

Violating Capacity Design in Structures with Viscous Damping and Hysteretic Damping

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ABSTRACT: Capacity design, while protecting a structure against undesirable energy dissipations, has major implications on member sizes and overall cost. Furthermore, in some situations where protected elements possess some inelastic deformation capacity, it may be unwarranted. One of these situations is when the forces applied to the protected elements result from viscous dampers. This is because when viscous forces cause yielding in an element, the element deforms, so no deformation in the viscous damper is required. If no deformation is required, the velocity is zero, so there is no force. This implies that very little inelastic yielding is likely to occur in protected elements.

In order to investigate whether or not this is so, a single storey structure was designed and fitted with braces to reduce its response. Both hysteretic and viscous braces were used to obtain the same peak displacement response. The column strength was decreased by a fixed percentage and inelastic dynamic time history analysis was conducted. The amount of energy dissipated in the columns was then compared to determine whether hysteretic braces or viscous braces caused more column yielding so that appropriate over strength values could be developed for different braces types. It was found that the amount of energy absorbed by the column depends on the period but also on the brace design ductility. However, irrespective of the period or design ductility, the column hysteretic energy dissipated by a viscous brace was lower than that dissipated by a hysteretic brace. It follows that column yielding may be significantly less critical for viscous, rather than for hysteresis, braces structures.

1 INTRODUCTION

Capacity design is an integral part of structural design techniques and well accepted within the structural design community. Under the strong-column, weak-beam design methodology it is assumed that the columns will remain elastic during an earthquake response cycle, and that any inelastic response will be concentrated in the formation of a plastic hinge zone in the beam. In the case of structures with diagonal braces, with either viscous or hysteretic dampers, the damping forces impose an additional axial load in the columns which must be considered in design. If the additional axial loads in the columns due to the damping forces is neglected, axial column yielding may result. However, this violation of capacity design may actually lead to a desirable outcome and is the primary focus of this investigation.

In structural design, a braced moment resisting frame can be considered as a combination of a braced pinned frame (with no lateral stiffness other than that provided by the brace) and an unbraced moment resisting frame, as shown schematically in Figure 1. The brace may also contain a viscous damper or include some form of hysteretic energy absorption, possibly through friction of yielding steel.

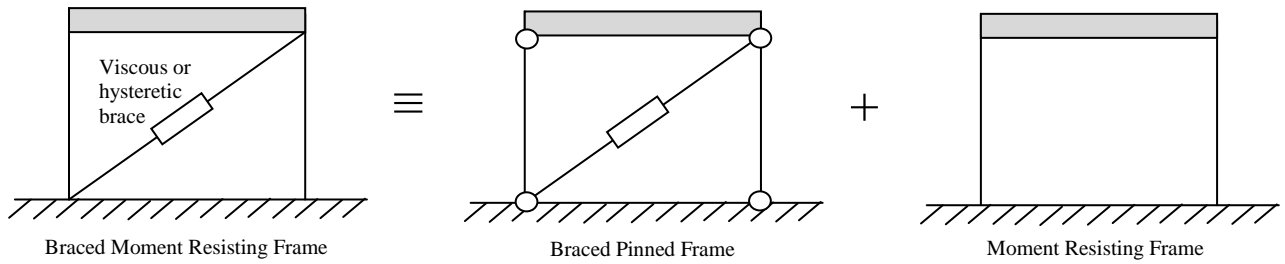


Figure 1: Schematic diagram of braced frame analysis approach.

The brace itself will likely have a linear force-displacement response, or if includes some form of yielding steel, a bilinear response. Figure 2 presents the response of a brace with a viscous damper. The elliptical damper response assumes a sinusoidal displacement input. If the brace and frame are well matched, then the peak force in the overall response will be roughly equal to that of the structure itself, as seen in Figure 2. However, if the damper is mismatched to the frame, or if we get much bigger velocities, then a much less well-balanced response can result. Figure 3 shows a schematic representation of such a mismatched response.

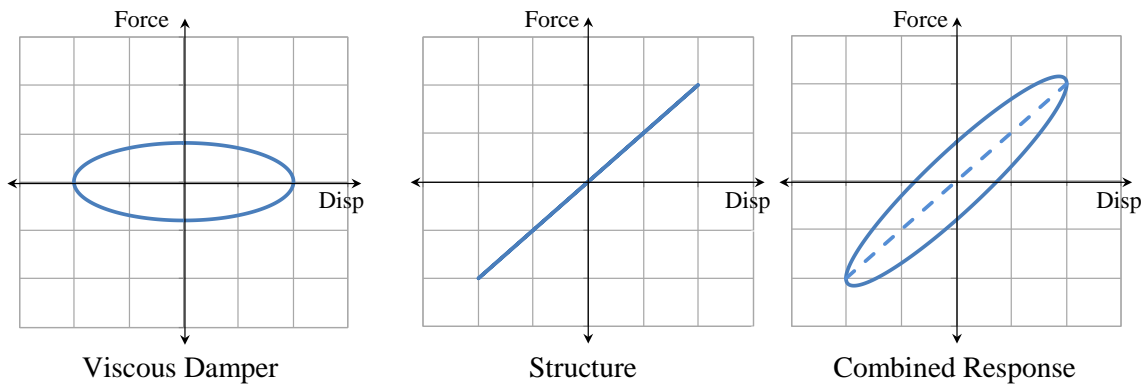


Figure 2: Schematic diagram of brace displacement response with well-matched damper and frame.

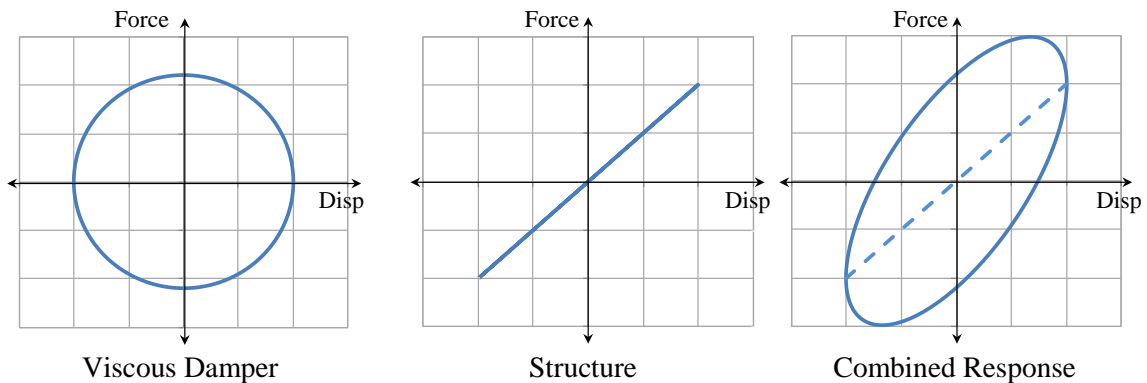


Figure 3: Schematic diagram of brace displacement response with poorly-matched damper and frame system, or a system that was subject to much larger velocities than expected.

The importance of this phenomenon is that while an overall ‘well-balanced’ damping system can be implemented with careful design, it is difficult to control the earthquake response velocity of the structure. Therefore, it is difficult to predict the maximum force that will be present within a viscous damper. Consequently, as the damper force induces additional axial column force, it is difficult to predict the maximum force that will be present in the columns. Moreover, if the behaviour of the frame system is considered, it can be seen that if the column yields axially, then the diagonal member does not lengthen with further displacement, as seen in Figure 4. This observation goes against the typical analysis approaches that are employed with braced frame systems.

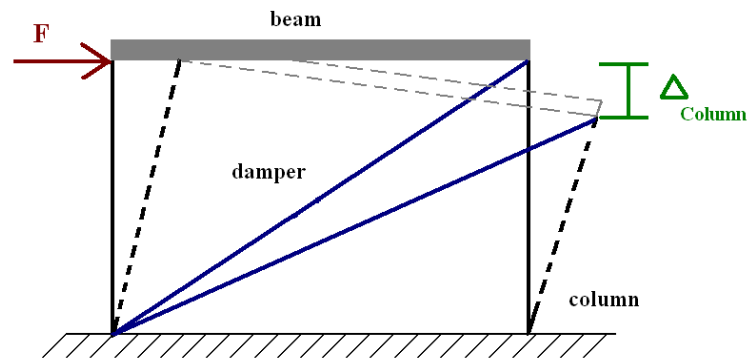


Figure 4: Schematic diagram of frame deflection, where axial column deflection results in no change in the length of the diagonal brace.

This observation gives rise to the question: *What if we just made the columns strong enough to resist the frame forces, and ignore the damper contributions?*

The conceptual advantage of this approach is that:

- if the column yields, the diagonal element will not change length with lateral frame deflection
- therefore, the velocity within the viscous damper goes to zero
- the force within the viscous damper goes to zero
- the column will not yield, because there is no additional column force from the damper
- for further displacements, forces are in the damper and there is no yielding

➔ A very small bit of axial column yielding may mitigate high damping effects.

2 MODEL DEVELOPMENT

2.1 General Model Information

An elementary model is modelled with SeismoStruct v5.0.4. The model consists of four elements: two vertical columns, one horizontal beam, and one main member that will be, for different analysis, an elastic flexible truss brace or a bilinear truss brace or a damper. Two concentrated mass element are fixed on nodes 3 and 4 following a lumped mass model. Rectangular solid sections are used for the four elements. Column and beam have a square section of 0.3 x 0.3 m and 0.1 x 0.1 m for the braces.

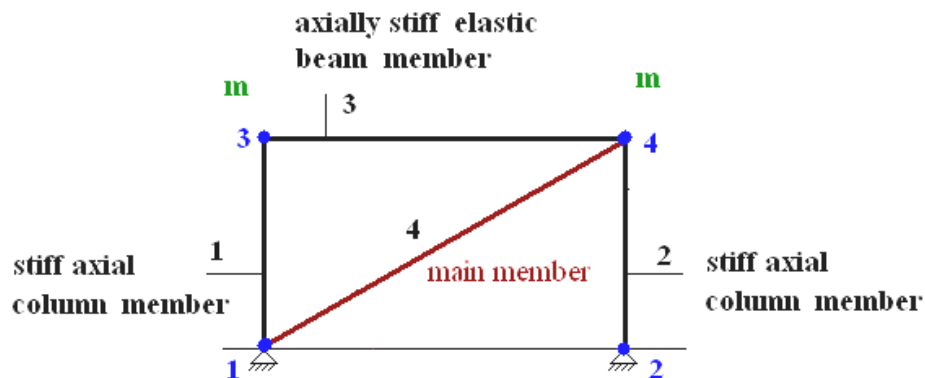


Figure 5: Schematic representation of the frame model.

2.2 Applied Loading

The input ground motion for the analysis are the odd numbered records from the LA 10 in 50 records from the SAC suites of ground motions (Somerville, 1997). The time-history curve derived from the text file will be applied to the node 1 and 2 in the horizontal direction. Rayleigh damping is applied to the simulations, with 5% inherent structural damping assumed.

3 METHODS

3.1 Elastic Truss Brace

For the initial structure with an elastic truss brace, the period was determined to be 4.1s through an eigenvalue analysis.

3.2 Bilinear Truss Brace

Initially, the bilinear truss brace is implemented and an eigenvalue analysis is used to find the period (T_2) of this structure. This period will be the period (**Error! Bookmark not defined.** T_2) of the mode 2 for the Rayleigh damping in the global damping parameter. A dynamic time-history analysis is then performed (with the previous structure) to look at the global response parameters. Several global response parameters results can be output; structural displacements, forces and moments at the supports, nodal velocities/accelerations and hysteretic curves. The peak force in the brace (F_{brace}) will be determined from the hysteresis curves. The yield strength of the brace will be modified by dividing through by a lateral force reduction factor, R , to give a new brace yield strength, F_B , where:

$$F_B = \frac{F_{brace}}{R} \text{ where } R \text{ is a coefficient } \geq 1.0 \quad (1)$$

The dynamic time-history analysis is then re-run and the peak displacement of the node 4 (d_{max}) and the peak force in the column (F_{column}) is determined. The yield strength of the column is modified to F_C where:

$$F_C = F_{column} * \alpha \quad (2)$$

where α is the fraction by which the column is understrength, and takes a value between 0 and 1 (excluded). α is defined as the ratio: ($Column_strength/Column_strength_{elastic}$).

For a range of values of R and α it is possible to plot the hysteretic curve of the brace and use the hysteretic curve of the column to calculate the cumulative inelastic column displacement.

3.3 Damper Truss Brace

In this analysis, a dashpot is added to the structure with an elastic truss brace. The dynamic time-history analysis is run and damping values of the dashpot are changed to find the same peak displacement of node 4 (d_{max}) than the structure with the bilinear truss brace. Once the peak displacement is matched, the yield force in the column is again modified by Equation (2).

For a range of α , the structural hysteretic energy, E , is calculated as before. Peak damper force cannot be found directly in Seismostruct and were calculated from other response variables.

4 RESULTS AND DISCUSSION

The analyses of a viscous brace structure are run with a range of different effective damping ratios. Figure 6 presents the structural response to the 'la02' earthquake from the SAC suite which was recorded in Imperial Valley. Figure 6a compares the response of a structure with no supplemental damping ($\xi = 0\%$) to that of a structure with 20% effective damping ($\xi = 20\%$). Figure 6b presents the comparison between the structure with no added damping ($\xi = 0\%$) to the structure with viscous brace with 100% effective damping ($\xi = 100\%$). It can be seen that the structure in Figure 6a has a good design balance and that the peak overall force is roughly similar to the structure without supplemental damping.

However, in Figure 6b, the lateral displacement response is noticeably less than that of the structure without the added damping, but the peak overall force is much higher. It is also important to note that the peak force, which occurs at the peak velocity, does not occur at the zero displacement position, which could be expected from a standard harmonic response. This results indicates that the peak velocity induced within the damper, and therefore the peak resistive force imparted into the structure may be difficult to predict. This observation within the results is similar to the concept presented schematically in Figures 2 and 3.

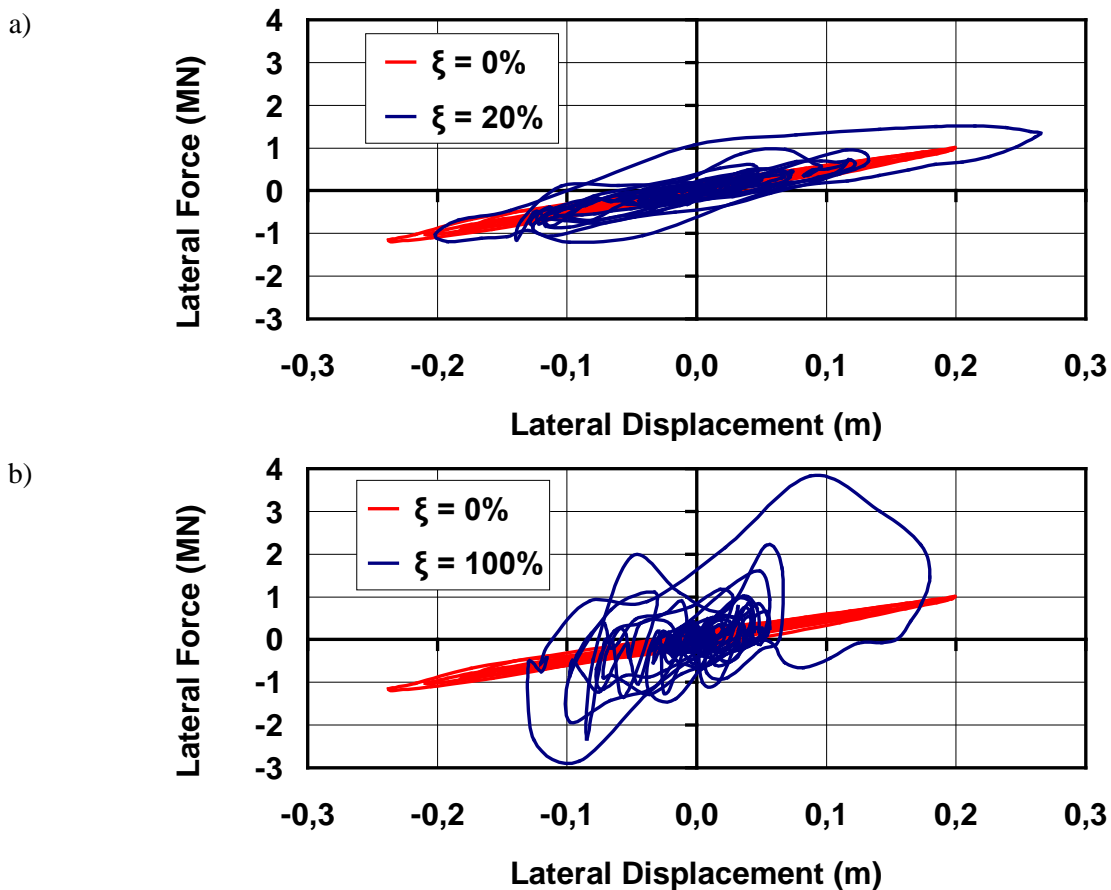


Figure 6: Lateral force versus displacement for a viscous brace structure with different effective damping and a column strength coefficient α of 1.0. The ground motion record used in these simulations was la02.

Figure 7 presents the cumulative column displacement versus the column strength coefficient, α , for both the hysteretic brace and the viscous damper brace for a structure with a period of 4s. The results in Figure 7 are a median result of 10 ground motion records, made up of the even numbered records of the medium suite of the SAC suite of records. It is clearly evident in Figure 7 that the structure with the viscous damper brace has significantly lower cumulative displacement than the structure with the hysteretic brace. This result can be explained by the initial concept presented in the introduction, whereby axial column yielding reduces the velocity within the damper and acts as a stabilising mechanism to minimise the amount of column yielding.

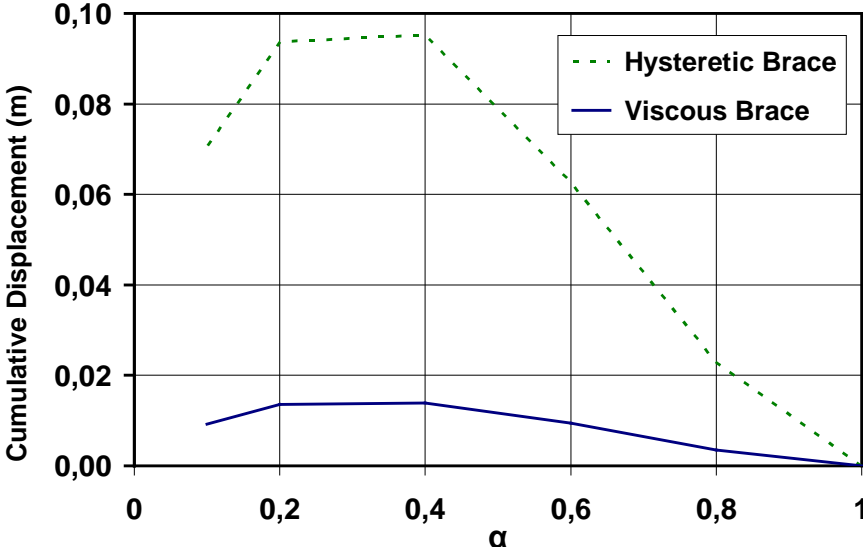


Figure 7: Median value of the column cumulative displacement versus the column strength coefficient α for a structure period T of 4s, a brace strength coefficient R of 3, an effective damping ζ of 35%.

Figure 8 presents the same analysis as the in Figure 7, but for a structure with a period of 2s. Again, there is a clear difference between the results for the hysteretic and damper braces, with the hysteretic brace resulting in significantly larger cumulative column displacement. It is also evident that as the column strength coefficient, α , is increased, the amount of cumulative displacement is initially increased, but that the overall trend is a reduction as a approaches 1.0. This is an expected result, as the lower column strength will naturally lead to an increase in yield displacement.

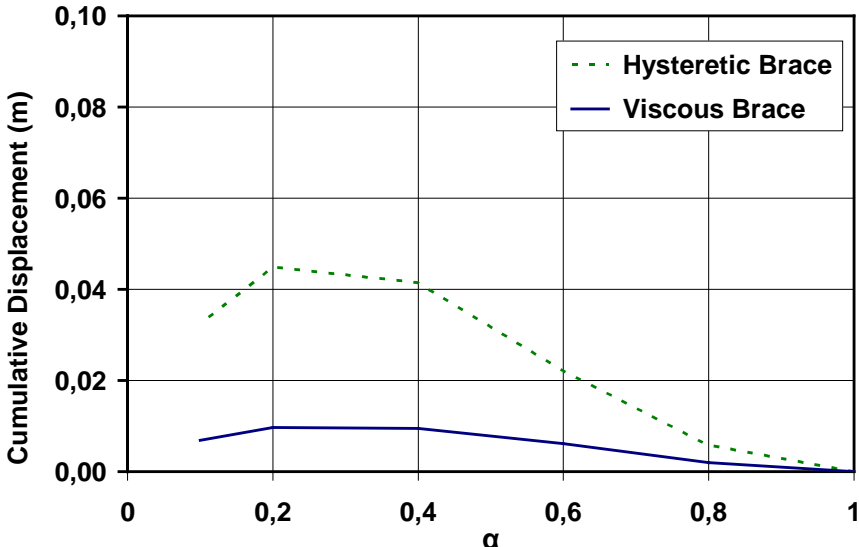


Figure 8: Median value of the column cumulative displacement versus the column strength coefficient α for a structure period T of 2s, a brace strength coefficient R of 3, an effective damping ζ of 35%.

Overall, the analytical investigation has confirmed the initial hypothesis that the use of viscous damping and the possible violation of capacity design methods have the potential to provide a self-stabilising system, where the onset of axial column displacement can reduce damping forces and prevent further yielding of the column.

In practice, the inclusion of axial column yielding may not be a desirable trait, and may be an aspect that provides a barrier to the consideration and uptake of such an approach in design. In practice, it may be much more desirable to include a sacrificial steel fuse connection at the end of the damper-brace in series with the damper. This steel fuse element could be sized to prevent yielding of the column under large drifts and act as a genuine fuse element, rather than as a primary form of energy dissipation for response reduction, such as that typically done with so-called 'yielding steel fuse bars' (Bradley et al, 2008, Rodgers et al, 2008).

5 CONCLUSION

This research presented a novel and perhaps provocative concept of how structures with viscous bracing may benefit from violation of traditional capacity design techniques. The onset of axial column yielding can lead to lateral frame deflection resulting in no extension of a diagonal brace element. Therefore, a viscous damper placed within this diagonal will experience zero velocity, eliminating damping forces and potentially eliminating column yielding, in a manner that may lead to self-protecting behaviour. This concept is introduced within this paper and initial simulations indicate from cumulative inelastic column displacement that the penalty for violating capacity design requirements of a viscous system is much less than for a traditional hysteretically braced system. However, further studies are required to obtain robust design recommendations for viscous systems.

REFERENCES:

- Bradley, B. A., Dhakal, R. P., Mander, J. B., Li, L. (2008). "Experimental multi-level seismic performance assessment of 3D RC frame designed for damage avoidance." *Earthquake Engineering & Structural Dynamics* **37**(1): 1-20. Doi: 10.1002/eqe.741
- Rodgers, G. W., Solberg, K. M., Chase, J. G., Mander, J. B., Bradley, B. A., Dhakal, R. P., Li, L.. (2008). "Performance of a damage-protected beam-column subassembly utilizing external HF2V energy dissipation devices." *Earthquake Engineering & Structural Dynamics* **37**(13): 1549-1564 Doi: 10.1002/eqe.830
- Somerville, P., Smith, N., Punyamurthula, S., and Sun, J. (1997). "Development of Ground Motion Time Histories For Phase II Of The FEMA/SAC Steel Project, SAC Background Document Report SAC/BD-97/04."