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SUSTAINABILITY IN SOFT WEAK OPEN FRONT BUILDINGS

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Soft weak open front (SWOF) buildings often perform poorly in earthquakes. Two examples are buildings with a street facing garage, or commercial facilities with extensive open display windows. The poor performance of SWOF structures can consist of complete loss of use or even total collapse. This paper presents an approach to protecting such structures via the addition of an energy dissipation system (viscous dampers) such that peak inter-story drifts are limited to about 1% under relatively severe seismic events, thus keeping the deformations within the elastic range. With this addition of damping, earthquake survivability of this class of structures increases significantly. A series of seismic analyses are presented herein to demonstrate the potential performance of the damping system. In addition, a variety of damper installation configurations that provide enhanced energy dissipation are discussed.

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Soft weak open front (SWOF) buildings often perform poorly in earthquakes. Two examples are buildings with a street facing garage, or commercial facilities with extensive open display windows. The poor performance of SWOF structures can consist of complete loss of use or even total collapse. This paper presents an approach to protecting such structures via the addition of an energy dissipation system (viscous dampers) such that peak inter-story drifts are limited to about 1% under relatively severe seismic events, thus keeping the deformations within the elastic range. With this addition of damping, earthquake survivability of this class of structures increases significantly. A series of seismic analyses are presented herein to demonstrate the potential performance of the damping system. In addition, a variety of damper installation configurations that provide enhanced energy dissipation are discussed.

Introduction

For the past 80 years, the minimum requirement for seismic design of typical structures has been to ensure "life-safety" under the Design-Basis Earthquake (DBE) (about 500-year return period) with an implicit performance level of "collapse prevention" under the Maximum Considered Earthquake (MCE) (about 2500-year return period). Recent changes to the building code implicitly define the "life-safety" performance level as being associated with a scaled version of the MCE event, the scaled version roughly corresponding to a DBE event. In spite of adherence to the building code, after a major earthquake, large numbers of people are without shelter, water, sanitary facilities, etc., for an extended period of time. This paper describes how the addition of an energy dissipation system can increase the seismic survivability of wood-framed buildings and thus mitigate the disastrous effects of an earthquake.

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Sustainability of Soft Story Buildings

Sustainability is increasingly becoming a concern relative to housing performance during significant earthquakes. As noted above, after a major earthquake, large numbers of people are without shelter, water, sanitary facilities, etc., for an extended period of time. Sustainability in this context refers to the design or retrofit of a structure such that, in spite of being damaged during a major earthquake, the damage is of such a nature that it can be repaired within a relatively short period of time without the need to displace the building occupants.

Most designs of multi-story, multi-family residential structures utilize an equivalent lateral force procedure where the base shear, V (total force acting on the structure) is equal to a design spectral acceleration multiplied by the seismic weight, W, and reduced by a factor, R, to account for inelastic behavior and a factor to account for the relative importance, I, of the structure ($V = S_{ds} \times W \div R/I$). The coefficient S_{ds} is the design spectral acceleration (force coefficient) associated with short period (stiff) structures at a particular location. Values of the spectral acceleration are obtained from ground motion “contour” maps where the mapped values depend on the local geology and proximity to known fault sources. The particular soil conditions at the building site are used to modify the value of the spectral acceleration. The importance factor is generally 1.0 for residential buildings. The response modification factor, R, for light-framed walls sheathed with wood structural panels or steel sheets is 6.5. This means that, if the building were to remain completely elastic (no damage), a design earthquake would induce forces on the building which are 6.5 times the code design forces. However, conventional design dictates that, under a design-level earthquake, the structure will experience inelastic behavior and thus dissipate energy via inelastic action, thereby reducing the required forces for design. As an illustration of this, the code allowable strength for 15/32-inch plywood with 10d common nails at 4" on center is 510 lbs per foot and for 10d at 3" on center is 665 lbs per foot. However, based on the lateral force capacity curves provided in the FEMA P-807 report [1], for a drift ratio of 1% the corresponding maximum values are 1250 and 1700 lbs. per foot (see Fig. 1), which are approximately 2.5 times the allowable values. With a drift ratio of 3%, the corresponding values are 1450 and 1950, which are 2.8 & 2.9 times the allowable values. An inter-story drift of 1% is generally the accepted value above which damage starts to become significant. This means that base shear forces greater than about 2.5 times the design forces will result in significant damage to the building and thus the building may not be regarded as having a sustainable design.

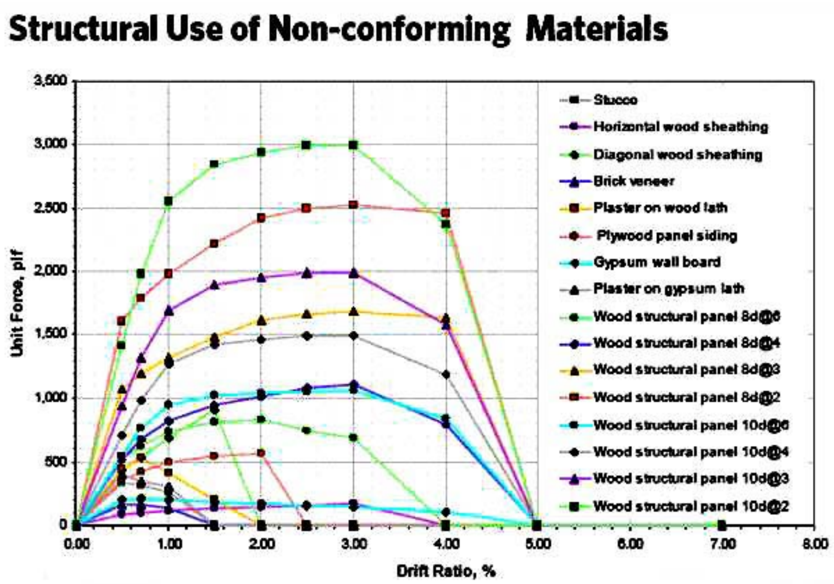

Figure 1. Unit load-drift curves for sheathing materials with high displacement capacity. Wall panel nailing notation: nail size @ center-to-center distance of nails in inches (from [1]).

A class of building systems which have not performed well in recent earthquakes are buildings with a soft weak open front (SWOF). This often occurs with wood frame structures, especially those with "tuck-under" construction to accommodate a street facing garage, or a commercial facility requiring extensive open display windows. Their poor performance, including total collapse, has resulted in code modifications that increase the retrofit requirements for these structures, and often prohibits these systems in new structures.

There are basically two methods for improving the performance of a structure subjected to loads caused by an earthquake. The first is to increase the capacity of the structure by designing for greater force levels. The second is to reduce the loading imposed on the structure by incorporating a flexible interface between the ground and the structure via a base isolation system and/or by dissipating energy within the structure via a damping system. For SWOF buildings, the following options are available to the building owner: 1) Tear down the structure and design and build a new structure which meets the current code requirements, 2) Supplement the existing lateral force resisting system (add new lateral force resisting elements) to be reasonably sure that the sum of the existing and new systems will be able to resist the design loads, 3) Upgrade the existing system (strengthen the elements in the existing system) so that it meets the design load requirements, and 4) Install a base isolation or damping system to reduce the design loads below the capacity of the existing supplemented or upgraded lateral force resisting system.

Options 2 & 3 utilize a rigid lateral force resisting systems with limited flexibility (includes rigid frames, braced frames, shear walls, etc.) where the only significant difference between the systems is the response modification factor R.

Option 4 allows the structure above the ground floor to move relative to the ground (base isolation) or to absorb energy through a damping system. In either case, the interface between the moving portion of the structure and the rigid foundation absorbs energy using elastomeric bearing pads, friction pendulum system bearings, etc., and/or some type of dampers.

The base isolation solution requires that two separate foundations be constructed. The bottom foundation moves with the ground while the upper supports the structure and is isolated from the lower with isolator units which are horizontally flexible and vertically stiff, permitting large lateral deformations under the design seismic loads. This approach to seismic protection may be challenging to implement due to the requirement for a rigid foundation above the isolator units [2,3].

The damping solution requires that damping devices be installed within one or more stories of the building. The inter-story drift of the stories is used to induce deformation in the damping devices and thus to dissipate energy. At a minimum, it is expected that a SWOF building would require 4 shear panels (one on each side of the building) which have built-in dampers that are designed to limit the drift to 1% or less. The dampers would be expected to reduce the earthquake forces imparted on the structure, to absorb a significant portion of the seismic input energy (which would otherwise be dissipated by the structure via inelastic response) and to significantly reduce damage to the structure [3,4].

Application of Dampers to SWOF Buildings

If a building owner opts for a higher performance level than that implicitly defined by the building code (e.g., immediate occupancy), the designer can incorporate a conventional rigid lateral force resisting system which limits deflections to that which the finish materials can undergo without appreciable damage. Alternatively, the designer could incorporate a damping system which dissipates energy, reducing the forces on the structure and limiting deflections to a level which would cause minimal or no damage to the finish materials in the structure [5].

The City of San Francisco is attempting to address the SWOF issue by using a reduced version of the FEMA P-807 [1] recommendations to reduce the damage or collapse potential of such buildings during a seismic event. The proposed San Francisco solution is as follows: Install a seismic retrofit system in the ground story of multi-story (2-5 stories) buildings which have a soft, weak open front framing system in the ground story. The basic criteria is to create a system which does not transfer ground level forces up to the second level which are greater than the strength (strength, not stiffness) of the existing second story lateral force resisting system, thereby preventing or minimizing damage in the upper stories, such damage being costly to repair given that occupants must be displaced.

When a conventional retrofit (shear walls, rigid frames, diagonal bracing, etc.) is installed in the ground story, the mass at the second level is subjected to accelerations which are generally much larger than they would be if the ground acceleration were applied directly to the mass (increased stiffness reduces period and thereby increases spectral acceleration). Since the inertial force acting on the mass is equal to the mass multiplied by the acceleration, with a given mass and a defined force capacity, the allowable acceleration at the 2nd level can be determined. To determine the corresponding allowable ground acceleration, the allowable acceleration at the 2nd level is divided by the amplification factor previously established. This determines the magnitude of the ground acceleration for the governing earthquake based on the use of a conventional retrofit system.

If on the other hand, the designer selects a retrofit system which includes dampers, the accelerations at the second floor are expected to only be slightly increased, or even decreased, relative to the ground level accelerations and thus the solution with dampers would require much larger ground accelerations (larger earthquake) to reach the limiting strength of the lateral force resisting system at the second story. Linear viscous dampers are one type of damper that could be used in such retrofits. Such dampers develop forces that are proportional to velocity and thus the damper force is 90 degrees out of phase with respect to the displacement-dependent restoring forces in the structure. Therefore, the peak forces in the damper are not additive to the peak restoring forces in the structure. The effect of linear viscous fluid dampers on the seismic performance of representative soft weak open front buildings is presented below. As will be shown, the seismic force resisting capacity of such buildings can often be increased by a factor of 3 to 4 over a building that utilizes a conventional retrofit system.

Performance Assessment

The representative SWOF buildings considered in this performance assessment have effective weights of 48.7 k, 77.9 k, 107.6 k & 137.3 k to simulate 2, 3, 4 & 5 story buildings. The allowable 2nd story drift ratios are defined as 1% and 2.5% (story height assumed to be 12 ft). The selected strength value for the second story was taken as 34.4 k. The buildings were subjected to the Treasure Island (TI) and Outer Harbor Wharf (OHW) ground motion records from the 1989 Loma Prieta earthquake and the El Centro record from the 1940 Imperial Valley earthquake. A typical set of results is shown in Figure 2 wherein the required ground acceleration to reach the strength of the second story is shown for both the case of the building with and without the damping system. A summary of the results from all simulations is provided in Table 1.

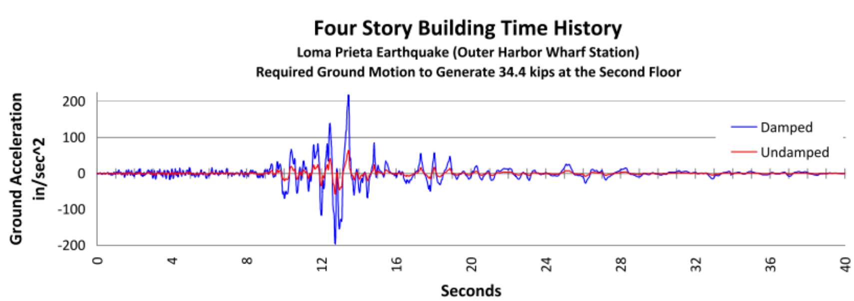

Figure 2 Required ground acceleration to reach strength of second-story of structure with and without damping system

Table 1. Required ground acceleration to reach strength of second story for representative buildings with and without a damping system

1% Allowable Drift Ratio = 1.44 in				
	Two Story	Three Story	Four Story	Five Story
Tributary Weight	48.2 kips	77.9 kips	107.6 kips	137.3 kips
2nd Floor Acceleration (required to generate 34.4 kips)	0.71g	0.44g	0.32g	0.25g
El Centro Maximum Ground Accelerations				
Undamped	0.23g	0.16g	0.16g	0.17g
Damped	0.92g	0.71g	0.59g	0.50g
Loma Prieta (TI) Maximum Ground Accelerations				
Undamped	0.32g	0.14g	0.12g	0.09g
Damped	0.81g	0.52g	0.41g	0.32g
Loma Prieta (OHW) Maximum Ground Accelerations				
Undamped	0.37g	0.25g	0.12g	0.11g
Damped	0.85g	0.58g	0.45g	0.37g

2.5% Allowable Drift Ratio = 3.60 in				
	Two Story	Three Story	Four Story	Five Story
Tributary Weight	48.2 kips	77.9 kips	107.6 kips	137.3 kips
2nd Floor Acceleration (required to generate 34.4 kips)	0.71g	0.44g	0.32g	0.25g
El Centro Maximum Ground Accelerations				
Undamped	0.43g	0.26g	0.24g	0.29g
Damped	1.34g	0.98g	0.81g	0.68g
Loma Prieta (TI) Maximum Ground Accelerations				
Undamped	0.24g	0.13g	0.12g	0.13g
Damped	0.91g	0.63g	0.49g	0.40g
Loma Prieta (OHW) Maximum Ground Accelerations				
Undamped	0.32g	0.17g	0.17g	0.18g
Damped	1.00g	0.70g	0.56g	0.48g

The tributary weight shown in Table 1 can be modifed (by varying the distance between shear walls) to limit the force demands and thus establish the strength available at the 2nd story for any particular structure. Of course, a continuous load path must be ensured between the existing structural system and the new lateral force resisting elements.

Practical Issues for Damper Implementation

One of the controlling factors in any retrofit design is the vertical (tie down) forces at each end of the new lateral force resisting element. The upward vertical component must be resisted by the tributary weight of the building and the downward forces must be resisted by the bearing capacity of the foundation. One of the major concerns in developing an acceptable seismic retrofit is to provide adequate resistance to the vertical component of the forces in any diagonal braces. Besides the obvious issue of resisting overturning of the bracing system, it should be noted that when the vertical component of the forces in the damper system is resisted by a relatively flexible beam, the effective stroke of the horizontally installed damper is reduced by the vertical deflection of the beam multiplied by the co-tangent of the angle of inclination of the brace (relative to the horizontal) (see Figure 4). When the damper is installed in a diagonal brace, the effective stroke of the damper is decreased by the vertical beam deflection divided by the sine of the angle. Such issues can be addressed by: 1) Using a chevron brace system that applies no net vertical force to the beam, 2) Designing diagonal braces so that they intersect at beam-column connections, 3) Ensuring that the beam is stiff enough that the vertical deflection is insignificant, 4) Increasing the stroke of the damper to allow for the horizontal displacement due to the vertical deflection of the beam caused by the vertical component of the diagonal force, and 5) Using a shear panel framed with metal and incorporating a viscous damper to absorb energy (see Figure 4).

For a damper installed in a diagonal brace, as the angle of damper inclination (relative to the horizontal) increases, the force in the damper increases, the displacement in the damper decreases and the vertical component of the force increases. This increases the required capacity and cost of the dampers and may require more dampers to limit the vertical "tie down" force to that which the building weight and/or foundation capacity can resist. At some point (with increased damper inclination angles), it becomes necessary to incorporate the damper within a displacement amplification system (e.g., via toggle-bracing or scissor-jack bracing [6,7]) to amplify the displacement in the damper and thus to optimize their effectiveness relative to their costs (see Figure 4 for concepts for providing displacement amplification). Of particular concern with narrow damper panels is the aforementioned issue of the vertical force component at the ends of any diagonal bracing.

San Francisco has a large stock of tall, narrow residential structures which have a street facing garage, making them highly vulnerable to torsional seismic response. The two side walls and the back wall of the building are generally stiff while the ground level wall with the garage doors has almost no stiffness. This asymmetry in stiffness distribution can be compensated for by adding damper panels, although the only available space for installing the dampers is within the narrow panels on either side of the garage doors. These panels are commonly only 24 inches wide by 80 inches tall, making it difficult to utilize a standard toggle brace (see Figure 4) since the angle of inclination becomes too large. One approach to addressing this situation is to increase the stiffness of the columns on either side of the damper panel and provide greater multiplication in the linkage. Increased column stiffness is provided by either adding steel reinforcement to the wood columns or by adding a light gage steel moment frame. Figure 4 shows a number of concepts for increasing the damper stroke with a multiplication linkage.

Another approach is to install garage-width damper panels at the back of the car stalls and parallel to the open side (see Figure 3). These panels would be capable of resisting the lateral loads from the front of the building to midway between the new panels and the next lateral force resisting element toward or at the rear wall. In addition, damper panels would be installed between each parking stall to resist torsion.

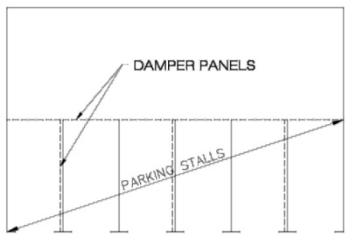

Figure 3. Plan view showing possible damper panel installation

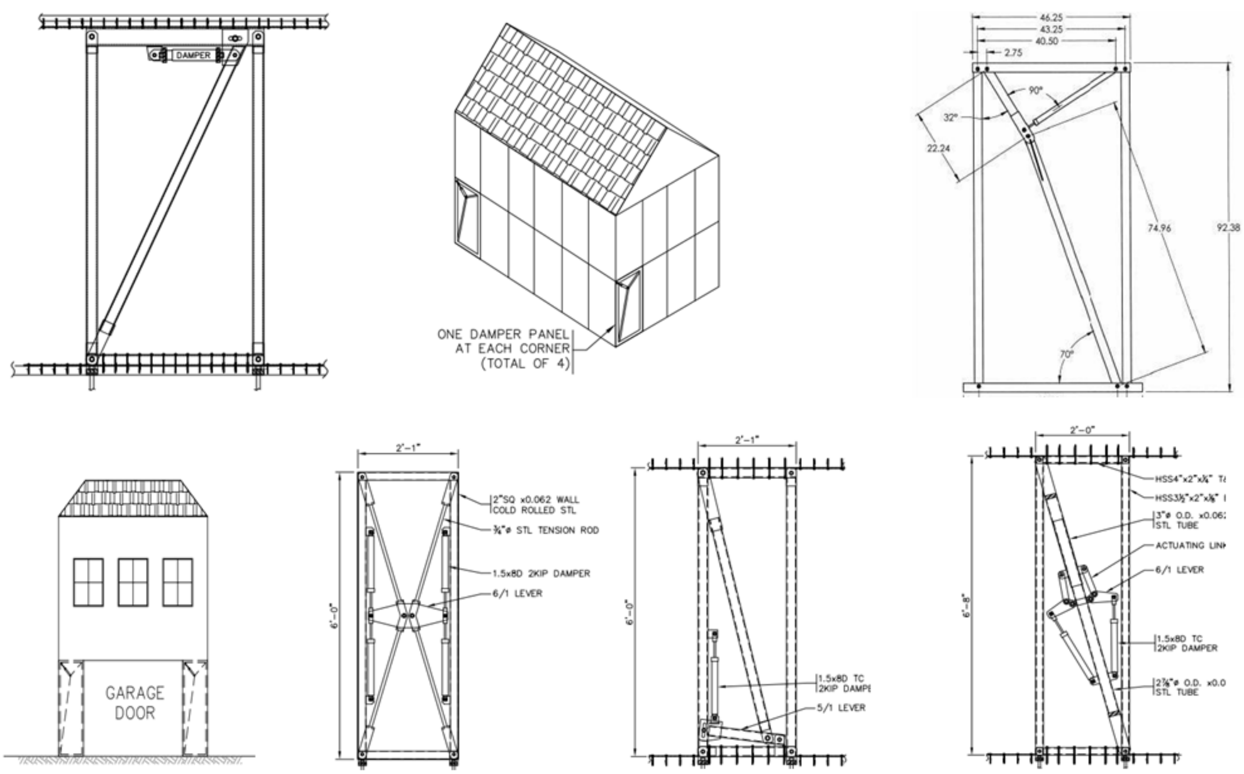

Figure 4 Possible damper panel configurations for amplifying damper displacements in narrow wall panels.

Conclusions

This paper presented an approach to protecting Soft Weak Open Front (SWOF) buildings via application of an energy dissipation system (viscous dampers) to the ground story, the objective being to limit peak inter-story drifts to about 1% under relatively severe seismic events, thus keeping the deformations largely within the elastic range. With this addition of damping, earthquake survivability of this class of structures increases significantly. Based on the analysis presented herein, it is evident that installing such a system in the building can significantly increase the ground acceleration required to reach the limiting force capacity of the second story as compared to the use of a strength-based retrofit. Furthermore, practical issues related to implementation of such systems were discussed along with an illustration of potential installation configurations for narrow wall panels.

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