SEISMIC REHABILITATION OF PRE-NORTHRIDGE STEEL MOMENT FRAME BUILDINGS INCORPORATING FLUID VISCOUS DAMPERS

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ABSTRACT

This paper discusses the seismic retrofit of 2-5 story steel moment frame buildings possessing Pre-Northridge beam-column connections. As part of a complete campus building remodel and renovation project in the San Francisco Bay Area, which included one 2 story, one 3 story and two 5 story steel moment frame buildings, our task was to review the seismic vulnerability of the existing buildings relative to the known deficiency of the Pre-Northridge beam-column connections, and develop a campus-wide rehabilitation scheme for mitigating the deficiencies.

As part of the campus expansion, the client desired to add approximately 75,000 gsf between two existing buildings, hereinafter referred to as the connector building. Initial discussions with the architect and owner indicated the desire to have the connector building possess an extremely high volume space between the 1st and 3rd floors. Additionally, diagonal bracing was not desired in the connector, thus representing a likely LFRS for the connector consisting of either interior hidden shear walls or very heavy steel moment frames along the building perimeter. With either system, it would be necessary to have very large seismic separations between the existing buildings and the new connector building. Additionally, both scenarios were cost-estimated and, with such high floor-to-floor heights, deemed too costly.

Rather than constructing a building with both large seismic separations and a costly LFRS, the three buildings were attached, thus eliminating the seismic joints, and possibly reducing the cost of the LFRS associated with the connector. The concept of incorporating Fluid Viscous Dampers (FVD's) as part of the selected seismic rehabilitation of the existing buildings was introduced, which would mitigate building drift, and therefore joint rotation. By adding the connector building and attaching it to the existing buildings, we increased the mass beyond that of the original two buildings by approximately 45% and reduced overall building drifts by approximately 40-60%.

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Introduction

As part of a complete campus building remodel and renovation project in the San Francisco Bay Area, which included one 2 story, one 3 story and two 5 story steel moment frame buildings, PARADIGM was selected to provide structural consulting to assist their client in attaining the client's desired post-earthquake performance objectives while meeting very stringent budgetary constraints. PARADIGM's task was to review the seismic vulnerability of the existing buildings relative to the known deficiency of the "Pre-Northridge" beam-column connections and develop a campus wide rehabilitation scheme for mitigating the deficiencies. Although the "Pre-Northridge" moment frame connections have been determined to possess greater capacity than once believed, and repair of the connections consistent with FEMA (FEMA 351, 2000) recommendations improve the connection performance, such repair does little to improve overall building performance. Thus, when considering the cost associated with removal of architectural finishes, removal of fire proofing finishes and subsequent replacement of both, only to achieve a limited building performance, alternative schemes for seismic rehabilitation were explored and implemented.

The performance objective selected by the owner for this building was to attain a "Life Safety" performance objective under a design basis earthquake for the existing Buildings B1 and B2, with a similar performance objective for the new connector building. Repair of the "Pre-Northridge" moment frame connections consistent with FEMA (FEMA 351, 2000) could provide the desired performance objective as defined by FEMA for the existing buildings, however, the existing Buildings B1 & B2 would not meet current drift limits as specified in the CBC (ICBO, 2001) for a "Life Safe" condition for a new building. Additionally, the combination of the "high bay" first floor of the connector building combined with the desire of the owner to eliminate diagonal bracing and/or shear walls as part of the new construction, was leading to a relatively costly LFRS for the connector building. The addition of the new connector building represents an increase of approximately 45% to the overall Buildings B1 & B2 mass. Such increases in building mass can be a major design obstacle for traditional LFRS in regions of high seismicity, especially when increasing mass to a structure or structures which are drifting in excess of 4%. The incorporation of FVD's as part of the LFRS can be a very efficient way of reducing such significant building drifts. The 1999 SEAOC Blue Book states that when incorporating FVD's, the LFRS shall be designed for strength requirements only, while it places no limitation on drift demands (SEAOC, 1999). The FVD's are added to control building drifts to acceptable levels, which, in turn, both meet the desired performance objective while also reducing the joint rotation of the previously deficient joints of the existing buildings. This case study showed that this design philosophy is a very effective way of adding mass and at the same time, reducing building drift to allow the previously "deficient" joints to perform to the desired performance objective.

Building Description

The building is a composition of 3 buildings. See Figure 1. Buildings B1 & B2 possess a floor plate of 25,600 ft.^{2} (2378 m^2) with 5 floors totaling 128,000 ft.^{2} (11890 m^2) per building, or 256,000 ft.^{2} (23,780 m^2) between the two buildings. The connector building possesses a floor plate of 18,000 ft.² (1674 m²) per floor with 5 floors totaling approximately 75,000 ft.² (6967 m²) for a composite Building 12C of 331,000 ft.² (30,751 m²). As previously mentioned, the addition of the connector building increased the original Building B1 and B2 mass by approximately 45%. Floor-to-floor heights are typical office building floor to floor heights of 14'-0".

Exterior column spacing is typically 15'-0" on center which we will define as a "half bay", thus a full bay will be a bay consisting of 3 columns and 2 diagonals. The existing building and new building foundation system consists of a pre-cast concrete driven piles extending down approximately 40'-0". The LFRS consists of 16 three story and 4 single story total bays of dampers for a total of 104 dampers. The dampers selected were all designed as 200 kip dampers with varying damping coefficients. The dampers were incorporated in the axis of the diagonal elements which created an "inverted chevron" appearance at some perimeter bays of Buildings B1 & B2. As indicated in the attached table, the fundamental period of the original Buildings B1 & B2 are 2.0 seconds with a code prescribed base shear of 0.13g, while the combined Building 12C period of 1.0 seconds and a corresponding code prescribed base shear of 0.11g.

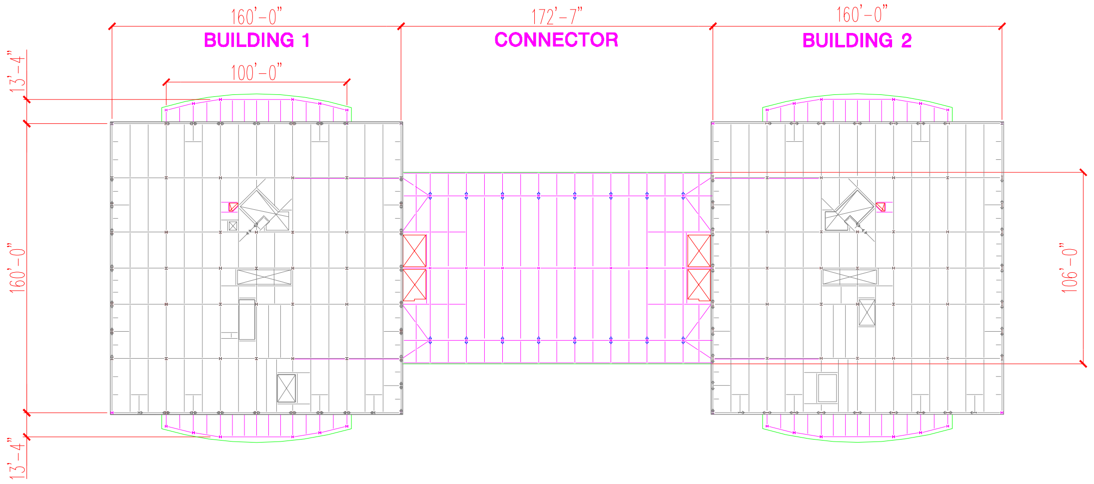

Figure 1. Building plan.

Retrofit Scheme

The desired performance objective for the composite building was selected as a "Life Safety" performance objective consistent with the requirements of FEMA (FEMA 273, 1997. FEMA 351, 2000), as well as the 2001 California Building Code (ICBO, 2001).

Fluid viscous dampers were chosen over more traditional LFRS to achieve this objective for a variety of reasons. When working with an existing building, “stiffening” the building in a conventional manner, such as would be the case with concrete shear walls or diagonally braced frames, increases the force delivered to the building, and thus the collectors and the associated connections require significant alterations. In order to ensure mobilization of the dampers, PARADIGM elected to ensure little to no yielding of the foundation system by supplementing the original foundation with site cast auger piles. Increasing the building stiffness further would have significantly increased the overturning forces, which would have necessitated an even greater quantity of auger cast piles.

Input Time Histories

The building is located 12 km from the Peninsula segment of the San Andreas Fault and approximately 10 km from the South segment of the Hayward Fault. A site-specific probabilistic seismic hazard analysis (PSHA) was performed to estimate the magnitude of ground acceleration at the site. The PSHA modeled the faults in the Bay Area as linear sources and assigned earthquake activities to the faults. Site-specific spectra at the ground surface were estimated using stiff soil attenuation relationships consistent with the subsurface conditions encountered at the site. The DBE is defined as a 500 year return event (10% in 50 years) and the MCE is defined as a 2500 year return event (2% in 50 years) (FEMA 351, 2000). Spectral matching was performed to provide appropriate time histories for both DBE and MCE levels. A site specific response spectrum for a DBE - 5% damped system is shown in Figure 2. Time histories were selected based upon similarities in magnitude and distance to the target spectra. Both FEMA and the 1999 SEAOC Blue Book allow the designer to incorporate the maximum of the time histories should only a suite of 3 time histories be selected or the average of the time histories should a suite of 7 be selected (FEMA 273, 1997. FEMA 356, 2000. SEAOC, 1999). For this project, it was determined that it would be adequate and not overly conservative to incorporate a suite of 3 time histories. The suite of 3 time histories incorporated earthquakes in each principal direction for each level of seismic hazard. For the DBE, the maximum values for acceleration, velocity and displacement were incorporated into the design of the LFRS. See the Time History values given in Table 1 (Golesorkhi, 2004).

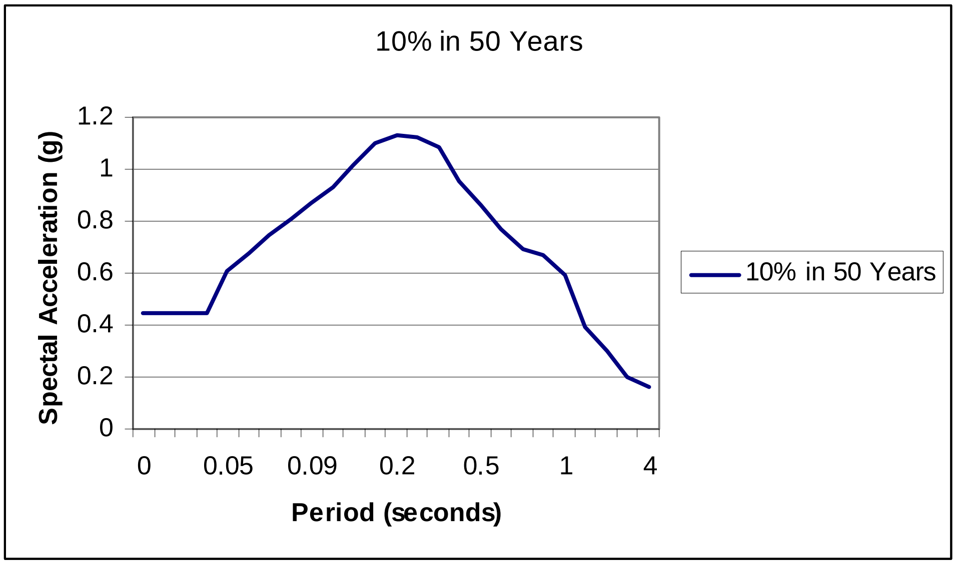

Figure 2. 5% damped spectral acceleration of DBE event

Table 1. Time history properties

Hazard Level	Earthquake	Magnitude	Time History	Component Relative to True North	Epicentral Distance (km)	Closest Distance to Rupture (km)	Peak Acceleration (g)
DBE	Loma Prieta	6.9	Los Gatos PC	0 deg.	23	6	0.470
				90 deg.			0.455
DBE	Landers	7.4	Yermo	270 deg.	84	15	0.414
				360 deg.			0.468
DBE	Kocaeli	7.4	Duzce	180 deg.	90	13	0.484
				270 deg.			0.492
MCE	Loma Prieta	6.9	Los Gatos PC	0 deg.	23	6	0.569
				90 deg.			0.566
MCE	Landers	7.4	Yermo	270 deg.	84	15	0.503
				360 deg.			0.564
MCE	Kocaeli	7.4	Duzce	180 deg.	90	13	0.573
				270 deg.			0.582

Design and Analysis Procedure

A bare frame model, based on the existing building story stiffness and existing building mass was developed in order to obtain an initial estimate of the required number of dampers simply for the existing building. A similar model was developed for the composite building, Building 12C. This determined the critical damper properties, including maximum force, damping coefficient and non-linear damping exponent, also known as the “alpha” (α) factor (although it was later determined that for the selected manufacturer, a linear damper was appropriate). Time History Analysis (THA) was used to determine the seismic demand on the structure. The analysis was performed on a trial and error basis, with a final result consistent with our desired performance objective for allowable building drift.

The bare frame model was then transformed into a two-dimensional model and analyzed in ETABS (CSI, 1999) using the damper properties from the previous model. The number of dampers was selected based upon the damping coefficient required to limit the building displacement to an acceptable drift. Additionally, the quantity of dampers was selected based upon a desire to provide a relatively uniform distribution of dampers throughout the buildings and in an attempt to limit the damper size to 200 kips under the DBE event. This 200 kip damper level was determined to be an economical damper size for this building. The building performance was then analyzed utilizing the suite of three DBE time histories to compare drifts with desired drift parameters. Although FEMA 351 (FEMA 351, 2000) prescribes a particular drift response parameter value, PARADIGM used a more stringent target of 1% drift during the analysis to facilitate a more elastic response. Additionally, although the performance objective desired was determined to be life safety resulting from a DBE, a brief comparison was made to ensure the performance objective of collapse prevention was attained under an MCE event. The beams and columns were modeled as liner elements and their demand to capacity ratios were determined and compared to the element demand modifier value to ensure elastic behavior was maintained under the DBE event.

Analysis Results

Based upon FEMA 351 requirements for an existing moment frame building incorporating FVD's, the building performance after upgrade is judged based on the two response parameters of inter-story drift and column axial forces (FEMA 351, 2000). For the design criteria of achieving life safety under a DBE event, the inter-story drift response parameter was approximately 2%. Building drifts for the upgraded building were less than or equal to 1.6%, with the maximum drift occurring at the 4th story. Drifts under the MCE event were also less than the design parameter for that level. For the design criteria of achieving collapse prevention under an MCE event, the inter-story drift response parameter was approximately 2.4%. Building drifts for the upgraded building were less than or equal to 2.0%, with the maximum drift occurring at the 5th story. The demand-to-capacity values for both columns and beams were less than the element demand modifier (m) values. Depending on the axial stress in the columns, the m value was between 2 and 6. The greatest ratio of column D/C ratio to m value was 1.9 to 2.3, or 0.85. The m value for beams was generally 6. All beam D/C ratios were less than 6. All deformation and force level results corresponded to a better than life safe performance level for the DBE level, and the LFRS remained essentially elastic throughout the DBE event.

In addition to upgrading the two existing buildings to achieve the desired performance level, the connector was successfully incorporated between the two existing buildings without requiring its own independent LFRS. See Table 2.

Table 2. Comparison of weight and displacements of existing and new buildings.

	Weight (kips)			**Maximum Displacement ‘X’ (Inches)**			**Maximum Displacement ‘Y’ (Inches)**		
Story	**Single Building 1 Or 2**	**Combined Building 12C**	**% Increase**	**Single Building 1 Or 2**	**Combined Building 12C**	**% Decrease**	**Single Building 1 Or 2**	**Combined Building 12C**	**% Decrease**
Roof	1025	2800	280	22.4	11.4	49.0	29.8	10.7	64.1
5TH	1950	6100	313	19.5	9.2	52.8	24.5	8.9	63.7
4TH	1950	6100	313	15.2	6.8	55.3	18.9	6.4	66.1
3RD	1950	6100	313	10.2	4.4	58.8	12.6	4.1	67.5
2ND	1950	5000	256	4.9	2.0	59.2	6.0	2.0	66.7

Recommendations regarding thermal movement as contained in AISC-LRFD Third Edition (AISC-LRFD, 2001) were considered and reviewed. Historical review of temperature fluctuations in the Santa Clara area indicate a maximum temperature variance within a given month of 26 degrees F with a maximum variance from January to July of 43 degrees F. AISC-LRFD Third Edition (AISC –LRFD, 2001) Figure 2-7, with a temperature variance of approximately 45 degrees, allows spacing between expansion joints for generally rectangular shaped buildings, of approximately 520 feet. Additionally, a 15% increase is allowed per note 2 which increases the maximum recommended spacing to approximately 600 feet. The maximum overall dimension of the subject building is 492 feet, and thus within an acceptable range so as not to require expansion joints in accordance with AISC.

Comparison to CBC-Designed Building

In order to validate that the composite building satisfied the design requirements for a new building in accordance with the CBC (ICBO, 2001), the force and displacement output of the composite Building 12C achieving a life safe performance objective under the DBE was compared to the force and displacement output of a building designed to current code. In other words, a comparison was made with the demand to capacity ratios DCR’s of the composite building under the DBE event, with the DCR’s of a hypothetical building under a CBC event. If the FVD system were not included, the size of the columns necessary to meet the drift limits provided by current code requirements would have been significantly larger. As it was, the existing columns in conjunction with the FVD’s possessed acceptable DCR’s along with achieving the desired drift limit. Thus, although the existing buildings recognized a substantial increase in mass, the overall building displacement was reduced by approximately 40% while still meeting the life safe performance objective.

Assuming the cost to retrofit a single moment frame joint to be on the order of $5000 per joint, and assuming that each existing building possesses approximately 240 joints per building requiring strengthening, the total cost of rehabilitation for just the existing buildings would have approached \$1,200,000.00. Additionally, due to the drift limits for new buildings, the approximate added steel to provide a LFRS for the connector building would be approximately 4#/ sq. ft. or \$300,000.00. Lastly, it is generally accepted that a seismic separation joint between two buildings can range in cost from \$175 per lineal foot to \$300 per lineal foot, depending upon the conditions requiring the joint. The total lineal footage of separation between the two buildings is 76 lineal feet x 2 x 5 levels + 75 lineal feet x 4 = 1060 lineal feet, at a cost of \$200 per lineal foot equates to \$212,000.00. Therefore, to attain a current code complying new building, the connector, along with marginally retrofitted existing buildings, buildings B1 & B2, would have cost a minimum of \$1.7M. The cost of the structural rehabilitation and that portion associated with the LFRS including the dampers was \$1.6M. Thus, for \$100,000 less, the owner received a building which possesses an increased value, a reduction in insurance premiums and a composite building which exceeds the life safe performance objective generally reserved for new buildings.

Conclusion

The incorporation of FVD's into an existing building possessing "Pre-Northridge" moment frame connections can be a very cost effective and practical method of reducing construction dollars, while enhancing the overall building performance. Additionally, as was done for this project, the addition of the FVD's in the existing buildings also reduced the construction costs for the new building by eliminating the necessity of having large seismic separations, and therefore seismic joints, in addition to allowing the existing buildings to provide the necessary LFRS for the new construction.

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