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Pre-Northridge Steel Moment Frame Retrofit Design Guide

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The purpose of this document is to provide guidance on the use of fluid viscous dampers as a retrofit solution for Pre-Northridge Steel Moment Frames (PN-SMF). Supplemental damping can be used to reduce story drifts to a level where the existing PN-SMF beam connections are protected from brittle failure. Although columns and panel zones are not traditionally the critical components of PN-SMF structures, they can also be protected from failure. In comparison with other retrofit solutions, one major advantage of using dampers is that foundation retrofits are minimal, and in many cases can be avoided.

Damping Configuration

A typical layout for distributing dampers within a floor plan is to have two dampers on each side of the building's center of stiffness, configured to resist torsion, totaling eight dampers per floor as shown in Figure 1.

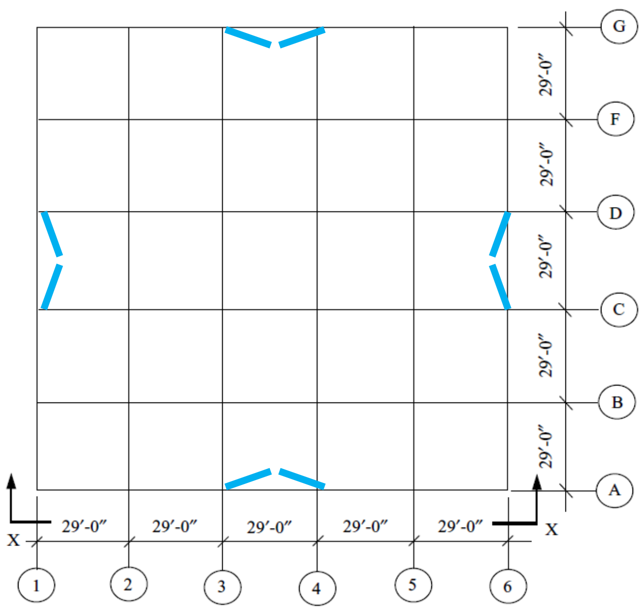


Figure 1: Damping Configuration - Horizontal Layout

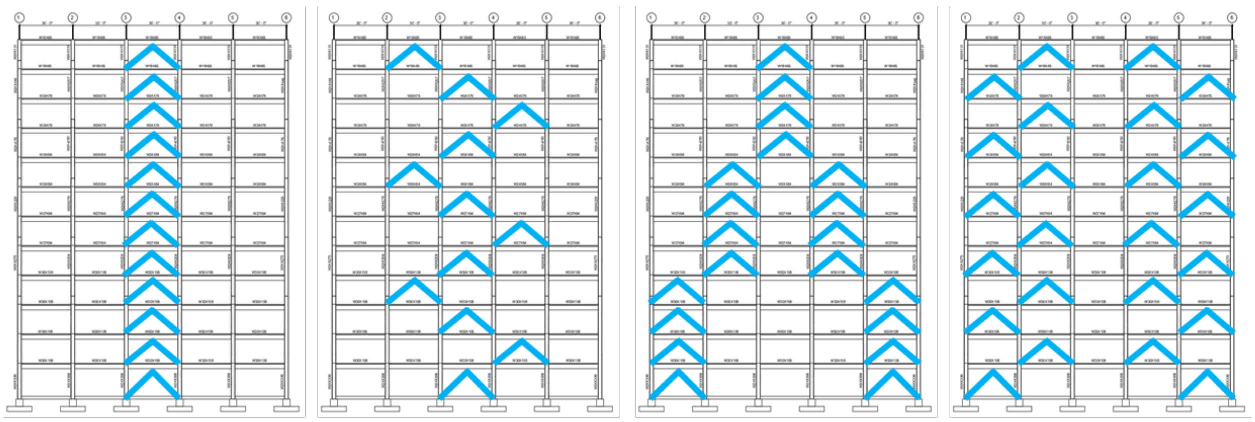
Following this arrangement avoids penalties on damper design force and stroke from ASCE 41-17 Section 15.2.2.4. It is important to configure viscous dampers to protect the floor diaphragm as well; more than eight dampers per floor level may be useful to reduce damper design forces and to protect the diaphragm from large demands induced by the horizontal component of the damper force.

For small to mid-rise building structures without vertical irregularities in floor stiffness, viscous dampers are typically placed on each floor level, excluding penthouse levels. Generally, as buildings get taller, above around fifteen stories, viscous dampers can be excluded from the top stories. Deciding when to exclude dampers from upper floors is beyond the scope of this design guide, but the reader is referred to Wang & Mahin's paper on the seismic retrofit of high-rise PN-SMF systems[1].

Within each bay of the damping system, the vertical component of the damper force is carried by the supporting columns, and to protect the columns and foundations from excessive demands, different vertical layout schemes can be used as shown in Figure 2. The "Stacked" configuration would place the

[1] Wang, Shanshan, and Stephen A. Mahin. "Seismic retrofit of a high-rise steel moment-resisting frame using fluid viscous dampers." *The Structural Design of Tall and Special Buildings* 26.10 (2017): e1367.

largest accumulated axial demand on the existing columns and foundations, whereas the other three schemes would distribute that load across multiple columns and foundations. While it may not be the most convenient solution, best practice is to distribute the dampers throughout the structure.



Stacked   Offset Stacked   Offset Tower   Checkerboard

Figure 2. Damping Configuration Variations- Vertical Layout

Preliminary Specification and Sizing of Viscous Dampers

In this design guide a simple method is outlined for determining the required viscous damping, β_v, in the structure's fundamental mode of vibration in each principal direction such that a desired structural performance can be achieved. This simplified procedure includes the following steps:

1. Determining target damping based upon chosen performance criteria.
2. Determining damper properties using a stiffness-proportional damping distribution.
3. Designing the damping system for load effects from the damper elements.

The Damping Coefficient, B_1, as defined in ASCE 41-17 Section 2.4.1.7 is a function of the structure's effective damping, β, and can be used to estimate the reduction in spectral response acceleration, which generally coincides with selected performance criteria, such as global displacement response, moment connections, or other component actions. This relationship between B_1 and the selected performance criteria will be used to determine the required viscous damping.

$$B_1 = 4/[5.6 - \ln(100\beta)] \qquad \text{(Eqn. 1)}$$

Where the effective damping ratio, β, is defined as:

$$\beta = \beta_i + \beta_v \qquad \text{(Eqn. 2)}$$

Where,

β_i is the inherent damping ratio, 0.025 or 2.5% in a structure with supplemental viscous damping
β_v is the viscous damping ratio

With a retrofit solution utilizing supplemental viscous damping, the post-retrofit action on any selected performance criteria can be estimated using the following equation:

$$Q_{ER} = Q_E/B_1 \qquad \text{(Eqn. 3)}$$

Where,

Q_{ER} is the performance criteria for the structure considering supplemental viscous damping
Q_E is the performance criteria for the structure without supplemental viscous damping

Substituting Equation 1 into Equation 3 and solving for the viscous damping ratio yields the following equation:

$$\beta_v = -\beta_i + \frac{1}{100} e^{\left[5.6 - 4\frac{Q_{ER}}{Q_E}\right]} \qquad \text{(Eqn.4)}$$

Note that β_v should be limited to 30% for this preliminary analysis in alignment with ASCE 41-17 § 15.9.2.4; future analysis using Nonlinear Response History Analysis may justify damping ratios above 30%.

Required Viscous Damping in the Fundamental Mode

Three different methods to estimate the required viscous damping are provided based upon performance criteria for 1) story drift, 2) moment connection capacity or 3) roof target displacement. The level of damping is set to modify the performance in one of these three areas from an analysis without dampers to a response that is below a desired threshold (e.g. limiting story drift below 1.5% or connection demand below capacity).

1. Story Drift Limit Method – Generally, a moment frame is designed such that the maximum story drift ratio under the Design Basis Earthquake (DBE) is less than or equal to 2%. This might equate to a 3% story drift ratio for the BSE-2X ground motion as is the case shown in Figure 3. In a general sense, most PN-SMF experience significant damage somewhere between 1.5% and 2.0% drift; this sets up a target drift for the retrofit scheme to protect key PN-SMF components. The design should target maximum story drifts of 1.5% to 2% in the damped structure under BSE-2X level demands. With reference to Equation 4, the target story drift for the post-retrofit structure $\Delta_{ER} = Q_{ER}$ and the Linear Dynamic Procedure (LDP) based on the site-specific response spectrum can be used to calculate elastic story drifts, $\Delta_E = Q_E$. Rewriting Equation 4 using the story drift limits gives:

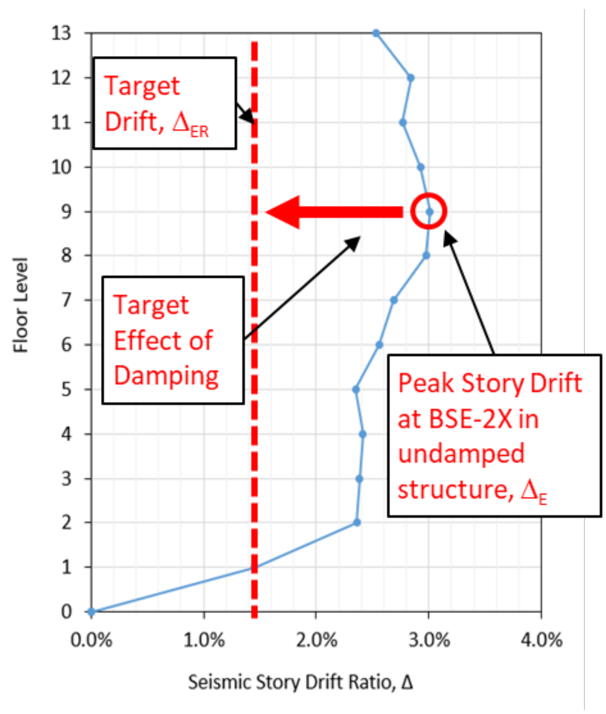


Figure 3. Story Drift Plot Before Damping and Target Drift

$$\beta_v = -\beta_i + \frac{1}{100} e^{\left[5.6 - 4\frac{\Delta_{ER}}{\Delta_E}\right]} \qquad \text{(Eqn.5)}$$

2. Moment Connection Capacity Method – The moment connection demand is frequently the limiting factor in the analysis of existing PN-SMF structures. Viscous damping can be tuned to reduce demands on the connections to acceptable levels based upon the highest connection Capacity-to-Demand Ratio (CDR). Using Equation 7-36 from ASCE 41-17, the CDR can be written as:

$$CDR = m\kappa M_{CE}/M_E = Q_{ER}/Q_E \qquad \text{(Eqn. 6)}$$

Where,

M_{CE} = expected connection strength
M_E = maximum connection moment from LDP of pre-retrofit structure
m = component capacity modification factor (ASCE 41-17 Table 9-6) with reductions per ASCE 41-17 §9.4.2.4.2 (4).
κ = knowledge factor (ASCE 41-17 §5.2.6)

Substituting into Equation 4 gives:

$$\beta_v = -\beta_i + \frac{1}{100} e^{\left[5.6-4\frac{m\kappa M_{CE}}{M_E}\right]} \qquad \text{(Eqn. 7)}$$

3. Target Displacement Reduction Method: When the Nonlinear Static Procedure is used to evaluate the PN-SMF, the required viscous damping can be determined to reduce the target displacement below the global displacement capacity of the structure in accordance with ASCE 41-17 Section 15.10.2. Substituting the desired roof displacement for the post-retrofit structure, δ_C, into Equation 4 for Q_{ER} and the target displacement determined in accordance with ASCE 41-17 Equation 7-18 of the pre-retrofit structure, δ_t, for Q_E gives the following:

$$\beta_v = -\beta_i + \frac{1}{100} e^{\left[5.6-4\frac{\delta_C}{\delta_t}\right]} \qquad \text{(Eqn. 8)}$$

Viscous Damping Specifications

Using the principles of the modal strain energy method, a stiffness-proportional damping distribution for the required viscous damping ratio, β_v, in the fundamental mode in each principal direction can be provided using the following equation:

$$C_{(L)ji} = \beta_v * \frac{k_i}{n_i} * \frac{T}{\pi} * \frac{1}{\cos^2\theta_{ji}} \qquad \text{(Eqn. 9)}$$

Where,

$C_{(L)ji}$ = Linear damping constant for the j^{th} damper on the i^{th} floor
k_i = i^{th} level floor story stiffness in the direction of interest
n_i = number of dampers on the i^{th} floor level in the direction of interest
T = fundamental period in the direction of interest[2]
θ_{ji} = angle of the j^{th} damper on the i^{th} floor level

[2] Note that is it important to select the modal period which is in the primary direction of consideration and has significant mass participation in that direction. Avoid using torsion dominant periods which sometimes may be the first or second modal period

While Equation 9 provides the formulation for determining the damping constant of a linear damper, it is standard practice to utilize dampers with nonlinear force-velocity relationships as shown in Figure 4. The damper output force for a nonlinear damper is given in Equation 10.

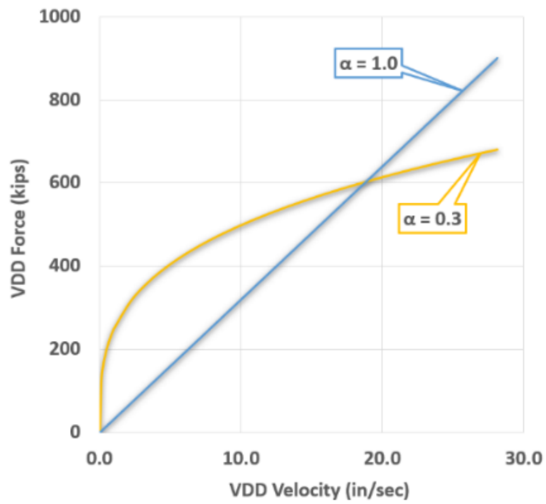


Figure 4. Linear and Nonlinear Damper Force-Velocity Relationship

$$F = CV^{\alpha} \qquad \text{(Eqn. 10)}$$

Where,

C = Damping constant
V = Velocity or deformation rate
α = Velocity exponent

Taylor can customize damper force-velocity relationships to exact specifications of C and α. In general, for seismic applications, α values between 0.3 and 0.5 are optimal for PN-SMF applications. If an alpha value of 0.4 is used (which is typical for most structural jobs), then the matrix given in Table 1 summarizes a selection of damping constants (C) and damper force capacities which are typical in seismic applications for structures. Note that the shaded cells are just highlighting typical values we would expect to see in retrofit applications for PN-SMFs based upon expected velocities of these types of building but are not a limitation on damper availability. Some projects may have higher or lower velocities which would produce results outside of the shaded region.

Table 1. Damper Properties Selection Table - Force Capacity and Damping Constant for α = 0.4

		Damping Constant, C (kip-(s/in)^{a})																												
		23	26	30	35	40	46	53	61	70	81	93	107	123	141	162	186	214	246	283	325	374	430	495	569	654	752	865	995	1144
Force (kips)	55																													
	110																													
	165																													
	220																													
	330																													
	440																													
	575																													
	750																													
	975																													
	1350																													

The energy dissipated by viscous dampers is velocity-dependent, and for velocity exponent values other than 1.0, a nonlinear damping constant, $C_{(N)}$, can be determined using the principle of equivalent energy dissipation. The corresponding damping constant for a nonlinear damper which has the equivalent energy dissipation as with the linear damping constant can be determined with Equation 9.

First, the peak inter-story velocities of the fundamental mode response, v_i, can be approximated using the Linear Dynamic Procedure calculated story drift, Δ_i.

$$v_i = \frac{2\pi}{T}\left(\frac{\Delta_i}{B_1}\right) \quad \text{(Eqn. 11)}$$

Then the required damping constant for each nonlinear damper, $C_{(N)ji}$, can be determined using the following equation:

$$C_{(N)ji} = C_{(L)ji} * \frac{\pi}{\lambda} * \left(v_i * \cos\theta_{ji}\right)^{(1-\alpha)} \quad \text{(Eqn. 12)}$$

Where,

$$\lambda = 4 * 2^{\alpha}\left(\frac{\Gamma^2\left(1+\frac{\alpha}{2}\right)}{\Gamma(2+\alpha)}\right) \quad \text{(Eqn. 13)}$$

The determination of lambda uses the gamma function; if α=0.4, then λ=3.582.

Damping System Design Considerations

The damping system is defined as both the viscous dampers and the structural components that transfer the viscous damper demands to the foundation and to the PN-SMFs. This includes the damper extender braces (where present), connections, beams, columns, diaphragms, and foundations. ASCE 41-17 Section 15.2.2.4 states that “the components and connections of the damping devices shall be designed to remain linearly elastic” for demands associated with 130% of the maximum velocity at BSE-2X with at least four dampers in each principal direction at each floor or 200% of the BSE-2X demands if less than four dampers are provided. Standard of practice is to consider the dampers, extender braces, connections and gusset plates as the elements to remain linearly elastic, not the entire damping system (i.e. beams, columns and diaphragms are allowed to yield in alignment with ASCE 41-17 limits).

Previously, it was demonstrated how peak inter-story floor velocities based on the fundamental mode response, v_i, could be approximated (Equation 11). This approximation, however, ignores contributions from higher modes which may be significant in taller buildings. To account for higher mode effects, an amplification factor, A_v, is used to modify the peak inter-story floor velocity approximation:

$$v_{i\text{-peak}} = A_v * v_j \quad \text{(Eqn. 14)}$$

Where,

$$A_v = 1 + 0.1*(n_s-1) \quad \text{(Eqn. 15)}$$

n_s = number of stories in the building

The maximum demand on the viscous damper under the considered hazard can then be determined as:

$$F_{ji} = C_{(N)ji} * (A_{ds} * v_{i\text{-peak}} * \cos\theta_{ji})^{\alpha} \quad \text{(Eqn. 16)}$$

Where,

A_{ds} = Damping System amplifier per ASCE 41-17 §15.2.2.4
= 1.3 if ≥ 4 dampers and at least 2 dampers on either side of the center of stinffness are provided at i^{th} floor in direction of interest
= 2.0 if < 4 dampers or fewer than 2 dampers on either side of the center of stiffness are provided at i^{th} floor in direction of interest.

Three different conditions must be considered when analyzing seismic demands on the damping system; demands associated with 1) the Maximum Displacement Stage, 2) the Maximum Velocity Stage, and 3) the Maximum Acceleration Stage.

1. Maximum Displacement Stage: This stage captures the condition where the building is at its maximum deflection and therefore maximum strain. The load effects from this stage would be those typically considered in a seismic analysis without additional forces caused by the dampers. The damping system demands associated with this stage are denoted as “E” in this design guide for load combination purposes.

2. Maximum Velocity Stage: This stage captures the condition where the dampers are experiencing their highest velocities and therefore the largest damper force output, captured by Equation 16 above. Component actions on beams, columns, panel zones, extender braces, connections and foundations in this stage can be estimated by considering the maximum viscous damper demands on the frame shown in Figure 5. The damping system demands associated with this stage are denoted as “E_{TD}” in this design guide for load combination purposes. The peak velocity occurs out-of-phase with the maximum displacement, therefore seismic demands caused by strain do not have to combined with demands caused by the dampers in this stage.

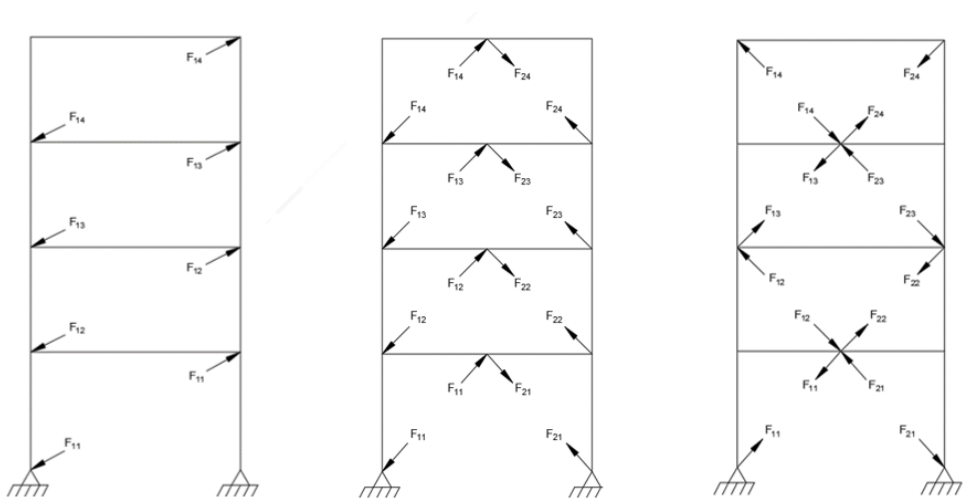


Figure 5. Damping System Demands from Viscous Damper Forces

3. Maximum Acceleration Stage: This stage captures the condition where floors reach their maximum acceleration. Demands on the damping system can be captured by a combination of the demands associated with the maximum displacement stage (E) and a portion of the demands associated with the maximum velocity stage (E_{TD}). This will be the most critical stage for damping system design. This load combination is dependent on the velocity exponent, α, selected for the viscous dampers.

   Generally, when a damper is selected with a velocity exponent, $\alpha = 0.4$, the damping system elements should be designed for the maximum seismic demands from $E+0.7E_{TD}$. When higher velocity exponents are selected for the dampers, the contribution of E_{TD} can be reduced.

Foundation System Considerations

The methods used to analyze existing foundations for seismic demands can vary significantly in accordance with ASCE 41-17. ASCE 41-17 Section 8.4 permits foundations to be modeled as either fixed-base foundations or as flexible-base foundations where foundation and soil stiffness are considered.

For preliminary foundation checks/sizing, the loads determined in accordance with the maximum acceleration stage would be sufficient in combination with the gravity loads per ASCE 41-17 Chapter 8. This approach would follow the ASCE 41 procedures and acceptance criteria associated with a Linear Dynamic Procedure.

A Brief Note on Modeling Critical PN-SMF Components

This section briefly highlights some key practices in modeling critical PN-SMF components to have reliable results for the Nonlinear Dynamic Procedure. Table 2 summarizes key considerations for different moment frame components, including PN-SMFs (Adopted from NIST GCR 17-917-46v2[3]). The choice to model PN-SMF elements with concentrated hinges for the nonlinear analysis should be driven by results from a Linear Dynamic Procedure in accordance with ASCE 41-17. Elements with m-factors greater than 1 (or DCR values greater than 1 when m-factors are ignored – i.e. elements experience yielding and nonlinear behavior) should be considered to have concentrated hinge elements in the nonlinear analysis. Note, however, that ASCE 41-17 permits elements which remain “essentially elastic” after damping has been added in the retrofit scheme to be modeled using linear elements (ASCE 41-17 §15.5.1) where “essential elastic” is typically taken as a DCR of 1.5.

[3] Applied Technology Council (2017) *Guidelines for Nonlinear Structural Analysis for Designs of Buildings: Part IIa – Steel Moment Frames*. National Institute of Standards and Technology Report GCR 17-917-46v2

Table 2. Nonlinear Behavioral Effects to Consider in Nonlinear Analysis (Table 2-1 from NIST GCR 17-917-46v2)

		Moment Frame Types				
Component	Nonlinear Behavior	*SMF*	*PN-SMF*	*IMF*	*OMF*	*NR-MF*
Beam	Yielding followed by gradual deterioration due to local buckling and/or lateral-torsional buckling and ductile tearing					
	SAME AS ABOVE – except with rapid deterioration					
	Yielding followed by rapid deterioration due to sudden connection fracture (ductile and/or brittle fracture)					
	No yielding or limited yielding followed by rapid deterioration due to local buckling and/or lateral-torsional buckling and ductile fracture					
	No yielding or limited yielding followed by rapid deterioration due to sudden connection fracture (ductile and/or brittle					
Column	Yielding followed by gradual deterioration due to local buckling and/or limited lateral-torsional buckling with possible ductile fracture (tearing)					
	SAME AS ABOVE – except with rapid deterioration					
	Yielding followed by rapid deterioration due to sudden column splice fracture (ductile and/or brittle fracture)					
	No yielding or limited yielding followed by rapid deterioration due to local buckling and/or lateral-torsional buckling and ductile fracture					
	No yielding or limited yielding followed by rapid deterioration due to sudden column splice fracture (ductile and/or brittle					
Panel Zone	Limited yielding and strain hardening after beams have begun to yield					
	Significant yielding prior to yielding in adjacent beams or columns					
Gravity Beam-Column Connection	Significant rotation capacity prior to gradual loss of flexural and/or shear capacity					
	Limited rotation capacity prior to sudden loss of flexural and/or shear capacity					
Column Base	Full fixity with limited yielding and deformation					
	Partial fixity with gradual yielding and deformation and significant rotation capacity					
	Partial or full fixity with sudden deterioration due to fracture or rupture in the plate					

Shading Legend
- *unlikely that phenomena will occur*
- *uncertain whether phenomena will occur*
- *very likely that phenomena will occur*

Beam Hinges:

Beam hinges should be provided using the guidelines of ASCE 41-17 and will follow the generalized backbone curve shown in Figure 6.

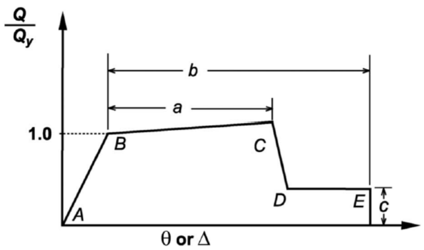


Figure 6. Generalized Force-Displacement/Rotation Backbone Curve for Steel Components (Adopted from ASCE 41-17)

ETABS has built-in functions to calculate the M3 hinge parameters that are easy to assign to steel moment frame beams (shown in Figure 7), although there is no option for defining hinges as having Pre-Northridge performance. Furthermore, in PN-SMF structures, it is often not the beam but the weld capacity of the beam-column connection which governs.

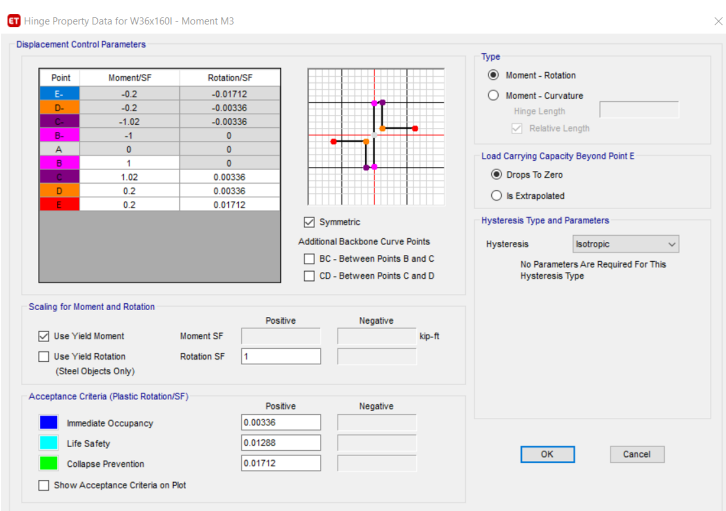


Figure 7. Example ETABS Hinge Element Input Parameters

In some cases, it might be useful to assign the automatic hinges, then export to the excel database, and modify moments and rotations accordingly before importing back in to ETABS. For PN-SMF, the NIST GCR 17-917-46v2 Appendix A document provides recommendations for calculating the median critical hinge rotation ($\theta_{p,fr}$) at fracture as:

$$\theta_{p,fr} = \alpha_f e^{(3.6+0.04d)} \qquad \text{(Eqn. 17)}$$

Where,

α_f = 1 for bottom beam flanges and 2 for top beam flanges

d = beam depth in inches

To check lower bound effects, the same document recommends reducing this plastic hinge capacity to 30% of the value calculated in Equation 17.

Panel Zones:

Including concentrated hinges for panel zones can be a critical piece of a nonlinear analysis of PN-SMFs. If panel zone hinges are excluded, but column hinges are included, the model may inaccurately overload and fail columns when in fact the panel zones would yield first, soften the structure and protect columns prior to failure. ETABS' built-in functions for panel zone hinges work well and are easy to apply to beam-column joint elements. The ETABS Performance Checks feature can be used to quickly look at panel zone hinge performance compared to capacities determined in alignment with ASCE 41-17. Also note that the limits used in calculating the capacity of panel zones was significantly altered between the ASCE 41-13 and ASCE 41-17 code cycles. Equation 9-19 of ASCE 41-17 limits panel zone rotation for PN-SMF systems to half of the rotation given in Table 9-7.2.

Column Hinges:

If the linear analysis showed columns to be at risk of failure or if strong column-weak beam criteria are not met, ETABS Parametric P-M2-M3 hinges column hinges can be used to capture nonlinear behavior of the columns. Note that, in general, PN-SMFs that utilize corner columns with biaxial demand tend to be overstressed and require retrofit. If possible, one should design the retrofit such that the columns are protected from failure and Parametric P-M2-M3 hinges need not be modeled, as these hinge elements are sensitive to input variables, may hinder analysis convergence, and present a non-ideal failure mechanism.