3. CAPACITIES OF STRUCTURE COMPONENTS

3.1 Displacement Capacity of Ductile Concrete Members

3.1.1 Ductile Member Definition

A ductile member is defined as any member that is intentionally designed to deform inelastically for several cycles without significant degradation of strength or stiffness under the demands generated by the MCE.

3.1.2 Distinction Between Local Member Capacity and Global Structure System Capacity

Local member displacement capacity, Δ_c is defined as a member's displacement capacity attributed to its elastic and plastic flexibility as defined in Section 3.1.3. The structural system's displacement capacity, Δ_C is the reliable lateral capacity of the bridge or subsystem as it approaches its Collapse Limit State. Ductile members must meet the local displacement capacity requirements specified in Section 3.1.4.1 and the global displacement criteria specified in Section 4.1.1.

3.1.3 Local Member Displacement Capacity

The local displacement capacity of a member is based on its rotation capacity, which in turn is based on its curvature capacity. The curvature capacity shall be determined by M-ϕ analysis, see Section 3.3.1. The local displacement capacity Δ_c of any column may be idealized as one or two cantilever segments presented in equations 3.1-3.5 and 3.1a-3.5a, respectively. See Figures 3.1 and 3.2 for details.

$$\Delta_c = \Delta_Y^{col} + \Delta_p \tag{3.1}$$

$$\Delta_Y^{col} = \frac{L^2}{3} \times \phi_Y \tag{3.2}$$

$$\Delta_p = \theta_p \times \left(L - \frac{L_p}{2} \right) \tag{3.3}$$

$$\theta_p = L_p \times \phi_p \tag{3.4}$$

$$\phi_p = \phi_u - \phi_Y \tag{3.5}$$

$$\Delta_{c1} = \Delta_{Y1}^{col} + \Delta_{p1} \quad , \quad \Delta_{c2} = \Delta_{Y2}^{col} + \Delta_{p2} \tag{3.1a}$$

$$\Delta_{Y1}^{col} = \frac{L_1^2}{3} \times \phi_{Y1} \quad , \quad \Delta_{Y2}^{col} = \frac{L_2^2}{3} \times \phi_{Y2} \tag{3.2a}$$

$$\Delta_{p1} = \theta_{p1} \times \left(L_1 - \frac{L_{p1}}{2} \right) \quad , \quad \Delta_{p2} = \theta_{p2} \times \left(L_2 - \frac{L_{p2}}{2} \right) \tag{3.3a}$$

$$\theta_{p1} = L_{p1} \times \phi_{p1} \quad , \quad \theta_{p2} = L_{p2} \times \phi_{p2} \tag{3.4a}$$

$$\phi_{p1} = \phi_{u1} - \phi_{Y1} \quad , \quad \phi_{p2} = \phi_{u2} - \phi_{Y2} \tag{3.5a}$$

Where:

L = Distance from the point of maximum moment to the point of contra-flexure

L_P = Equivalent analytical plastic hinge length as defined in Section 7.6.2

Δ_p = Idealized plastic displacement capacity due to rotation of the plastic hinge

Δ_Y^{col} = The idealized yield displacement of the column at the formation of the plastic hinge

ϕ_Y = Idealized yield curvature defined by an elastic-perfectly-plastic representation of the cross section's M-ϕ curve, see Figure 3.7

ϕ_p = Idealized plastic curvature capacity (assumed constant over L_p)

ϕ_u = Curvature capacity at the Failure Limit State, defined as the concrete strain reaching ε_{cu} or the confinement reinforcing steel reaching the reduced ultimate strain ε_{su}^R

θ_p = Plastic rotation capacity

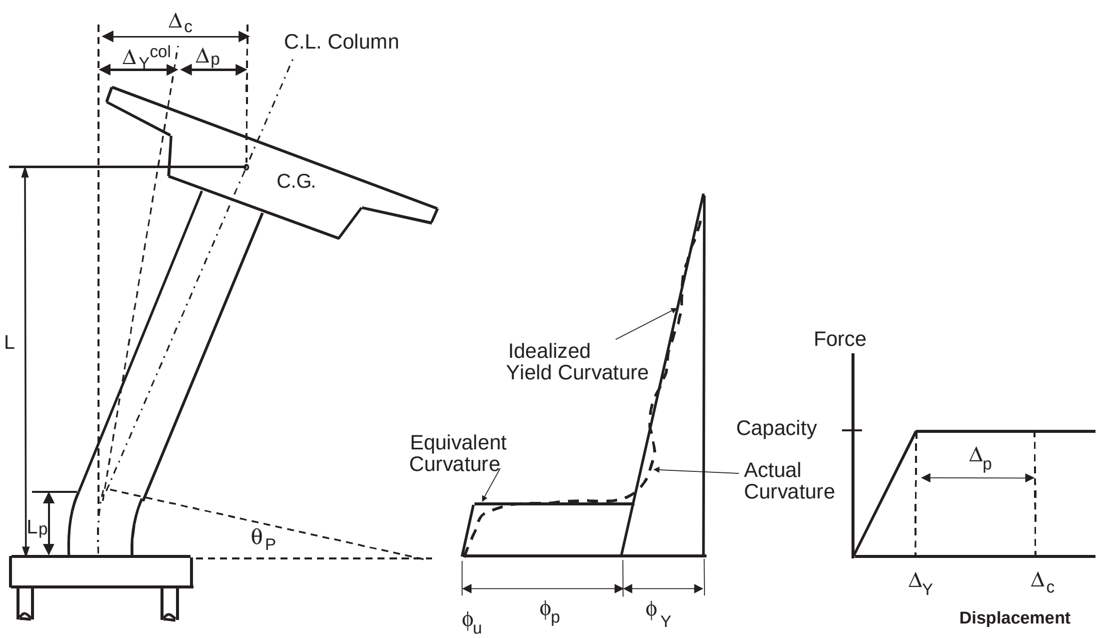

Figure 3.1 Local Displacement Capacity - Cantilever Column w/ Fixed Base

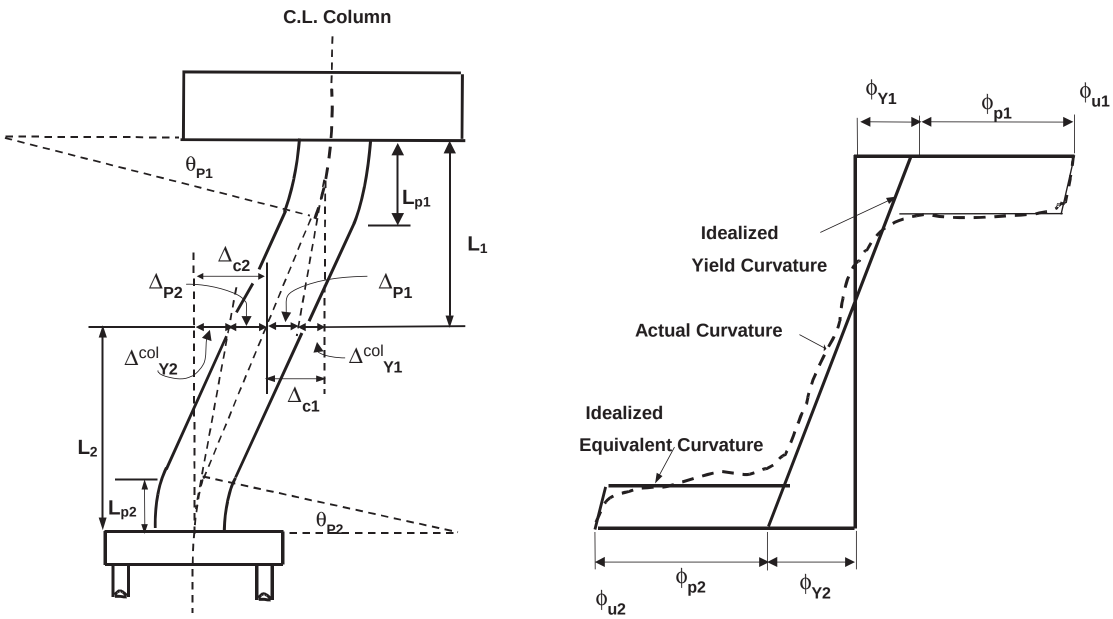

Figure 3.2 Local Displacement Capacity - Framed Column, Assumed as Fixed-Fixed

3.1.4 Local Member Displacement Ductility Capacity

Local displacement ductility capacity for a particular member is defined in equation 3.6.

$$\mu_c = \frac{\Delta_c}{\Delta_Y^{col}} \quad \text{for Cantilever columns,}$$

$$\mu_{c1} = \frac{\Delta_{c1}}{\Delta_{Y1}^{col}} \quad \& \quad \mu_{c2} = \frac{\Delta_{c2}}{\Delta_{Y2}^{col}} \text{ for fixed-fixed columns} \tag{3.6}$$

3.1.4.1 Minimum Local Displacement Ductility Capacity

Each ductile member shall have a minimum local displacement ductility capacity of $\mu_c = 3$ to ensure dependable rotational capacity in the plastic hinge regions regardless of the displacement demand imparted to that member. The local displacement ductility capacity shall be calculated for an equivalent member that approximates a fixed base cantilever element as defined in Figure 3.3.

The minimum displacement ductility capacity $\mu_c = 3$ may be difficult to achieve for columns and Type I pile shafts with large diameters $D_c > 10$ ft, (3m) or components with large L/D ratios. Local displacement ductility capacity less than 3 requires approval, see MTD 20-11 for the approval process.

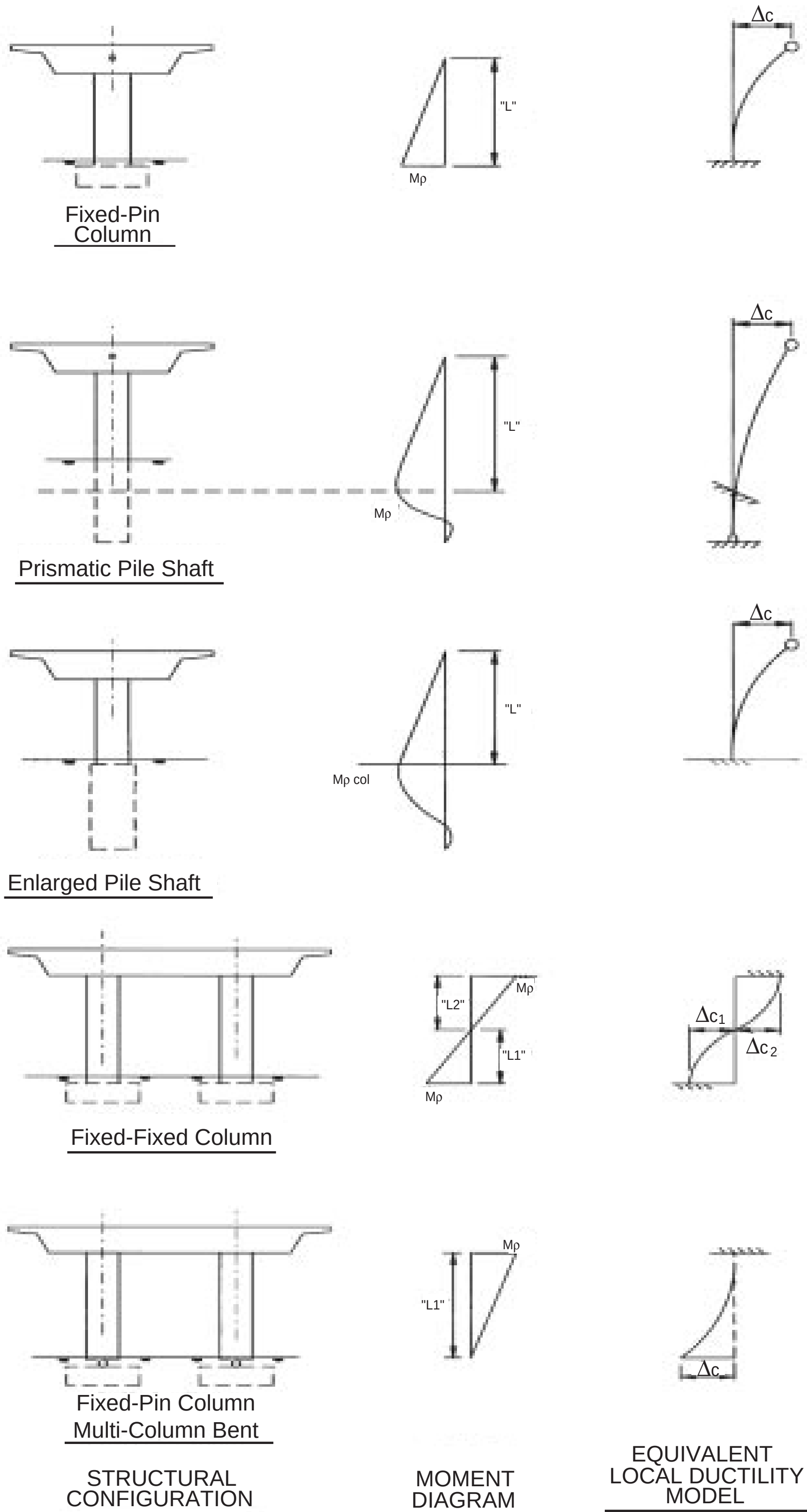

Figure 3.3 Local Ductility Assessment

3.2 Material Properties for Concrete Components

3.2.1 Expected Material Properties

The capacity of concrete components to resist all seismic demands, except shear, shall be based on most probable (expected) material properties to provide a more realistic estimate for design strength. An expected concrete compressive strength, f'_{ce} recognizes the typically conservative nature of concrete batch design, and the expected strength gain with age. The yield stress f_y for ASTM A706 steel can range between 60 ksi to 78 ksi. An expected reinforcement yield stress f_{ye} is a "characteristic" strength and better represents the actual strength than the specified minimum of 60 ksi. The possibility that the yield stress may be less than f_{ye} in ductile components will result in a reduced ratio of actual plastic moment strength to design strength, thus conservatively impacting capacity protected components. The possibility that the yield stress may be less than f_{ye} in essentially elastic components is accounted for in the overstrength magnifier specified in Section 4.3.1. Expected material properties shall only be used to assess capacity for earthquake loads. The material properties for all other load cases shall comply with the Caltrans Bridge Design Specifications (BDS). Seismic shear capacity shall be conservatively based on the nominal material strengths defined in Section 3.6.1, not the expected material strengths.

3.2.2 Nonlinear Reinforcing Steel Models for Ductile Reinforced Concrete Members

Reinforcing steel shall be modeled with a stress-strain relationship that exhibits an initial linear elastic portion, a yield plateau, and a strain hardening range in which the stress increases with strain.

The yield point should be defined by the expected yield stress of the steel f_{ye}. The length of the yield plateau shall be a function of the steel strength and bar size. The strain-hardening curve can be modeled as a parabola or other non-linear relationship and should terminate at the ultimate tensile strain ε_{su} . The ultimate strain should be set at the point where the stress begins to drop with increased strain as the bar approaches fracture. It is Caltrans' practice to reduce the ultimate strain by up to thirty-three percent to decrease the probability of fracture of the reinforcement. The commonly used steel model is shown in Figure 3.4 [4].

3.2.3 Reinforcing Steel A706/A706M (Grade 60/Grade 400)

For A706/A706M reinforcing steel, the following properties based on a limited number of monotonic pull tests conducted by Materials Engineering and Testing Services (METS) may be used. The designer may use actual test data if available.

Modulus of elasticity	$E_s = 29{,}000$ ksi	200,000 MPa
Specified minimum yield strength	$f_y = 60$ ksi	420 MPa
Expected yield strength	$f_{ye} = 68$ ksi	475 MPa
Specified minimum tensile strength	$f_u = 80$ ksi	550 MPa
Expected tensile strength	$f_{ue} = 95$ ksi	655 MPa
Nominal yield strain	$\varepsilon_y = 0.0021$	
Expected yield strain	$\varepsilon_{ye} = 0.0023$	

Ultimate tensile strain

$$\varepsilon_{su} = \begin{cases} 0.120 & \#10\ (\#32m)\ \text{bars and smaller} \\ 0.090 & \#11\ (\#36m)\ \text{bars and larger} \end{cases}$$

Reduced ultimate tensile strain

$$\varepsilon_{su}^{R} = \begin{cases} 0.090 & \#10\ (\#32m)\ \text{bars and smaller} \\ 0.060 & \#11\ (\#36m)\ \text{bars and larger} \end{cases}$$

Onset of strain hardening

$$\varepsilon_{sh} = \begin{cases} 0.0150 & \#8\ (\#25m)\ \text{bars} \\ 0.0125 & \#9\ (\#29m)\ \text{bars} \\ 0.0115 & \#10\ \&\ \#11\ (\#32m\ \&\ \#36m)\ \text{bars} \\ 0.0075 & \#14\ (\#43m)\ \text{bars} \\ 0.0050 & \#18\ (\#57m)\ \text{bars} \end{cases}$$

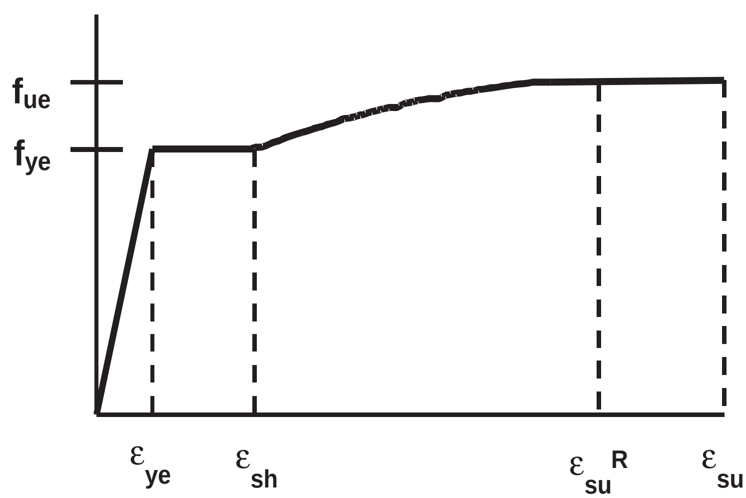

Figure 3.4 Steel Stress Strain Model

3.2.4 Nonlinear Prestressing Steel Model

Prestressing steel shall be modeled with an idealized nonlinear stress strain model. Figure 3.5 is an idealized stress-strain model for 7-wire low-relaxation prestressing strand. The curves in Figure 3.5 can be approximated by equations 3.7 – 3.10. See MTD 20-3 for the material properties pertaining to high strength rods (ASTM A722 Uncoated High-Strength Steel Bar for Prestressing Concrete). Consult the OSD Prestressed Concrete Committee for the stress-strain models of other prestressing steels.

Essentially elastic prestress steel strain

$$\varepsilon_{ps,EE} = \begin{cases} 0.0076 & \text{for } f_u = 250\,\text{ksi} \quad (1725\,\text{MPa}) \\ 0.0086 & \text{for } f_u = 270\,\text{ksi} \quad (1860\,\text{MPa}) \end{cases}$$

Reduced ultimate prestress steel strain $\varepsilon^R_{ps,u} = 0.03$

250 ksi (1725 MPa) Strand:

$$\varepsilon_{ps} \le 0.0076: \; f_{ps} = 28{,}500 \times \varepsilon_{ps} \;\text{(ksi)} \qquad f_{ps} = 196{,}500 \times \varepsilon_{ps} \;\text{(MPa)} \tag{3.7}$$

$$\varepsilon_{ps} \ge 0.0076: \; f_{ps} = 250 - \frac{0.25}{\varepsilon_{ps}} \;\text{(ksi)} \qquad f_{ps} = 1725 - \frac{1.72}{\varepsilon_{ps}} \;\text{(MPa)} \tag{3.8}$$

270 ksi (1860 MPa) Strand:

$$\varepsilon_{ps} \le 0.0086: \; f_{ps} = 28{,}500 \times \varepsilon_{ps} \;\text{(ksi)} \qquad f_{ps} = 196{,}500 \times \varepsilon_{ps} \;\text{(MPa)} \tag{3.9}$$

$$\varepsilon_{ps} \ge 0.0086: \; f_{ps} = 270 - \frac{0.04}{\varepsilon_{ps} - 0.007} \;\text{(ksi)} \qquad f_{ps} = 1860 - \frac{0.276}{\varepsilon_{ps} - 0.007} \;\text{(MPa)} \tag{3.10}$$

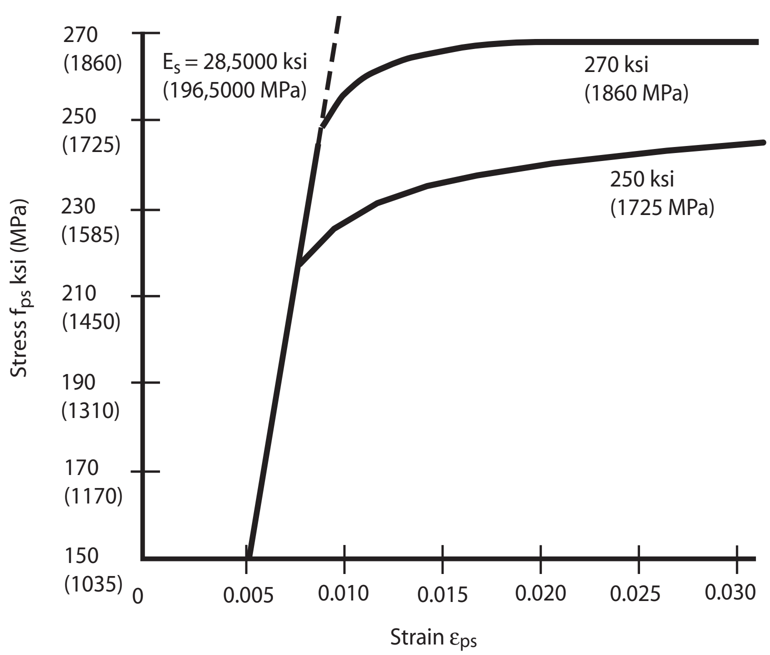

Figure 3.5 Prestressing Strand Stress Strain Model

3.2.5 Nonlinear Concrete Models for Ductile Reinforced Concrete Members

A stress-strain model for confined and unconfined concrete shall be used in the analysis to determine the local capacity of ductile concrete members. The initial ascending curve may be represented by the same equation for both the confined and unconfined model since the confining steel has no effect in this range of strains. As the curve approaches the compressive strength of the unconfined concrete, the unconfined stress begins to fall to an unconfined strain level before rapidly degrading to zero at the spalling strain ε_{sp}, typically $\varepsilon_{sp} \approx 0.005$. The confined concrete model should continue to ascend until the confined compressive strength f'_{cc} is reached. This segment should be followed by a descending curve dependent on the parameters of the confining steel. The ultimate strain ε_{cu} should be the point where strain energy equilibrium is reached between the concrete and the confinement steel. A commonly used model is Mander's stress strain model for confined concrete shown in Figure 3.6 [4].

3.2.6 Normal Weight Portland Cement Concrete Properties

Modulus of Elasticity , $E_c = 33 \times w^{1.5} \times \sqrt{f'_{ce}}$ (psi) $E_c = 0.043 \times w^{1.5} \times \sqrt{f'_{ce}}$ (MPa) (3.11)

Where w = unit weight of concrete is in lb/ft³ and kg/m³, respectively. For w = 143.96 lb/ft³ (2286.05 kg/m³), Equation 3.11 results in the form presented in other Caltrans documents.

Shear Modulus

$$G_c = \frac{E_c}{2 \times (1 + \nu)} \qquad (3.12)$$

Poisson's Ratio $\nu = 0.2$

Expected concrete compressive strength f'_c = the greater of:

$$\begin{cases} 1.3 \times f'_c \\ or \\ 5000\,(\text{psi})\ 34.5\,(\text{MPa}) \end{cases} \qquad (3.13)$$

Unconfined concrete compressive strain at the maximum compressive stress $\varepsilon_{c0} = 0.002$

Ultimate unconfined compression (spalling) strain $\varepsilon_{sp} = 0.005$

Confined compressive strain $\varepsilon_{cc} = *$

Ultimate compression strain for confined concrete $\varepsilon_{cu} = *$

* Defined by the constitutive stress strain model for confined concrete, see Figure 3.6.

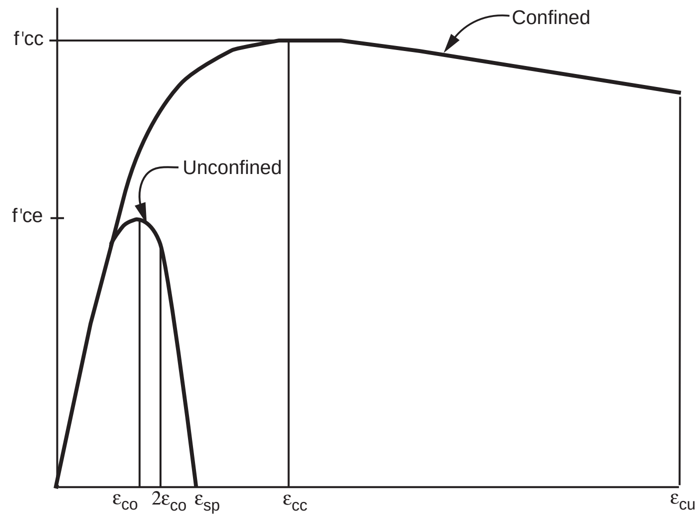

Figure 3.6 Concrete Stress Strain Model

3.2.7 Other Material Properties

Inelastic behavior shall be limited to pre-determined locations. If non-standard components are explicitly designed for ductile behavior, the bridge is classified as non-standard. The material properties and stress-strain relationships for non-standard components shall be included in the project specific design criteria.

3.3 Plastic Moment Capacity for Ductile Concrete Members

3.3.1 Moment Curvature ($M\!-\!\phi$) Analysis

The plastic moment capacity of all ductile concrete members shall be calculated by $M\!-\!\phi$ analysis based on expected material properties. Moment curvature analysis derives the curvatures associated with a range of moments for a cross section based on the principles of strain compatibility and equilibrium of forces. The $M\!-\!\phi$ curve can be idealized with an elastic perfectly plastic response to estimate the plastic moment capacity of a member's cross section. The elastic portion of the idealized curve should pass through the point marking the first reinforcing bar yield. The idealized plastic moment capacity is obtained by balancing the areas between the actual and the idealized $M\!-\!\phi$ curves beyond the first reinforcing bar yield point, see Figure 3.7 [4].

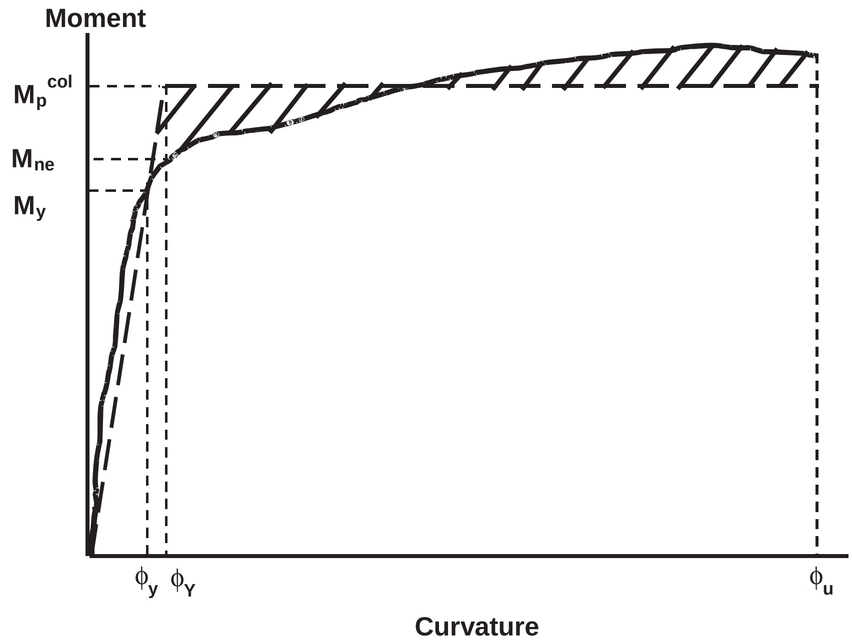

Figure 3.7 Moment Curvature Curve

3.4 Requirements for Capacity Protected Components

Capacity protected concrete components such as footings, Type II pile shafts, bent cap beams, joints, and superstructure shall be designed flexurally to remain essentially elastic when the column reaches its overstrength capacity. The expected nominal moment capacity M_{ne} for capacity protected concrete components determined by either $M\text{–}\phi$ or strength design, is the minimum requirement for essentially elastic behavior. Due to cost considerations a factor of safety is not required. Expected material properties shall only be used to assess flexural component capacity for resisting earthquake loads. The material properties used for assessing all other load cases shall comply with the Caltrans design manuals.

Expected nominal moment capacity for capacity protected concrete components shall be based on the expected concrete and steel strengths when either the concrete strain reaches 0.003 or the reinforcing steel strain reaches ε_{su}^R as derived from the steel stress strain model.

3.5 Minimum Lateral Strength

Each column shall have a minimum lateral flexural capacity (based on expected material properties) to resist a lateral force of $0.1 \times P_{dl}$, where P_{dl} is the tributary dead load applied at the center of gravity of the superstructure.

3.6 Seismic Shear Design for Ductile Concrete Members

3.6.1 Nominal Shear Capacity

The seismic shear demand shall be based on the overstrength shear V_o associated with the overstrength moment M_o defined in Section 4.3. The shear capacity for ductile concrete members shall be conservatively based on the nominal material strengths.

$$\phi V_n \geq V_o \qquad \phi = 0.85 \tag{3.14}$$

$$V_n = V_c + V_s \tag{3.15}$$

3.6.2 Concrete Shear Capacity

The concrete shear capacity of members designed for ductility shall consider the effects of flexure and axial load as specified in equation 3.16 through 3.21.

$$V_c = v_c \times A_e \tag{3.16}$$

$$A_e = 0.8 \times A_g \tag{3.17}$$

- Inside the plastic hinge zone

$$v_c = \begin{cases} \text{Factor 1} \times \text{Factor 2} \times \sqrt{f'_c} \leq 4\sqrt{f'_c} & \text{(psi)} \\ \text{Factor 1} \times \text{Factor 2} \times \sqrt{f'_c} \leq 0.33\sqrt{f'_c} & \text{(MPa)} \end{cases} \tag{3.18}$$

- Outside the plastic hinge zone

$$v_c = \begin{cases} 3 \times \text{Factor 2} \times \sqrt{f'_c} \leq 4\sqrt{f'_c} & \text{(psi)} \\ 0.25 \times \text{Factor 2} \times \sqrt{f'_c} \leq 0.33\sqrt{f'_c} & \text{(MPa)} \end{cases} \tag{3.19}$$

$$\text{Factor 1} = \begin{cases} 0.3 \leq \dfrac{\rho_s f_{yh}}{0.150} + 3.67 - \mu_d < 3 & \text{(English Units)} \\ 0.025 \leq \dfrac{\rho_s f_{yh}}{12.5} + 0.305 - 0.083\mu_d < 0.25 & \text{(Metric Units)} \end{cases} \tag{3.20}$$

In equation (3.20), f_{yh} is in ksi [MPa]

$$\text{Factor 2} = \begin{cases} 1 + \dfrac{P_c}{2000 \times A_g} < 1.5 & \text{(English Units)} \\ 1 + \dfrac{P_c}{13.8 \times A_g} < 1.5 & \text{(Metric Units) (MPa)} \end{cases} \tag{3.21}$$

In equation (3.21), P_c is in Lb (N), and Ag in in^2 (mm^2)

For members whose net axial load is in tension, $v_c = 0$.

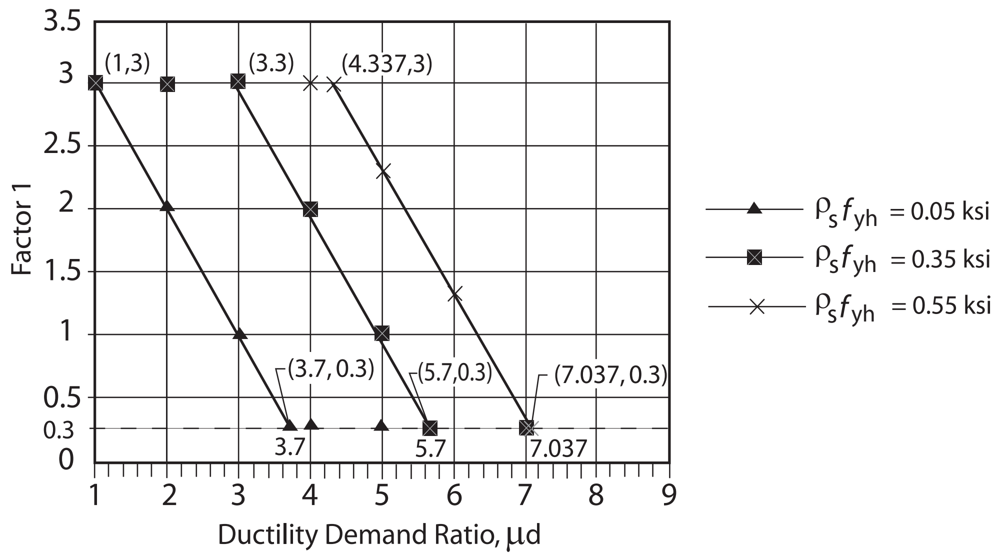

Figure 3.8 Concrete Shear Factors

The global displacement ductility demand μ_D shall be used in the determination of Factor 1 provided a significant portion of the global displacement is attributed to the deformation of the column or pier. In all other cases a local displacement ductility demand μ_d shall be used in Factor 1 of the shear equation.

3.6.3 Shear Reinforcement Capacity

For confined circular or interlocking core sections

$$V_s = \left(\frac{A_v \; f_{yh} \; D'}{s} \right), \quad \text{where } A_v = n * \left(\frac{\pi}{2} \right) * A_b \tag{3.22}$$

n = number of individual interlocking spiral or hoop core sections.

For pier walls (in the weak direction)

$$V_s = \left(\frac{A_v \; f_{yh} \; D'}{s} \right) \tag{3.23}$$

A_v = Total area of the shear reinforcement.

Alternative methods for assessing the shear capacity of members designed for ductility must be approved through the process outlined in MTD 20-11.

3.6.4 *Deleted*

3.6.5 Maximum and Minimum Shear Reinforcement Requirements for Columns

3.6.5.1 Maximum Shear Reinforcement

The shear strength V_s provided by the reinforcing steel shall not be taken greater than:

$$8 \times \sqrt{f'_c} A_e \quad \text{(psi)} \qquad 0.67 \times \sqrt{f'_c} A_e \quad \left(\frac{\text{N}}{\text{mm}^2}\right) \tag{3.24}$$

3.6.5.2 Minimum Shear Reinforcement

The area of shear reinforcement provided in columns shall be greater than the area required by equation 3.25. The area of shear reinforcement for each individual core of columns confined by interlocking spirals or hoops shall be greater than the area required by equation 3.25.

$$A_v \geq 0.025 \times \frac{D' s}{f_{yh}} \quad (\text{in}^2) \qquad A_v \geq 0.17 \times \frac{D' s}{f_{yh}} \quad (\text{mm}^2) \tag{3.25}$$

3.6.5.3 Minimum Vertical Reinforcement in Interlocking Portion

The longitudinal rebars in the interlocking portion of the column shall have a maximum spacing of 8 inches and need not be anchored in the footing or the bent cap unless deemed necessary for the flexural capacity of the column. The longitudinal rebar size in the interlocking portion of the column shall be chosen correspondingly to the rebars outside the interlocking portion as follows:

Size of rebars required inside the interlocking portion	Size of rebars used outside the interlocking portion
#6	#10
#8	#11
#9	#14
#11	#18

3.6.6 Shear Capacity of Pier Walls

3.6.6.1 Shear Capacity in the Weak Direction

The shear capacity for pier walls in the weak direction shall be designed according to Section 3.6.2 & 3.6.3.

3.6.6.2 Shear Capacity in the Strong Direction

The shear capacity of pier walls in the strong direction shall resist the maximum shear demand specified in Section 2.3.2.2.

$$\phi V_n^{pw} > V_u^{pw} \qquad \phi = 0.85 \tag{3.26}$$

Studies of squat shear walls have demonstrated that the large shear stresses associated with the moment capacity of the wall may lead to a sliding failure brought about by crushing of the concrete at the base of the wall. The thickness of pier walls shall be selected so the shear stress satisfies equation 3.27 [6].

$$\frac{V_n^{pw}}{0.8 \times A_g} < 8 \times \sqrt{f_c'} \quad \text{(psi)} \qquad \frac{V_n^{pw}}{0.8 \times A_g} < 0.67 \times \sqrt{f_c'} \quad \text{(MPa)} \tag{3.27}$$

3.6.7 Shear Capacity of Capacity Protected Members

The shear capacity of essentially elastic members shall be designed in accordance with BDS Section 8.16.6 using nominal material properties.

3.7 Maximum and Minimum Longitudinal Reinforcement

3.7.1 Maximum Longitudinal Reinforcement

The area of longitudinal reinforcement for compression members shall not exceed the value specified in equation 3.28.

$$0.04 \times A_g \tag{3.28}$$

3.7.2 Minimum Longitudinal Reinforcement

The minimum area of longitudinal reinforcement for compression members shall not be less than the value specified in equation 3.29 and 3.30.

$$0.01 \times A_g \quad \text{Columns} \tag{3.29}$$

$$0.005 \times A_g \quad \text{Pier Walls} \tag{3.30}$$

3.7.3 Maximum Reinforcement Ratio

The designer must ensure that members sized to remain essentially elastic (i.e. superstructure, bent caps, footings, enlarged pile shafts) retain a ductile failure mode. The reinforcement ratio, ρ shall meet the requirements in BDS Section 8.16.3 for reinforced concrete members and BDS Section 9.19 for prestressed concrete members.

3.8 Lateral Reinforcement of Ductile Members

3.8.1 Lateral Reinforcement Inside the Analytical Plastic Hinge Length

The volume of lateral reinforcement typically defined by the volumetric ratio, ρ_s provided inside the plastic hinge length shall be sufficient to ensure the column or pier wall meets the performance requirements in Section 4.1. ρ_s for columns with circular or interlocking core sections is defined by equation 3.31.

$$\rho_s = \frac{4A_b}{D' s} \tag{3.31}$$

3.8.2 Lateral Column Reinforcement Inside the Plastic Hinge Region

The lateral reinforcement required inside the plastic hinge region shall meet the volumetric requirements specified in Section 3.8.1, the shear requirements specified in Section 3.6.3, and the spacing requirements in Section 8.2.5. The lateral reinforcement shall be either butt-welded hoops or continuous spiral.[3]

3.8.3 Lateral Column Reinforcement Outside the Plastic Hinge Region

The volume of lateral reinforcement required outside of the plastic hinge region, shall not be less than 50% of the amount specified in Section 3.8.2 and meet the shear requirements specified in Section 3.6.3.

3.8.4 Lateral Reinforcement of Pier Walls

The lateral confinement of pier walls shall be provided by cross ties. The total cross sectional tie area, A_{sh} required inside the plastic end regions of pier walls shall be the larger of the volume of steel required in Section 3.8.2 or BDS Sections 8.18.2.3.2 through 8.18.2.3.4. The lateral pier wall reinforcement outside the plastic hinge region shall satisfy BDS Section 8.18.2.3.

3.8.5 Lateral Reinforcement Requirements for Columns Supported on Type II Pile Shafts

The volumetric ratio of lateral reinforcement for columns supported on Type II pile shafts shall meet the requirements specified in Section 3.8.1 and 3.8.2. If the Type II pile shaft is enlarged, at least 50% of the confinement reinforcement required at the base of the column shall extend over the entire embedded length of the column cage. The required length of embedment for the column cage into the shaft is specified in Section 8.2.4.

3.8.6 Lateral Confinement for Type II Pile Shafts

The minimum volumetric ratio of lateral confinement in the enlarged Type II shaft shall be 50% of the volumetric ratio required at the base of the column and shall extend along the shaft cage to the point of termination of the column cage.

If this results in lateral confinement spacing which violates minimum spacing requirements in the pile shaft, the bar size and spacing shall be increased proportionally. Beyond the termination of the column cage, the volumetric ratio of the Type II pile shaft lateral confinement shall not be less than half that of the upper pile shaft.

Under certain exceptions a Type II shaft may be designed by adding longitudinal reinforcement to a prismatic column/shaft cage below ground. Under such conditions, the volumetric ratio of lateral confinement in the top segment $4D_{c,max}$ of the shaft shall be at least 75% of the confinement reinforcement required at the base of the column.

[3] The SDC development team has examined the longitudinal reinforcement buckling issue. The maximum spacing requirements in Section 8.2.5 should prevent the buckling of longitudinal reinforcement between adjacent layers of transverse reinforcement.

If this results in lateral confinement spacing which violates minimum spacing requirements in the pile shaft, the bar size and spacing shall be increased proportionally. The confinement of the remainder of the shaft cage shall not be less than half that of the upper pile shaft.