Performance-Based Design of an Essential Hospital with Supplemental Viscous Damping in a High Seismic Zone

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SUMMARY:
Since 1973, California has mandated that "hospitals …which must be completely functional to perform necessary services … after a disaster, shall be designed and constructed to resist, insofar as practical, the forces generated by earthquakes…" With advances in performance-based design methodologies, nonlinear response history analysis, and supplemental damping, engineers can better meet this legislated mandate.

This paper describes the analysis of a 94,000 sq. meter, high-rise urban hospital in California using steel moment resisting frames with supplemental damping devices to control drift. Full-scale test results of Viscous Wall Dampers and steel beam-column moment resisting connections are used to validate the Non-Linear Response History analysis assumptions. The building seismic performance is evaluated using 10 strike-slip ground motion records.

Keywords: damping, viscous, hospitals, NLRH.

1. INTRODUCTION – PERFORMANCE-BASED DESIGN FOR HOSPITALS IN CALIFORNIA

Since the 1971 Sylmar Earthquake (M 6.6), the State of California has regulated the design and construction of California hospitals as essential facilities. The Hospital Facilities Seismic Safety Act (1973) provided a very general performance goal, albeit incomplete, that required hospitals "must be reasonably capable of providing services to the public after a disaster" and that these facilities must be "designed and constructed to resist, insofar as practical, the forces generated by earthquakes, gravity and winds (State of California, California Statues, 1973)." Until 2002, the regulations to meet such a performance goal were generally based on the Uniform Building Code (ICBO) with amendments developed by the Office of Statewide Health Planning and Development (OSHPD), the state agency responsible for the enforcement of the Act. Since 2002, the regulations have been based on the International Building Code (IBC) and its reference standards.

For most hospitals designed since the passage of the Act, meeting the performance goal was predicated on meeting prescriptive seismic requirements the Code. These prescriptive regulations were generally established as a more stringent set of rules than those required for ordinary occupancy buildings. The prescriptive rules were based on common standards of practice, using linear elastic analysis techniques with pseudo-seismic force levels to design a facility that would respond in the inelastic domain when subjected to real seismic forces with the expectation that the hospital building would remain in service after a major earthquake. This prescriptive design standard employs conventional building systems, increased design forces, and decreased allowable drift levels in conjunction with detailed plan review and construction inspection practices.

The new 555-bed hospital, shown in Figure 1 is 15 stories above grade with two below-grade levels for parking. From the foundation level to Level 5, the building footprint occupies the entire site, which measures 117 meters by 84 meters. The building plan area reduces above Level 5, and reduces again above Level 7 to form the nursing bed tower. The building roof is approximately 76 meters above sloping grade. Floor-to floor heights vary from 4.4 meters in the upper half of the building to 5.2 meters in the lower half in order to accommodate the large utility requirements including mechanical ductwork, piping, conduits, etc., above the ceilings. The site is located in San Francisco, approximately 11 kilometers from San Andreas Fault and 18 kilometers from the Hayward Fault.

The lateral force resisting system above grade consists of steel moment resisting frames with 153 supplemental Viscous Wall Dampers, provided by Dynamic Isolation Systems, Inc. In order to analyze and design this system, full-scale prototype testing of the viscous wall dampers and steel moment frame connections were required.

Figure 1. Architectural rendering (courtesy of SmithGroupJJR)

2. PERFORMANCE-BASED DESIGN REQUIREMENTS

For an essential acute care hospital, the lateral-force resisting system must meet Immediate Occupancy performance goals for the Design Earthquake and Life-Safety performance for the Maximum Considered Earthquake. These goals are not prescriptively defined in the California Building Code and must be inferred from various requirements and descriptions provided in available guidelines such as Chapter 1 of ASCE 41 – *Seismic Rehabilitation of Existing Buildings.*

Given the proximity of the new hospital to the San Andreas Fault and the overall height, massing and floor-to-floor heights of the building, it was very apparent that it would difficult to meet the Code maximum allowable interstory drift of 1% for an essential building (equal to one-half the allowable interstory drift of a normal occupancy building) without providing a very significant, stiff lateral load resisting system. Stiff lateral systems such as braced frames or concrete shear walls reduce the program flexibility to change functions in the hospital throughout the life of the building. Therefore, we proposed a supplemental damping system with a steel moment resisting frame system; the supplemental damping to reduce interstory drift and steel moment resisting frames to provide seismic strength.

A project-specific Design Criteria document was developed combining information from:

1. California Building Code (based on the 2006 International Building Code)
2. ASCE 7-05 – Minimum Design Loads for Buildings and Other Structures, Chapter 16 – Seismic Response History Analysis, Chapter 18 – Seismic Design Requirements for Structures with Damping Systems, and
3. ASCE 41-06 Seismic Rehabilitation of Existing Buildings, Chapters 2 – General Requirements, Chapter 4, Foundations and Geologic Site Hazards, Chapter 5 Steel.

Nonlinear element modeling parameters from ASCE 41 were used for the modeling and acceptance criteria for the steel moment resisting frame including girders, columns and column panel zones. Foundation spring elements were developed from geotechnical information and ASCE 41, Chapter 4. An independent peer review panel reviewed the project Design Criteria and its implementation throughout the process.

3. GROUND MOTIONS

In order to evaluate the nonlinear response history performance of lateral force resisting system, the owner's geotechnical engineer, Treadwell & Rollo, developed site-specific horizontal response spectra and associated ground motion time histories to represent the Maximum Considered Earthquake (MCE). The MCE is defined as the lesser spectrum of the probabilistic seismic hazard analysis (PSHA) for the 2% probability of exceedence in 50 years (2475 year mean return period) or the 84% percentile of the deterministic earthquake from the governing fault. (Both spectra are based on the maximum rotated ground motion component.) Using EZFRISK 7.26, site-specific spectra with the maximum rotated component were developed using four Next Generation Attenuation (NGA) relationships (PEER) and modifications provided by OSHPD for use in hospital design. The MCE and DE spectra are shown below in Figure 2.

For the deterministic earthquake, the governing earthquake scenario was the San Andreas event with a moment magnitude, M_w, of 7.9 occurring at 11.1 km from the site.

The project geotechnical engineer provided ten ground motion records, shown in Table, 1 for amplitude scaling to develop site specific ground motions at the MCE level. Design Earthquake time histories were developed by scaling the MCE by a 2/3 factor.

Each ground motion record included two orthogonal horizontal components. In applying the records in the Non-Linear Response History (NLRH) analysis, five records were applied with the stronger component in the North-South direction while the other five records were applied with the stronger component in the East-West direction. By using more than seven ground motion records, the Code permits establishing the design member forces, Q_{Ei}, and design interstory drift values, Δ_i, as the average of the maximum values obtained from each response history.

Table 1. Selected Earthquakes to develop site specific ground motions

	Earthquake	Mag.	Time History	Directivity	Epicentral Distance (km)	Closest Distance to Rupture (km)	MCE Scale Factor
1	Loma Prieta	6.9	Los Gatos PC	Neutral	23	6	0.88
2	Denali	7.9	Pump Sta. #10	Backward	85	3	0.69
3	Landers	7.4	Yermo	Forward	84	25	1.81
4	Landers	7.4	Joshua Tree	Backward	15	12	2.07
5	Duzce	7.1	Duzce	Forward	2	17	1.03
6	Kocaeli	7.4	Gebze	Backward	50	17	1.80
7	Kocaeli	7.4	Duzce	Forward	90	13	1.40
8	Imperial Valley-1979	6.6	Holtville	Forward	20	8	1.52
9	Manjil	7.4	Abbar	Forward	40	13	1.58
10	Imperial Valley-1940	7.0	El Centro	Neutral	13	6	2.53

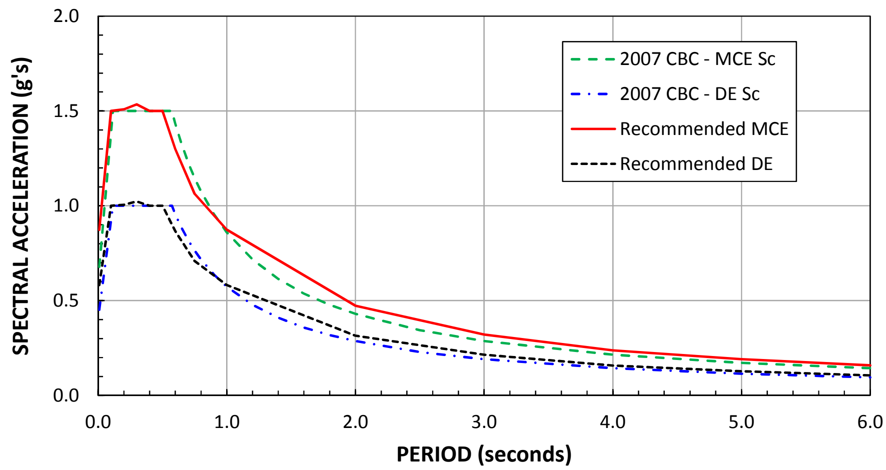


Figure 2. MCE and DE spectra

4. SUPPLEMENTAL VISCOUS DAMPING

4.1. Introduction

Viscous Wall Dampers (VWDs), originally developed and used in Japan, consist of a stiffened steel tank filled with viscous "fluid" and one or more steel vanes that extend into the tank of viscous fluid. The VWD tank connects to a girder within a steel moment resisting frame line at one level and the vane assembly to the girder at the level above (see Figure 3). Interstory drift results in in-plane displacement and velocity of the vane within the tank. Viscous shear of the fluid relative to the tank wall and vane provides damping and energy dissipation (Newell, et al. 2011).

4.2. Damper Model

The damper model consists of a linear spring in series with a nonlinear damper (Maxwell Model). This damper model was implemented with the PERFORM-3D viscous bar element made up of a fluid damper and linear elastic bar component. The damper force versus velocity relationship was represented by Equation 4.1 and an additional maximum force cap.

$$F = CV^{\alpha} \tag{4.1}$$

where: F = damper force, C = damping coefficient, V = damper velocity, and α = damper velocity exponent. The damper velocity exponent, α = 0.7, was selected based on typical Japanese practice and fit of the damper model to the prototype test data. The force versus velocity relationship is modeled with a multi-linear discretization. The force cap, or maximum, was based on observations from prototype testing.

The compound viscous bar element (dashpot and linear elastic spring) was located at story mid-height and is connected to the beam above and below with rigid vane and tank elements. Interstory drift causes relative displacement between the two damper nodes and activates the viscous bar element. Stiffness of the VWD vane and tank components was included in the dashpot-spring damper model properties that were determined based on the results of prototype testing. Therefore, the component properties of the elastic vane and tank elements in the model were considered to be essentially rigid.

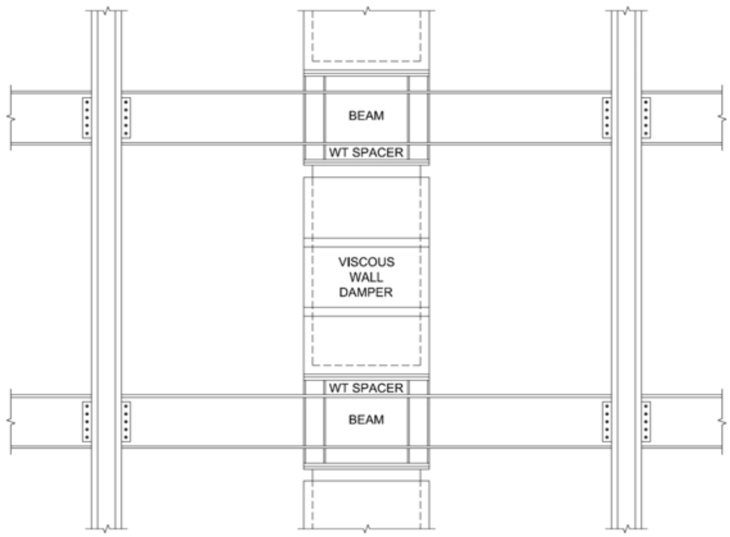


Figure 3. Frame and Viscous Wall Damper elevation

4.3. Damper Property Variation

Damper response is affected by variations in damper properties due to 1) first-cycle effect, 2) temperature variation, 3) aging, and 4) specification tolerance. In order to account for the expected range of building response due to these variations, an upper-bound and lower-bound damper properties analysis scheme was adopted. The nominal damper properties determined from prototype testing were modified by a lower-bound or upper-bound damper property modification factor. Displacement-based demands were typically controlled by analysis with lower bound damper properties while force-based demands were controlled by upper bound damper property analysis. Combining the four modification factors resulted in lower-bound damper properties about 75% of nominal properties and upper-bound damper properties between 190% and 220% of nominal properties, depending on the size of the damper. The upper bound properties modification factors are dominated by the first-cycle effect. This increase was necessary to determine the maximum forces that would be generated by the damper on the girders that supported the dampers. For both MCE and DE level events, analyses were run for upper and lower-bound damper property assumptions. Nominal property analysis was not generally used as this case did not govern strength or displacement limits states.

4.4. Cyclic Load vs. Deformation Response

As observed in Figures 4 and 5 below, VWDs typically exhibit a higher force capacity on the first excursion to large displacements (cyclic-history dependence). This first-cycle phenomenon is not captured by the damper model implemented in PERFORM-3D. A first-cycle effect upper-bound damper property modification factor (>1.0) is used to ensure the structure is adequately designed for the increased force demand resulting from the first-cycle effect. To establish the appropriate values for first-cycle factors maximum positive and maximum negative force data from prototype testing was compared with maximum values from NLRH simulation of the tests. The maximum value of the test force divided by NLRH simulation force was used to establish the first-cycle factor for a given damper size and earthquake demand level.

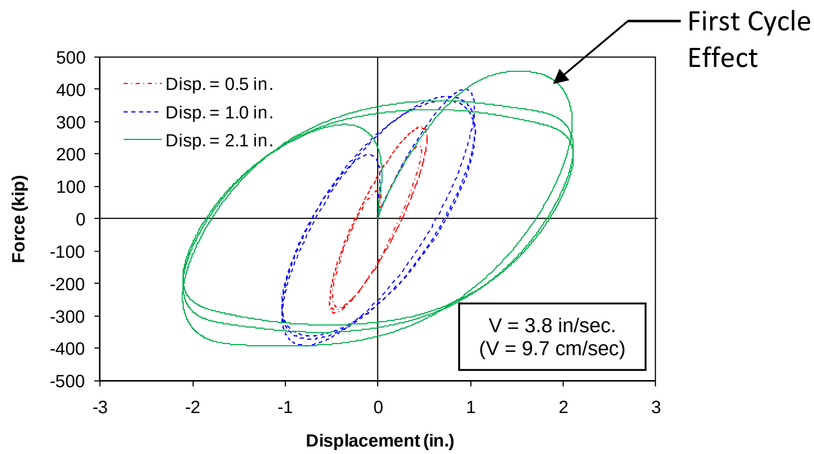


Figure 4. Displacement-dependent cyclic load vs. deformation response

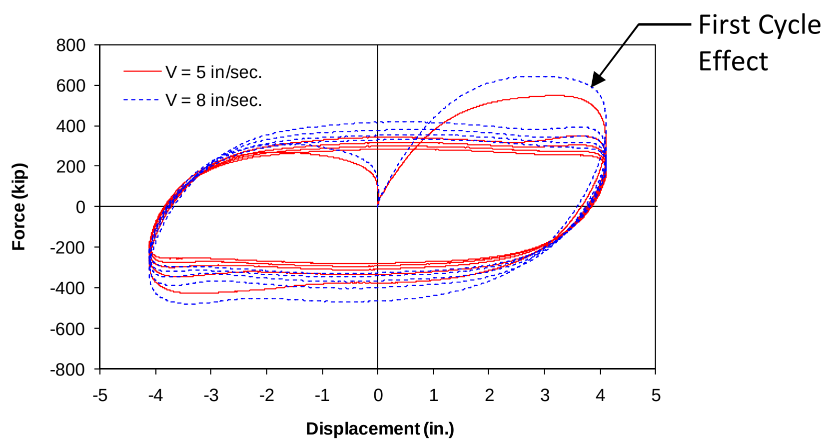


Figure 5. Velocity-dependent cyclic load vs. deformation response

5. STEEL MOMENT RESISTING FRAME

5.1 Introduction

As a result of the 1994 Northridge Earthquake and lessons learned regarding the design and construction of Special Moment Resisting Frames (SMRF) connections, the use of these connections has been limited to those connections that have been shown through full-scale testing to provide predictable strength and ductility at interstory drifts of 4%. Based on substantial testing and research, there are a number of moment connection types now used in SMRF systems. However, in order to use these connections, it is required to show through full-scale testing that the connections meet the performance based requirements. Based on previous testing results, a number of connection types for a limited number of girders are considered as Prequalified Connections (ANSI/AISC 358) and can be used without additional testing. Typical limitations include that the girder size may not exceed 223 kg/m (150 lbs./ft.), flange thickness may not exceed 25mm (1 in.), and the clear span-to-depth ratio must be 7 or greater for special moment frame systems.

With the viscous damper located at mid-span of the moment frame bay, the girders are subjected to large flexural moments and large axial collector forces simultaneously. The usual assumption of a point of inflection at mid-bay was not applicable; the girder segment on each side of the damper can develop a double-curvature deformation pattern. The Welded Unreinforced Flange – Welded Web (WUF-W) connection as outlined in FEMA 350 was chosen in order to maintain large axial force capacity in the presence of large flexural forces at the face of the column. (The WUF-W connection is now published in AISC 358-10 as a prequalified connection. This document was in progress during the design stages of this project.)

The girder sizes necessary for this project exceeded weight, flange thicknesses limitations for prequalified connection consideration. Additionally, the span-to-depth ratio, varying between 4.2 to 5.6, did not comply with the minimum span-to-depth value of 7. Therefore, three full-scale prototypes were tested at U.C. Berkeley to validate the strength, stiffness, and ductility assumptions used in the NLRH model.

The primary sources of inelastic behaviour in the moment connection included the beam-column panel zone and the beam plastic hinge zone located close to the column face. The NLRH model included nonlinear panel zone elements based on Krawinkler model (3D PERFORM Components and Elements Manual) with initial elastic stiffness, yield strength and post yield stiffness and ductility assumptions. Strength and stiffness values were calculated using the full depth of the column. Moment hinge strength, stiffness and ductility assumptions were based on ASCE 41 Modeling Parameters and Acceptance Criteria for Nonlinear Procedures (Table 5-6).

Acceptance criteria for the prototype connections were based off the results of the NLRH analyses. Connections must be capable of sustaining

1. Two complete cycles at the MCE interstory drift without strength degradation
2. Two complete cycles at 150% MCE interstory drift without complete failure of the connection

5.2 Prototype Test Results and Conclusions

The prototype tests indicated that the WUF-W connections chosen for this project met the required performance goals based on the NLRH analysis. Furthermore, the overall connections performed better than assumed in the NLRH analysis based on the ASCE 41 modeling parameters. One primary difference between the ASCE 41 modeling parameters and the test results was the performance of the panel zone compared with the analytic model. The panel zone analytic model was stiffer and stronger than the tested specimens. Figure 6 shows the actual test results for a panel zone with the predicted results plotted for comparison. Based on the test results, modified strength and stiffness parameters were back-calculated to better match the experimental results. Figure 7 shows the originally assume backbone curve and the revised curve. It is clear that substantial additional inelastic deformation occurred in the test specimen than predicted using the original Krawinkler model for the NLRH analysis.

This differences are likely related to the analytic assumption that the full depth of the column web, d_c, participates in the strength and stiffness of the panel zone. This assumption may be reasonable for columns with relatively thin flanges, less than 2.5 cm. (Kim and Engelhardt 2002). For these heavy column sections, the flanges were significantly larger, reducing the depth of the panel zone.

Based on comparisons between the actual tested connection behaviour and the NLRH analytic models of the test specimens, we concluded for all connection specimens:

1. The initial stiffness and yielding strength were overestimated in the original NLRH model.
2. The ductility and residual strength were underestimated by the original NLRH model.
3. The initial elastic stiffness, yield strength and post-yield stiffness were overestimated in the original NLRH model.
4. The energy dissipated by the test specimens was underestimated by the original NLRH models. Significantly greater energy was dissipated in the panel zones of the prototype test specimens than predicted by the models.
5. The increased inelastic behaviour of the panel zone decreased ductility demand in the girder hinge zone.

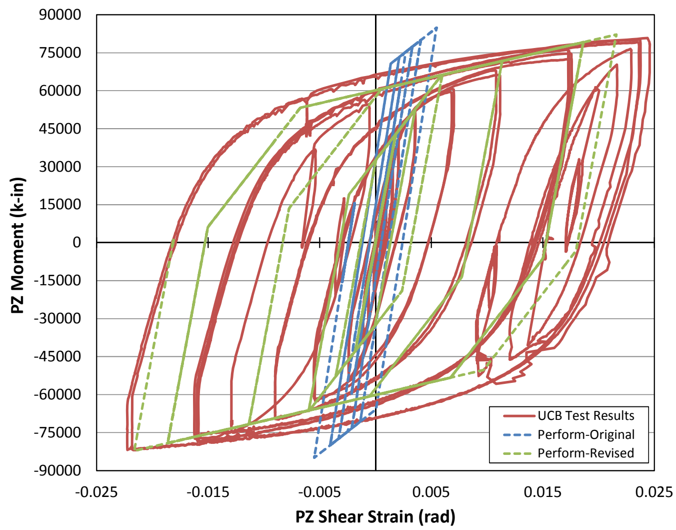


Figure 6. Panel zone moment vs. strain – comparison of test results with original and revised models (Spec. 1)

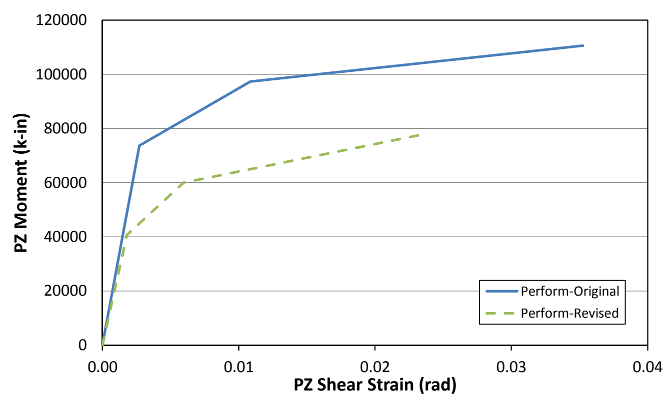


Figure 7. Original and revised analytic panel zone backbone curves
[Spec. 1, W14x550 column with (2) 1-1/4” doubler plates]

5.3 Global Response Based on Prototype Test Results

While the nonlinear modelling of individual connections required substantial modification to produce analytic predictions that better matches the test results, the overall global response was not sensitive to these local connection modelling revisions. Comparison of 2-D frame NLRH models with original connection modelling assumptions and frame models with revised connection modelling indicates very little change in interstory drift as shown in Figure 8. Figure 9 shows the change in energy dissipation between the various elements due to the revised modeling of the connections. The revised model indicated some improvement in global drift behaviour due to the enhanced ductility provided by the panel zones and girder moment hinges. This enhanced ductility modelling resulted in fewer cases of predicted strength loss for some time history records that contributed most to the drift prediction.

The supplemental Viscous Wall Damper with a steel moment resisting frames provides an efficient lateral load resisting system by reducing interstory drift and floor accelerations. As shown in Figure 10, at MCE level ground motions, the interstory drift is less than 2%. Floor accelerations remain fairly constant up the height of the building in contrast to the expectation for increasing floor accelerations with building height.

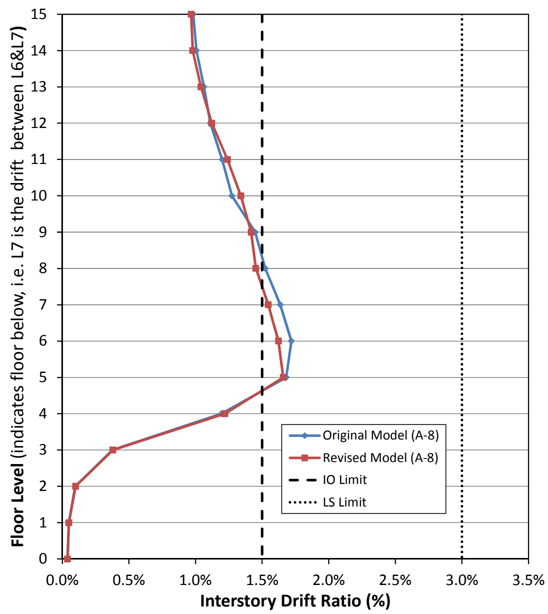
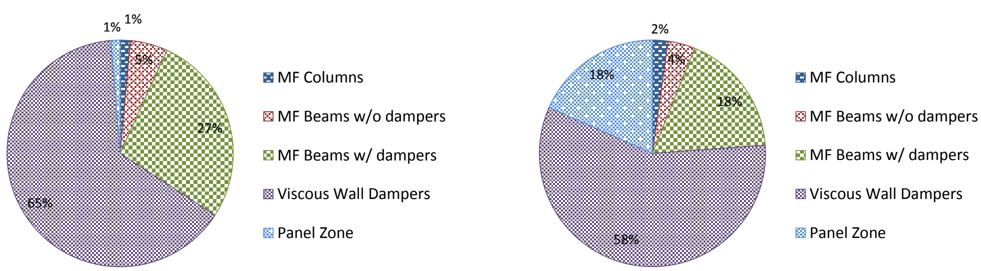


Figure 8. Comparison of interstory drifts with original and revised connection modeling (2D Line A model, lower bound damper properties, average of 10 MCE records)



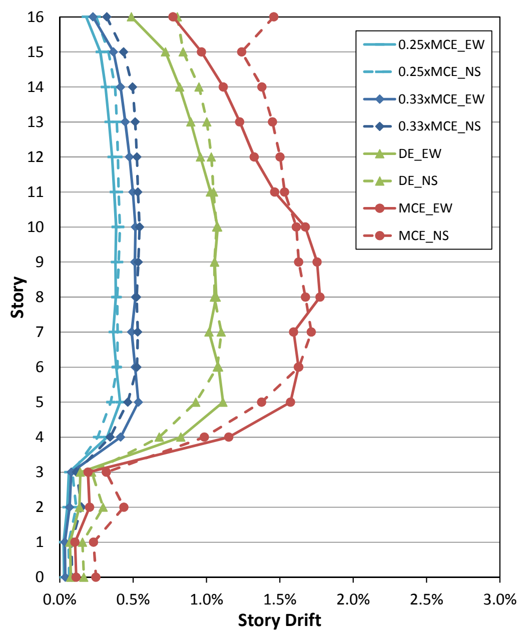
(a) Original model

(b) Revised model

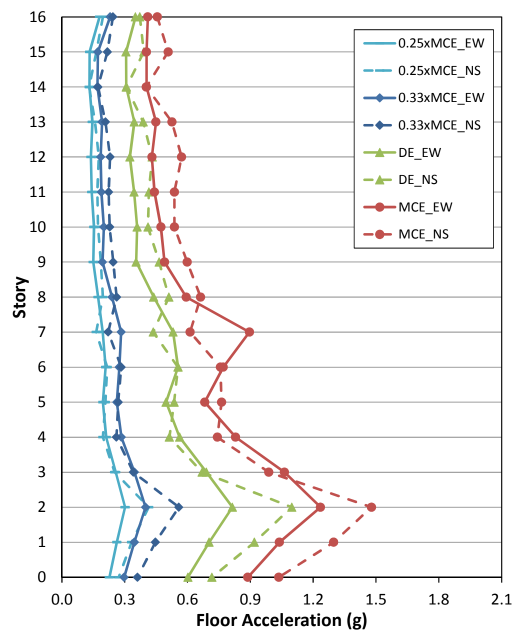
Figure 9. Comparison of energy dissipation by element group with original and revised connection modeling (Record E05A)

6. CONCLUSIONS

Using supplemental damping within a moment-resisting frame system, combined with site-specific ground motions and NLRH analysis techniques, it is feasible to design a structurally efficient essential facility such as a hospital in areas of high seismicity. The efficiency of the designed structural system is evidenced by the reduced steel tonnage of the building. The seismic force resisting system for the building weighs approximately 54 kg/m^2 (11 psf) of floor area and the total steel weight is approximately 93 kg/m^2 (19 psf). This essential facility meets the Performance goals of Life-Safety at the MCE and Immediate Occupancy at the Design Earthquake. The Viscous Wall Dampers, newly introduced to the US by the project team, provide a superior solution to the conventional Special Moment Resisting Frame (SMRF) by dissipating considerable amounts of energy that would otherwise be dissipated by the steel framing elements as permanent damage, and permit fewer and lighter elements to be used. The combined cost of the moment frame plus damping devices was estimated to be less than that of a much heavier conventional SMRF designed to meet the same code requirements. The dampers also reduce seismic floor accelerations that would otherwise damage contents and disrupt hospital operations in the critical post-earthquake period. The completed structural design therefore occupies the rare solution space described simply as "both better and cheaper".



(a) Story drift



(b) Floor acceleration

Figure 10. Average interstory drift and floor acceleration for various earthquake levels (lower bound damper properties)

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