Practical Design Procedure for Steel Moment Frames with Fluid Viscous Dampers

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Abstract

This paper outlines a practical design procedure for steel moment frames with fluid viscous dampers. The design procedure is being developed in accordance with International Code Council Evaluation Service (ICC-ES) AC 494, "Acceptance Criteria for Qualification of Building Seismic Performance of Alternative Seismic Force-Resisting Systems." The new design procedure decouples the design of the moment frames and the damping system to minimize model complexity and design iteration. Notably, the design of the moment frames follows typical moment frame design procedures found in AISC 341 and ASCE/SEI-7 chapter 12, but with reduced strength and drift requirements to account for reduction in the seismic response provided by the damping system.

Through state-of-the-art FEMA P-695 incremental dynamic analysis on a suite of nearly 100 archetype designs, the design procedure is shown to produce steel moment frame designs that meet the seismic collapse safety requirements of FEMA P-695 and ASCE/SEI-7, while also significantly reducing steel tonnage when comparing to traditional steel moment frames.

Introduction

Implementing fluid viscous dampers (FVDs) in structures is an effective way to dissipate seismic energy while avoiding significant damage to the structural system and reducing inter-story drifts and floor accelerations. The performance achieved with FVDs improves structural resiliency and can be used to achieve functional recovery goals in critical buildings.

FVDs are velocity dependent devices, governed by the force-velocity relationship:

$$F = CV^{\alpha}$$

where,

F = output force
C = damping constant
V = input velocity
α = velocity exponent.

Currently, engineers who wish to use FVDs in their design fall under ASCE/SEI-7 (ASCE, 2017) Chapter 18. While the code does allow linear methods to be used for designs with FVDs, the linear approaches in the code are complex and therefore nonlinear response history analysis (NLRHA) tends to be the most practical approach and most commonly used by engineers. Furthermore, all designs with FVDs require peer review. While the use of NLRHA and requirement for peer review provide confidence in the integrity of the design, they can also be barriers for many engineering firms. Both elements can increase cost and schedule, causing engineers to avoid using FVDs, especially for the first time. Additionally, the design of systems with FVDs can be confusing. There is little direction from the code on how to select key damper design parameters (such as C and α) and the process is iterative, where the design of the moment frame and damper frame are integral with each other.

To simplify the design process and decrease barriers to entry, a simplified procedure for the design of steel moment frame structures with supplemental damping has been developed. This procedure mirrors ASCE/SEI-7 Chapter 12 approaches, utilizing Modal Response Spectrum Analysis and decoupling the moment frame design from the damper design. Use of this procedure precludes the need for NLRHA and peer review, relying upon the rigorous FEMA P-695 (FEMA, 2009) procedures to support the adequacy of the system.

Further, the prescriptive approach gives direct guidance on the selection of the damper properties, C and α. Alpha (α) is fixed at 0.4, which provides optimized energy dissipation while minimizing output forces. Required damping constants, C, are calculated through the procedure and can be smoothed out, within bounds, to choose standardized values from Taylor Devices.

The design procedure is developed and validated according to AC494 (ICC-ES, 2018), which relies on the FEMA P-695 methodology (FEMA, 2009). An overview of the FEMA P-695 methodology is shown in Figure 1.

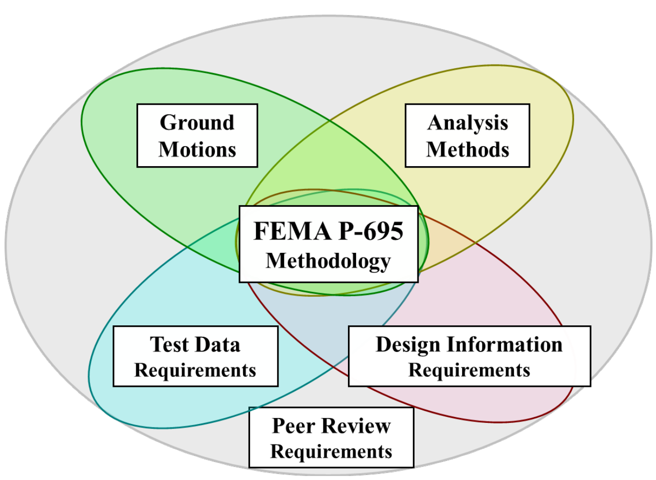

Figure 1. Overview of the FEMA P-695 methodology (FEMA, 2009)

The FEMA P-695 methodology was developed primarily for determining design factors (*e.g.*, the response modification coefficient, *R*) for new structural systems, so that their collapse resistance meets the safety objectives of ASCE/SEI 7. The FEMA P-695 method consists of numerous aspects of structural design considerations and nonlinear response history analysis (NLRHA), in order to validate the collapse resistance of a new structural system. Due to the complexities of numerical modeling and execution of NLRHA, as well as the intricacies of interpreting the quality of test data and adequacy of design criteria, the entire process is required to have independent peer review.

This paper covers the process used to develop this simplified design procedure, including an overview of the design procedure, a description of the archetype design space that was used to validate the design procedure, a description of the nonlinear modeling used in the FEMA P-695 analysis and a summary of key results. The full procedure and supporting documentation will be available through the ICC-ES website and Taylor Devices upon final approval of the system.

Overview of the Design Procedure

The design procedure for steel moment frames with fluid viscous dampers is developed for damping devices produced by Taylor Devices, with the combined lateral force-resisting system being termed a Taylor Damped Moment Frame (TDMF™).

Scope/Limitations

The design criteria are limited to buildings in which each of the following conditions are satisfied:

- Floor diaphragms are rigid as defined by ASCE/SEI 7 Section 12.3.1.2.
- Buildings do not have horizontal irregularity type 1b, extreme torsional irregularity, as defined in ASCE/SEI 7 Table 12.3-1.
- In each principal direction, the damping system has at least two damping devices in each story above the base, configured to resist torsion.
- Height limit of 300 ft.

Moment Frame Design

The design of the moment frame (MF) is carried out according to the linear analysis design procedures outlined by ASCE/SEI 7 (ASCE, 2017) Chapter 12 with slight modifications to account for the addition of supplemental viscous damping. The modifications are:

- Seismic forces and displacement responses shall be determined using the Modal Response Spectrum Analysis (MRSA) procedure of ASCE/SEI 7 §12.9.1 assuming 5%-damped spectral demands.

- The seismic base shear coefficient (C_s) (ASCE/SEI 7 Eqns. 12.8-2, 12.8-3, 12.8-4, 12.8-5 and 12.8-6) shall be reduced by 25% for *strength design*.
- The deflection amplification factor (C_d) for special steel moment frames is taken as 4.5 for calculating drifts.
- Minimum base shear coefficient for scaling drift response ($C_{s,d}$) uses a modified version of ASCE/SEI 7 Eqn. 12.8-6 that includes a site-specific term, but does not receive a 25% reduction as in strength design:

$$C_{s,d} = 0.35 S_{D1}/(R/I_e) \leq 0.5\, S_1/(R/I_e)$$

- The redundancy factor (ρ) is taken as unity (*i.e.*, 1.0).
- Maximum permissible stability coefficient (ASCE/SEI 7 §12.8.7) for checking P-Delta effects (θ_{max}) is 0.25.
- Accidental torsion considers the effect of damper forces using a conservative simplification that avoids iteration.

Considering the exceptions listed above, the design procedure of MF structural elements adheres to AISC requirements for Special Steel Moment Frames, namely AISC 360, AISC 341 and AISC 358 (AISC, 2016a; AISC, 2016b; AISC, 2018).

Initial comparative calculations indicate that the current design procedure can reduce required steel tonnage for the MF by 30% to 40% compared to traditional steel moment frames. The reduction is a direct result in the reduced response of the MF due to coupling with the dampers within the DF sub-system.

Damper Frame Design

The sizing of dampers and subsequent design of the damper frame (DF) is carried out based on a pre-determined set of supplemental damping design assumptions:

- The target supplemental damping ratio (β_v) is taken as 0.25 (*i.e.*, 25%) at the fundamental period T_1 in each principal direction for the design earthquake (DE).
- Supplemental damping is provided by nonlinear viscous dampers with a velocity exponent (α) of 0.4.
- Damping constants (C) are approximately proportioned to the elastic story stiffness (*i.e.*, $k_i = C_d(V_i/\Delta_i)$) of the *moment frame*, as determined from an equivalent lateral force (ELF) procedure analysis, where V_i and Δ_i are story shear and story drift, respectively. Determination of the damper constants is prescriptive and non-iterative (unless the MF design changes).

This set of underlying assumptions is intended to limit design decisions and avoid iteration. The damper frame design considers two stages: maximum velocity and maximum displacement. The maximum velocity stage is used for assigning damping constants and estimation of overstrength forces to be resisted by the DF structural elements (*e.g.*, beams and columns). The maximum displacement stage is used to obtain required damper strokes.

Maximum Velocity Stage

The maximum velocity stage analysis is prescriptive and can be easily programmed into a spreadsheet.

First, the linear damping constants ($C_{ji(L)}$) at each story i and damper j is calculated as a function of story stiffness (k_i, based on story shear and displacement from an equivalent lateral force procedure analysis of the MF), fundamental period (T_1), and the number (N_i) and angle of the damper with the horizontal at the story of interest (θ_{ji}). For dampers that are aligned with the principle direction (which is the most common case), the equation for the linear damping constant is:

$$C_{ji(L)} = 0.25\left(\frac{k_i}{N_i}\right)\left(\frac{T_1}{\pi}\right)\left(\frac{1}{\cos^2\theta_{ji}}\right)$$

In cases where some dampers may not be aligned with the principle axes, an alternative formulation for the linear damping constant is provided in the design procedure.

Two different relative story velocities (v_{ji} and v^*_{ji}) for the design earthquake are calculated using the story drift of the moment frames from MRSA, expressed in terms of local damper displacement (d_{ji}). These are converted to velocity by multiplying by the fundamental angular frequency (*i.e.*, $\omega_1 = 2\pi/T_1$) in the direction of interest. The first relative story velocity (v_{ji}), is used to calculate the nonlinear damping constants ($C_{ji(L)}$) required to provide 25% supplemental damping in the first mode. This is based on the design earthquake MRSA drift response without the consideration for minimum base shear for drift checks as shown in the equation below:

$$v_{ji} = \omega_1 d_{ji} \cdot \min\left[1.0, V_t/\left(C_{s,d}W\right)\right]$$

where V_t is the modal base shear from ASCE/SEI-7 §12.9.1.4.2 and W is the seismic weight of the building. Linear damping constants ($C_{ji(L)}$) are converted to equivalent nonlinear damping constants ($C_{ji(NL)}$) using an equal energy-based approach (Ramirez *et al.*, 2001) according to the following relationship:

$$C_{ji(NL)} = C_{ji(L)} \frac{\pi}{\lambda} \left(v_{ji}\right)^{(1-\alpha)}$$

where α is 0.4 and λ is 3.582. The specified nonlinear damping constants ($C_{ji(NL)spec}$) are allowed to vary between 0.9 and 1.3 times the calculated $C_{ji(NL)}$ values to allow smoothing of the damper properties specifications within the building.

The design relative story velocities used for calculating damper forces (v^*_{ji}) include the minimum base shear for checking drifts (if applicable) and also consider the amplification of velocities due to higher mode effects using the following relationship:

$$v^*_{ji} = A_v \omega_1 d_{ji}$$

The influence of higher modes is accounted for by a factor (A_v) which is a function of the number of stories in the building. Specified damper capacity (*i.e.*, nominal force) is calculated at the risk-targeted maximum considered earthquake (MCE_R) intensity. This is done by scaling the DE relative velocity (v_{ji}) by a factor of 1.5. The DE and MCE_R damper forces are denoted by f_{ji} and $f_{MCE,ji}$, respectively.

$$f_{ji} = C_{ji(NL)spec}\left(v^*_{ji}\right)^{\alpha}$$
$$f_{MCE,ji} = C_{ji(NL)spec}\left(1.5 v^*_{ji}\right)^{\alpha}$$

Overstrength damper forces (F_{ji}) are used to design connecting elements of the DF and shared elements of the MF (if applicable). The forces consider a factor R_v of 1.15 to account for variation in the damper force-velocity relationship due to environmental and manufacturing factors; this factor is used to amplify the damping constants. A velocity overstrength factor (Ω_v) of 2.5 is used to ensure that elements surrounding the dampers will remain elastic at forces beyond those expected at the MCE_R intensity to ensure that the dampers remain effective beyond MCE_R. The overstrength damper forces are calculated as:

$$F_{ji} = R_v C_{ji(NL)spec}\left(\Omega_v v^*_{ji}\right)^{\alpha}$$

The overstrength damper forces (F_{ji}) are used to design elements of the DF (*i.e.*, beams and columns) using simplified analysis similar to buckling restrained brace design. Figure 2 shows an example set of damper forces for a chevron DF. The damper-induced seismic load effect (Q_{TD}) on elements of the DF are used in combination with the maximum displacement stage demands from MRSA of the MF.

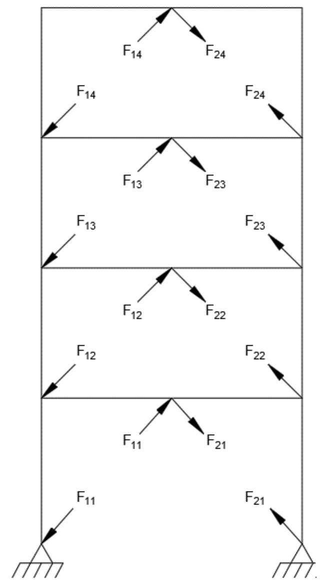

Figure 2. Damper overstrength forces used for simplified analysis of a one-bay chevron damper frame

Maximum Displacement Stage

The required damper stroke (s_{req}) is a function of building importance factor (I_e), the design earthquake damper displacement (d_{ji}) from MRSA of the MF and a damper stroke overstrength factor (Ω_d). The damper stroke overstrength factor Ω_d ranges from 2.5 to 3.5 as a function of the number of stories (n_s) and the Seismic Design Category (SDC).

For SDC D and lower:

$$\Omega_d = \begin{array}{cc} 3.5 & n_s \leq 4 \\ 4.0 - (0.125 n_s) \quad for & 5 < n_s < 12 \\ 2.5 & n_s \geq 12 \end{array}$$

For SDC E:

$$\Omega_d = \begin{array}{cc} 3.5 & n_s \leq 8 \\ 4.0 - (0.125 n_s) \quad for & 8 < n_s < 14 \\ 2.75 & n_s \geq 14 \end{array}$$

The resulting required damper strokes (s_{req}) for typical story levels are calculated according to the following relationship:

$$s_{req} = I_e \Omega_d d_{ji}$$

One additional requirement is that the damper stroke at the first story ($s_{req,1}$) must accommodate 85% of the design story drift in the second story above:

$$s_{req,1} = 0.85 I_e \Omega_d d_{j1} \left(\frac{\Delta_2 / h_{s,2}}{\Delta_1 / h_{s,1}} \right)$$

The additional stroke requirement considers that first-story drifts can be reduced using a linear design model with fixed-base columns; a reduction that diminishes with base column yielding in a large earthquake.

Archetype Design Space

A large suite of buildings (*i.e.*, archetype design space) have been designed according to the simplified design procedure. The current archetype design space consists of nearly 100 different designs. Collapse resistance of each archetype is determined according to the FEMA P-695 procedure (summarized in the next section) in order to test the effectiveness of the design procedure.

The archetype design space targets cases most common in practice while also capturing a broad range of buildings that may be designed with the simplified procedure, especially those that could potentially have problematic behavior. The moment frame designs for this project are carried out using a modified version of the "Automated Seismic Design and Analysis Platform" (*AutoSDA*) design module (Guan *et al.*, 2020) that adheres to the TDMF™ requirements.

Buildings range in number of stories from 2 to 20. Typical plan dimensions are 120 feet by 180 feet that are resisted by either two perimeter moment frames (2- or 4-bay) per direction or assume a space frame layout. Story heights are typically 16 feet for the first story and 14 feet for upper stories, and select cases investigate the influence of a taller first story (*e.g.*, 22 feet). Bay lengths are commonly 30 feet with additional archetypes that investigate the influence of 25- and 35-foot bay spacing.

Different system types based on the amount of coupling between the moment frame (MF) and the damper frame (DF) are considered. Type I is a fully decoupled system between MF and DF, as shown in Figure 3. Type II systems have fully shared frames and Type III systems have partially shared elements. The archetype space focuses mostly on Type I systems (fully decoupled MF and DF), yet considers different degrees of shared elements by analyzing Type II and III systems.

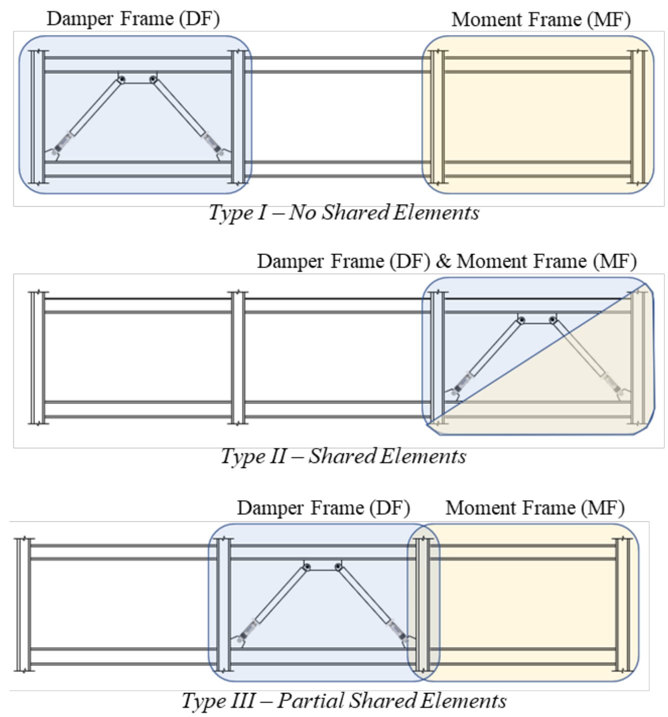

Figure 3. Illustration of different TDMF™ system types

The type of beam-to-column connections within the MF design include: reduced beam section (RBS) without doubler plates, RBS with doubler plates, welded unreinforced flange – welded web (WUF-W) without doubler plates, and SidePlate® (www.sideplate.com) connections. The design procedure is not limited to these connection details, yet this set of connection types was investigated to produce a range of strength hierarchies between MF beams, columns and panel zones. MF column section depth class (*e.g.*, W14, W27, etc.) is varied from W12 to W36 depending on the number of stories and site seismicity. MF base column connectivity is commonly treated as fixed, yet the influence of pinned base columns is investigated for shorter (e.g., 2- and 4-story) archetypes.

Damper frame (DF) layouts are commonly two single-bays per direction of the building. Damper configurations include chevron, diagonal and 2-story X layouts (see Figure 4). Additional damper frame layouts include changing the DF bay size (producing different inclination angles) and adding additional DF bays (more dampers per direction). Variations in specified nonlinear damping constants ($C_{ji(NL)spec}$) investigate the upper and lower bounds of the acceptable range from calculated values (*i.e.*, $0.9C_{ji(NL)}$ is the lower bound for $C_{ji(NL)spec}$), and select cases implement a more realistic smoothed set of damping constants that leverage the bounds on $C_{ji(NL)spec}$.

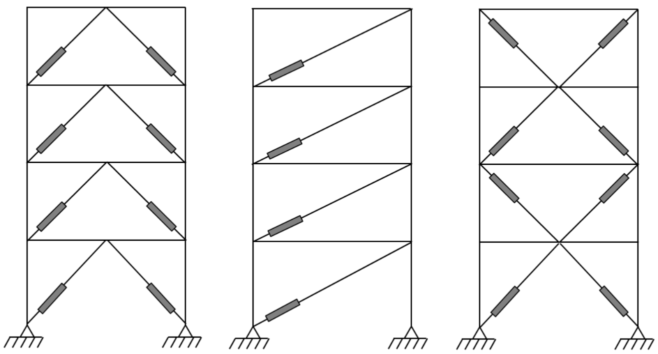

Figure 4. Different damper configurations considered in the archetype design space

The Seismic Design Category (SDC) for archetypes focuses mainly on SDC D_{max} as defined in FEMA P-695 (S_{DS}=1.0, S_{D1}=0.6). Additionally, SDC D_{min} (S_{DS}=0.5, S_{D1}=0.2) and a far-field SDC E (S_{DS}=1.5, S_{D1}=1.0) is considered, with the latter not a requirement of FEMA P-695, but included to be more in-line with increased seismic demands in recent versions of ASCE/SEI-7. Archetype designs are typically designed as Risk Category II structures, but a subset of designs adhering to Risk Category IV design requirements is also included.

Nonlinear Modeling for Time History Analysis

Nonlinear structural models of the each archetype design are required for performing the FEMA P-695 assessment. Note, nonlinear models are *not* required for using the design procedure. These are only necessary for qualifying the system with ICC-ES AC 494.

Nonlinear models were developed using the *OpenSees* analysis platform (McKenna *et al.*, 2010). Models are developed and analyzed assuming a representative 2D planar structure, with underlying moment frame (MF) designs neglecting the effects of biaxial loading and accidental torsion, to be consistent with the modeling procedure.

Modeling of the MF sub-system follows the ATC-114/NIST nonlinear modeling guidelines (NIST, 2017) and includes recent research published following the release of the ATC-114/NIST guidelines. The MF models include three sources of nonlinearity: beam-to-column connections, column hinging, and panel zones. Relationships used to estimate the nonlinear behavior are based on regression analysis of experimental testing (Lignos and Krawinkler, 2011; Lignos *et al.*, 2019; Skiadopoulos *et al.*, 2021). Beam-to-column connection and column hinges include hysteretic energy-based degradation using the *BiLin* material in *OpenSees* (Ibarra *et al.*, 2005).

Dampers are modeled using a Maxwell model approach, where the assembly stiffness of the damper (K_{TD}) is considered in series with the extender brace stiffness (K_E). An illustration of the Maxwell model is shown in Figure 5. The damper model uses a combination of a *truss* element with a *Viscous* material in series with an *elasticBeamColumn* element that provides the assembly stiffness (see K_A in Figure 5).

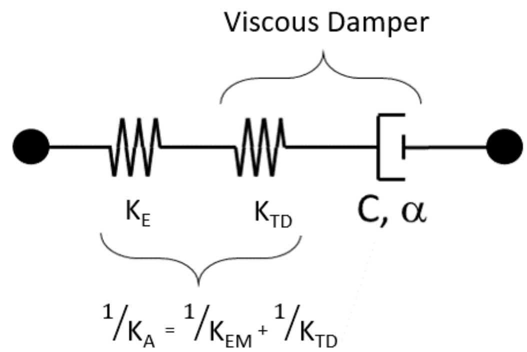

Figure 5. Maxwell model used for modeling nonlinear viscous dampers

The basic model configuration for a Type I system with a single-bay chevron damper frame (DF) configuration is shown in Figure 6. The moment frame (MF) is connected to the DF using rigid link *truss* elements. Similarly, a leaning P-Δ column is placed on the opposite side of the MF in order to capture gravity load effects from loads that are not tributary to the MF or DF. Typical modeling assumptions for Type I systems treats the DF column bases and beam connections as pinned, neglecting any strength or stiffness provided by these connections (see Figure 6). Inherent damping of the structure is modeled as 2% Rayleigh damping applied at the first and third mode periods (second mode for 2-story archetypes). Stiffness proportional Rayleigh damping is only applied to linear elastic elements, and the implemented formulation in *OpenSees* follows the recommendations of Zareian and Medina (2010) for lumped plasticity models.

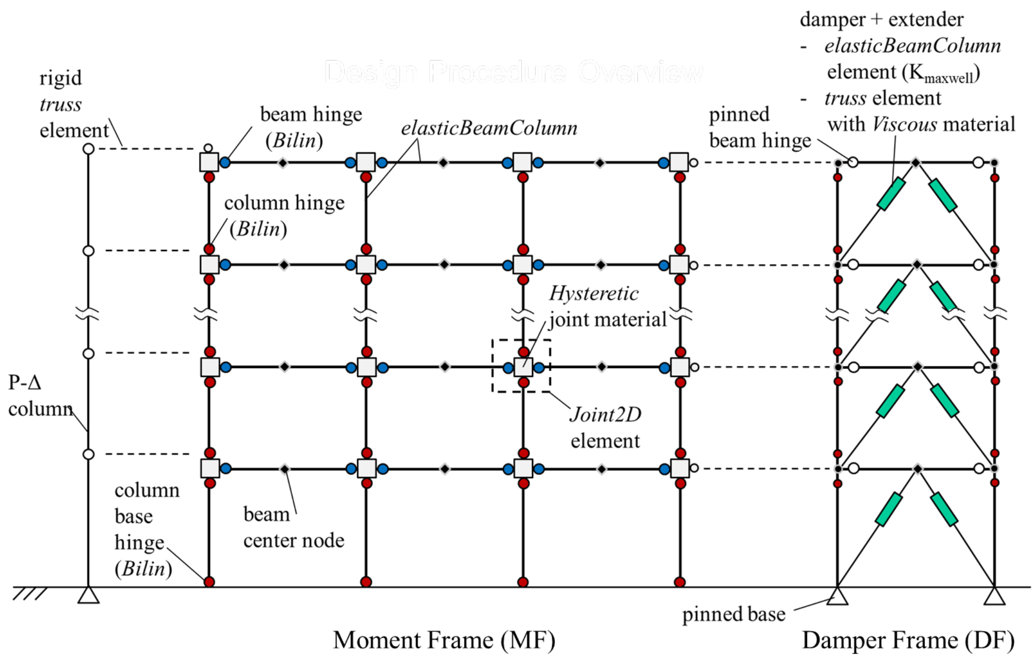

Figure 6. Nonlinear numerical model for a Type I TDMF™ system

Application of FEMA P-695 Analysis for Steel Moment Frames with Viscous Dampers

The implementation of the numerical analysis portion of the FEMA P-695 methodology (FEMA, 2009) consists of four basic steps for each archetype design/model:

- Run Incremental Dynamic Analysis (IDA) and record collapse statistics (*i.e.*, Sa(T) at collapse);
- Determine the median collapse intensity (S_{CT});
- Adjust the median for appropriate spectral shape and apply the total system uncertainty;
- Evaluate the adjusted probability of collapse at the MCE_R level and compare with acceptable collapse performance thresholds.

Incremental Dynamic Analysis (Vamvatsikos and Cornell, 2002) is conducted using a suite of 44 accelerograms (22 horizontal component pairs; see FEMA P-695 Appendix A). As per FEMA P-695, these ground motions are scaled using the spectral acceleration ($Sa(T)$) at the upper code-based period ($T=T_aC_u$) as an intensity measure until collapse of the system is reached. Figure 7a shows an example set of IDA curves for an 8-story archetype.

A key assumption for quantifying the collapse performance of the current archetype designs is that collapse is conservatively considered the intensity at which the required damper stroke (s_{req}) is exceeded at any location in the structure. This is treated as a non-simulated collapse mode (see FEMA P-695 Chapter 5) since actual damper response beyond the stroke capacity is not simulated. Typical IDA implementations consider collapse to be the intensity that causes a global sidesway mechanism, where the IDA curves become essentially flat (see Figure 7a). The use of stroke exceedance as a collapse mode causes additional conservatism to be incorporated into the design procedure, where many archetypes reach the ultimate damper stroke well before the displacement capacity of the moment frame is exhausted. Figure 7b illustrates the stroke exceedance story drift ratio (SDR) profile that is used for the collapse threshold.

For the example 8-story archetype in Figure 7b, the SDR response conditioned on stroke exceedance is shown to be controlled by the bottom story, although each archetype is not always controlled by the response of a single story. Following post-processing of the IDA results, the collapse intensity statistics allow for the estimation of the median collapse intensity (S_{CT}). This is compared with the MCE_R intensity (S_{MT}) in Figure 7b. The ratio of these two values (S_{CT}/S_{MT}) is the Collapse Margin Ratio (CMR) according to FEMA P-695, shown in Figure 8.

Since a generic set of ground motions (as opposed to site- and period-specific) is used for all archetypes, the median collapse intensity estimated from IDA (S_{CT}) must be adjusted for spectral shape to avoid unwarranted conservatism in the median collapse prediction (Baker and Cornell, 2006; Haselton *et al.* 2011). This is carried out within the FEMA P-695 methodology using a Spectral Shape Factor (SSF) that is estimated as a function of period-based ductility (μ_T) which is a proxy for period-elongation at the point of collapse. Period-based ductility is typically calculated using global effective yield and 80% post-peak strength roof displacements from pushover analysis (i.e., $\mu_T = d_{ult}/d_{y,eff}$). The current implementation for the archetype design space assumes that the ultimate roof displacement is that found from IDA at the point of stroke exceedance. This was found to give results that are similar or more conservative than using the default FEMA P-695 calculations from pushover analysis. The adjusted median collapse capacity ($S_{CT,adj}$) is simply S_{CT} multiplied by SSF. The influence of the spectral shape factor on the median collapse intensity is shown for an example 8-story archetype in Figure 8.

The final step includes the estimation of the total system uncertainty (β_{TOT}) that includes uncertainty due to record-to-record variability (β_{RTR}), design requirements (β_{DR}), test data (β_{TD}) and numerical modeling (β_{MDL}). The total system uncertainty is taken as 0.5 for all archetypes based on numerous factors, including: nature of the system, the design requirements and definition of collapse within the study. Figure 8 shows the final collapse fragility for an 8-story archetype defined by $S_{CT,adj}$ and β_{TOT}.

The adjusted collapse margin ratio (ACMR) is computed as $S_{CT,adj}/S_{MT}$ and then compared with the acceptable collapse margin ratios according to FEMA P-695, which is a function of the β_{TOT} for the system. For typical Risk Category II structures, a performance group of archetypes must have an average ACMR that gives 10% or less probability of collapse at the MCE_R intensity (*i.e.*, P[C|MCE] $\leq$ 10%). Single archetypes must have individual ACMRs that correspond to P[C|MCE] of less than or equal to 20%. For an example 8-story archetype, the ACMR is 2.32 and the P[C|MCE] is 4.6% which is within the FEMA P-695 requirements (see Figure 8).

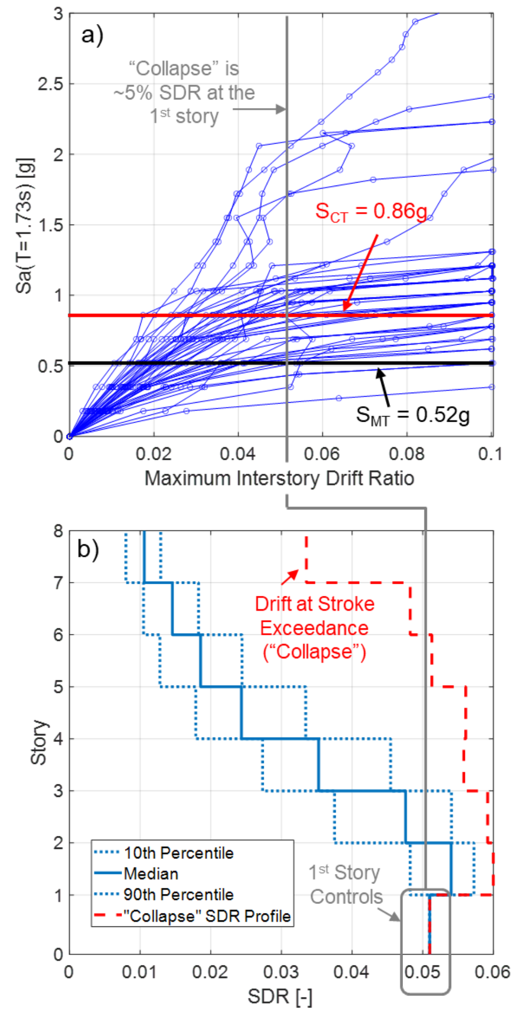

Figure 7. a) Example IDA curves for an 8-story archetype, b) Story drift ratio profile corresponding to damper stroke exceedance (dashed red) compared with median response conditioned on stroke exceedance (solid blue). The gray line between the figures highlights that stroke exceedance governs collapse rather than a full sidesway mechanism

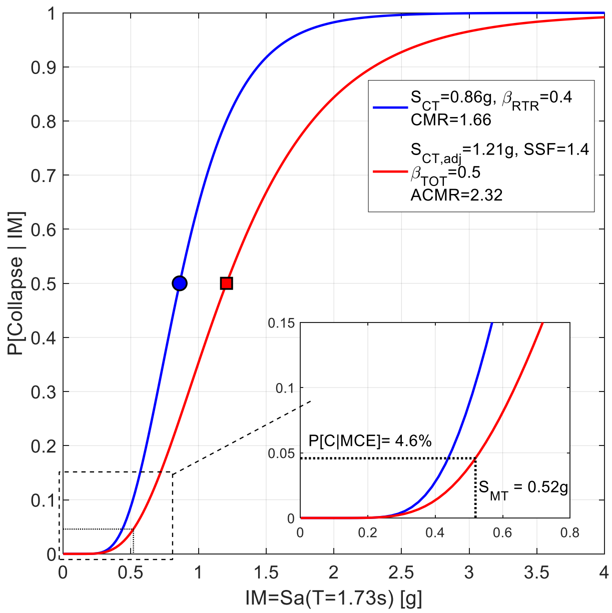

Figure 8. Example performance summary for an 8-story archetype

Performance Summary

At the writing of this paper, a majority of the ~100 archetypes have been designed and their collapse resistance assessed; the results to date show that the TDMF™ design procedure produces buildings that reliably meet the explicit life safety collapse resistance goal of ASCE/SEI 7 (*i.e.*, ≤10% probability of collapse given MCE_R shaking intensity).

The development of the design procedure for steel moment frames with fluid viscous dampers following the AC494 requirements and FEMA P-695 methodology entailed conducting over 40,000 nonlinear response history analyses. This large numerical effort helped fine-tune the design procedure while also expanding the archetype design space to explore its range of applicability. Some of the extended archetype design space studied in this project, such as SDC E and Risk Category IV buildings are not required by AC494 and FEMA P-695, but are included to make the study more complete.

Conclusion

A linear design procedure, which follows ASCE/SEI 7 Chapter 12 with minor modifications, is developed for steel moment frames with fluid viscous dampers, using ICC-ES AC494. The final ICC-ES report for this design procedure is expected near the end of 2022. The design procedure is prescriptive in nature and relatively simple to use. Notably, it removes two of the main barriers to designing steel moment frames with dampers:

1. Nonlinear response history analysis is no longer needed; and
2. Peer review is not required.

This project demonstrates the feasibility of codifying new lateral systems and/or design procedures through ICC-ES AC 494, which is a practical option for bringing innovation to practice in a relatively short time-frame (approximately two years in this case). Additionally, the new design procedure for steel moment frames with fluid viscous dampers has the potential to positively impact society by making resilient design more feasible, because supplemental damping is one of the few ways to reduce both displacements *and* floor accelerations, both of which are important for resilient design.

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