Best Practices in Dam and Levee Safety Risk Analysis

V-1. Reinforced Concrete Failure Mechanisms

22 April 2015

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Introduction

- Chapter V-1 presents the failure mechanisms of reinforced concrete sections such as spillway piers, walls, slabs, and buttresses.
  - ✓ factors influencing the strength and stability of the reinforced concrete sections
  - ✓ how to consider National code requirements in the context of risk
  - ✓ considerations when determining risk analysis failure probabilities based on structural analysis results
  - ✓ a typical event tree of the progression of failure

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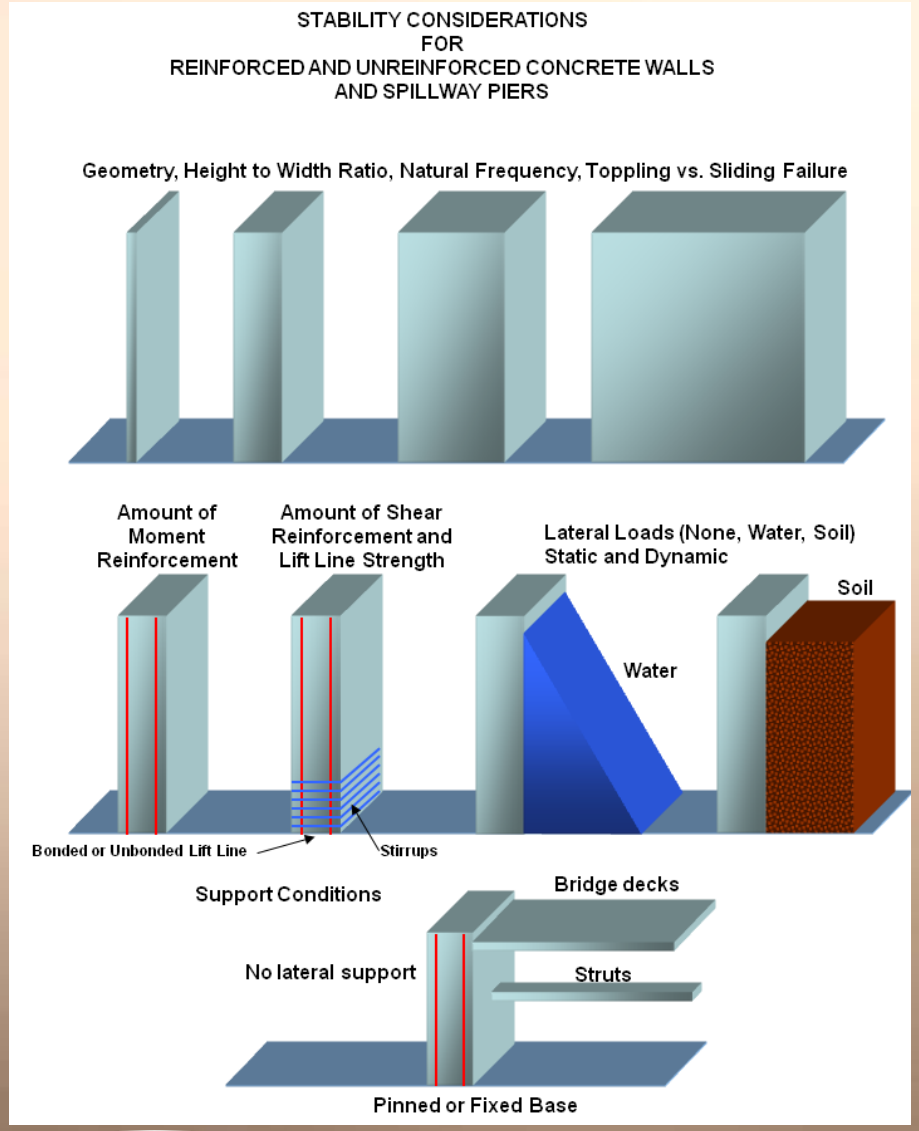
Factors Influencing the Strength and Stability of Reinforced Concrete Sections

- Geometry and support conditions of the section
- Material properties of the reinforcement
- Material properties of the concrete
- Amount and detailing of the reinforcement
- Type and duration of loading
- Location of the reinforced concrete members relative to the entire structure

Geometry and Support Conditions



- Reinforced concrete sections in hydraulic structures vary greatly in size and shape
  - ✓ Spillway walls can be very tall and narrow
  - ✓ Spillway piers tend to be shorter and wider than walls
  - ✓ Buttresses can vary from very thin tall sections to more stout sections
- The geometry of the concrete section can have a significant impact on how the section fails
- Sections with height to width ratios of 4:1 or less tend to slide more than rotate or bend while sections with height to width ratios more than 4:1 tend to bend, rotate and topple (deep beam criteria in ACI Code 318)

Geometry and Support Conditions - Piers

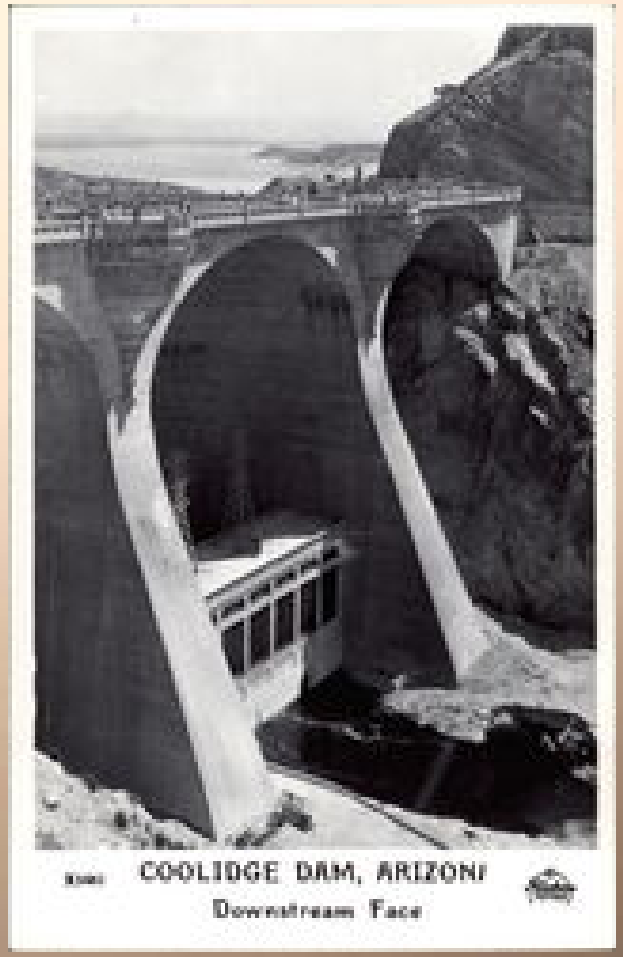
Glen Canyon Dam Spillway Gate Piers

Canyon Ferry Dam Spillway Gate Piers

Minidoka Dam Canal Headworks Gate Piers

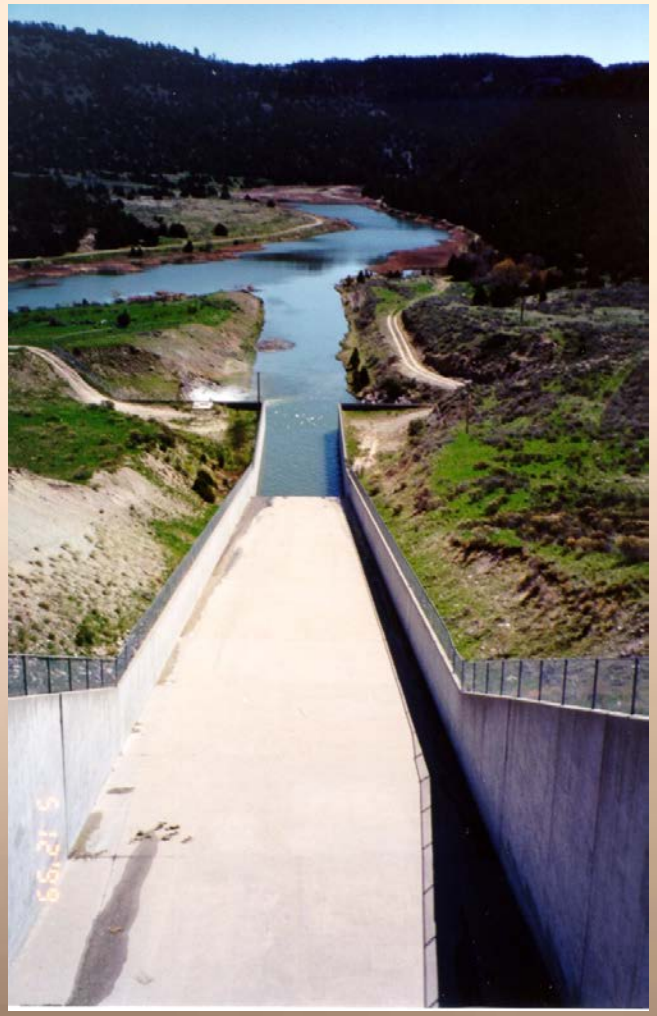
Geometry and Support Conditions - Buttresses

Stony Gorge Dam Thin Buttress Construction



Coolidge Dam Thick Buttress Construction

Geometry and Support Conditions - Spillways



Glendo Dam Chute Walls

Stampede Dam Inlet Control Structure

Stampede Dam Stilling Basin

- Examples typically not considered a reinforced concrete PFM
- Generally only consider gated spillway crest structure
- However failure could contribute to another PFM

Geometry and Support Conditions

- Structures have definite, signature dynamic characteristics
- The geometry greatly affects the natural frequency of the reinforced concrete member
- The natural frequency of the member decreases as the height to width ratio increases.

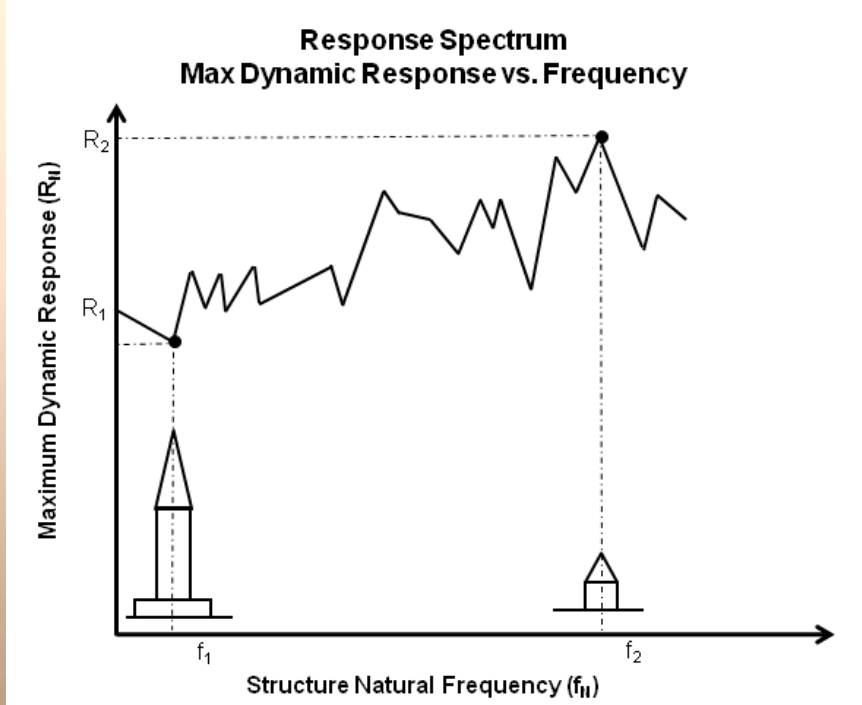


Figure modified from ASCE *Seismic Design and Performance of Building Structures*, 2008

Geometry and Support Conditions

NATURAL FREQUENCIES OF VARIOUS SYSTEMS
GRAVITY WALLS [R1]

GRAVITY WALLS [R1]

$H := 30 \cdot ft$

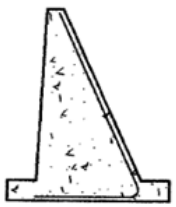
$S := 0.7$

$B := H \cdot S$ $\qquad B = 21 \cdot ft$

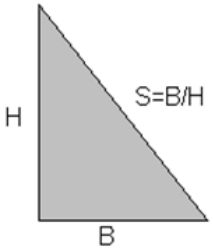
$F := 0.000425 \frac{sec}{ft}$

$T := \frac{F \cdot H^2}{T\,B}$ $\qquad T = 0\,s$

$f := \frac{1}{T}$ $\qquad f = 54.9\,Hz$



REINFORCED CONCRETE
RIGID–GRAVITY WALL



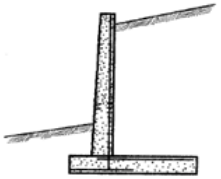
CANTILEVER WALL FIXED AT BASE (B/H < 0.5) [R1]

$H := 276\,ft$ $\qquad B := 8.33\,ft$

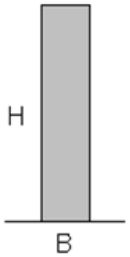
$F := 0.000643 \frac{sec}{ft}$

$T := \frac{F \cdot H^2}{T\,B}$ $\qquad T = 5.8$

$f := \frac{1}{T}$ $\qquad f = 0.1$



REINFORCED CONCRETE CANTILEVER
SEMI–GRAVITY WALL



Geometry and Support Conditions

- Structural response to seismic loading will be different for sections:
  - ✓ on rock foundations compared to soil foundations
  - ✓ founded on the top of a dam where ground motions are generally amplified

Geometry and Support Conditions

$S_{MS} = F_a S_s$ = site adjusted mapped short period MCE_R spectral response acceleration
$S_{M1} = F_v S_1$ = site adjusted mapped 1-sec period MCE_R spectral response acceleration
$S_{DS} = 2/3 S_{MS}$ = design short period spectral response acceleration parameter
$S_{D1} = 2/3 S_{M1}$ = design 1-sec period spectral response acceleration parameter

TABLE 11.4-2 SITE COEFFICIENT, F_v

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \le 0.1$	$S_1 = 0.2$	$S_1 = 0.3$	$S_1 = 0.4$	$S_1 \ge 0.5$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7				

NOTE: Use straight-line interpolation for intermediate values of S_1.

TABLE 11.4-1 SITE COEFFICIENT, F_a

Site Class	Mapped Maximum Considered Earthquake Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.5$	$S_S = 0.75$	$S_S = 1.0$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7				

NOTE: Use straight-line interpolation for intermediate values of S_S.

Site Class

A. Hard rock

B. Rock

C. Very dense soil and soft rock

D. Stiff soil

E. Soft clay soil

F. Soils requiring site response analysis in accordance with Section 21.1

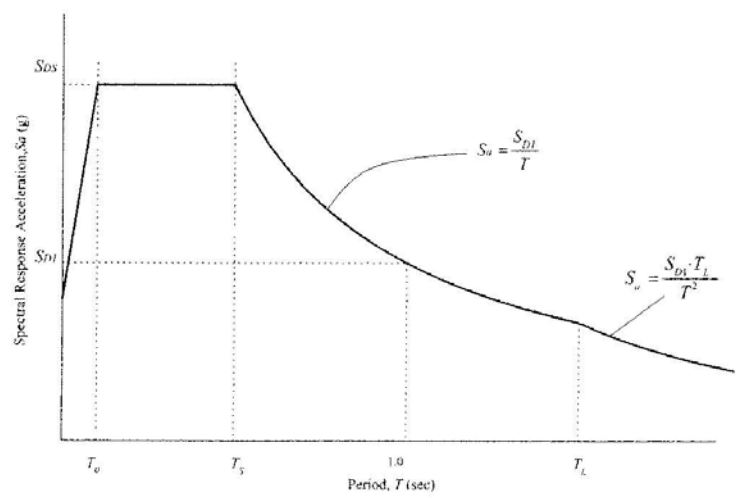


FIGURE 11.4-1 DESIGN RESPONSE SPECTRUM

Reinforcement Material Properties

- Material properties of the reinforcement directly contribute to the strength of the concrete section. While the modulus of elasticity of steel is fairly consistent at 29,000,000 lb/in^2, the yield strength of the reinforcement depends on the era of the structure.

Steel Grade	Yield	Ultimate	Years	
			From	To
33	33,000	55,000	1911	1966
40	40,000	70,000	1911	present
50	50,000	80,000	1911	present
60	60,000	90,000	1959	present
70	70,000	80,000	1959	present
75[1]	75,000	100,000	1959	present

[1]Excludes the years from 1966 through 1987

- Historical reinforcement availability and yield properties can be found in CRSI Engineering Data Report No. 48 and ASCE 41 Seismic Rehabilitation of Existing Buildings.
- The shear strength of the reinforcement is typically taken as the yield strength.

Concrete Material Properties

- Key contributors to:
  - ✓ Member strength
  - ✓ Structural response

- Required concrete properties to estimate reinforced concrete member strength and structural response include:
  - ✓Density
  - ✓Modulus of elasticity
  - ✓Compressive strength
  - ✓Tensile strength
  - ✓Shear strength

- Standard or assumed values for concrete material properties can be used in preliminary structural evaluations (Reference ASCE 41)
  - ✓Uncertainty
  - ✓Unconservative results
  - ✓Concrete coring and lab testing may be required

Time Frame	Footings	Beams	Slabs	Columns	Walls
1900-1919	1,000-2,500	2,000-3,000	1,500-3,000	1,500-3,000	1,000-2,500
1920-1949	1,500-3,000	2,000-3,000	2,000-3,000	2,000-4,000	2,000-3,000
1950-1969	2,500-3,000	3,000-4,000	3,000-4,000	3,000-6,000	2,500-4,000
1970-Present	3,000-4,000	3,000-5,000	3,000-5,000	3,000-10,000	3,000-5,000

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Concrete Material Properties

- Construction joints
  - ✓Unbonded -> No tensile strength/reduced shear resistance
  - ✓Often adversely located in structure

Construction joint at geometric discontinuity

Reinforcement Details

- Ductile vs. brittle failures

  - ✓Ductile failure much better than brittle failures
  - ✓Ductile failures occur much slower than brittle failures
  - ✓Ductile failures provide evidence of structural distress prior to failure
  - ✓Ductile failures allows time for repair or evacuation prior to failure
  - ✓ Shear failures tend to be more sudden than ductile type bending or tensile failures

- Ductile sections

  - ✓Require reinforcement design details per ACI code
  - ✓Detailing examples
    - stirrups will confine areas of damaged concrete and help maintain post-seismic structural integrity
    - $A_{s(min)} = 200b_w d/f_y$
    - Shear strength based exclusively on V_c is okay provided $A_s \geq A_{s(min)}$ and $\rho \leq 0.75\rho_b$

Reinforcement Details

✓Older hydraulic structures were not designed for current seismic loads

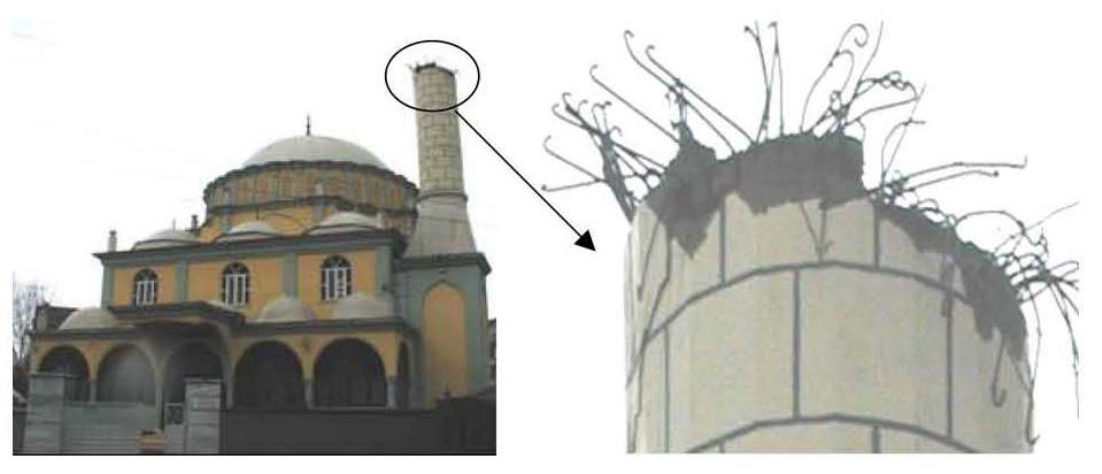
✓Seismic detailing requirements have changed dramatically over the last several decades

✓Insufficient embedment lengths, splice lengths or hook details can result in sudden pullout failures

✓Massive hydraulic structures are typically lightly or under-reinforced and can be greatly overstressed by large earthquakes and can yield and deflect excessively

Reinforcement Details

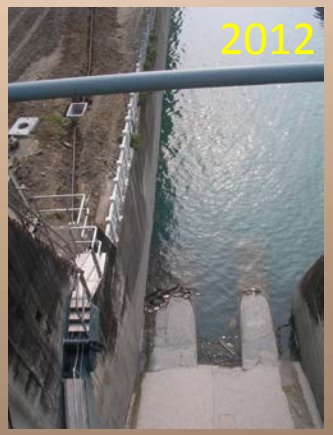
1999 Kocaeli, Turkey Earthquake



Embedment/Splice Lengths

Reinforcement Details

1999 Kocaeli, Turkey Earthquake



Shi-Kang Dam - 1999 Chi-Chi, Taiwan Earthquake

Type and Duration of Load

- Static loads
  - ✓ Examples - hydrostatic or soil pressures
  - ✓ Typically act for long durations - sustained loads
  - ✓ There may be no mechanism to stop or resist a section in the process of failing if the static loads exceed the capacity of the structure
- Earthquake loads
  - ✓ are cyclical and change direction rapidly
  - ✓ sections may not crack through the member thickness even though the tensile capacity is exceeded for short durations
  - ✓ the seismic load may not have sufficient duration or have enough significant stress peaks to completely strain a section to failure
  - ✓ as the member cracks and changes frequency, the response of the structure may change the seismic loads and failure potential
- Post-seismic stability must consider the ability of a damaged section to carry static loads

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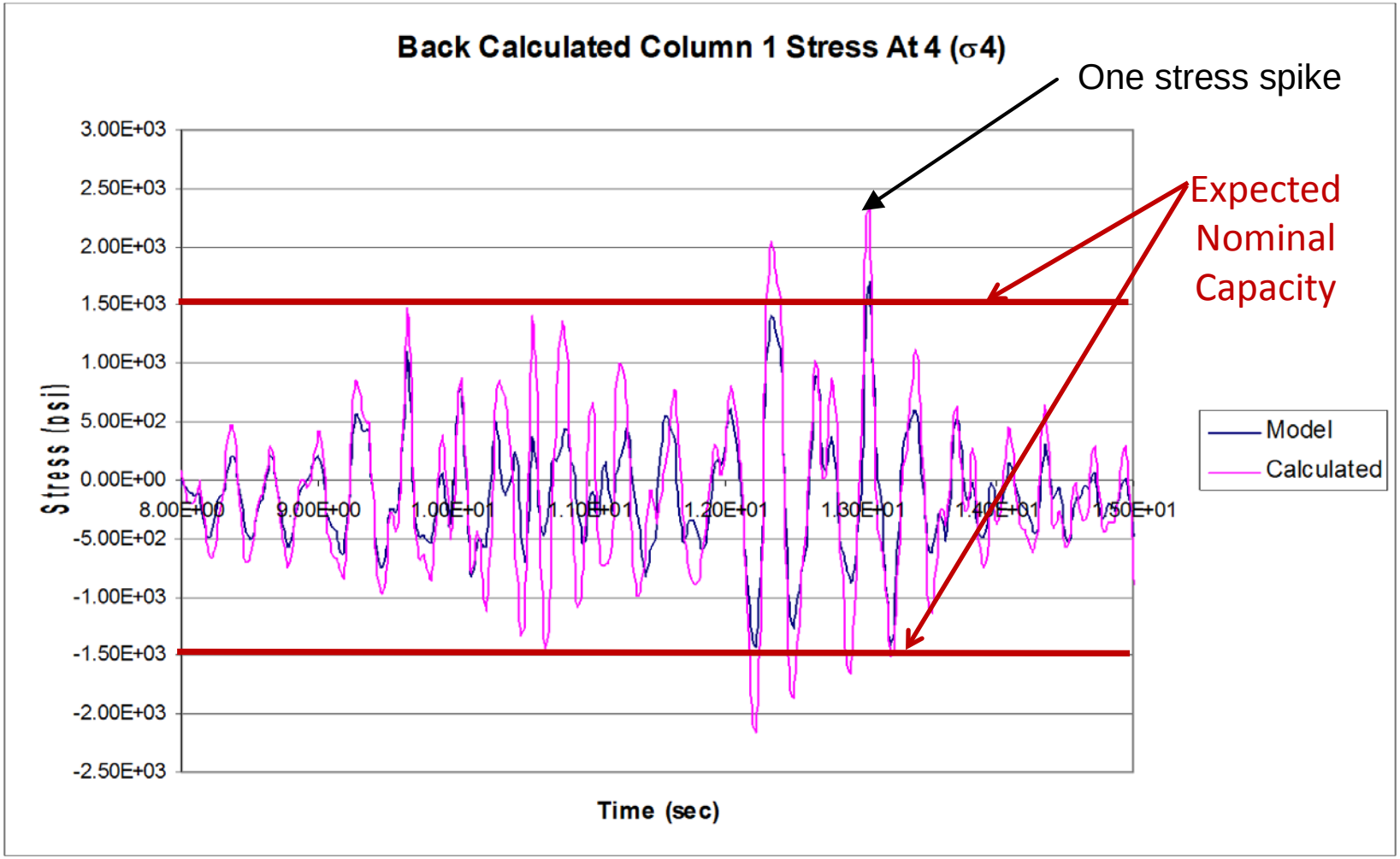
Type and Load Duration



Structural System Considerations

- Structural systems that perform well during earthquakes
  - ✓Dissipate energy through inelastic deformation
  - ✓Alter dynamic properties (period shift)
  - ✓Mobilize additional strength elsewhere in the system (highly redundant)
- Hydraulic structures are generally not highly redundant
- However, retaining walls have historically performed very well during earthquakes
- Seismic loads extend beyond performance database

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Structural System Considerations

- Concept of ASCE 7 response modification coefficients (R) that vary with the type of seismic force-resisting systems

- However, there are specific detailing requirements that must be met in order to utilize R coefficients to effectively reduce computed seismic demands

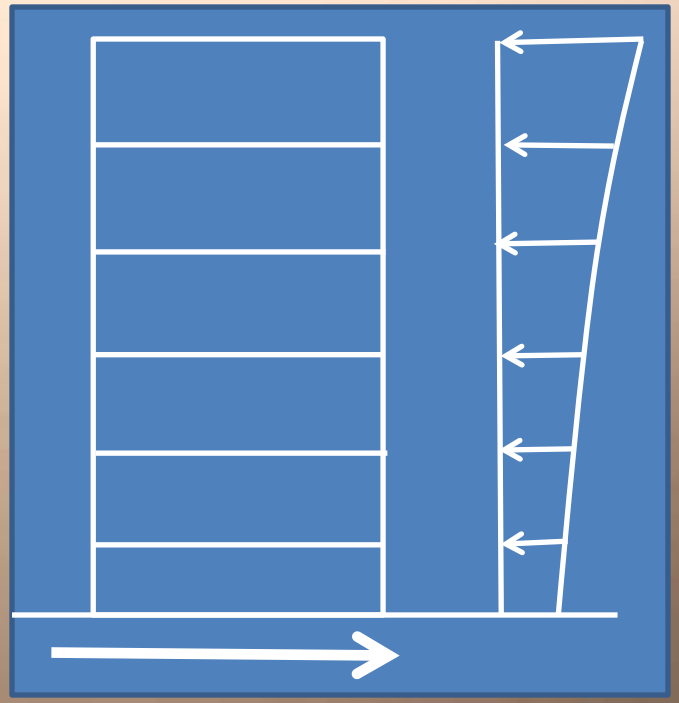
Lateral Force Resisting System Type

Detailing Requirements

R Value

TABLE 12.2-1 DESIGN COEFFICIENTS AND FACTORS FOR SEISMIC FORCE–RESISTING SYSTEMS

Seismic Force–Resisting System	ASCE 7 Section where Detailing Requirements are Specified	Response Modification Coefficient, R^a	System Overstrength Factor, $\Omega_0{}^g$	Deflection Amplification Factor, $C_d{}^b$	Structural Sys and Building H		
					Seismic Des		
					B	C	D[d]
A. BEARING WALL SYSTEMS							
1. Special reinforced concrete shear walls	14.2 and 14.2.3.6	5	2½	5	NL	NL	160
2. Ordinary reinforced concrete shear walls	14.2 and 14.2.3.4	4	2½	4	NL	NL	NP
3. Detailed plain concrete shear walls	14.2 and 14.2.3.2	2	2½	2	NL	NP	NP
4. Ordinary plain concrete shear walls	14.2 and 14.2.3.1	1½	2½	1½	NL	NP	NP
5. Intermediate precast shear walls	14.2 and 14.2.3.5	4	2½	4	NL	NL	40[k]
6. Ordinary precast shear walls	14.2 and 14.2.3.3	3	2½	3	NL	NP	NP
7. Special reinforced masonry shear walls	14.4 and 14.4.3	5	2½	3½	NL	NL	160
8. Intermediate reinforced masonry shear walls	14.4 and 14.4.3	3½	2½	2¼	NL	NL	NP
9. Ordinary reinforced masonry shear walls	14.4	2	2½	1¾	NL	160	NP
10. Detailed plain masonry shear walls	14.4	2	2½	1¾	NL	NP	NP
11. Ordinary plain masonry shear walls	14.4	1½	2½	1¼	NL	NP	NP
12. Prestressed masonry shear walls	14.4	1½	2½	1¾	NL	NP	NP
13. Light-framed walls sheathed with wood structural panels rated for shear resistance or steel sheets	14.1, 14.1.4.2, and 14.5	6½	3	4	NL	NL	65
14. Light-framed walls with shear panels of all other materials	14.1, 14.1.4.2, and 14.5	2	2½	2	NL	NL	35
15. Light-framed wall systems using flat	14.1, 14.1.4.2,	4	2	3½	NL	NL	65



$V = C_s W$

$C_s = S_{DS}/(R/I)$

Analysis Results Considerations

- When evaluating D/C ratios, it is important to evaluate values representative of the structure as a whole and not just localized maxima
- A progressive failure may occur if a localized area is overstressed, but this will take time under multiple earthquake peaks if there is potential for load redistribution
- Displacement criteria should be used to evaluate inelastic behavior of reinforced concrete members
- Biggest challenge for RA team
  - ✓Severe damage may result from many cycles of demand exceeding capacity
  - ✓The remaining strength of the damaged section is primarily a judgment call of the RA team

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Code Considerations

- Caution should be exercised when using National codes like ACI or AASHTO to compute the capacity of reinforced concrete sections

- Most codes are for new designs and assume ductile sections with adequate reinforcement details
  - ✓Adequate lap splices
  - ✓Appropriate confining reinforcement – closed ties or stirrups
  - ✓Proper anchorage of ties and hooks – 135° seismic hooks

- Consider looking at a code or reference for evaluation of existing structures
  - ✓ASCE 31 – Seismic Evaluations of Existing Buildings

Code Considerations

- Load factors and strength reduction (ϕ) factors
  - ✓Used for new designs to
    - Address analysis and design uncertainties and assumptions (LF)
    - Account for variations in materials (ϕ)
    - Account for variations in construction (ϕ)
    - Generally build-in factors of safety
  - ✓Do not apply for risk analyses of existing reinforced concrete structures
    - Compute the demand or load on the section without load factors
    - Compute the "true" or "expected" capacity of the section without ϕ
- During the risk analyses team members should consider:
  - ✓The condition of the concrete and reinforcement
  - ✓Severity of the environment
  - ✓Deterioration due to alkali-aggregate reaction
  - ✓Evidence of freeze-thaw deterioration
  - ✓Evidence of corrosion

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Typical Event Tree for Reinforced Concrete Columns, Piers, and Buttresses

Revision date: March 24, 2015

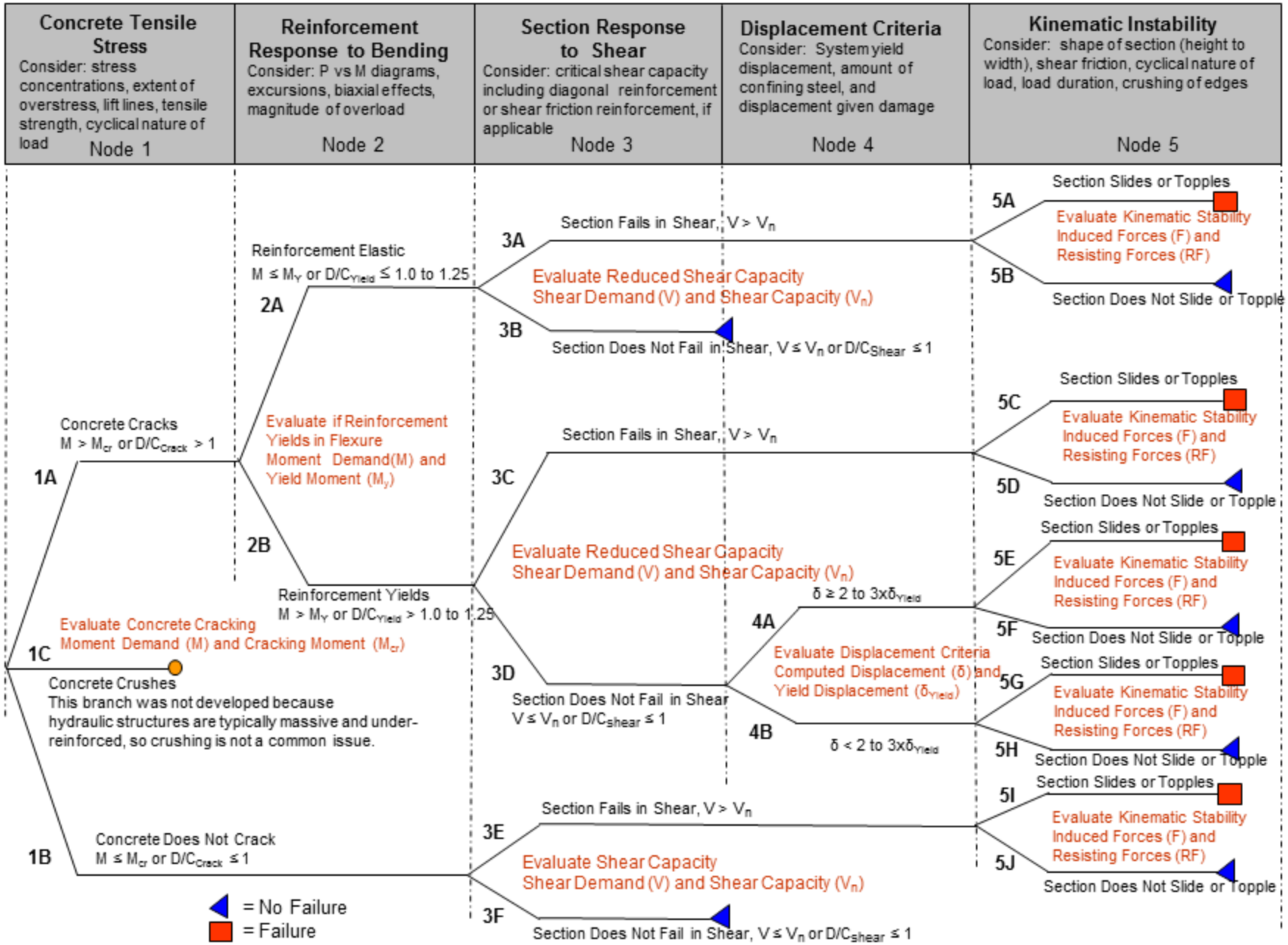


Figure 10-9 – Example Event Tree for Failure of a Reinforced Concrete Member

Node 1 – Concrete Stress

- Cracking moment criteria - compare moment demand (M) to cracking moment capacity (M_{CR})
  - ✓ $M \leq M_{cr}$

  where:

  $M_{cr} = f_t I_g / y_t$ (modified ACI Eq. 9-9)

  f_t = concrete tensile strength per Chapter 20

  I_g = moment of inertia of the gross concrete section

  y_t = distance from the section centroid to the extreme tension fiber

  - ✓ Tensile stresses from axial loads compared to f_t
  - ✓ Concrete crushing due to compressive stresses is unusual

Node 2 – Reinforcement Response to Bending

- Yield moment criteria - compare moment demand (M) to yield moment capacity (M_y)
  - ✓ $M \leq M_y$
  - ✓ $M_n \leq M_y \leq M_{pr}$

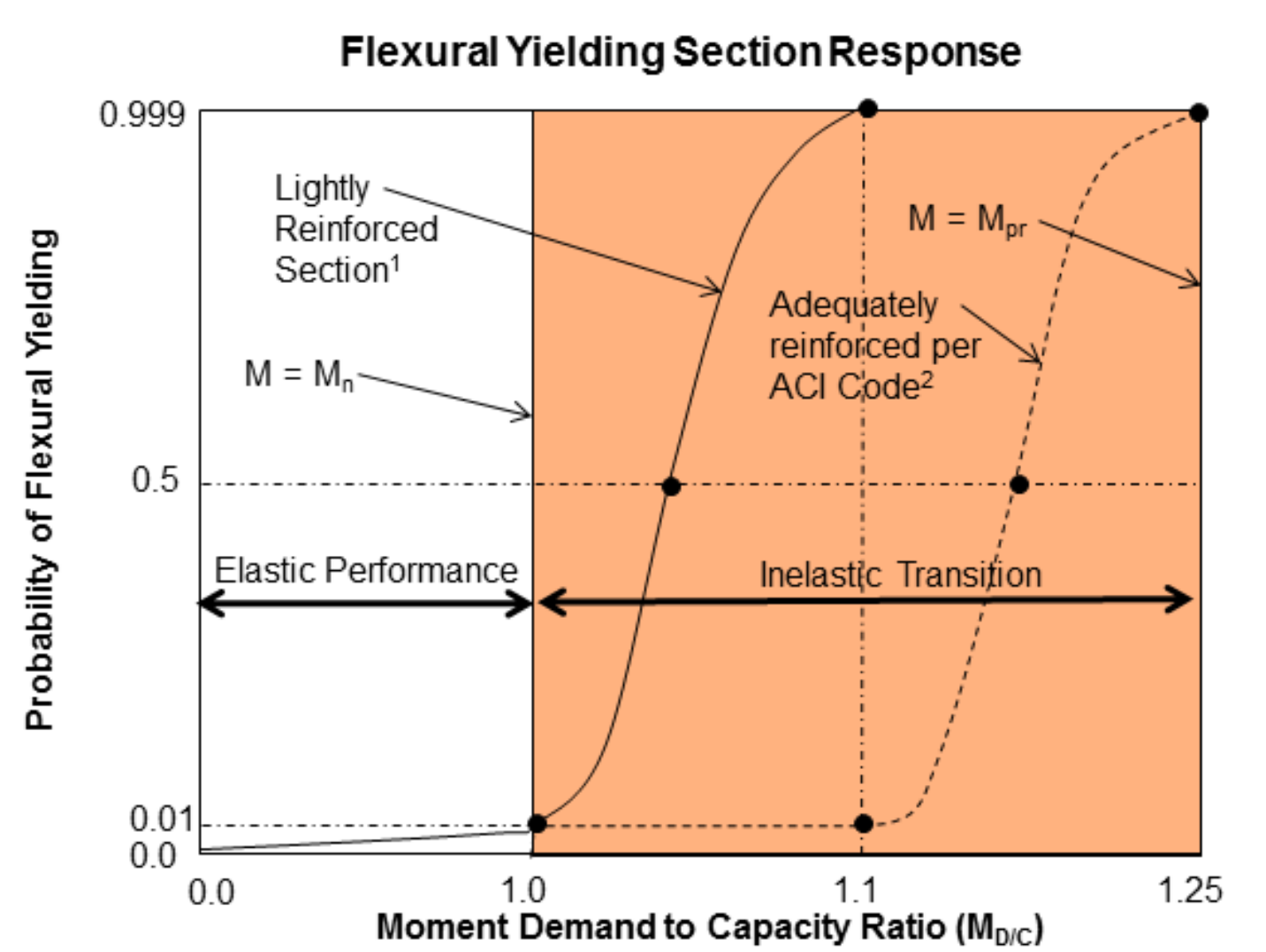
  where:

  M_y = section yield moment

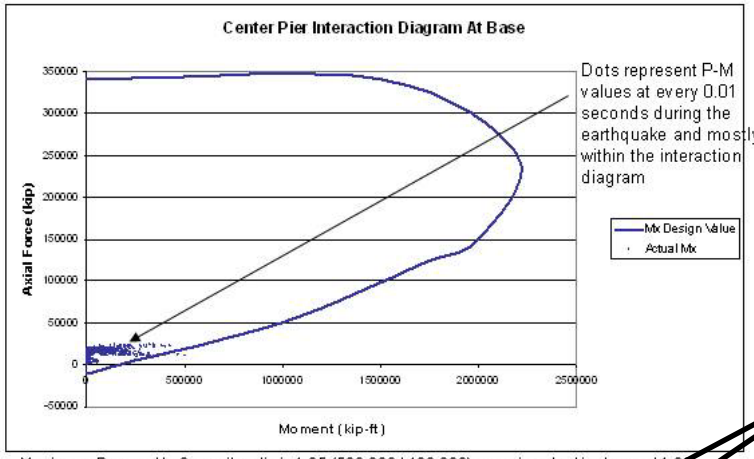
  $M_n = A_s f_y (d-a/2)$ = nominal moment capacity

  $M_{pr} = A_s (1.25 f_y)(d-a/2)$ = probable moment strength at plastic hinging

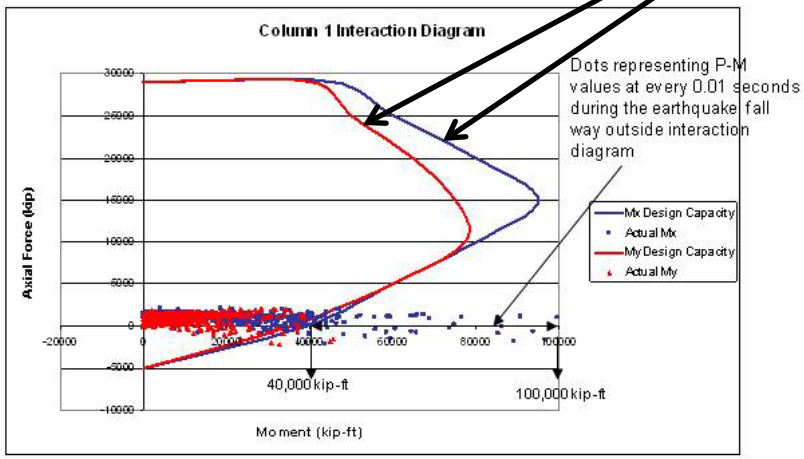
Node 2 – Reinforcement Response to Bending



Node 2 – Reinforcement Response to Bending



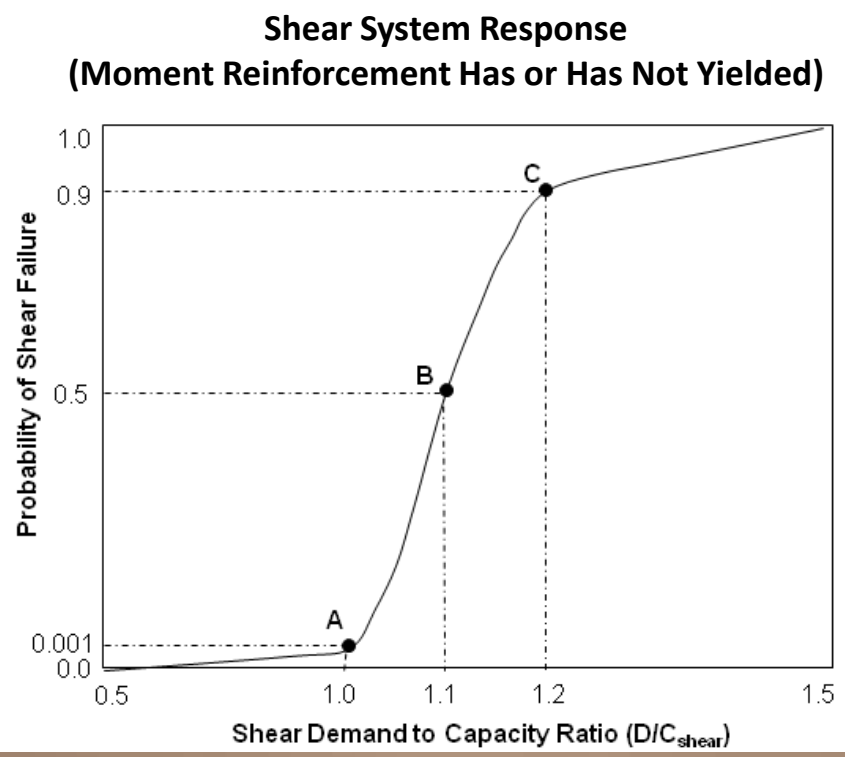
Maximum Demand to Capacity ratio is 1.25 (500,000 / 400,000) meaning steel is stressed 1.25 times beyond its yield



Maximum Demand to Capacity ratio is 2.5 (100,000 / 40,000) meaning steel is stressed 2.5 times beyond its yield

spColumn – interaction diagrams for member subjected to both axial load and flexure

Node 3 - Section Response to Shear



- Response curve more representative of lightly or unreinforced sections - shear reinforcement will add ductility

- For slender members (>4H:1W)
  - ✓ $V_n = V_c + V_s$
    - V_c = concrete shear strength
    - V_s = reinforcement shear strength

- Shear friction reinforcement
  - ✓ Need to consider type of shear failure when evaluating shear capacity – diagonal crack or horizontal crack
  - ✓ Should be supplemental to primary flexural reinforcement

Node 3 - Section Response to Shear

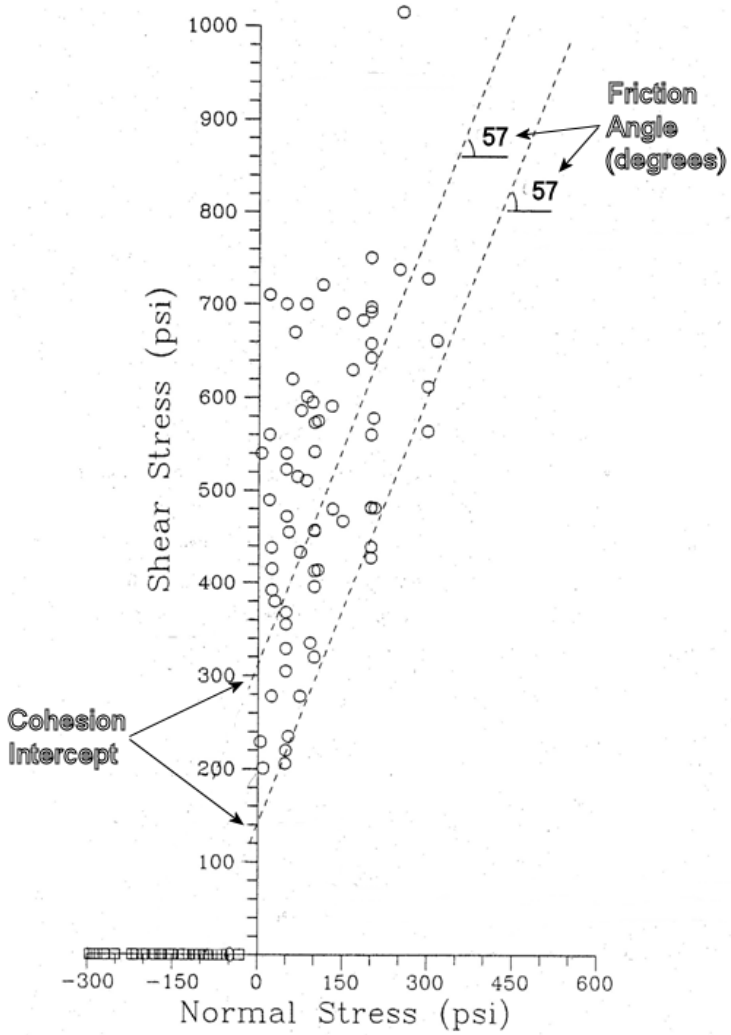
- Sliding

$$SF = (N - U)\mu + CA$$

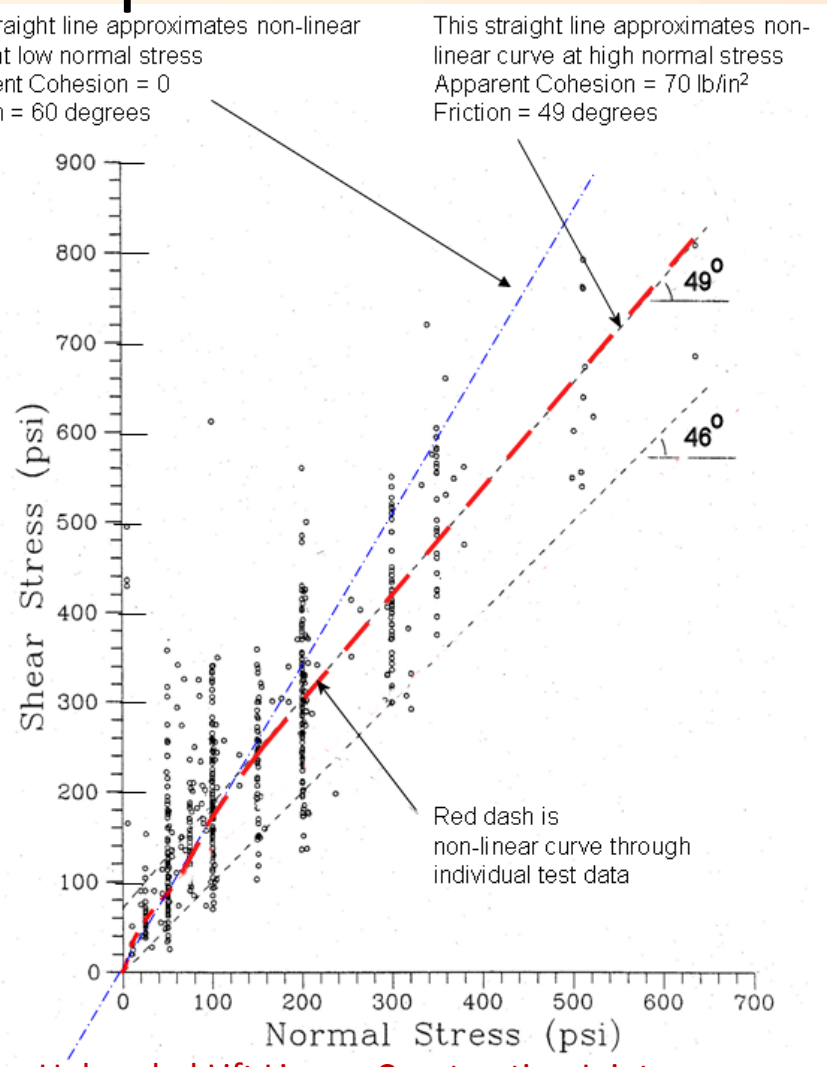
where:
SF = Shear resistance
N = Normal force on the sliding plane
U = Uplift forces along sliding plane
μ = Friction coefficient (tangent of the friction angle)
C = Cohesion (or apparent cohesion)
A = Area of slide surface

Node 3 - Section Response to Shear

Bonded Lift Line or Construction Joint



Peak Shear Strength of Concrete-Lift Joints



Unbonded Lift Line or Construction Joint

Sliding Friction Shear Strength of Concrete-Lift Joints (Unbonded joints)

Node 4 - Displacement Criteria

- Based on research at the University of Illinois at Champagne-Urbana by Mete Sozen
- Considers nonlinear behavior of section within structural system
- Determine nonlinear displacements in reinforced concrete system
- Structure may be viable if: $\delta / \delta_{yield} \leq 2$ to 3

Node 4 - Displacement Criteria

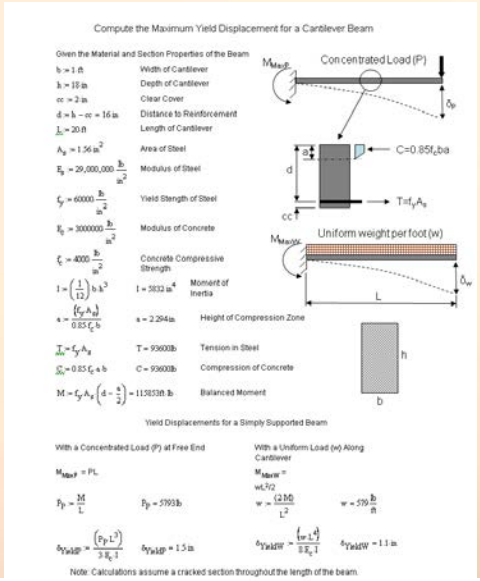
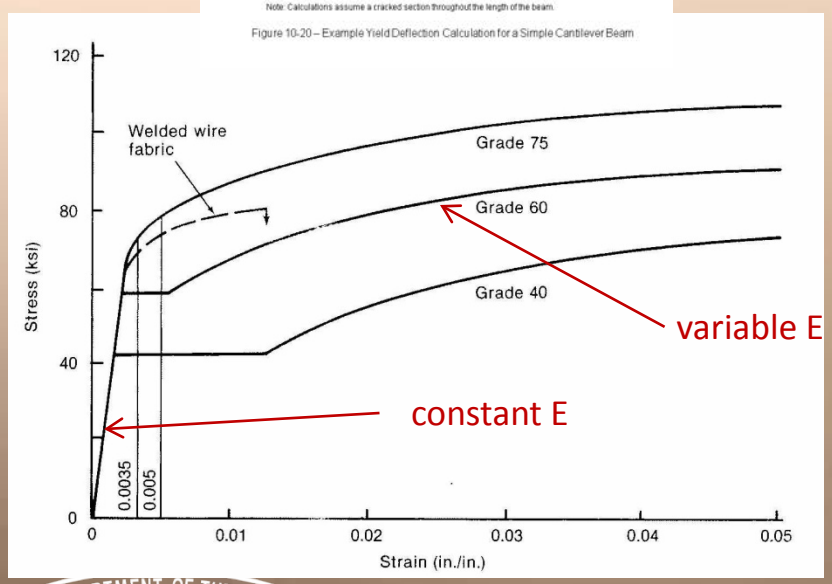
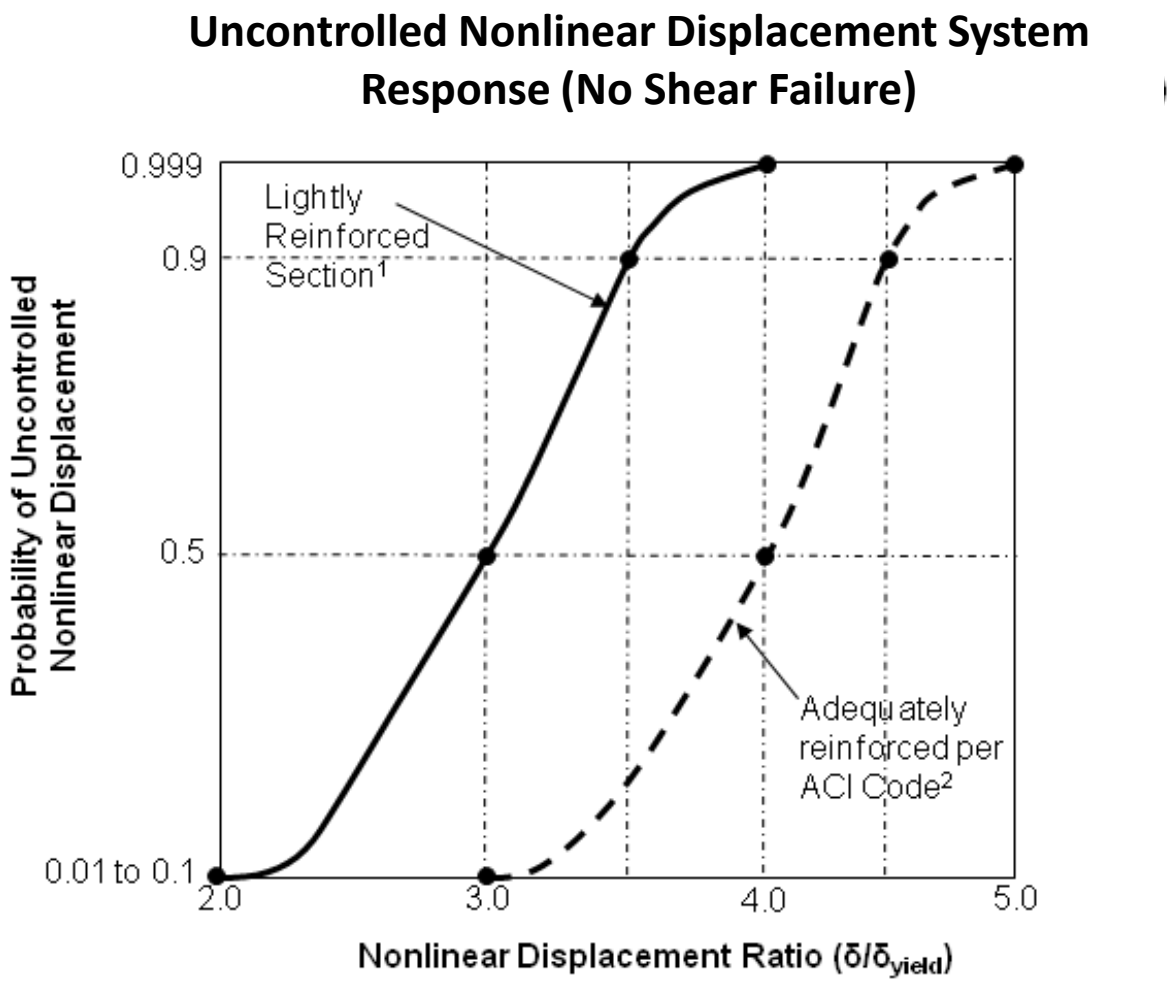


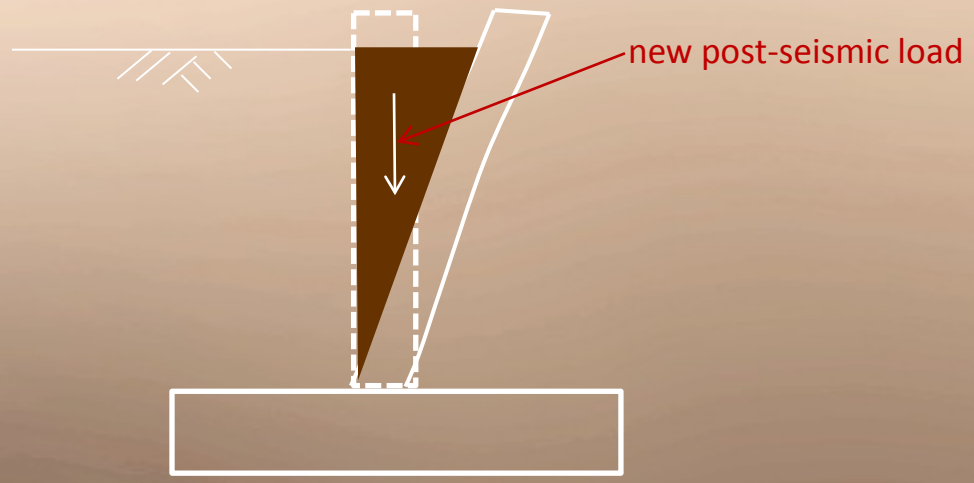
Figure 10-20 – Example Yield Deflection Calculation for a Simple Cantilever Beam



- δ_{yield} calculation
  - ✓Straightforward – constant E
  - ✓ Actual yield deflections will likely be larger since moment of inertia will be that for a cracked section (method is conservative)
- δ calculation
  - ✓Not so easy – variable E
  - ✓ Non-linear FEA most accurate approach
  - ✓Simplified approach use ⅓ to ½ E_c
  - ✓System secondary (P-δ) analysis

Node 4 - Displacement Criteria



Node 5 - Kinematic Instability

- Three cases to consider
  - ✓ Independent concrete block separated from structure by shear failure (sliding)
  - ✓ Uncontrolled displacement of yielded member (toppling)
  - ✓ Post-seismic instability of yielded member