A prescriptive method for the design of new steel moment frame structures with supplemental damping

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ABSTRACT

This paper presents a prescriptive method for the design of new steel moment frame structures with supplemental damping using fluid viscous dampers (FVDs). The prescriptive method uses Modal Response Spectrum Analysis (MRSA) instead of Nonlinear Response History Analysis (NLRHA) as the basis for evaluation. Additionally, the procedure decouples the design of the moment frame from the damper frame, reducing design iterations. This makes the use of FVDs in new buildings easier to evaluate at schematic level design and significantly reduces the effort for full design. The procedure produces a 25% viscous damping ratio at the Design Earthquake (10% probability of exceedance in 50 years) and requires the use of nonlinear dampers with a damping exponent, alpha of 0.4.

The design procedure has been developed and validated through the AC494 and FEMA P-695 procedures and has been approved by the ICC Evaluation Services process. An overview of the procedure development and approval process will be provided, including a description of the evaluation archetype space, nonlinear modelling and collapse probability evaluation using incremental dynamic analysis. While the design procedure has been written in alignment with the United States' ASCE/SEI 7 provisions, the ICC-ESR is applicable in both New Zealand and Australia. While presenting the prescriptive method design procedure, this paper presents appropriate parallels between the ASCE/SEI 7 code approach and the New Zealand Standard NZS 1170.5.2004.

1 INTRODUCTION

The use of Fluid Viscous Dampers (FVDs) for energy dissipation is a very effective way to improve the performance and resiliency of new buildings or can be used in new construction to reduce steel tonnage and foundation costs. FVDs utilize the inter-story movement of a building to drive a machined piston head through a viscous fluid, generating heat which is dissipated into the atmosphere. The use of FVDs is increasing around the world for both new and retrofit applications, but, for some engineers, the requirements of nonlinear time history, peer review, and site-specific ground motions can be barriers to the use of FVDs. To help reduce or eliminate these barriers, a prescriptive method for the design of new steel moment frames with supplemental damping has been developed.

This design procedure, named the Taylor Damped Moment Frame™ (TDMF™), utilizes Modal Response Spectrum Analysis (MRSA) along with a decoupled design approach to simplify the design of new structures with FVDs. The result is a design procedure which can be used to quickly evaluate a damped moment frame solution for early schematic level comparison to other structural systems. Primarily in the United States, this is being used to reduce steel and foundation costs, though the same procedure could be used to produce structures which remain essentially elastic in design level earthquakes or achieve other seismic performance metrics.

FVDs work to dissipate energy by using inter-story drift to drive a machined piston head through a viscous fluid which generates heat. The fundamental equation describing the behaviour of a FVD is

$$F = CV^{\alpha} \tag{1}$$

where F = damper output force; C = damping coefficient; V = velocity; and α = damping exponent.

The TDMF™ design procedure guides the design engineer into the selection of the damping constants, C, corresponding to a nonlinear damper with a damping exponent, $\alpha = 0.4$ which achieves a 25% viscous damping ratio.

1.1 System Description

The primary seismic force resisting system under this design procedure is a steel moment frame (MF) which meets high ductility detailing requirements – a "Steel Special Moment Frame" in the ASCE/SEI 7 (ASCE 2017) classification system. Supplemental damping is provided by FVDs which, along with the complete load path of the dampers (e.g. extender braces, connections, beams, columns, diaphragm and foundation), make up the damper frame (DF) and is designed separately from the MF. Physically, the MF and DF may be separate or integrated, as shown in Figure 1.

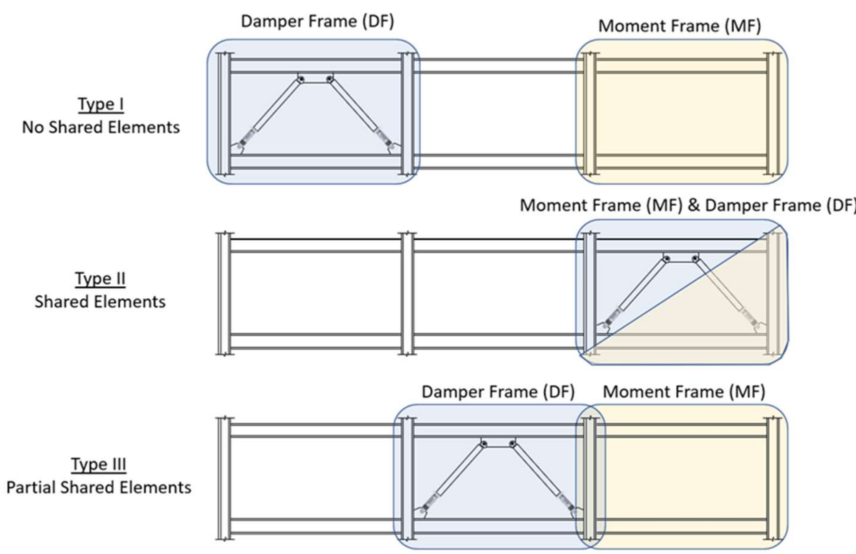

Figure 1: MF and DF relationship diagrams

The DF may also be laterally offset from the MF, as illustrated in Figure 2. In the case of Type II and Type III configurations, the force produced by the dampers must be integrated back onto the design of the MF elements which are shared between the MF and DF, requiring an iterative step in the MF design. Type I systems are fully decoupled in the design process.

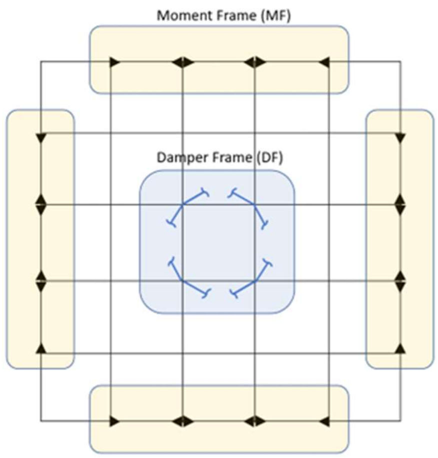

Figure 2: Illustration of MF and DF with lateral offset

Dampers in the DF may be configured in several different orientations, shown in Figure 3. In general, damper performance is optimized when the dampers are oriented horizontally, although these configuration (Figure 3 f to h) also tend to require more enhanced detailing for localized forces and to restrain out-of-plane movement of the frame. Different damper configurations may be interspersed throughout a structure.

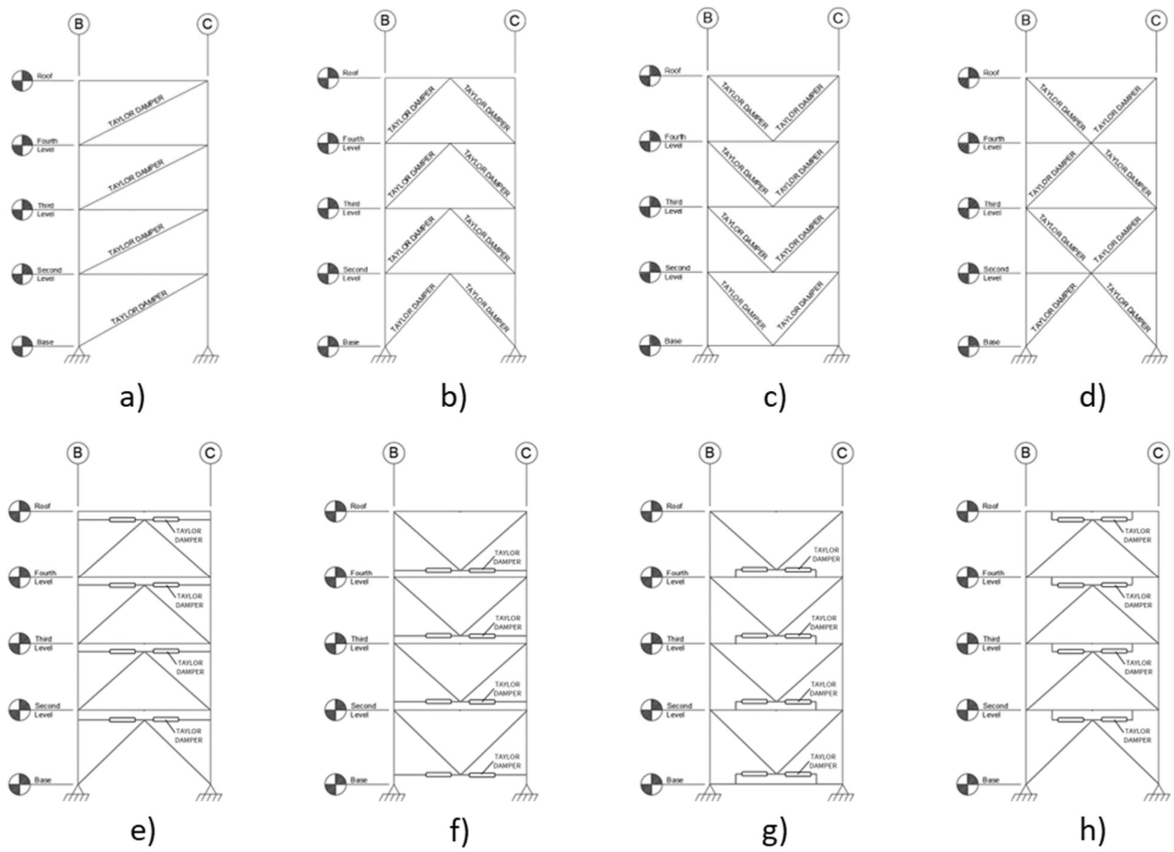

Figure 3: Approved damper configurations – a) diagonal, b) chevron, c) inverted chevron (V-type), d) 2-story X-type, e) column connected modified chevron, f) column connected modified V-type, g) beam connected modified V-type, h) beam connected modified chevron.

Use of this prescriptive design approach carries the following system requirements:

- Maximum building height of 91.4 m (300 ft.)
- Flexible floor diaphragms as defined by ASCE/SEI 7-16 Section 12.3.1.1 (Diaphragm Response NZS 1170.5.2004 Clause 6.1.4.1) are not permitted.

- Buildings must not have an Extreme Torsional Irregularity as defined by ASCE/SEI 7-16 Table 12.3-1 (Torsional Sensitivity Irregularity, NZS 1170.5.2004 Clause 4.5.2.3).
- At least two dampers must be provided on each floor above the seismic base in each principal direction, configured to resist torsion (i.e. one on either side of the centre of stiffness). Damper lines, however, do not have to stack vertically with the stories above or below, allowing for horizontal staggering of the dampers up the height of the building.
- Taylor Devices FVDs must be used.

Buildings which do not meet these requirements may still be designed using FVDs, but their design would require the use of nonlinear time history analysis in alignment with traditional design approaches rather than the prescriptive approach outlined herein.

2 PROCEDURE APPROVAL PROCESS

Validation of the developed design procedure was achieved through the International Code Council's Evaluation Services (ICC-ES) procedure, specifically using AC494: Acceptance Criteria for Qualification of Building Seismic Performance of Alternative Seismic Force-Resisting Systems (ICC-ES 2018). A key component of this acceptance criteria is to show that buildings designed following the proposed procedure meet the intention of the code with respect to collapse, namely:

- The adjusted collapse margin ratio (ACMR) for a class of structures designed using the procedure should not exceed 10% at the Risk-Targeted Maximum Considered Earthquake (MCE_R – 2% probability of exceedance in 50 years) level ground motion (AC494 Section 3.4.5.1).
- The ACMR for any individual structure designed using the procedure should not exceed 20% at the MCE_R level ground motion (AC494 Section 3.4.5.2).

The evaluation process was overseen by a review panel of topic experts and a representative from the ICC-ES and includes quality assurance provisions for the damper manufacturing process (AC10).

The FEMA P-695 (FEMA 2009) methodology was used to evaluate the collapse margin ratios for structures designed using the TDMF™ procedure. Briefly stated, a significant collection of archetype structures were designed in alignment with the proposed design procedure. Two-dimensional nonlinear models were created for each structure using the OpenSees analysis platform (McKenna et al., 2010) and included concentrated plasticity hinges for the beams, panel zones and columns. A leaning column was used to capture gravity loads and P-Delta effects and dampers were modelled as nonlinear elements with behaviour in alignment with Equation 1. Each structure was evaluated using Incremental Dynamic Analysis (Vamvatsikos and Cornell, 2002). "Collapse" was defined as the point at which the first dampers in the building exceeded their provided stroke limit – a conservative assumption as the building will still have capacity after this occurs, but holding this assumption avoids the need to model the post-stroke exhaustion characteristics of a damper. For more detailed information on the modelling and analysis of the structures for this study see (Welch et al., 2022). Collapse margin ratios were then calculated with results from a suite of 44 ground motions for each individual structure and averaged over all structures within a given performance group. The design procedure was then modified in an iterative process until the AC494 criteria were met.

2.1 Archetype Design Space

The archetype design space for this study consisted of 101 structures. The following design variations were considered:

- Number of stories: 2, 4, 6, 8, 12 & 20
- Story height and bay length

- Damping system (Type I, Type II & Type III) and damper configuration (e.g. diagonal)
- Moment frame connection details: WUF-W, RBS with and without doubler plates and SidePlate®
- Specified damping constant including upper and lower performance bounds (including temperature effects) of the design range
- Damper frame element detailing
- Perimeter and Space Frame moment frame layout
- Level of gravity loading and seismic mass resisted by the moment frames
- Seismic Design Category: D_{min} (S_{DS} = 0.50, S_{D1} = 0.20), D_{max} (S_{DS} = 1.00, S_{D1} = 0.60) and E (S_{DS} = 1.50, S_{D1} = 1.00)
- Building Risk Category II and IV (Corresponds to IL2 and IL4 structures in the NZS)

It is worth noting that Seismic Design Category E and Risk Category IV structures were added to the archetype design space, beyond the requirements of AC494. This was done because it is expected that this system will be utilized for many essential facilities (Risk Category IV) as well as an effective method to cope with poor soil conditions (Seismic Design Category E). Understanding the system performance under these conditions was important to the developers of the design procedure.

The archetype structures were organized into 12 performance groups with a minimum of three structures in each cluster. An example design group included five structures, all two-story, Type I with chevron damper configurations, Risk Category II and Seismic Design Category D_{max} with varying MF connections (WUF-W, RBS with and without doubler plates) and varying base condition (fixed and pinned variants with the RBS with doubler plate connection type). Each structure was designed by Englekirk Structural Engineers and the collapse analysis was conducted by Haselton Baker Risk Group, LLC.

3 DESIGN PROCEDURE OVERVIEW

The following sections outline the major elements of the design procedure. There are, however, nuances and alternative approaches which are not discussed and would be best learned from the ICC approved design procedure and accompanying commentary. For example, an asymmetric placement of the dampers about the centre of stiffness may be necessary due to architectural restraints and the output damper force could be balanced with varying C values so that the dampers don't contribute torsion to the floor; the process to do this is omitted from this summary.

3.1 Moment Frame Design

The process for designing the Special Steel Moment Frames mirror closely the linear dynamic procedures of ASCE/SEI 7. Moment frames must meet all the requirements of AISC 360, AISC 341 and AISC 358 (NZS 3404: Part 1: 1997)– seismic detailing requirements to ensure proper ductile behaviour and to avoid undesired yielding and failure. The moment frame demands shall be derived from Modal Response Spectrum Analysis (MRSA) following ASCE/SEI 7 Section 12.9.1 (NZS 1170.5: 2004 Section 6.3), assuming 5% damped spectra, but with the following adjustments:

- For *strength design*, the base shear coefficient (C_s) from ASCE/SEI 7 Equations 12.8-2 through 12.8-6 is reduced by 25%.
  - An equivalent approach for the NZS would be to reduce the scale factor, k, from NZS 1170.5:2004 Clause 5.2.2.2 by 25%.
- The deflection amplification factor (C_d) is decreased from 5.5 to 4.5. This factor is used to amplify the elastic deformation derived from the MRSA.

- An equivalent approach for the NZS would be to modify the ductility factor, μ, in NZS 1170.5:2004 Clause 7.2.1.1 by a factor of 4.5/5.5 = 0.818.

- The minimum base shear coefficient for scaling drift response ($C_{s,d}$ – ASCE/SEI 7 Section 12.8.6.1) does not receive the 25% reduction used for strength, but is instead modified to include the site-specific spectral acceleration (S_{D1}):

  $$C_{s,d} = 0.35S_{D1}/\left(\frac{R}{I_e}\right) \leq 0.5S_1/\left(\frac{R}{I_e}\right) \tag{2}$$

  where S_{D1} = site-specific design response acceleration parameter at a period of 1 second; R = Response Modification Coefficient; I_e = Seismic Importance Factor; and S_1 = mapped MCE_R spectral response acceleration parameter at a period of 1 second.

- The redundancy factor (ρ) is taken equal to one.

- The maximum permissible stability coefficient (ASCE/SEI 7 Section 12.8.7) for checking P-Delta effects is 0.25.
  - This corresponds to the stability factor in NZS 1170.5:2004 Clause 6.5.2(c).

Preliminary studies comparing standard moment frames with moment frames designed according to this procedure show a 30-40% reduction in the steel tonnage in the moment frame. This reduction is due to both the base shear reduction for strength and the lower displacement amplification factor, as well as producing moment frame buildings with longer periods than undamped moment frame structures. It should be noted that this preliminary work was focused on utilizing this procedure to reduce material costs and not with the objective of achieving improved building performance (although that does occur as well through reduced nonlinear behaviour in the moment frame connections).

The design of the seismic force resisting system in the ASCE code occurs at the Design Earthquake, which has a probability of exceedance of 10% in 50 year (approximately a 1/500 year event). This corresponds to the Ultimate Limit State (ULS) for an Importance Level (IL) 2 building in the NZS. The 1/2500 year return period for an IL4 building at ULS is similar also to a Risk Category IV designation in the ASCE code, where demands are amplified by a factor of 1.5 (DE*1.5 ≈ MCE ≈ 1/2500 year event). In this way, the modifications to the TDMF™ approach to moment frame design suggested above should hold for NZS linear dynamic procedures at ULS for IL 2 and IL4 structures as a preliminary design approach.

3.2 Damper Frame Design

As stated before, design of the damper frame (DF) is procedurally decoupled from the design of the MF. For the Type II or Type III configuration, where the DF and MF share common elements, an iterative process is required to introduce the damper force back into the MF. The following section outlines the selection of the damper properties, the damper force, required damper stroke and the design of the load path of the damper.

3.2.1 Damping Constant Determination

The damper properties are determined through a prescriptive method whereby the damping constant is proportional to the story stiffness. For simplicity, an outline of the process is provided here and readers are encouraged to see the full design procedure and associated commentary for specifics.

- The damping exponent (α) is fixed at 0.4.

- The target supplemental damping ratio (β_v) is set to 0.25 (i.e. 25%) at the fundamental period, T_1. A fixed amount of target damping must be used for a prescriptive approach, though future iterations may provide alternative procedures to achieve other degrees of damping.
- Calculate a linear damping constant ($C_{(L)}$) based upon the stiffness at each story, the damping ratio, the fundamental period and the angle of the damper with respect to the principal direction (horizontally and laterally).
- Calculate a pseudo-velocity based upon the angular frequency and damper displacement determined from the MRSA results.
- Convert the linear damping constant to a nonlinear damping constant (with $\alpha = 0.4$) using approximately equivalent energy dissipation at the pseudo-velocity previously calculated.

The specified nonlinear damping constant for each damper at each floor must be within -10% and +30% of the calculated value, allowing for "smoothing" of the damper properties over multiple stories. The design procedure allows for many configurations of the dampers and gives flexibility to the designer in determining the damping constant. When dampers are all aligned with the principal axes and geometrically balanced around the centre of stiffness, the equations provided simplify down to give dampers with the same properties on a given floor in a given direction. An alternative formulation for the damping constant equation is also provided for conditions where the dampers are not all aligned with the principal directions or there are advantages to using different damping constants on the same floor, perhaps to avoid damper induced torsion if dampers are asymmetric to the centre of stiffness.

3.2.2 Damper Force Determination

The design procedure uses the fundamental damper equation (Equation 1) along with the specified damping constant to determine the damper force.

- The pseudo-velocity used to convert the linear damping constant to a nonlinear damping constant is modified to account for higher modes and nonlinear behaviour in the structure. This modification was derived from the thousands of nonlinear time histories conducted as part of the validation process for this system.
- A damper force at the DE/ULS level (f_{ji}) is first computed with Equation 1.
- A damper force at the MCE level ($f_{MCE,ji}$) is determined by multiplying the modified velocity by 1.5. Note that because the modification is on velocity, the 1.5 factor is raised to the power of 0.4 (i.e. $f_{MCE,ji} = (1.5)^{0.4} * f_{ji} = 1.18 * f_{ji}$).
- $f_{MCE,ji}$ corresponds to the rated value of dampers and is used to specify the dampers to the damper manufacturer. A minimum factor of safety on force of 1.6 is provided based upon the rated force and provides additional reserve capacity and redundancy.
- Finally, an overstrength damper force (F_{ji}) is calculated which includes the upper bound amplifier ($R_v = 1.15$) and an overstrength factor of 2.5 on velocity, which results in a 66% increase from the DE/ULS damper force (i.e. $F_{ji} = 1.66 f_{ji}$).
- The overstrength damper force (F_{ji}) is used in the design of the load path of the damper, including connections, extender braces, beams, columns and diaphragms.

3.2.3 Required Damper Stroke

As mentioned earlier, the definition of "collapse" used to guide the validation of this system was taken as the point at which the first dampers reached their provided stroke limit. Because of this definition, an amplification on the damper stroke was developed to reduce the probability of collapse. The minimum stroke required is calculated as

$$s_{req,ji} = A_1 I_e \Omega_d d_{ji} \tag{3}$$

where A_1 = modifier for the first story only based on the inter-story drift ratio; I_e = seismic importance factor; Ω_d = stroke amplifier based upon Seismic Design Category and number of stories; and d_{ji} = damper displacement determined from MRSA results.

A minimum stroke corresponding to the damper displacement for a 3% inter-story drift ratio also applies as a lower limit for supplied damper stroke. The calculated stroke is typically rounded up to the nearest 25mm for production.

3.2.4 Damper Frame Considerations

The overstrength damper force (F_{ji}) is used in the design of the components of the damper frame, combined with any gravity or seismic loading depending on the element. For components with both seismic and damper induced loads, a load combination of 70% on the overstrength damper induced force can be used in combination with the seismic force. For foundation design, only the DE/ULS level damper force needs to be used, not the overstrength force.

The design procedure provides a minimum stiffness for the extender braces which connect the dampers to the frame. This minimum stiffness ensures that the dampers perform as expected and that energy losses due to the elastic deformation of the brace are minimized.

Finally, the design procedure provides an optional reduction in the damper induced axial demand in columns which continuously support damper frames above. This reduction begins in columns with four or more continuous damper frames above and, in taller buildings, can be as high as a 50% reduction in the accumulated damper force.

4 CONCLUSIONS

The information presented here provides an overview of a prescriptive method for the design of new steel moment frame structures with supplemental damping. This procedure has been developed and validated through the rigorous AC494 and FEMA P-695 procedures, providing evidence that buildings designed in alignment with this procedure meet the intended safety margins of the current code. This approach allows for the rapid design of buildings with dampers to help engineers quickly make decisions about appropriate building systems at a schematic level. Additionally, this procedure can be used to quickly design new buildings which meet enhanced performance objectives through the use of dampers. Buildings designed with this procedure will incur less structural yielding than a code-compliant moment frame only counterpart, decreasing damage and downtime for the building following major earthquakes. For smaller earthquakes, it is expected that the moment frames will remain largely elastic, relying solely on the dampers to dissipate the seismic energy.

This paper has provided suggestions for the translation of the design procedure to align with the NZS code. Simple modifications to the MRSA base shear modification factor and the structure ductility factor can be made to develop equivalent moment frame designs which account for the impact of the dampers without having to explicitly model them. It is our hope that these simple adjustments will provide an important tool for New Zealand structural engineers to make damper design of new buildings more accessible and widespread. It is acknowledged that this method would be considered an alternative design method in New Zealand and would therefore be subject to peer review, at a minimum, and possibly still require NLTHA for the damper design. It is the intention of the authors to present his method, for the very least, as a preliminary design tool for engineers who wish to use dampers in new construction applications.

5 ACKNOWLEDGEMENTS

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