Seismic Rehabilitation of “Pre-Northridge” Steel Moment Frame Building to Achieve Immediate Occupancy

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ABSTRACT

This paper discusses the seismic retrofit of a steel moment frame building possessing “Pre-Northridge” beam column connections with the goal of achieving an “Immediate Occupancy” performance objective.

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, we designed a rehabilitation scheme which included Fluid Viscous Dampers. Our initial task was to rehabilitate the 2 story building, then address similar deficiencies in the other buildings and identify potential deficiencies. Several scenarios for mitigating the beam column joint deficiencies were presented which would improve the beam column joint performance, however, they would not significantly improve or enhance the buildings beyond mitigating the deficiency of the joints. Upon understanding the client’s expectation of the level of post earthquake performance desired for the 2 story building, we believed it to most closely resemble the “Immediate Occupancy” performance objective.

Although the building possesses “Pre-Northridge” moment frame connections, there is no code mandated seismic retrofit for the deficiencies, and thus, identification of the client’s desired “Performance Objective” was required. The 2 story building consists of 65,000 gsf floor plates for a total of approximately 130,000 gsf and incorporates open web steel joists and girders with non-concrete filled metal deck for the gravity system at the roof with similar framing at the second floor, however, concrete filled metal deck is used as at this level. Steel moment frames comprise the Lateral Force Resisting System (LFRS) which possess “Pre-Northridge” beam-column connections.

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 desired post earthquake performance objectives while meeting very stringent budgetary constraints. PARADIGM’s task was to review the seismic vulnerability of the existing

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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 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, alternative schemes for seismic rehabilitation were explored and implemented.

The desired performance objective selected by the owner for this building was to attain an "Immediate Occupancy" performance level under an MCE. The owner's desire for the equipment housed within the building to be fully accessible and functioning after a major event led to their decision to strive for an "Immediate Occupancy" performance. Although numerous discussions occurred in which the major difficulties in attempting to achieve "Immediate Occupancy" for this existing building under an MCE were identified, mainly due to the inability of existing non-structural components to perform in a similar manner without special attention and upgrading, the owner remained with the desire to achieve the stated performance objective for the existing building.

Consideration was given to repair of the "Pre-Northridge" moment frame connections consistent with FEMA (FEMA 351, 2000), however such rehabilitation of the joints, although beneficial, would not adequately limit building drift and thus not provide the desired performance objective as defined by FEMA for the existing building. Alternative schemes for reducing building drift and thus achieving the IO level under an MCE were considered. Such schemes consisted of the addition of Reinforced Concrete Shear Walls, Special Concentric Braced Frames – SCBF's, Buckling Restrained Braced Frames - BRBF's, Base Isolation and incorporation of Fluid Viscous Dampers – FVD's. All schemes had benefits as well as limitations which will be discussed later in the paper.

The incorporation of FVD's as part of the LFRS can be a very efficient way of reducing significant building drifts and thus allowing a structure to attain an enhanced performance objective. 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 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 building. This case study showed that this design philosophy is a very effective way of reducing building drift to allow the previously "deficient" joints to perform to the desired enhanced performance objective.

Building Description

The building is a 2 story structure, possessing an irregular plan shape floor plate of approximately 65,000 ft.^{2} (6,039 m^2) for a total building area of approximately 130,000 ft.^{2} (12,078 m^2). Floor to floor heights are 16'-0". The typical second floor framing consists of 2½" normal weight concrete fill over a 3” metal deck, supported by open web steel joists and girders. The roof consists of a non concrete filled metal deck supported by similar open web steel joists and girders. Steel moment frames were sporadically located along discrete areas at both the building perimeter and along two interior column lines. Refer to Figure 1.

Interior and Exterior column spacing at the moment frames are 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. Although moment frames existed along the building perimeter, there was a strong desire by the owner to limit the visual impact of the selected retrofit system on the exterior of the building. Therefore, although moment frames existed, in some locations along the perimeter we were discouraged from adding diagonal elements.

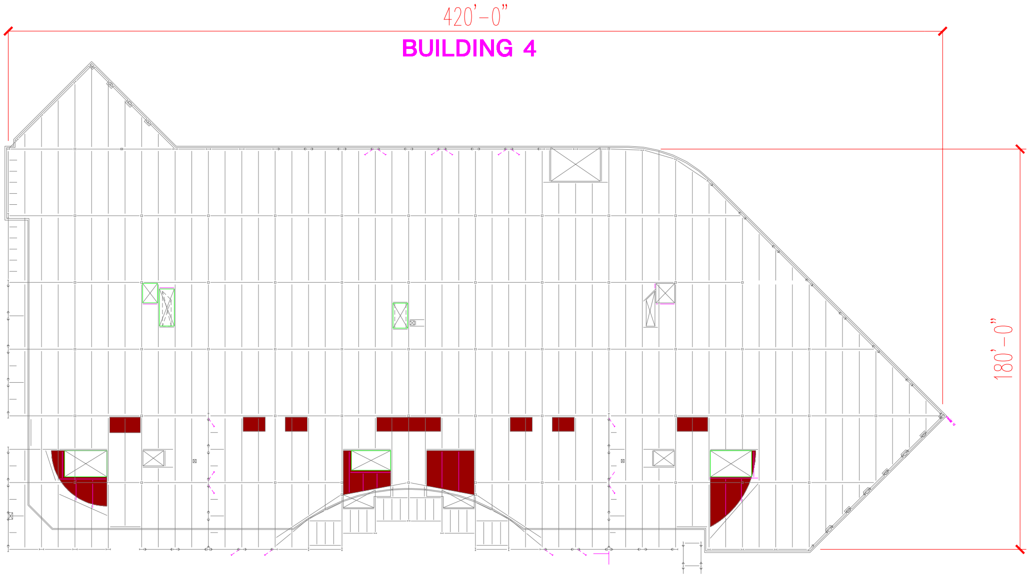

Figure 1. Building plan.

The LFRS consists of five 2 story bays, 2 single story bays and 2 single story “half bay frames” of dampers for a total count of 26 dampers. Due to the limitation of the possible locations for the dampers, the dampers selected were designed as 200 kip dampers from the second floor to the roof at all locations, 660 kip dampers along the longitudinal axis of the building from the ground to second floor and 985 kip dampers along the transverse axis at the two interior frames. All dampers have varying damping coefficients, however we maintained an α=0.3. The dampers were incorporated in the axis of the diagonal elements which created an “inverted chevron” appearance at some bays. As indicated in the attached table, the fundamental period of the original building was 0.5 seconds with a code prescribed base shear of 0.13g. The building, incorporating the FVD’s, possesses a period of 0.3 seconds and a corresponding code prescribed base shear of 0.31g.

Retrofit Scheme

Fluid viscous dampers were selected over more traditional LFRS 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, and thus the collectors and the associated connections require significant alterations. As the second floor possesses open web joists and girders acting as collectors, significantly altering the collectors would have been difficult and definitely not desired by the client. Additionally, the existing building and new building foundation system consists of a pre-cast concrete driven piles extending down approximately 40’-0”. In order to ensure mobilization of the dampers, we elected to ensure little to no yielding of the foundation system by supplementing the original foundation with additional pile elements. Helical piles were selected to enhance the existing pile foundation system. Increasing the building stiffness further would have significantly increased the overturning forces, which would have necessitated an even greater quantity of helical piles. Furthermore, the more conventional systems generally increase the level of damage building contents may see resulting from either a DBE or MCE.

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 356, 2000). Spectral matching was performed to provide appropriate time histories for both DBE and MCE levels. A site specific response spectrum for a 5% damped DBE 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 allows the designer to incorporate the maximum of the time histories should only a suite of 3 be selected or the average of the times 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 MCE, the maximum values for acceleration, velocity and displacement were incorporated into the design of the LFRS. See the Time History properties provided in Table 1 (Golesorkhi, 2004).

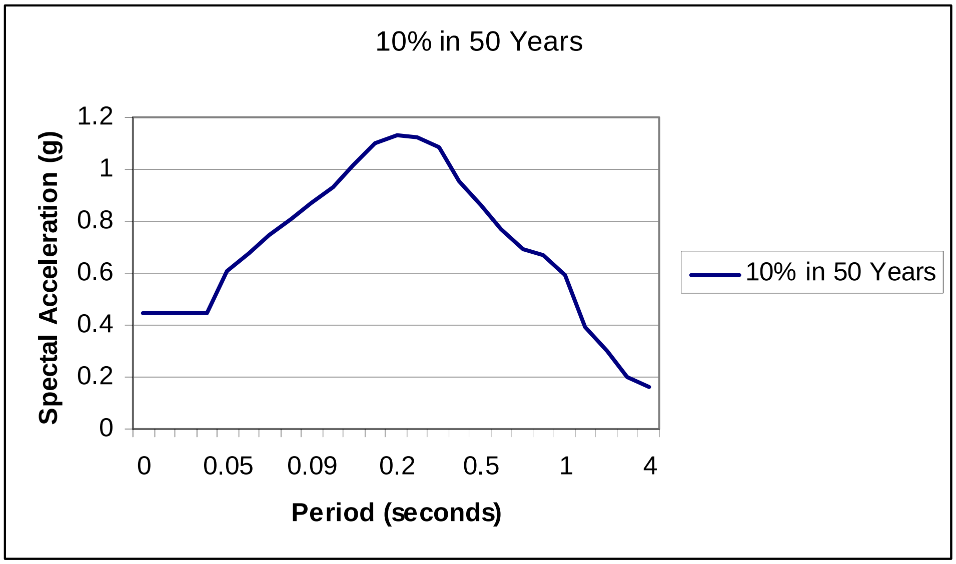

Figure 2. 5% damped spectral acceleration of DBE event

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. 90 deg.	23	6	0.470 0.455
DBE	Landers	7.4	Yermo	270 deg. 360 deg.	84	15	0.414 0.468
DBE	Kocaeli	7.4	Duzce	180 deg. 270 deg.	90	13	0.484 0.492
MCE	Loma Prieta	6.9	Los Gatos PC	0 deg. 90 deg.	23	6	0.569 0.566
MCE	Landers	7.4	Yermo	270 deg. 360 deg.	84	15	0.503 0.564
MCE	Kocaeli	7.4	Duzce	180 deg. 270 deg.	90	13	0.573 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. This determined the critical damper properties, including maximum force, damping coefficient and non-linear damping exponent, also known as the “alpha” (α) factor. 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. The quantity and location of dampers was limited by the owner based upon a desire to limit visual impacts on the structure. Although a higher quantity and more efficient placement of the dampers was recommended, the quantity of dampers was selected and located consistent with the owner's desires. Due to the limitation of the quantity and location of dampers, PARADIGM was unable to provide a relatively uniform distribution of dampers throughout the building. Therefore, in addition to the dampers having to limit direct displacements, they also had to limit rotational demands placed on the building due to the relative few dampers allowed. In an attempt to reduce costs, PARADIGM elected to limit the damper size to 200 kips under the MCE event at the roof. This 200 kip damper level was determined to be an economical damper size for this building at this level. The remaining few locations for the dampers necessitated significantly large dampers be incorporated at the lower levels. The model was then analyzed utilizing the suite of three time histories and results compared to the desired drift design parameters. The beams and columns were modeled as linear elements, and their demand to capacity ratios were compared to the element demand modifier (m) values prescribed by FEMA (FEMA 273, 1997).

Analysis Results

All elements that had a possibility of experiencing inelastic response were modeled as deformation controlled elements using nonlinear components. Inelastic limits were verified against the limits set forth in FEMA 273 requirements. All elements that were expected to remain elastic were modeled as force controlled elements using linear components. Force levels were checked for these elements based on standard steel design equations without a stress reduction factor.

All deformation and force level results corresponded to an immediate occupancy performance level for the MCE level. The LFRS remained essentially elastic throughout the MCE event. Based upon FEMA 351 requirements for this building with a 50% confidence level indicate an immediate occupancy performance objective is achieved if the maximum drift is less than 0.64% (FEMA 351, 2000). A maximum inter-story drift of approximately 0.75% occurred during the MCE event, which indicates the immediate occupancy performance objective has been achieved but at a lower confidence level of approximately 40%. 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 2.3 to 2.7, 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. The dampers were designed to possess a non-linear exponent as described above. This non-linearity limits the increase in axial force above the design value resulting from the MCE level event.

P-Δ effects were checked for gravity columns and determined to be negligible due to the displacement control provided by the FVD's.

Comparison to CBC-Designed Building

Although the building met the Immediate Occupancy performance objective for an MCE in accordance with FEMA 351, in order to validate that the building satisfied the design requirements for a new building in accordance with the CBC (ICBO, 2001), the force and displacement output of the building achieving a life safe performance objective under the DBE was compared to the force and displacement output of a building designed under current code. In other words, we compared the demand to capacity ratios DCR's of the 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 possessed acceptable DCR's along with achieving the desired drift limit.

Assuming the cost to retrofit a single moment frame joint to be on the order of $5,000.00 per joint, and assuming that the building possesses approximately 108 joints requiring strengthening, the total cost of rehabilitation of only the existing connections would have approached $540,000.00. However, strengthening of the joints would not improve the overall building performance beyond that of attaining life safety under a DBE. Building drift must be controlled in order to meet the immediate occupancy performance objective. Installation of more rigid elements would necessitate the addition of shear walls or braced frames which would require enhancements to the collectors and further enhancements to the foundations system.

Conclusion

Buildings possessing "Pre-Northridge" beam-column connections can economically be retrofitted to achieve enhanced levels of performance such as immediate occupancy or life safety at either DBE or MCE levels. Attention must be paid to the desired performance objective such that appropriate levels of enhancement are attained. Installation of FVD's allowed for a reduction in potential foundation upgrades which would have been necessary should more conventional strengthening systems have been incorporated.

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