Non-Ductile Concrete Moment Frame Retrofit Design Guide

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The purpose of this document is to provide guidance on the use of viscous dampers as a retrofit solution for Non-Ductile Concrete Moment Frames (NDCMF). NDCMFs are structures typically built in the 1980s or before and are characterized by insufficient detailing to support ductile behavior of the concrete columns, beams and beam-column connections. Supplemental damping can be used to reduce story drifts to a level where the existing NDCMF columns are protected from sudden and catastrophic failure. In comparison with other retrofit solutions, one major advantage of using dampers is that foundation retrofits are typically minimal, and in many cases can be avoided all together.

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 mass, configured to resist torsion, totaling eight dampers per floor as shown in Figure 1.

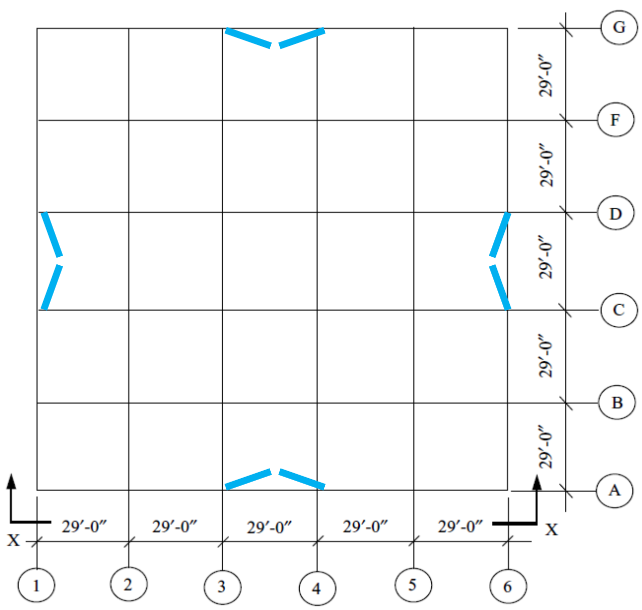


Figure 1: Damping Configuration - Horizontal Layout

Following this requirement avoids penalties on damper design force and stroke requirements 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 required to reduce damper design forces and protect the diaphragm from excessive demands.

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 12 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.

Within each bay of the damping system, the vertical component of the damper force is carried by the supporting columns, and, in order 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 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, rather than in a stacked configuration.

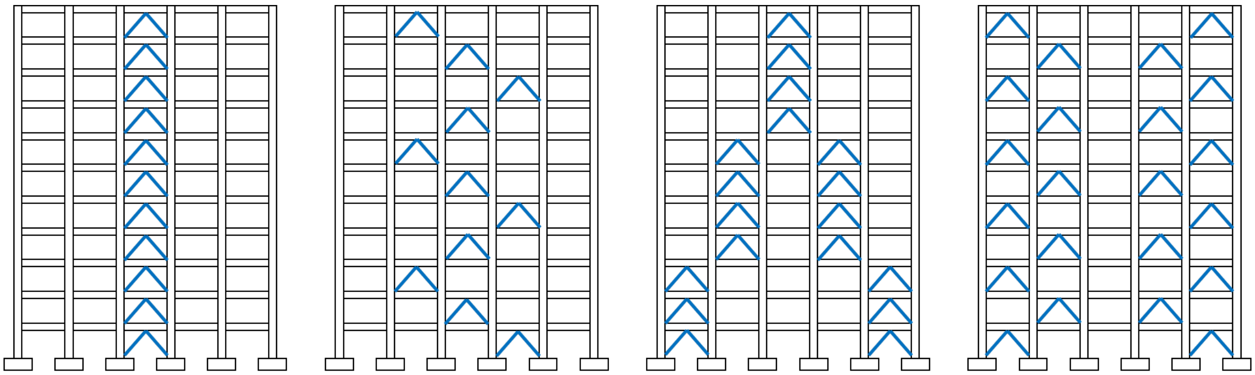


Figure 2. Damping Configuration Variations- Vertical Layout

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 <u>without</u> supplemental viscous damping

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

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

Note that β_v should be limited to 30% in alignment with ASCE 41-17 §15.9.2.4. It is mathematically possible that Equation 4 can produce recommendations for damping above this value, but it is important for the designer to limit the damping because of the relationship between the damped natural frequency and the undamped natural frequency shown in Figure 3 below. If dampers are sized to provide damping beyond the 30% limit, they begin to significantly impact the frequency of the structure. There also tends to be a functional limit on the improved performance from dampers alone and 30% is a good limit to hold at this preliminary stage..

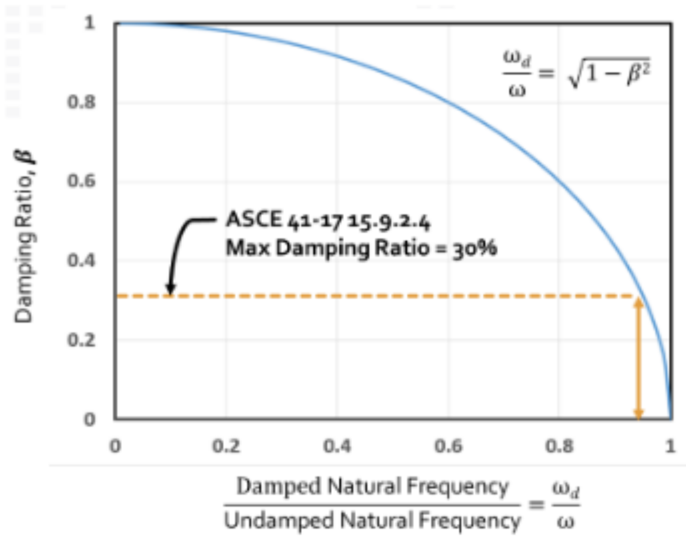


Figure 3. Damping impact on ratio between damped and undamped natural frequency

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) controlling column capacity or 3) roof target displacement. The level of damping is set to modify the performance in one of these areas (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 44. In a general sense, most NDCMFs experience significant damage somewhere between 1.0% and 1.5% drift[1]; this sets up a target drift for the retrofit scheme to protect key NDCMF components. 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:

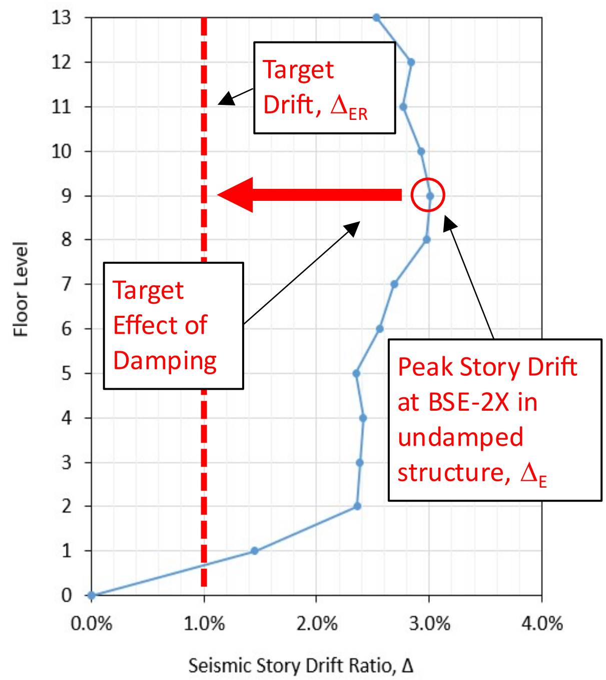


Figure 4. Story Drift Plot Before Damping and Target Drift

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

2. Moment Connection Capacity Method – When the moment connection demand is the limiting factor in the analysis of existing NDCMFs 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 for Deformation-Controlled actions or Equation 7-37 for Force-Controlled actions, the CDR can be written as:

$$\mathrm{CDR} = (m\kappa Q_{CE}/Q_{UD} \ \textbf{ OR } \ \kappa Q_{CL}/Q_{UF}) = Q_{ER}/Q_E \qquad \text{(Eqn. 6)}$$

Where,

Q_{CE} = expected component strength
Q_{UD} = deformation-controlled action from LDP of pre-retrofit structure
Q_{CL} = lower-bound component strength
Q_{UF} = force-controlled action caused by gravity and seismic forces

[1] SEAOSC & ICC (2016) *SEAOSC Design Guide – City of Los Angeles NDC Building Ordinance*

m = component capacity modification factor (ASCE 41-17 Chapter 10) with reductions per ASCE 41-17 §9.4.2.4.2 (4).
κ = knowledge factor (ASCE 41-17 §5.2.6)

Readers are directed to the SEAOSC Design Guide[2] for further discussion on the identification of deformation-controlled and force-controlled actions in NDCMFs.

Substituting into Equation 4 gives:

$$\beta_v = -\beta_i + \frac{1}{100} e^{[5.6-4(CDR)]} \qquad \text{(Eqn. 7)}$$

3. Target Displacement Reduction Method: When the Nonlinear Static Procedure is used to evaluate the NDCMF, 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[3]
θ_{ji} = angle of the j^{th} damper on the i^{th} floor level

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 55. The damper output force for a nonlinear damper is given in Equation 10.

[2] SEAOSC & ICC (2016) *SEAOSC Design Guide – City of Los Angeles NDC Building Ordinance*
[3] 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

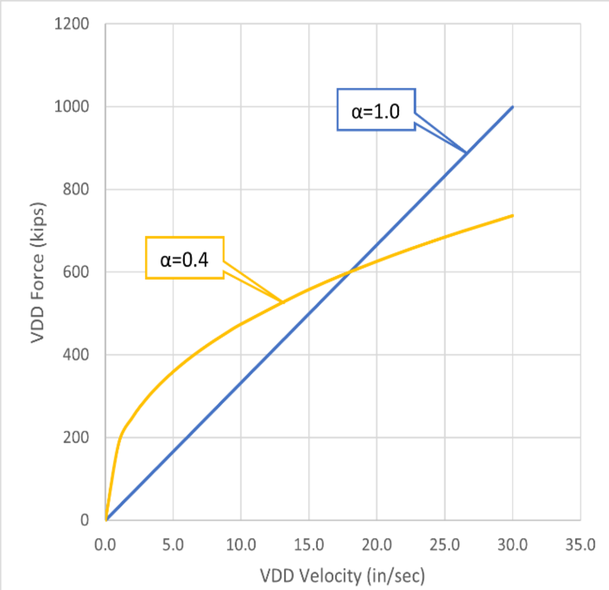


Figure 5. Linear and Nonlinear Damper Force-Velocity Relationship

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

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 NDCMF applications. If an alpha value of 0.4 is used (which is typical for most seismic applications), then the matrix given in Table 1 summarizes a selection of damping constants (C) and damper force capacities which also will work for most structural applications. Shaded regions in the table mark the range of typical velocities seen in structural projects and can help to correlate the expected range of C values associated with each damper rated force.

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

Force	Damping Constant, C																													
	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	1316
55																														
110																														
165																														
220																														
330																														
440																														
575																														
750																														
975																														
1350																														
1800																														

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.

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

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; Table 2 provides calculated values for common velocity exponents used in seismic applications.

Table 2. Values of Parameter λ for Typical Velocity Exponents

Velocity Exponent, α	λ
0.3	3.675
0.35	3.627
0.4	3.582
0.45	3.538
0.5	3.496
1	π = 3.14

Damping System Design Considerations

The damping system is defined as both the viscous dampers and the structural components that transfer the forces generated by the dampers to the foundation and to the NDCMFs. 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. “Components and connections” is typically taken to mean the dampers and all the components from one gusset to the other including bolts, welds, extenders and plates. The beams, columns, diaphragm and foundations do not have to remain elastic and can be evaluated in alignment with the deformation or force-controlled procedures of ASCE 41 with the amplified damper force added to the component demands.

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 and nonlinear effects 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,

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

n_s = number of stories in the building

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

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

Where,

A_{ds} = Damping System amplifier per ASCE 41-17 §15.2.24
= 1.3 if ≥ 4 dampers provided at i^{th} floor in direction of interest
= 2.0 if < 4 dampers 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. <u>Maximum Displacement Stage</u>: 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. <u>Maximum Velocity Stage</u>: 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 6. 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 be combined with demands caused by the dampers in this stage.

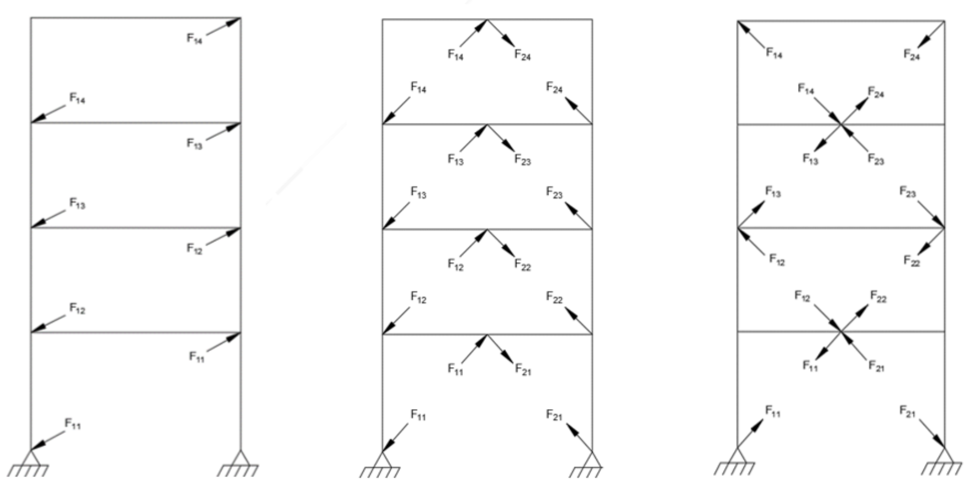


Figure 6. Damping System Demands from Viscous Damper Forces

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, α = 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 NDCMF Components

This section briefly highlights some key practices in modeling critical NDCMF components to have reliable results for the Nonlinear Dynamic Procedure. It is highly recommended that readers refer to the NIST GCR17-917-46v3[4] document's Appendix A for a more detailed discussion of modeling elements of nonductile concrete moment frames for nonlinear analysis. Some highlights from that document and personal experiences are included in this design guide.

Table 3 summarizes key considerations for different moment frame components, including NDCMFs (Adopted from NIST GCR17-917-46v3). The choice to model NDCMF 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 "essentially elastic" is generally taken to mean DCR values less than 1.5 when m-factors are ignored.

[4] Applied Technology Council (2017) *Guidelines for Nonlinear Structural Analysis for Design of Building: Part IIb – Reinforced Concrete Moment Frames* – NIST GCR 17-917-46v3

Table 3. Nonlinear Behavioral Effects to Consider in Nonlinear Analysis (Table 2-1 from NIST GCR17-917-46v3)

Table 2-1 Behavioral Effects to Consider in Nonlinear Analysis

		Structural System Types		
Component	*Nonlinear Response Phenomena*	*SMF*	*IMF*	*OMF / non seismic*
Beams of Moment Frames	Flexural yielding followed by **gradual lateral-strength** deterioration due to flexural failure modes, i.e., longitudinal bar buckling and/or concrete core deterioration			
	Flexural yielding followed by **rapid lateral-strength deterioration** due to flexural failure modes, i.e., longitudinal bar buckling/fracture and/or concrete core deterioration			
	Flexural yielding followed by **rapid lateral-strength deterioration** due to brittle failure modes, i.e., shear failure, anchorage failure, splice failure, and/or sudden loss of confinement caused by ties opening or fracturing			
	Rapid lateral-strength deterioration prior to flexural yielding due to shear failure, anchorage failure, and/or splice failure			
	Beam elongation due to yielding and resulting effects on restraining slab			
Columns of Moment Frames	Flexural yielding followed by **gradual lateral-strength deterioration** due to flexural failure modes, i.e., bar buckling and/or concrete core deterioration			
	Flexural yielding followed by **rapid lateral-strength deterioration** due to flexural failure modes, i.e., longitudinal bar buckling/fracture and/or concrete core deterioration; more likely with higher axial load			
	Flexural yielding followed by **rapid lateral-strength deterioration** due to brittle failure modes, i.e., shear failure, anchorage, splice failure, and/or sudden loss of confinement caused by tie opening/fracturing; more likely with higher axial load			
	Flexural yielding followed by **axial-strength deterioration** due to flexural failure modes, i.e., longitudinal bar buckling/fracture and concrete core deterioration; more likely with higher axial load			
	Flexural yielding followed by **axial-strength deterioration** due to brittle failure modes, i.e., shear failure, anchorage failure, splice failure, and/or sudden loss of confinement caused by tie opening/fracturing; more likely with higher axial load			
	Rapid lateral-strength deterioration prior to flexural yielding due to brittle failure modes, i.e., shear failure, anchorage failure, and/or splice failure			
	Axial-strength deterioration prior to flexural yielding due to brittle failure modes, i.e., shear failure, anchorage failure, and/or splice failure			
Beam-Column Joints	**Gradual loss of shear stiffness** after beam and/or column flexural yielding			
	Rapid loss of shear strength and stiffness after beam and column flexural yielding due to shear failure and/or anchorage failure			

Shading Legend	
(blank)	*unlikely that phenomena will occur*
(hatched)	*uncertain whether phenomena will occur*
(gray)	*very likely that phenomena will occur*

In modeling any concrete structure for linear elastic or linear dynamic procedures, it is critical to reduce the section properties to account for cracked concrete behavior. Table 10-5 from ASCE 41-17, reproduced below with notes as Table 4, makes suggestions for cracked section properties. As this will impact the building stiffness and period, this is critical to include from the beginning of the analysis of any concrete structure.

Table 4. ASCE 41-17 Table 10-5: Effective Stiffness Values

Component	Flexural Rigidity	Shear Rigidity	Axial Rigidity
Beams—nonprestressed[a]	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$	—
Beams—prestressed[a]	$E_{cE}I_g$	$0.4E_{cE}A_w$	—
Columns with compression caused by design gravity loads $\geq 0.5A_g f'_{cE}$ [b]	$0.7E_{cE}I_g$	$0.4E_{cE}A_w$	$E_{cE}A_g$
Columns with compression caused by design gravity loads $\leq 0.1A_g f'_{cE}$ or with tension[b]	$0.3E_{cE}I_g$	$0.4E_{cE}A_w$	$E_{cE}A_g$ (compression) $E_{sE}A_s$ (tension)
Beam–column joints	Refer to Section 10.4.2.2.1		$E_{cE}A_g$
Flat slabs—nonprestressed	Refer to Section 10.4.4.2	$0.4E_{cE}A_g$	—
Flat slabs—prestressed	Refer to Section 10.4.4.2	$0.4E_{cE}A_g$	—
Walls—cracked[c]	$0.35E_{cE}A_g$	$0.4E_{cE}A_w$	$E_{cE}A_g$ (compression) $E_{sE}A_s$ (tension)

[a] For T-beams, I_g can be taken as twice the value of I_g of the web alone. Otherwise, I_g should be based on the effective width as defined in Section 10.3.1.3.

[b] For columns with axial compression falling between the limits provided, flexural rigidity shall be determined by linear interpolation. If interpolation is not performed, the more conservative effective stiffnesses shall be used. An imposed axial load N_{UG} is permitted to be used for stiffness evaluations.

[c] See Section 10.7.2.2.

Guidelines for the backbone curve parameters can be found in both ASCE 41-17 and ACI 369.1-17. These standards address failure modes of shear failure, axial failure and anchorage or splice failure. The choice of modeling approaches (i.e. concentrated hinges vs. fiber-type vs. continuum finite elements) is beyond the scope of this design guide, but are critical decisions that impact the validity of the results and should be tied to each individual examination. The reader should consult Chapter 3 of the NIST GRC 17-917-46v3 document when considering different approaches.

Connecting Dampers to the Existing NDCMF Structure

There are two common methods for connecting dampers to existing NDCMFs, however, alternative connections are always available at the discretion and design of the structural engineer. One approach is to provide and steel frame within the existing concrete moment frame. In this approach, the dampers connect to the steel frame in a fairly straight forward manner and the steel frame can connect to the existing concrete MF through anchors spaced over the entire beam and column lengths. This approach is helpful to avoid congested reinforcement in the beam-column joints. The second approach uses localized connections in the beam-column joint where steel embed plates are attached to the concrete with epoxy anchors or through bolts.

Figure 7. Damper Connection: Through Bolt (8 Willis St, Wellington, NZ)

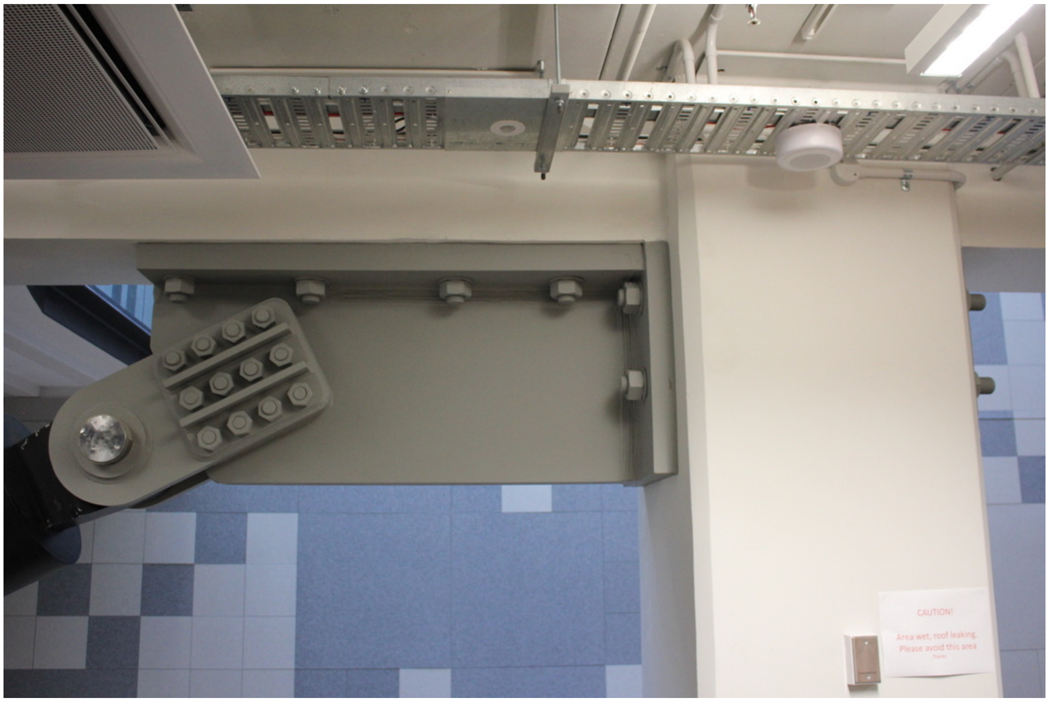
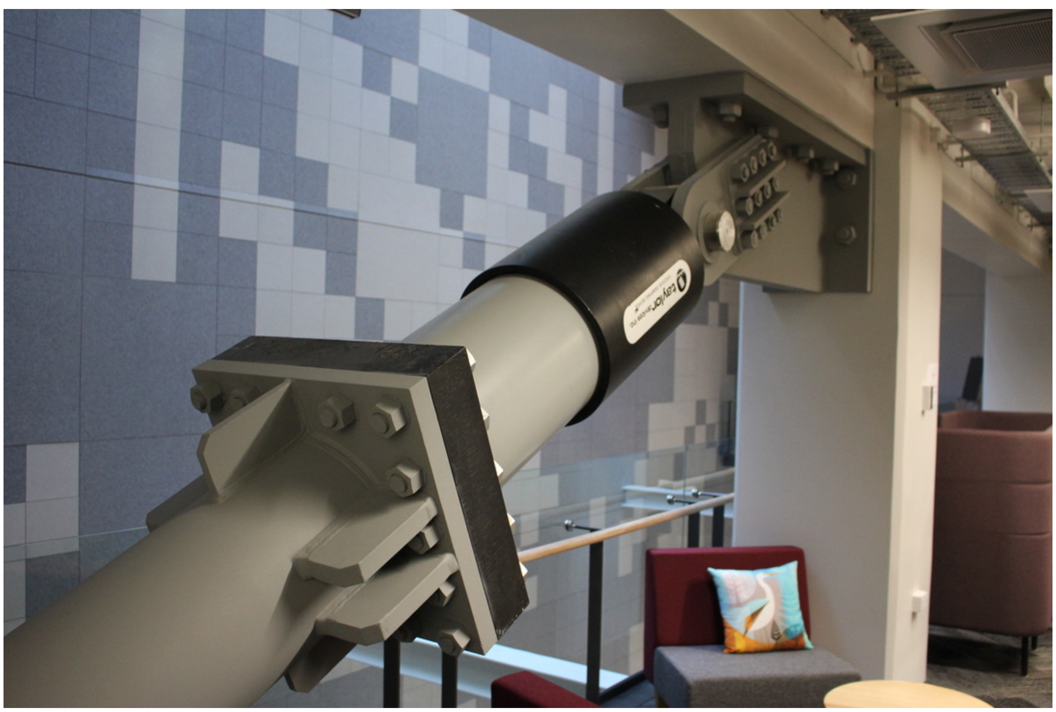


Figure 8. Damper Connection (8 Willis St, Wellington, NZ)



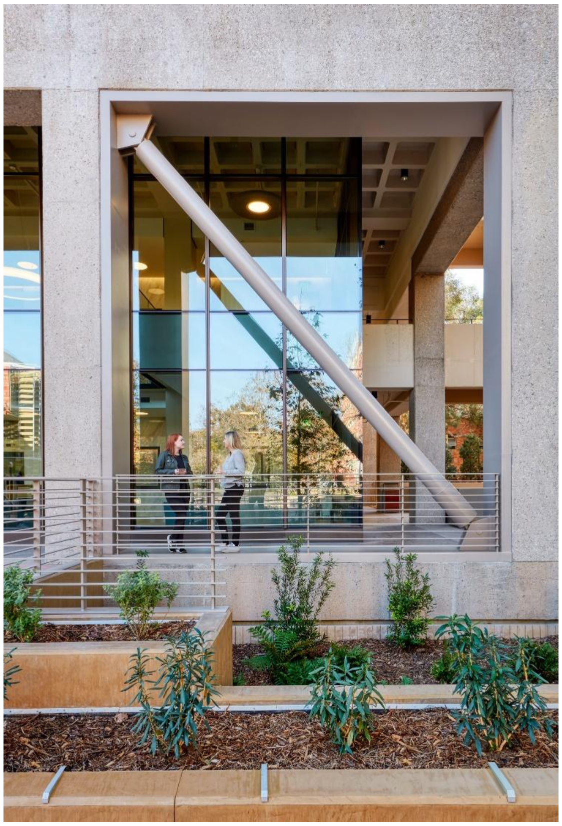


Figure 9. Installed Damper in Concrete Frame (UCLA Franz Hall)

Figure 10. Damper Connection: Through Bolt (UCLA Franz Hall)