PRELIMINARY "GUIDELINES" FOR THE NON-LINEAR ANALYSIS AND DESIGN OF HYSTERETIC (DISPLACEMENT DEPENDENT) ENERGY DISSIPATION DEVICES IN BUILDINGS

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INTRODUCTION

Much of the traditional seismic design of structures relies upon yielding of certain members to limit the forces in the structure. This yielding lengthens the effective period of the structure and dissipates energy, with the result that the displacements of the yielding structure are about the same as if the structure had remained elastic. Since the members that yield may also be part of the gravity load carrying portion of the structure and may be hard to repair in the event of an earthquake, there have been efforts made to introduce members or devices into the structures which are not a part of the gravity system, that will limit the forces and add damping and which do not need repair or can be easily repaired. Many ordinary braced systems are of this type. There are other methods of controlling the seismic response of structures, and devices to do this are broken into three broad categories: active, semi-active, and passive. Active devices measure the response of the building during the earthquake and use control devices to move masses to create inertia forces that reduce the structure response. They require a complicated control system and large amounts of energy. Semi-active devices tend to control the response by changing the stiffness or damping of some members. This also requires a control system but does not require as much energy. Passive systems can involve tuned mass dampers that don't involve energy dissipation, but the majority of passive systems rely on some sort of yielding/frictional system, or on viscous dampers.

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Passive energy dissipation devices can be broken down into two broad categories:

Displacement Dependent Devices

The response of these devices are essentially independent of the velocity of the structure, and their effect depends upon relative displacements in the structure. They generally add stiffness and increase the damping when inserted into a building frame. They are essentially "yielding" devices and dissipate energy by yielding or slipping and produce hysteresis loops based on a displacement cycle. Examples are metal yielding devices (ADAS elements or buckling prevented braces), friction dampers, eccentric-braced frames, etc.

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Velocity Dependent Devices

These devices are essentially displacement independent and rely on relative velocity differences in the structure, and are generally referred to as viscous dampers. They do not add stiffness in the normal sense but increase the damping of the structure, similar to shock absorbers in a car. They creep under static loads and do not carry any of the gravity or temperature induced loads. Viscoelastic dampers do have an elastic component as well, and so can add stiffness as well as damping.

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Both types of devices are most effective when they are used to control lateral deflections within a relatively flexiblestructure. They can be used with stiffer lateral systems but the size of the devices would have to be such that they might not be economical.In general:

Displacement Dependent Devices reduce deflections when added to a structure by adding stiffness, and in most cases by also adding damping. This added stiffness often increases the total base shear for the new structure, but forces in members outside the added lateral system may decrease because of the reduced displacements. Forces in members that are attached to the added lateral system, such as columns in a braced bay, may increase.

Velocity Dependent Devices do not add stiffness when added to an elastic structure but reduce deflections by adding damping. The viscous dampers themselves will have forces, which will be added to the building frame forces, but since the viscous forces are velocity dependent, they are out of phase with the displacement-induced forces in the frame if the structure remains elastic. However, if the structure yields the viscous forces can add to the yielding forces. Generally, forces in members outside the added lateral system may be reduced because of the reduced displacements. Forces in members that are part of the added lateral system often increase, but may decrease because of the out-of-phase nature of the loading.

This Guideline is not meant to apply to viscous or viscoelastic devices (velocity dependent devices) even though there are some issues in common with displacement dependent devices, and portions of the Guidelines might be applicable.

BACKGROUND INFORMATION

Preliminary work for his Guideline based upon several numerical parametric studies of single degree of freedom (SDOF) hysteretic systems using several earthquake input records (a small number of SDOF viscous damped parameter studies were also performed).

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Issues addressed were:

P-Δ effects.

Strain hardening.

Attached elastic systems of varying stiffness.

Force reduction factors (R) ranging from 1 to 16.

Multiple "spectrum matched" earthquake records for Vancouver.

Duration of records.

Responses for 1, 2 and 3 times the amplitudes or the records.

Different viscous damping values.

Building height variations.

Various periods.

A 10-storey steel building frame was also studied for the parameters listed above (except building height) for both:

Eccentric braced steel frames.

Friction braces.

This is not an extensive amount of study. For instance:

Only earthquake records for Vancouver were run.

The only model building run was a 10-storey frame. Other numbers of stories (i.e. 5 or 30) may lead to different conclusions.

While the above work forms the main source of information for this Guideline, there are US resource documents which have been helpful:

ATC33/FEMA 273 and 274/"Guidelines and Commentary for the Rehabilitation of Buildings". This document was published in October 1997 and has been around in draft forms since 1995. It treats non-linear analysis and has a chapter on energy dissipation devices.

NCEER/EERC – Seminar on "Passive Energy Dissipation for Seismic/Wind Design and Retrofit" – Seattle – Sept./96.

The FEMA documents are for "rehabilitation" of existing buildings so their philosophy is slightly different than that for "new building" codes. However, the technical information is common to both types of codes.

A summary of the findings for the braced system, based on the above, is:

Different earthquake records, even when matched to the Vancouver design spectrum, produce widely varying results.

Adding eccentric braces or friction braces to the bare frames reduced the bare frame displacement response and increased the system total base shear.

The "equal displacement' theory holds depending upon a variety of parameters. In some cases the response started to diverge with R values as low as 2, but most times it held up to R values of 4.

$P\Delta$ effects are very important.

Strain hardening is very important. Systems without it, such as friction systems, need an elastic system in parallel.

The results indicate little or no difference between an eccentric brace system or a friction brace system based on force levels and displacements.

There is effectively no reduction in displacements from the elastic response. In both the eccentric or friction brace systems, as the R values increase, the frame starts to bulge out at the lower levels and the displacements exceed the elastic displacements, even as the displacements reduce slightly near the top.

There is effectively no reduction in force levels beyond code levels. With the necessary elastic gravity frame in place, and with brace only load levels at R = 16 (which collapsed under some of the earthquake records used) the system R value was about 5. Here system R refers to the force reduction factor when the total base shear is considered. This includes forces in the gravity frame as well as the brace system. The R = 16 for the braces refers to the force reduction factor applied to the forces in the braces when the building is analyzed elastically.

Without the elastic gravity frame or strain hardening in the braces, the systems started to collapse at quite low R values. Even with an elastic gravity frame which takes at least 25% of the elastic base shear at the design earthquake, the performance became quite erratic at R values for the braces over about R = 6, particularly when the design ground motion was doubled.

Elastic base shears of the 10-storey buildings were less than code values mainly due to:

The code formula uses a response (the S factor) proportional to 1/sqrt(T). This tends to boost forces at the long period end to account for higher mode shapes and to also provide a degree of conservatism for tall "important(?)" buildings. The design spectrum for the dynamic analysis was proportional to 1/T for long period responses (T> 0.5 seconds).

The code period for the building analyzed is 1.0 seconds whereas the calculated first natural period is about 2.5 seconds. This discrepancy in period is not unusual, and for the building studied this has the effect of reducing the "force response" to about 60% of the code response. The code currently addresses this issue by limiting the force reduction for dynamic analysis to preserve the idea of "equivalent performance" between building designs, both current and historical. The 1995 NBCC requires the base shear to greater than 80% of the static provisions.

The above have been used to develop the following "Guidelines" for the design of buildings using non-strain hardening hysteretic energy dissipation devices. The "Guidelines" also try to take into account the very high levels of uncertainty associated with building behaviour and, particularly, ground motions. However, they are based upon a limited amount of information and both these results as well as any conclusions or recommendations drawn from them will undoubtedly be modified as more knowledge becomes available.

DRAFT DESIGN GUIDELINE - HYSTERETIC TYPE DAMPERS

DISCUSSION

A structure with a friction damped brace system responds much the same as a structure with yielding concentric or eccentric braces. The friction brace system adds stiffness to the structure and when the slip force in the brace is exceeded they slide and dissipate energy, much the same as yielding elements. Because the system is relatively new and has not been widely tested in real situations, and has not been addressed in codes to date, it is our belief that there should be some conservativeness in the design of friction damped systems as opposed to the more common yielding systems that have been codified.

In the NBCC load provisions the design base shear Vu is taken as the elastic base shear Ve multiplied by a calibration factor U=0.6, and then reduced further by the force reduction factor R. The origins of the calibration factor U are perhaps controversial, but in part it is said to represent the overstrength that most buildings possess. Part of this overstrength comes from the lateral force resisting system but another part comes from the gravity structure that is often ignored in designing the lateral system. In the proposed guidelines the gravity system is considered in the design and the friction devices would be constructed to slip at the design load, i.e. there should be no overstrength in the slip forces. Thus there should be little overstrength in a friction braced structure. For this reason the U=0.6 factor has been eliminated in this guideline, and the force reduction values are to be applied to the elastic base shear Ve, as well as using the elastic forces or compatible ground motions in calculating displacements.

Many friction braces systems have a very flat or constant force level during the sliding phase, i.e. in terms of an equivalent elastic-plastic response there is very little if any strain hardening. Without strain hardening, or some elastic stiffness in the gravity frame part of the structure, $P\Delta$ can become a problem; there is danger of a soft storey developing; and the structure response may be of a ratchet type where the structure sways to one side and the deformations continually increase in the one direction with each large cycle.

If the gravity frame remains elastic and is by itself stable up to say twice the gravity loads, then $P\Delta$ should not be a problem. However, even if the frame is stable and if the stiffness in one storey is much lower than the other storeys, and this typically happens at the first storey in a frame building, then a soft storey can still occur and the drift will become large in that storey. If the stiffness of the gravity frame is such that it will resist at least 10% of the total base shear at the first storey level, and if the stiffness of the bottom storey exceeds that of the floors immediately above, and if the gravity frame remains elastic, then the response should be well behaved (storey stiffness is not well defined but one definition which would be appropriate here would be to subject the structure to a horizontal load at the top and calculate the storey drifts, which can then be used to define the storey stiffness).

An effort should be made to reduce both the stiffness eccentricity as well as the strength eccentricity of the building, and to provide at least two lines of braces in each direction to form a torsionally redundant structure.

Two methods of analysis are discussed below, a Static Method for small buildings that essentially follows the NBCC static method, and a Dynamic Method where the slip of the braces is modelled explicitly. These are followed by General Requirements common to both methods of analyses.

STATIC DESIGN METHOD

Use of this method of analysis and design is limited to structures that:

Are less than 5 storeys or 20 m in height, with a fundamental period less than 0.5 seconds.

Are regular in plan and elevation.

Have 2 or more widely separated bays of friction braced frames in each direction.

Have small torsional eccentricity, both in stiffness and strength.

1. The strength and stiffness of the lateral force resisting system (LFRS) shall include, in addition to the strength and stiffness of the gravity frame, the slip force in the braces when calculating strength, and the stiffness of the braces assuming no slippage when carrying out an elastic analysis. The gravity frame should carry at its base at least 20% of the base shear in an elastic analysis, and the stiffness of the lowest storey should exceed that of the storeys immediately above. In this context the storey stiffness of the gravity frame can be determined by applying a horizontal force at the top of the structure (where the braces are omitted) and then dividing the storey shear by the storey drift.
2. The maximum value of R to be used in the design of the friction braces is R = 5. The brace slip forces are to be determined by dividing by R the brace forces from the elastic analysis of the LFRS for the elastic base shear (Ve).
3. Storey drifts of the elastic LFRS should not exceed 1.5% when calculated using the loads associated with the elastic base shear. If the structure is concrete then cracked section properties should be used in determining the displacements.
4. The gravity frame should remain elastic at 1.5 times the displacements calculated above.
5. Damper slip forces could vary up the height of the building in the same manner as the storey shears, or they could be the same at all levels. Changes in the distribution up the height appears to have little effect upon the displacements.
6. Torsion. Apply static code provisions.

DYNAMIC DESIGN METHOD

The guidelines for design using a dynamic analysis are meant as a guide covering input ground motions, some structure parameters, and some performance requirements. The usefulness of the dynamic analysis is to determine the structure displacements and drifts, and see how this affects the gravity load portion of the structure.

1. Design spectrum and ground acceleration records.

Design spectrum. Use code Sa, i.e. acceleration spectrum implied in the code commentary, scaled to v.

Foundation factor. Suggest use of code multiplier with a cutoff at the peak value of Sa, or use of program such as SHAKE to get surface motion.

Ground records for time history analysis. Records should be selected that are appropriate for the magnitude, distance and duration of the strong shaking for the site seismic hazard, and should come from different seismic events. The records should be fitted to or are compatible with the design spectrum. Compatible means that the average value of the resulting spectrum is generally greater than the design spectrum in the period ranges of interest, which includes the period from 1.5 times the first mode period to the third mode period. If a three dimensional analysis is being performed, pairs of horizontal ground motion records should be applied simultaneously. For each pair of horizontal ground records, the average value over the period range of interest, as described above, of the square root of the sum of the squares

(SRSS) of the scaled records should be greater than 1.4 times the design spectrum. (Note that this latter requirement does not imply that the records have spectra that are 1.4 times the design spectrum. If the two orthogonal records are identical then each record will essentially match the design spectrum).

If three time-history analyses are performed, then the maximum response shall be used. If seven or more time-history analyses are performed then average values may be used.

7. Analysis. Non linear analysis of entire structure.

PΔ. must be included, where the P is the total gravity load and not just the gravity load attributed to the braced bays. If the structure is concrete use cracked I based on CSA recommendations. Use soil springs if soil-structure interaction is deemed important when calculating displacements.

3. Viscous damping.

For viscous damping use 2% for steel structures and 3% for concrete structures. Don't assign any viscous damping to the members that contain the friction devices.

4. Gravity frame stiffness.

If the frame (without the brace devices) is able to counteract the PΔ effect, i.e., the frame is stiff enough so that it is stable under the gravity loads, then the effect of PΔ should be small.

Frames that performed well in our analyses picked up about 10-25% of the total base shear in a static elastic analysis (assuming the dampers didn't slip). These frames were stable under 1.6 or more times the gravity loads.

A requirement that the frame be stiff enough to pick up 10% of the base shear in an elastic analysis might be satisfactory for design, although it may be stiffer than necessary for short period and not stiff enough for long period structures. It may also be stiffer than necessary for high v zones and not stiff enough for low v zones.

Columns should be moment connected at base, or the stiffness of the first floor columns should be increased so that the stiffness of the first storey is greater than the storeys above. It could be that the restraint provided by axial loads on the column bases may be adequate to provide the stiffness required, but this is difficult to evaluate. A rotational stiffness at the base of the columns equivalent to that provided by beams the same size as the floor beams in the upper storeys, increased the first storey stiffness so that it was greater than the upper storeys, and reduced the soft storey effect considerably. Storey stiffness should not change abruptly between adjacent levels, and should be greater in the lower floors.

Based on the information we have at present, the frame stiffness should at least:

Provide 10% of the total shear at the base in an elastic analysis

Be stable under 2*DL (this assumes the structure is not a storage facility)

The storey stiffness in the first floor should be greater than those above. Storey stiffness should not change abruptly, and should be greater in the lower storeys.

5. Friction damper sliding force values.

Variation with height of building. Use distribution roughly corresponding to static code distribution of shears, dynamic elastic distribution of storey shears, or a uniform distribution. This does not appear to be a critical parameter but it is best if the slip forces do not change greatly from one storey to the next. It may be desirable to increase the damper slip force in the lower third of the building to counteract any tendency to form a soft storey in the lower floors, and reduce the deflections in these storeys which usually have the highest drifts.

The slip forces in the friction braces should be equal or greater than the brace forces found in an elastic dynamic analysis of the LFRS, scaled so that the base shear is equal to the code base shear Ve, divided by a force reduction factor R=7.

Code based shear is the elastic base shear Ve determined using the code period estimate (or 80% of Ve if applicable).

8. Response requirements.

Using the design slip values in the friction braces carry out nonlinear analyses to determine the displacements. The slip values determined in 5 above are deemed sufficient for the strength of the structure (unless wind governs), but the structure must also satisfy drift and displacement criteria. For the displacement calculations use structure properties that will increase the period, such as cracked stiffness estimates for concrete members, and soil springs if their use would make an appreciable change in the fundamental period.

9. Building should not collapse or show signs of significant displacements to one side for 2*design earthquake input.
10. Drift limits. Storey drift should not exceed 2.5% for 2*design earthquake.
11. Ductility demand in frame members should not exceed those associated with a displacement ductility demand of 1.5 at 2*design earthquake.
12. Frame should remain elastic for 1.5*design earthquake input.

It is preferable to keep the frame substantially elastic at 2*design earthquake so that it can still provide some lateral stiffness.

7. Torsion.

3D analysis. In both the design analysis (elastic) and the nonlinear analysis to check for displacements, do two analyses. Once with the mass centroid moved a distance of 0.1D towards the stiff sides of the building and the second with the mass centroid moved 0.1D towards the flexible sides of the building. The first analysis should produce higher forces and displacements on the stiff side of the structure, while the second should govern on the flexible side. In some torsionally stiff buildings the above procedure results in stiff side design forces less than what would be required if the building had no torsion (a torsionally balanced building), i.e. the building did not rotate but only translated. If these cases the buildings performed better if the design of the stiff side was based on the torsionally balanced analysis. In terms of ductility the above procedure usually produces a ductility demand in the flexible side less than the target ductility, while in the stiff side the ductility demand sometimes increases. However the displacements of the stiff side are usually small and so this may not be of concern.

When moving the mass centroid the question arises to how this affects the mass moment of inertia of the floors about a vertical axis. Most recent research on torsion has moved the mass centroid without changing the moment of inertia. Programs with rigid diaphragms usually allow you to input the total mass and moment of inertia, and then you can simply move the location of the mass centroid.

2D analysis. If an elastic 3D program is available design the braces as discussed above for the 3D analysis. If only 2D analysis programs are available use the static code provisions to determine the additional forces on the braces due to torsion.. When checking the displacements from the nonlinear analysis scale the deflections calculated by the dynamic analysis by the ratio (Ve + 2Vt)/Ve, where Ve is the elastic base shear and Vt is the additional slip force due to the torsional design in all the braces in the direction under consideration. Use these scaled deflections to check the drift limits.

GENERAL REQUIREMENTS

1. Detailing requirements.

   a) Frame should be detailed for 'nominal ductility' as defined in the CSA standards, i.e. the system should not be brittle.

   b) Frame should be detailed for slip forces of 1.5 times the specified values, i.e.the frame (including brace members, beams, columns, connections, drag struts and foundations) should be designed for a slip capacity of 1.5 times the design (or specified) slip forces.

   c) Dampers should be able to undergo slip displacements of 3 times the values calculated using the design earthquakes (analyzed using cracked section properties).

   d) Wind design – use 1.5 times code wind force with no slipping of the braces using a friction device ϕ=0.85.

   e) ϕ factors:

   ϕ = 1.0 for design of friction dampers except for 0.85 used in wind design.

   ϕ as defined in CSA for design of other structural members and consideration of overstrength.

2. Review

The analysis, design and testing should be subject to review by an independent third party professional engineer with substantial experience in design as per ATC 33 / FEMA 273.

3. Detailed system requirements (as per ATC 33 / FEMA 273).

   The following list some of the issues that need to be considered:

   - Effect of operating temperatures
   - Effect of environmental conditions (ageing, creep, fatigue, exposure to elements)
   - Access for inspection and replacement
   - Manufacturing quality control
   - Maintenance and ongoing testing

4. Required testing (as per ATC 33 / FEMA 273 with some reduction).

The following lists some of the tests that need to be carried out:

   - Loading, at the structure fundamental frequency, not less than 2000 fully reversed cycles at a load equal to the wind load.
   - Loading, at the structure fundamental frequency, not less than 10 fully reversed cycles at a displacement equal to that produced by an input twice the design spectrum.