c}{f_{yt}}\right)\left[\left(\frac{A_g}{A_{ch}}\right) - 1\right]$$

and

$$A_{sh} = 0.09 sb_c \frac{f'_c}{f_{yt}}$$

A_g = gross area of column
A_{ch} = area confined within the hoops
b_c = transverse dimension of column core measured center to center of outer legs

Second equation typically governs for larger columns

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Next the necessary column transverse reinforcement to provide confinement of the concrete core is calculated. The equation on the slide that results in the larger amount of confining steel will govern.

Column Transverse Reinforcement

Maximum spacing is smallest of:

- One quarter of minimum member dimension
- Six times the diameter of the longitudinal bars
- s_o calculated as follows:

$$s_o = 4 + \frac{14 - h_x}{3}$$

h_x = maximum horizontal center to center spacing of cross-ties or hoop legs on all faces of the column, not allowed to be greater the 14 in.

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This slide presents the rules to calculate maximum hoop spacings within the potential hinge region.

Column Transverse Reinforcement

For max s = 4 in.

$$A_{sh} = 0.3\left(sb_c \frac{f'_c}{f_{yt}}\right)\left[\left(\frac{A_g}{A_{ch}}\right) - 1\right] = 0.3\left(4 \cdot 26.5 \cdot \frac{4}{60}\right)\left(\frac{900}{702} - 1\right)$$

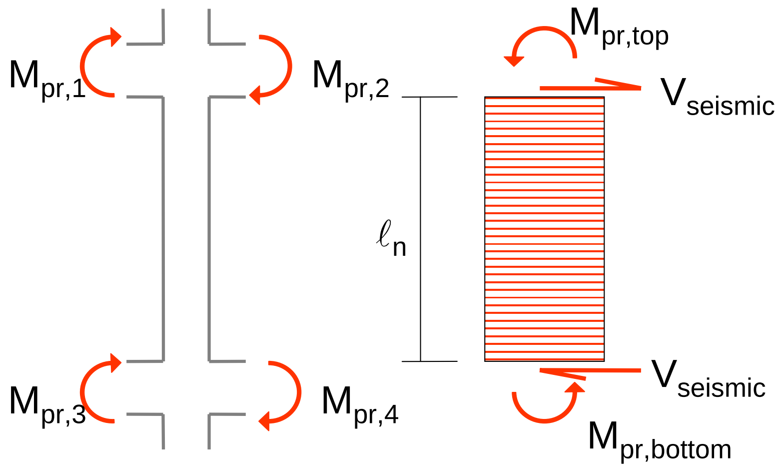
$$A_{sh} = 0.60\,\text{in}^2$$

and

$$A_{sh} = 0.09sb_c \frac{f'_c}{f_{yt}} = 0.09 \cdot 4 \cdot 26.5 \cdot \frac{4}{60} = 0.64\,\text{in}^2$$

Use 4 legs of #4 bar – A_{sh} = 0.80 in^2

Determine Seismic Shear



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The seismic shear in the columns is based on the probable flexural strength in the beams that frame into the column at the top and the bottom. After the probable moments in the beams are determined, the column moments above and below the joint are determined based on their relative flexibilities. Then the shear in the column is calculated based on the moments at the top and the bottom of the column and the column clear height.

Column Transverse Reinforcement Shear Demand from M_{pr} of Beams

$M_{pr,1}$ = 9000 in-k (2 #9 and 3 #8)
$M_{pr,2}$ = 7460 in-k (2 #7 and 3 #8)

Assume moments are distributed equally above and below joint

$$V_{seismic} = \frac{8230 \cdot 2}{(12.5 \cdot 12) - 32} = 139 \text{ kips}$$

Note $V_{seismic}$~100%V_u

$$V_c = 0, \text{if } P_{\min} < \frac{f'_c A_g}{20} = 180 \text{ kips}$$

For 30 in. square column
P_{min} = 266 kips OK

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In this slide the shear demand in the column is calculated. The probable moments in the beams framing into the joints at column line A’ are calculated (reinforcing layout was presented in Slide 83). It is assumed at a typical story, the beam moments will be distributed equally to the columns above and below the joint, and that the moments at the top and bottom of the column will be equal. This allows the seismic shear to be calculated as the sum of the moments at the top and bottom of the column divided by the clear column height. Note that since there is significant axial compression in the column, concrete shear strength may be used.

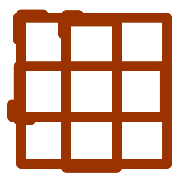
Column Transverse Reinforcement Shear Demand from M_{pr} of Beams

$$\phi V_c = \phi 2\lambda\sqrt{f'_c}bd = 0.75\cdot 2\cdot 0.85\sqrt{4000}\cdot 30\cdot 27.5 = 66.5\text{ kips}$$

$$\phi V_{s,required} = 139 - 66.5 = 72.5\text{ kips}$$

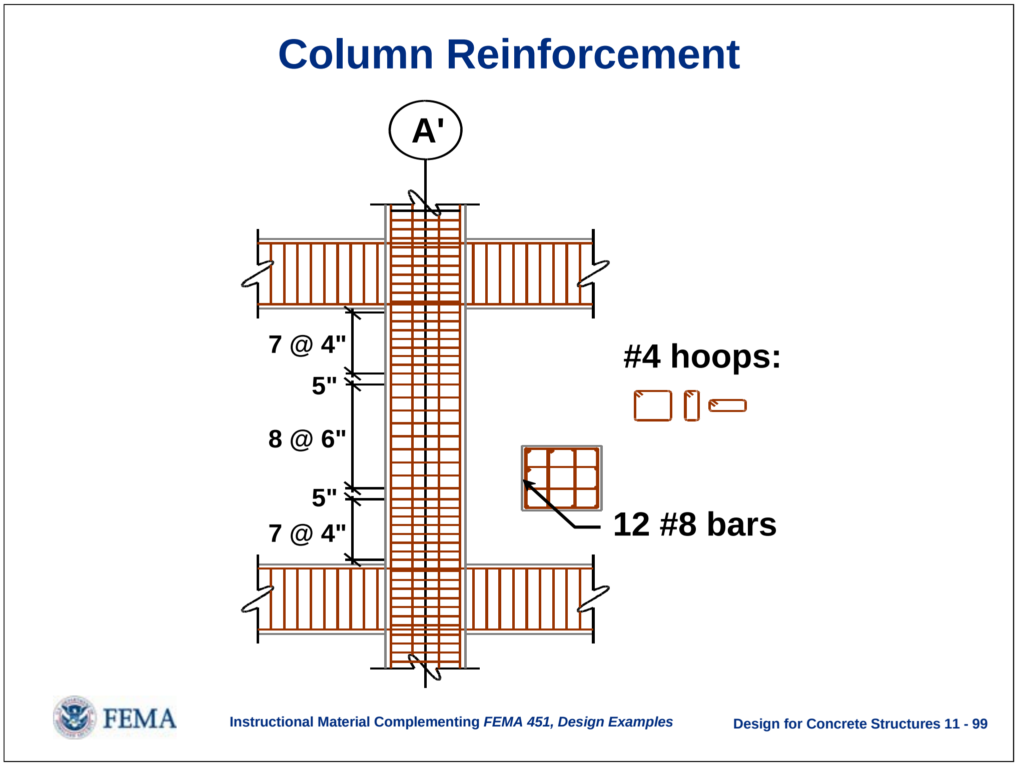
$$\phi V_{s,provided} = \phi\frac{A_v f_y d}{s} = 0.75\frac{4\cdot 0.2\cdot 60\cdot 29.6}{4} = 266.4\text{ kips}$$

Hoops



4 legs #4
s = 4”

This slide shows the calculation of the concrete contribution to column shear strength (66.5 kips). Note that a lightweight factor (λ) is required in the shear calculation. Then the required shear strength to be provided by the steel is determined. The shear strength provided by the stirrups spaced at 4in. (required spacing for confinement) is shown to be more than adequate. Therefore, the final hoop layout within the plastic hinge lengths at the tops and bottoms of the columns is set.

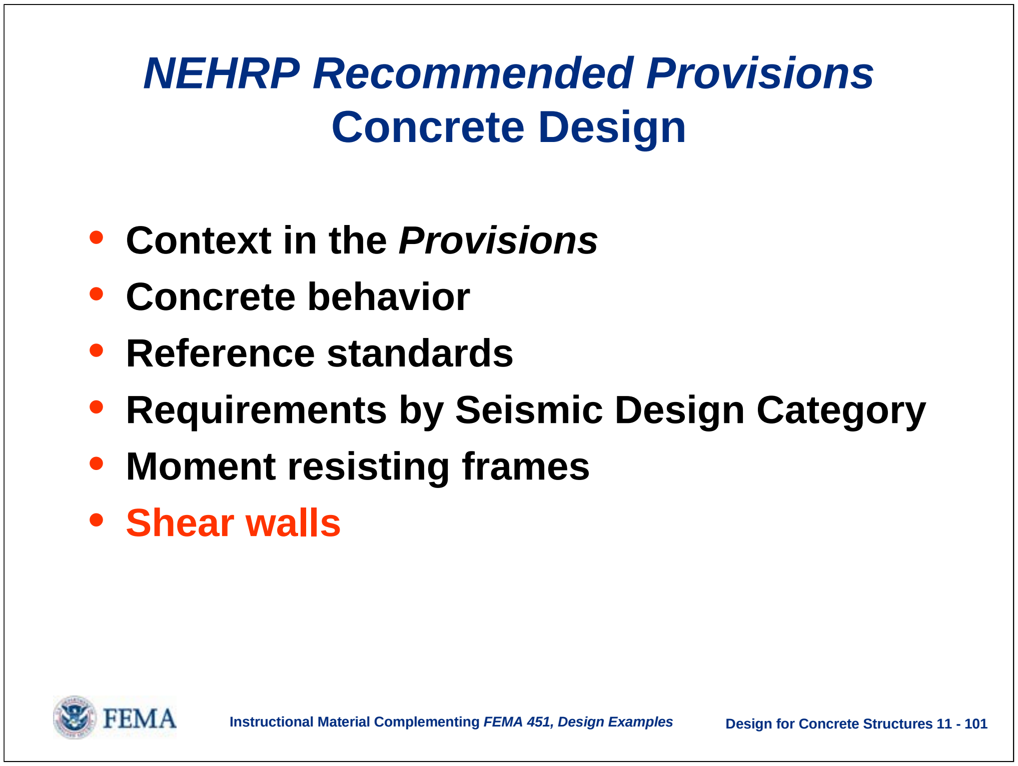


This slide shows the final column reinforcing layout. Outside of the plastic hinge length, the spacing of the hoops can be increased to 6 inches.

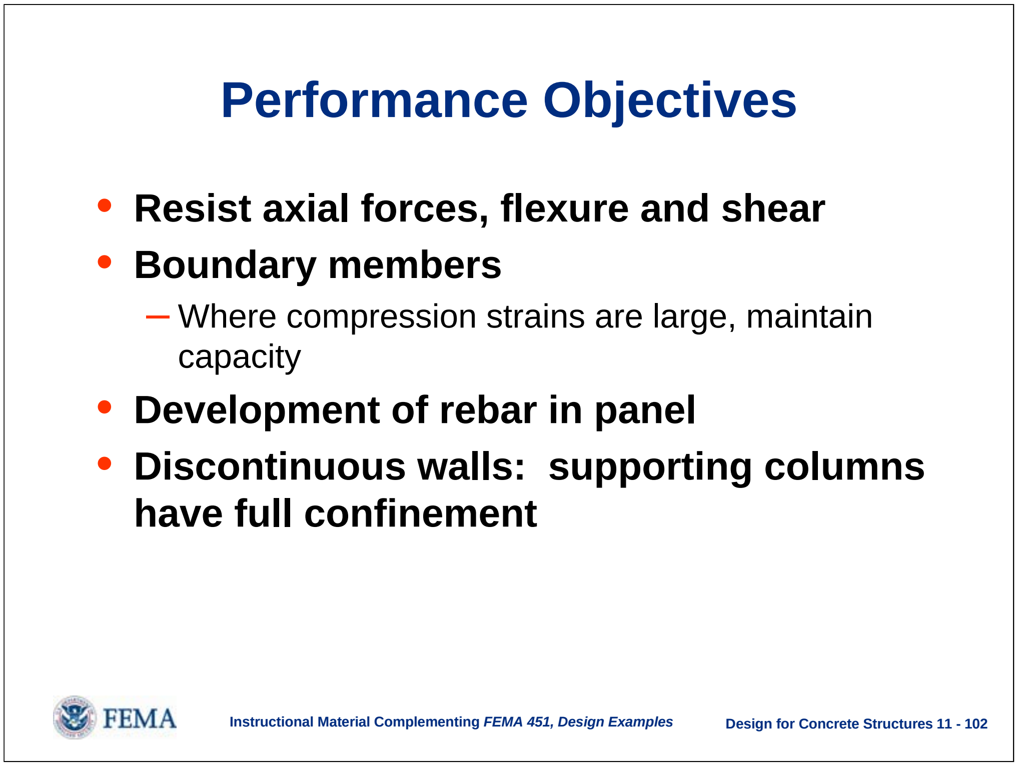
Levels of Seismic Detailing for Frames

Issue	Ordinary	Intermediate	Special
Hinge development and confinement		minor	full
Bar buckling		lesser	full
Member shear		lesser	full
Joint shear	minor	minor	full
Strong column			full
Rebar development	lesser	lesser	full
Load reversal	minor	lesser	full

This slide summarizes the differences in detailing requirements for the three types of frames. Special moment frames require special attention to each of the performance objectives. Intermediate frames have less strict requirements on all counts. Ordinary frames have no special requirements beyond those of ACI 318. Note that the detailing to avoid rebar congestion is important in special moment frames (use scaled drawings of joints for this task).



We will now discuss shear walls.



Design of shear walls for seismic resistance includes designing to resist axial forces, flexure, and shear. Boundary members to confine concrete in compression regions where stresses due to overturning are high may be required. In the past, boundary members had to take **all** the vertical resultants. As of ACOI 318-99, the boundary member requirements changed significantly. The boundary member requires transverse reinforcement capable of providing compression ductility through confinement, but it need not resist all the vertical resultants. In a panel, adequate rebar development must be provided since loads reverse and cover spalls. Under discontinuous walls, columns must have full confinement for compressive ductility.

Design Philosophy

- **Flexural yielding will occur in predetermined flexural hinging regions**
- **Brittle failure mechanisms will be precluded**
  - Diagonal tension
  - Sliding hinges
  - Local buckling

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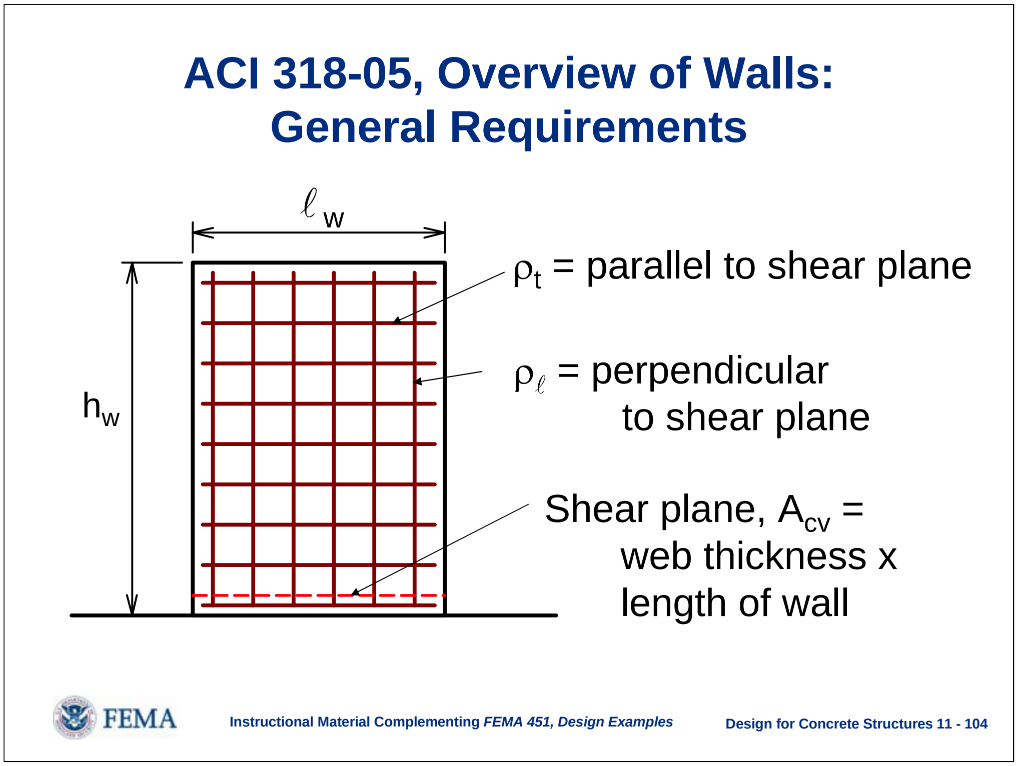
The design philosophy for walls is to ensure a ductile, flexural failure mechanism and preclude all brittle mechanisms.



This slide presents some of the ACI 318-05 notation for dimensions and reinforcing ratios in shear walls.

ACI 318-05, Overview of Walls: General Requirements

- ρ_ℓ and ρ_t not less than 0.0025 unless $V_u < A_{cv}\sqrt{f'_c}$ then as allowed in 14.3
- Spacing not to exceed 18 in.
- Reinforcement contributing to V_n shall be continuous and distributed across the shear plane

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This slide presents some of the general requirements for special shear walls.

ACI 318-05, Overview of Walls: General Requirements

- **Two curtains of reinforcing required if:**

$$V_u > 2A_{cv}\sqrt{f'_c}$$

- **Design shear force determined from lateral load analysis**

This slide presents more requirements for special shear walls.

ACI 318-05, Overview of Walls: General Requirements

- **Shear strength:**

$$V_n = A_{cv}\left(\alpha_c\sqrt{f'_c} + \rho_t f_y\right)$$

α_c = 3.0 for $h_w/\ell_w \leq 1.5$
α_c = 2.0 for $h_w/\ell_w \geq 2.0$
Linear interpolation between

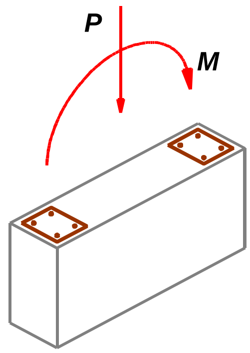
- **Walls must have reinforcement in two orthogonal directions**

Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 107

The equation for shear strength of walls recognizes the higher shear strength of walls with high shear-to-moment ratios. For stubby walls, ρ_ℓ shall not be less than ρ_t.

ACI 318-05, Overview of Walls: General Requirements

- **For axial load and flexure, design like a column to determine axial load – moment interaction**



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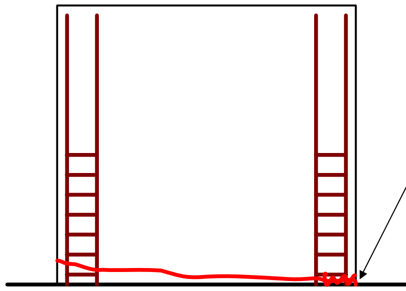
Instructional Material Complementing *FEMA 451, Design Examples*

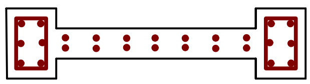
Design for Concrete Structures 11 - 108

To determine the required longitudinal reinforcement, the wall is treated like a column. An interaction diagram can be developed for the selected reinforcing layout, and checked against combinations of axial load and moment as determined from analysis.

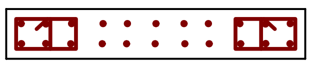
ACI 318-05, Overview of Walls: Boundary Elements



For walls with a high compression demand at the edges – Boundary Elements are required



Widened end with confinement



Extra confinement and/or longitudinal bars at end

If there is a high compression demand at the edges of the wall, boundary elements may be required. A boundary element is a portion of the wall which is strengthened with longitudinal and transverse reinforcement. Widening of the wall may or may not be required.

ACI 318-05, Overview of Walls: Boundary Elements

- **Boundary elements are required if:**

$$c \geq \frac{\ell_w}{600\left(\delta_u / h_w\right)}$$

δ_u = Design displacement

c = Depth to neutral axis from strain compatibility analysis with loads causing δ_u

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This slide presents one of the two methods ACI 318-05 presents to check if boundary elements are required.

ACI 318-05, Overview of Walls: Boundary Elements

- **Where required, boundary elements must extend up the wall from the critical section a distance not less than the larger of:**

$$\ell_w \quad \text{or} \quad M_u/4V_u$$

Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 111

If the method presented on the previous slide is used to determine if a boundary element is required, the method presented on this slide is used to determine at what height up the wall the boundary element can be terminated.

ACI 318-05: Overview of Walls Boundary Elements

- **Boundary elements are required where the maximum extreme fiber compressive stress calculated based on factored load effects, linear elastic concrete behavior and gross section properties, exceeds 0.2 f'_c**
- **Boundary element can be discontinued where the compressive stress is less than 0.15f'_c**

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This slide presents the second possible method for checking if boundary elements are required. This method has a different way to determine at what point on the wall the boundary element can be discontinued.

ACI 318-05: Overview of Walls Boundary Elements

- Boundary elements must extend horizontally not less than the larger of c/2 or c-0.1ℓ_w
- In flanged walls, boundary element must include all of the effective flange width and at least 12 in. of the web
- Transverse reinforcement must extend into the foundation

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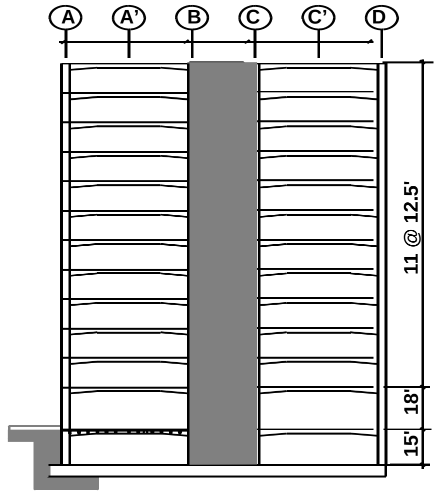
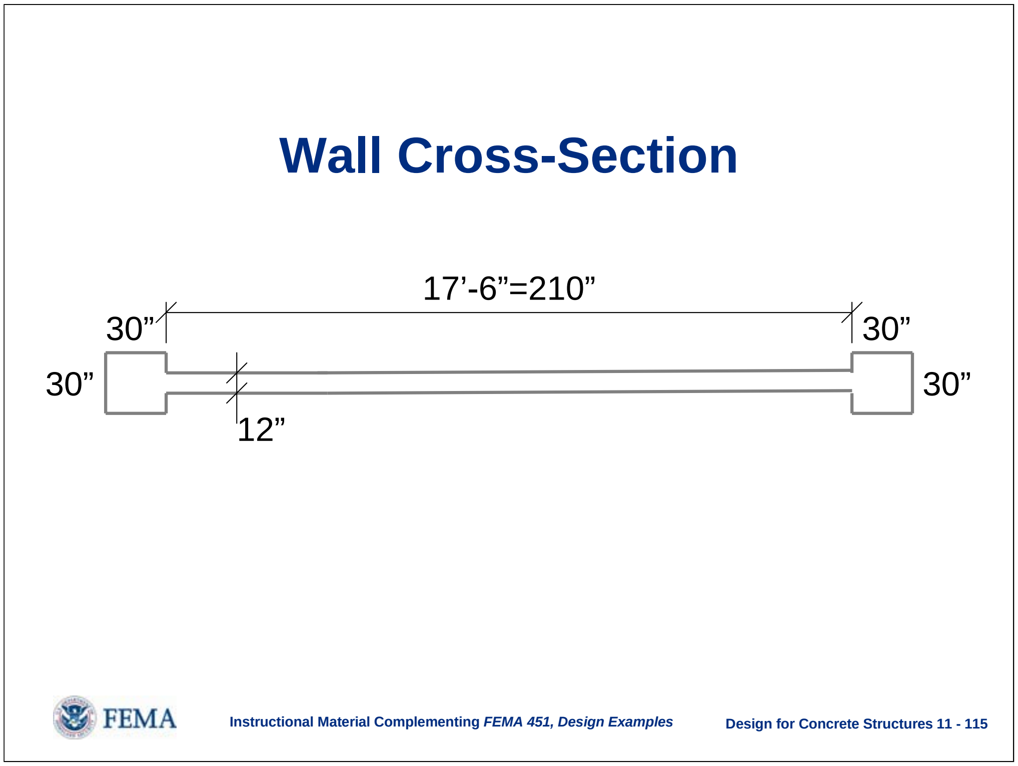
This slide presents several more requirements for boundary elements in special shear walls.

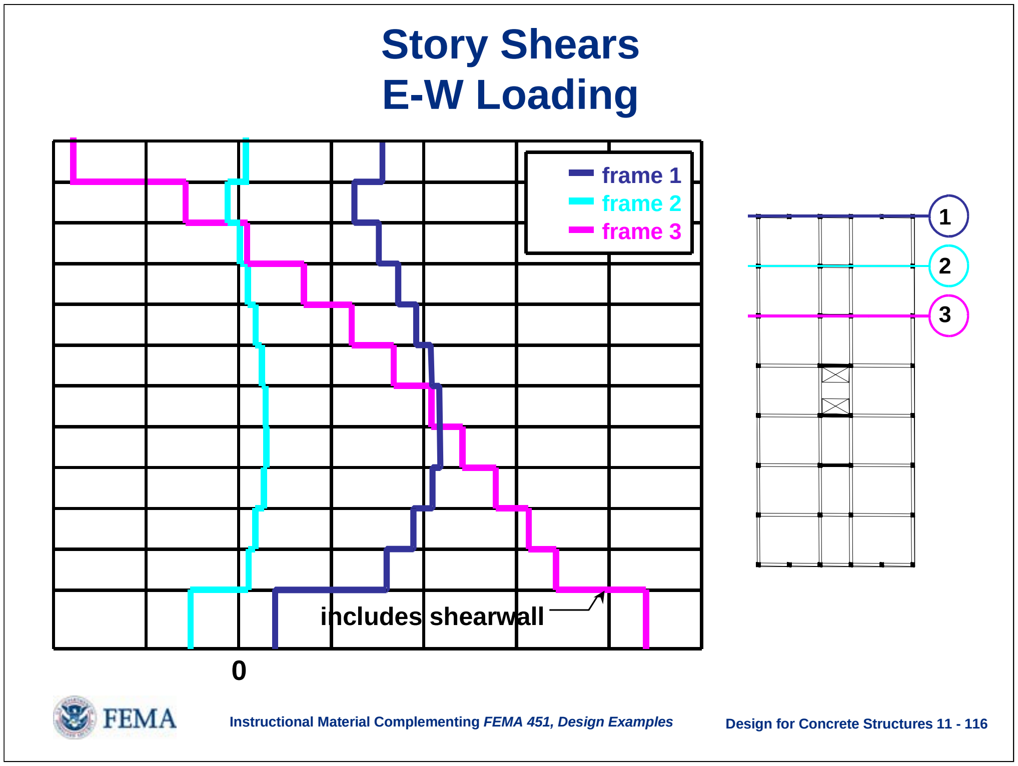
Wall Example



We will now look at a shear wall example. This slide shows the same building as the moment frame example. We will look at the wall in the frame at Column Line 3.



This slide shows the wall cross-section. At the ends of the wall are the typical 30 in. by 30 in. columns; in between the columns, the wall is 12 in. thick.



This slide shows the story shears for East-West load application. Note that Frame 3 includes the shear wall.

Boundary Element Check

Required if: $f_c > 0.2 f'_c$ based on gross concrete section

Axial load and moment are determined based on factored forces, including earthquake effects

At ground P_u = 5550 kip

M_u from analysis is 268,187 in-kip

The wall has the following gross section properties:

A = 4320 in^2 S = 261,600 in^3

f_c = 2.3 ksi = 38% of f'_c = 6 ksi

$\therefore$ **Need boundary element**

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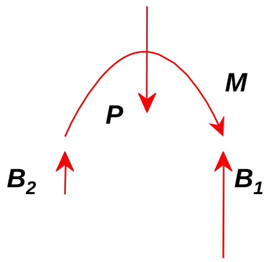
We will check whether a boundary member is required based on the second ACI recommended method. The axial load and moment come from the lateral and gravity load analysis of the building and the worst case load combination. The gross section properties are calculated, using the cross-section shown on Slide 114. Since a fairly large compressive stress is present, a boundary element is needed.

Boundary Element Design

Determine preliminary reinforcing ratio in boundary elements by assuming only boundary elements take compression



$$M = 268{,}187 \text{ in-k}$$

$$P = 5550 \text{ k}$$

$$B_1 = \frac{P}{2} + \frac{M}{240} = 3892 \text{ kip}$$

$$B_2 = \frac{P}{2} - \frac{M}{240} = 1658 \text{ kip}$$

Need $0.8P_o = 0.8(0.7)A_g\left[0.85 f'_c(1-\rho) + \rho f_y\right] > 3892 \text{ kip}$

For $A_g = 30(30) = 900 \text{ in}^2$
For $f_c' = 4$ ksi $\Rightarrow$ $\rho = 7.06\%$ Too large
For $f_c' = 6$ ksi $\Rightarrow$ $\rho = 4.18\%$ Reasonable; 24 #11

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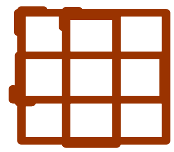
To determine an area of longitudinal reinforcement in the boundary elements, and to determine the required strength of the concrete in the wall, it will first be assumed that the boundary elements take the moment and axial load alone. This results in an approximate required reinforcement ratio in the column of 4% for f'_c of 6 ksi.

Boundary Element Confinement

Transverse reinforcement in boundary elements is to be designed essentially like column transverse reinforcement

$$A_{sh} = 0.09\, s b_c \frac{f'_c}{f_y} = 1.08 \text{ in}^2 \text{ at } s = 4"$$

4 legs of #5



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The confinement is designed essentially like column confinement. For a hoop spacing of 4 in., 4 legs of #5 bar are required.

Shear Panel Reinforcement

$$V_n = A_{cv}\left(2\lambda\sqrt{f_c^{'}} + \rho_t f_y\right)$$

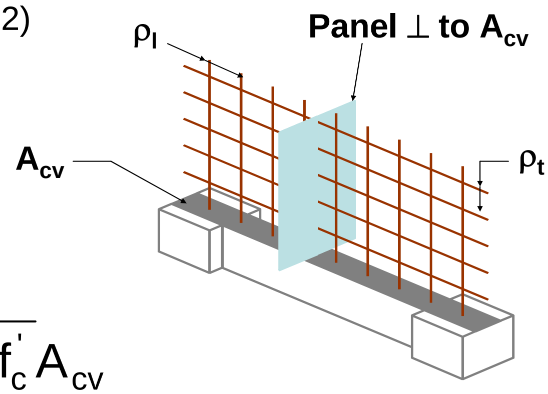
V_u = 539 kips (below level 2)

ϕ = 0.6 (per ACI 9.3.4(a))

ρ_t = 0.0036 for f_y = 40 ksi

Min ρ_ℓ (and ρ_t) = 0.0025

2 curtains if $$V_u > 2\sqrt{f_c^{'}}A_{cv}$$



Next the vertical and horizontal reinforcement required in the shear panel can be calculated. Note that the λ term is needed to reflect the lower shear strength of light weight concrete. The low phi factor is required because we will not provide enough shear reinforcement to enable the wall to develop it's full flexural strength without a shear failure. This would be an excessive amount of shear reinforcement so a lower phi factor is acceptable.

Shear Panel Reinforcement

Select transverse and longitudinal reinforcement:

longitudinal:

$$\#4@12'' \Rightarrow \frac{0.2 \cdot 2}{12 \cdot 12} = 0.0028 > 0.0025$$

transverse:

$$\#4@9'' \Rightarrow \frac{0.2 \cdot 2}{12 \cdot 9} = 0.0037 > 0.0036$$

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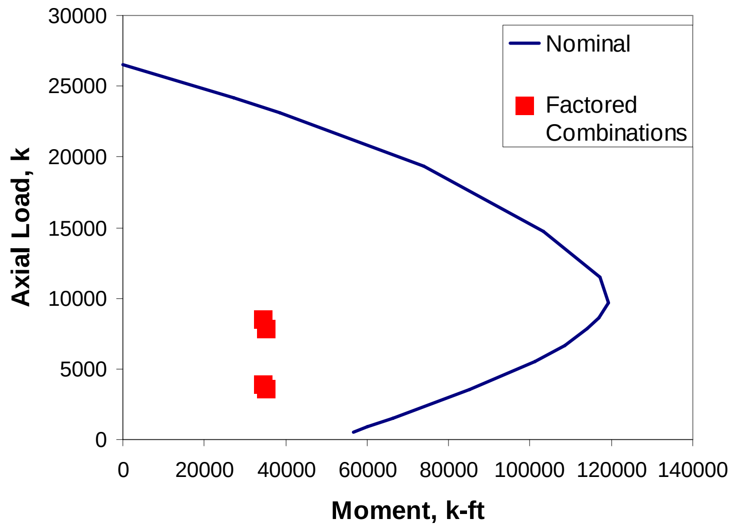
The longitudinal and transverse reinforcement can then be selected as shown in this slide.

Check Wall Design



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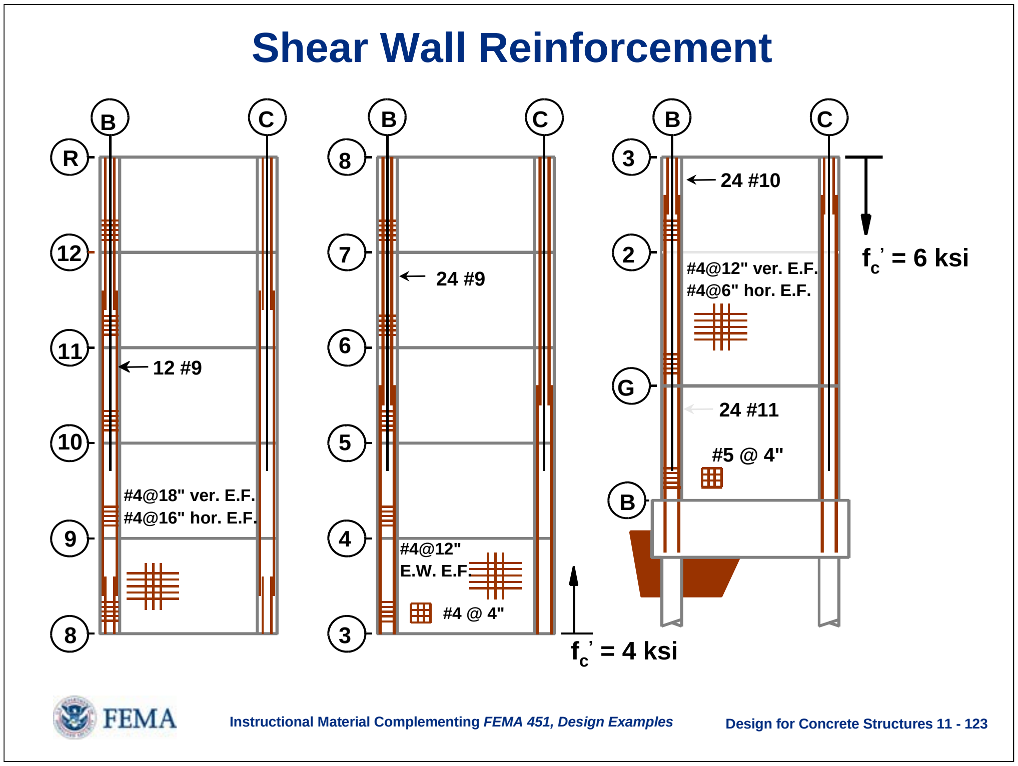
With the boundary element and shear panel reinforcement selected, a column interaction curve can be generated and compared against the factored combinations of axial load and moment (divided by appropriate phi factor). In this case, the wall is more than adequate for the first two levels. The design could be further optimized to reduce boundary element reinforcing congestion.



This slide shows a summary of the reinforcement for shear wall boundary members and panels. Note that the concrete strength can be reduced at Level 3. Also note how the boundary element and shear panel reinforcement can be reduced as you progress up the wall.

NEHRP Recommended Provisions Concrete Design

- Context in the *Provisions*
- Concrete behavior
- Reference standards
- Requirements by Seismic Design Category
- Moment resisting frames
- Shear walls
- Other topics

FEMA

We will now discuss other topics.

Members Not Part of SRS

- **In frame members not designated as part of the lateral-force-resisting system in regions of high seismic risk:**
  - Must be able to support gravity loads while subjected to the design displacement
  - Transverse reinforcement increases depending on:

    Forces induced by drift

    Axial force in member

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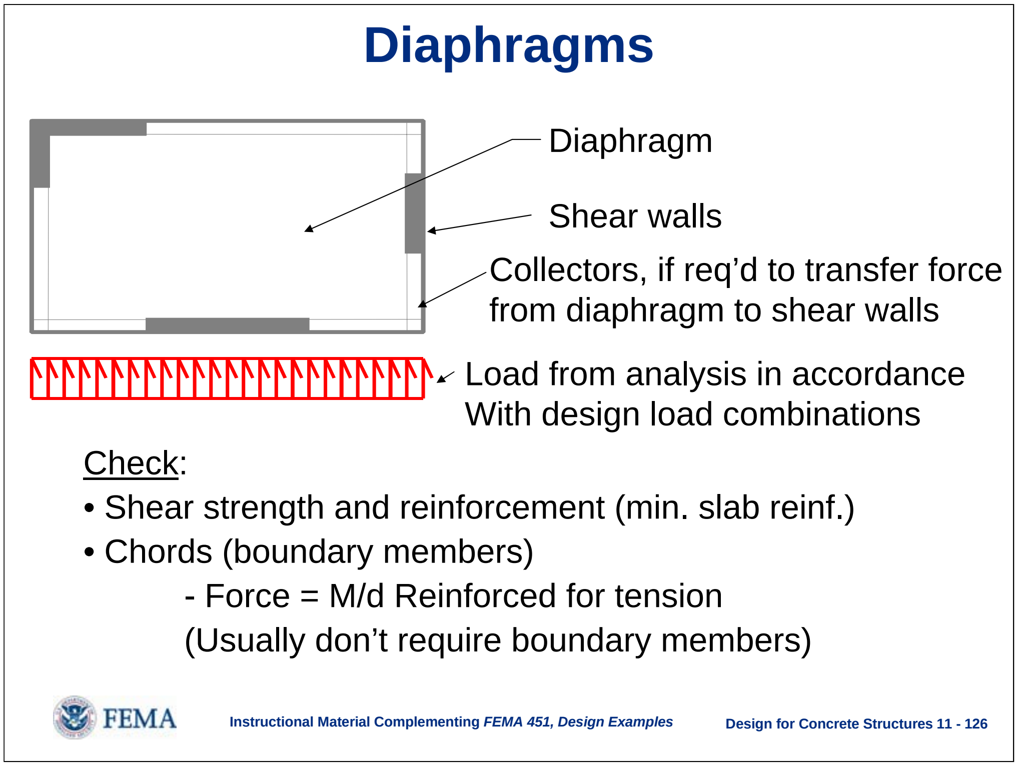
This important set of rules was implemented rapidly following the 1994 Northridge earthquake (in ACI 318-95) and has been modified with each code cycle.



This slide shows some aspects of diaphragm design. The slide shows where collectors might be needed to transfer forces from a long diaphragm into a narrow shear wall. The design load is to be from seismic analysis in accordance with the design load combinations. Slab reinforcement is based on shear stress or slab reinforcement minima (same as for slender structural walls). The chord (boundary element) of a diaphragm is designed to resist tension and compression of M/d. Diaphragms rarely require boundary members. There are also special considerations for topped and untopped precast diaphragms. Appendix A to Chapter 9 of the *Provisions* covers untopped precast diaphragms.

Struts and Trusses
performance objectives

- **All members have axial load (not flexure), so ductility is more difficult to achieve**
- **Full length confinement**

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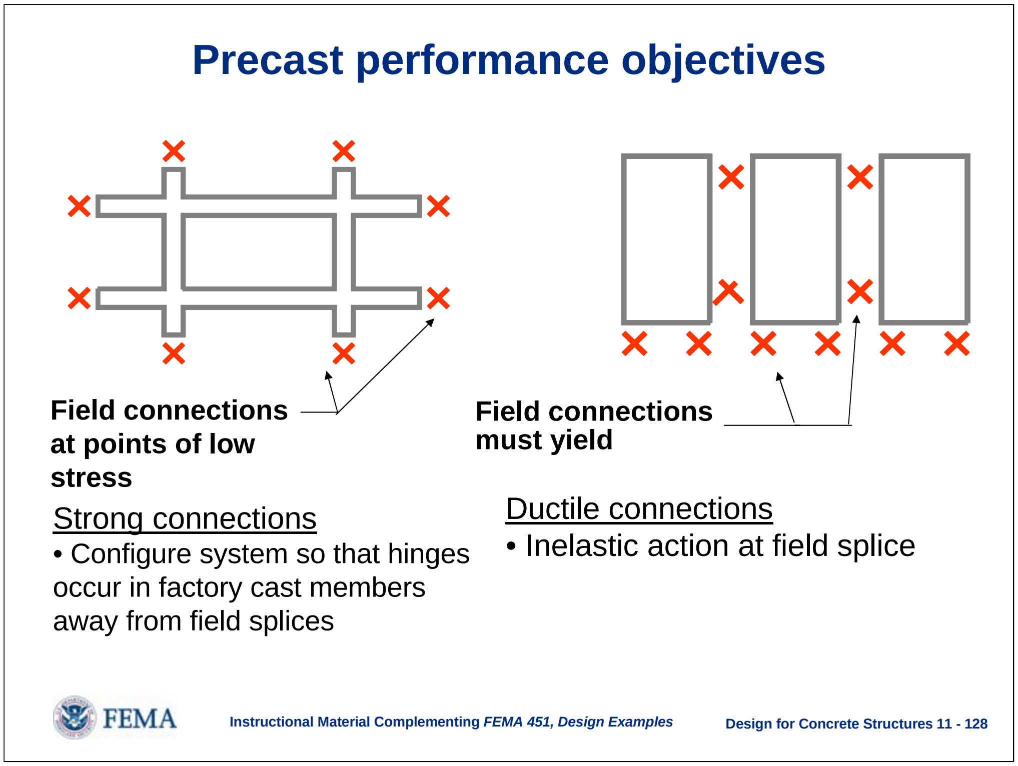
Truss systems of reinforced concrete are rarely used. Ductility, usually developed in flexure, is difficult to achieve. Every member in a truss is axially loaded; therefore, every member is designed and reinforced as a column. Full height confinement is used in all members. Anchorage is extremely important to assure adequate post yield response.



Frames can be based on one of two basic modes of behavior in precast buildings:

• Precast that emulates monolithic construction used for frames (strong connections). For this type of system, field connections are made at points of low stress, and the hinges will occur in factory cast members, not field splices.

• Jointed precast with ductile connections. For this type of system, yielding occurs in the field connections.

In the *NEHRP Provisions,* acceptance of special precast structural walls is based on validation testing.

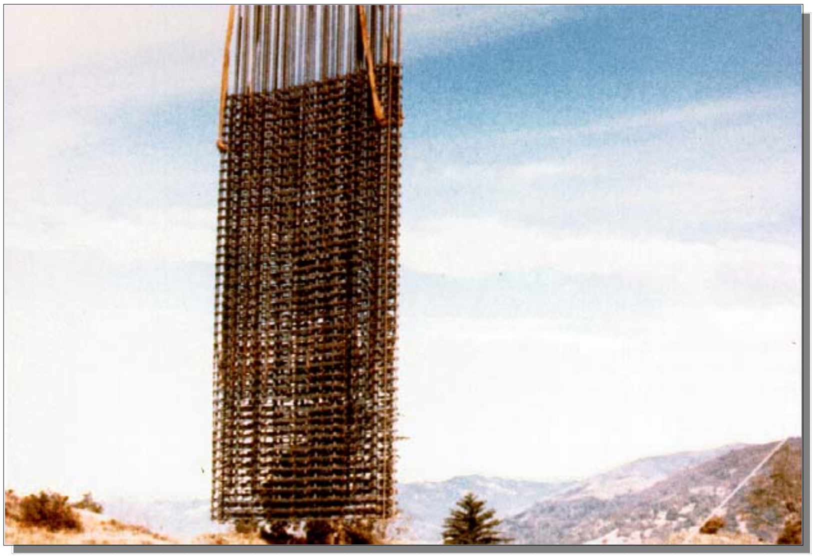
Quality Assurance
Rebar Inspection

- **Continuous**
  - Welding of rebar
- **Periodic**
  - During and upon completion of placement for special moment frames, intermediate moment frames and shear walls

Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 129

Quality assurance is covered in Chapter 2 of the *Provisions*. A quality assurance plan must be submitted for most seismic force resisting systems. Welding of rebar requires continuous special inspection. Periodic inspection is required during placement of rebar and upon completion of rebar placement for special moment frames, intermediate moment frames, and shear walls.

Shear panel reinforcement cage



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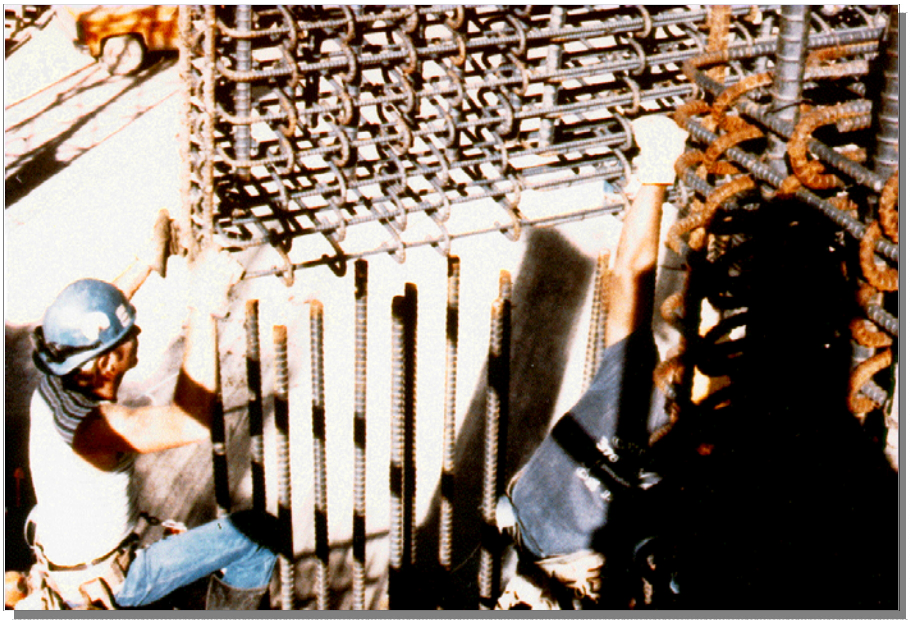
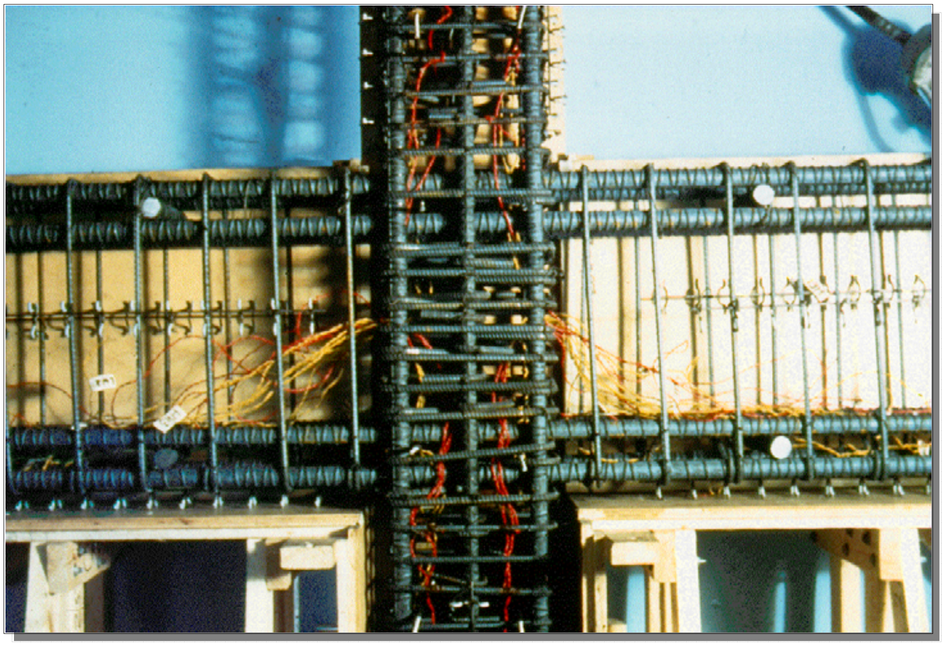
This photo shows a shear panel reinforcement cage. Inspection is extremely important to ensure that all required reinforcement is present and in its proper location.



Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 131

This photo shows a reinforcement cage for a shear wall. Note the dowels for the boundary element at one end of the panel reinforcement. The cage and the dowels must be inspected prior to concrete placement. Also it is important to inspect during placement to see that the concrete is being placed and vibrated properly in this congested area.



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This joint with hoops was created for use in laboratory tests. Note the tight spacing of steel, which will require careful placement and vibration of concrete. Similar joints, which will require careful inspection, may appear in real structures.

Quality Assurance: Reinforcing Inspection - Prestressed

- **Periodic**
  - Placing of prestressing tendons (inspection required upon completion)
- **Continuous**
  - Stressing of tendons
  - Grouting of tendons

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Steel in prestressed concrete has additional *Provisions* inspection requirements. For precast, prestressed concrete the inspections are typically done by the manufacturer. For cast-in-place, post-tensioned concrete, an on-site Special Inspector must do inspections. Continuous inspection is required during the stressing, and grouting of prestressing tendons.

Quality Assurance: Concrete Placement Inspection

- **Continuous**
  - Prestressed elements
  - Drilled piers
  - Caissons
- **Periodic**
  - Frames
  - Shear walls

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Placement of concrete requires inspection. The inspection must be continuous during concrete placement in prestressed elements, drilled piers, and caissons. Periodic inspection is required during placement of concrete in frames and shear walls. Frequency of periodic inspection will vary. Common practice is very frequent inspections early in a construction project and less frequent as the inspector verifies that the contractor's handling of concrete is acceptable.

Quality Assurance: Precast Concrete (plant cast)

- **Manufacturer may serve as special inspector if plant's quality control program is approved by regulatory agency**
- **If no approved quality control program, independent special inspector is required**

Most large precast operations have careful internal quality control programs. When a program has been approved by authority having jurisdiction, the manufacturer may be the owner's special inspector. Otherwise, the owner must have an independent special inspector visit the plant to make inspections.

Quality Assurance: PCI Certification Program

- **Review of plant operations**
  - Scheduled and surprise visits
  - Qualified independent inspectors
  - Observed work of in-plant quality control
  - Check results of quality control procedures
  - Periodic – specific approvals requiring renewal

The Prestressed Concrete Institute has a rigorous plant certification program. Features of the program are noted on this slide. Review of plant operations includes both scheduled and surprise visits by qualified, independent inspectors. They observe the work of the in-plant quality control personnel and check results of quality control procedures. Approvals issued for specific time periods; periodic reinspection and reapproval required.

Quality Assurance: ACI Inspector Certification

- **Specialized training available for:**
  - Laboratory and in situ testing
  - Inspection of welding
  - Handling and placement of concrete
  - Others

Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 137

This slide presents information about the ACI Inspector Certification program. Note the types of training that should be included in developing reinforced concrete inspectors. PCA and ICC also offer training and certification programs. Structural engineers are not commonly trained to be competent inspectors of welding, handling and placement of concrete (or to perform tests). An independent special inspector, trained by ACI or similar programs, performs these tasks.

Quality Assurance: Reinforcement Testing

- **Rebar**
  - Special and intermediate moment frames
  - Boundary elements
- **Prestressing steel**
- **Tests include**
  - Weldability
  - Elongation
  - Actual to specified yield strength
  - Actual to specified ultimate strength

FEMA Instructional Material Complementing *FEMA 451, Design Examples* Design for Concrete Structures 11 - 138

Rebar in special moment frames, intermediate frames and in boundary elements in shear walls must be tested for weldability, elongation, actual-to-specified yield, and ultimate strengths. The *Provisions* only requires weldability testing of steel used in certain types of structural systems. Weldability is important because welding embrittles steel unless steel is specifically manufactured to be weldable. Elongation is a measure of ductility. Actual-to-specified strength ratios are important because significantly higher strengths than specified may cause the concrete to fail first (sudden, abrupt failures). Higher steel strength usually means reduced elongation (reduced ductility). Certified mill tests may be accepted for prestressing steel and for ASTM A706 and ASTM A615 (if it will not be welded).

Quality Assurance: Concrete Testing

- **Sample and test according to ACI 318-05**
  - Slump
  - Air content
  - 7 and 28 day strengths
  - Unit weight
- **Rate**
  - Once per day per class

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The *Provisions* requires sampling and testing of structural concrete per ACI 318-05 for slump, air content, 7 and 28 day compressive strengths, and unit weight. Note that samples must be made once per day per class of concrete placed.

NEHRP Recommended Provisions: Concrete Design

- Context in the *Provisions*
- Concrete behavior
- Reference standards
- Requirements by Seismic Design Category
- Moment resisting frames
- Shear walls
- Other topics
- Summary

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This concludes our discussion of seismic design of reinforced concrete structures.