Performance Based Seismic Retrofit of a Pre-Northridge Steel Moment Frame Building in California

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SUMMARY:

The 1994 Northridge earthquake brought about the realization that Steel Moment Frames, as they were being built since the 1960's, were not the reliable lateral force resisting systems that most people in the earthquake engineering community believed them to be. Flaws in certain design and construction details led to a poor structural performance during this event. Even though no steel moment frame building collapsed during the Northridge earthquake, the damage observed in numerous connections was substantial enough to prompt a region-wide search for hidden structural damage in existing moment frame buildings. A broad effort was also initiated to significantly improve the analysis, design, retrofit and construction techniques for this type of lateral force resisting systems.

This paper describes the seismic retrofit of an eight story existing pre-Northridge steel moment frame building in California. The goal of the retrofit was to ensure that the building performs to a "Life-Safe" level when subject to a "Design Earthquake" level of demand. Due to limitations on the prescriptive approaches available for the retrofit of the building, a Performance-Based methodology, using nonlinear analysis techniques, was implemented. The basis for the approach was ASCE 41 "Seismic Rehabilitation for Existing Buildings", a standard that is now explicitly referenced by the California Building Code (CBC).

The retrofit was focused on improving the post-elastic performance of the building as well as reducing the overall drift of the structure. As part of the structural retrofit, top and bottom haunches were proposed for the retrofit of the existing connections. A component testing program was developed and implemented in order to validate the performance of these connections. Five full size specimens were tested at the University of California San Diego (UCSD) structural laboratory. Results of the testing program are also presented in this paper.

Keywords: Seismic retrofit, performance based design, nonlinear analysis, steel moment frames, testing.

1. INTRODUCTION

Like many other engineering disciplines, earthquake engineering learns from painful experiences about certain flaws of the conventional design approaches. The series of moderate to large earthquakes that hit California and Japan in the late nineteen eighties and mid nineties showed the earthquake engineering community that steel moment frames, a lateral force resisting system widely believed to be one of the most suitable for moderate to large seismic demands, were in fact exposed to severe earthquake induced damage. This was due in part to inadequate construction procedures, and in part to excessive extrapolation of observed and tested nonlinear response for beams, columns and beam-to-column connections in particular.

The sudden awareness of the seismic vulnerability of the existing building stock set off the implementation of new seismic regulations. The regulations were intended to address the shockingly excessive damage observed on existing buildings and their contents during the seismic events mentioned above. In California, hospital buildings were the subject of new seismic criteria to determine their suitability to remain offering critical care services to the community. After the Northridge earthquake, the state of California implemented Senate Bill 1953 (SB1953) which, among other things, created structural and non-structural seismic performance categories for existing hospital buildings, and set up deadlines for the seismic retrofit of buildings deemed in risk of collapse. SB 1953 drew a line between buildings built before and after the year 1972. Buildings built before 1972 were labeled “non-conforming” buildings, whereas buildings built after 1972 were labeled “conforming” buildings. Non-conforming buildings were deemed by default at-risk-of-collapse unless it could be demonstrated via a prescribed structural evaluation that they were life safe or better. Non-conforming buildings could also remain in operation if a seismic retrofit program is implemented to bring the building to a life-safe seismic performance level. The approach to life-safe retrofit of the buildings could be prescriptive or performance based.

2. BUILDING DESCRIPTION

The subject hospital building is an eight story plus mechanical penthouse steel moment frame, located in California. The building was built in 1972, and as discussed above, by default is a “non-conforming” hospital building. The floor plan of the building is enlarged from the third floor up, creating a severe vertical mass irregularity in this region. The lower three floors of the building are currently in operation for different hospital functions, while the upper five floors are vacant. Fig. 1 shows a 3-D view of the building. The building is surrounded by other hospital buildings of different ages.

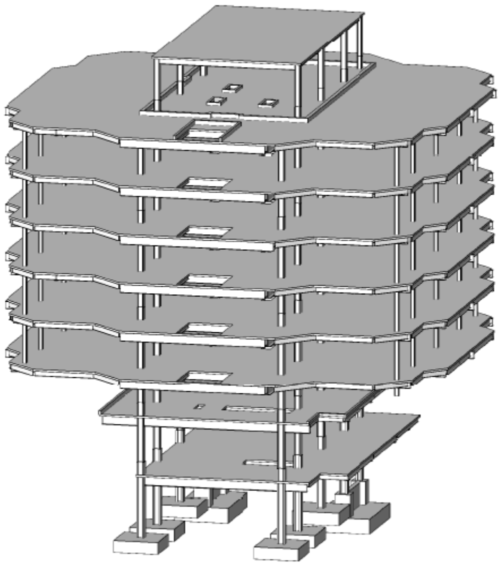

Figure 1. 3-D view of subject building

3. MAIN SEISMIC DEFICIENCIES OF THE LATERAL SYSTEM

A preliminary evaluation of the building was conducted to determine what seismic deficiencies it had. The main deficiencies that the building presented were, first, seismic performance of the beam column connections, which due to the vintage of the building, were expected to be inadequate for large ductility demands. These are typical pre-Northridge connections with potential brittle or limited ductility capacities. Second, the potential development of a soft story mechanism due to the vertical configuration of the frames. A third major deficiency recognized during a subsequent more detailed evaluation of the structure was the excessive lateral drift, which would lead to pounding into adjacent structures.

4. RETROFIT SCHEME USING CONVENTIONAL LINEAR ELASTIC APPROACH

The initial retrofit approach proposed for the building used a conventional linear elastic procedure. Prescriptions in the California Building Code required the existing lateral system to sustain a limited ductility demand. This could only be accomplished with extensive bracing and foundation work, the later to accommodate the increased overturning demands. As a result of the excessive retrofit scope to bring the building to a life-safe condition, it was determined that the upper five floors and mechanical penthouse would have to be demolished and only the bottom three floors would be saved. Fig. 2 shows the scope of demolition required to meet the prescriptive code criteria.

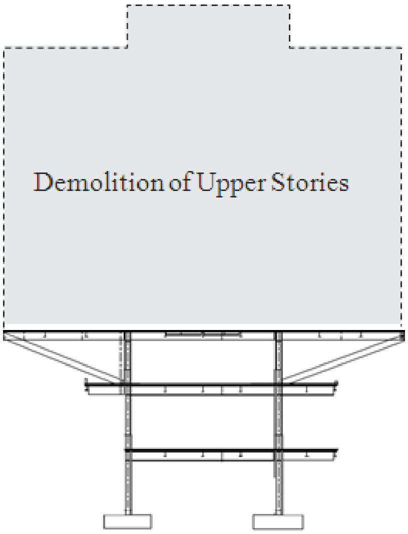

Figure 2. Retrofit scheme following conventional linear-elastic methodologies

5. PERFORMANCE BASED APPROACH

The potential lost of valuable square footage on the approach described above, led to the search for an alternative retrofit solution, this time using a performance based procedure. The basis for the approach was ASCE 41-06 "Seismic Rehabilitation for Existing Buildings", a standard that is now explicitly referenced by the California Building Code. ASCE 41-06 provided the guidelines for the nonlinear modeling of plastic hinges before and after the proposed retrofit. After an initial evaluation of the contribution of higher modes to the overall response of the building, it was determined that higher mode effects were small enough to allow the use of a Nonlinear Static Procedure (NSP) or Pushover analysis. The software used for the analysis was Perform 3D.

An initial nonlinear evaluation of the structure showed the tendency of the building to develop a soft story on the lower floors. It was also found that if the lower columns were strengthened to force a beam sway mechanism, the existing limited ductility of the beams would lead to insufficient overall ductility of the structure. It was determined, therefore that to meet the target "life safe" performance level the beam column connections should be retrofitted.

Three options were considered for the beam column connection retrofit: Reduced Beam Section (RBS), Welded Haunches, and Bolted Haunches. The three options are conceptually shown in Fig. 3. While the main goal was to improve the ductility of the plastic hinges expected to be formed on the beams with the minimum required scope of retrofit, it was also required that the overall performance of the building should be appropriate. As described above, one of the system deficiencies that needed to be mitigated was the excessive drift of the structure, such condition disqualified the use of an RBS repair. While less invasive than the other two options considered, the use of RBS was counter to the need to stiffen the lateral system. It was finally decided to use welded haunch connections, due to the perceived easier installation on an existing moment frame compared to the bolted haunches.

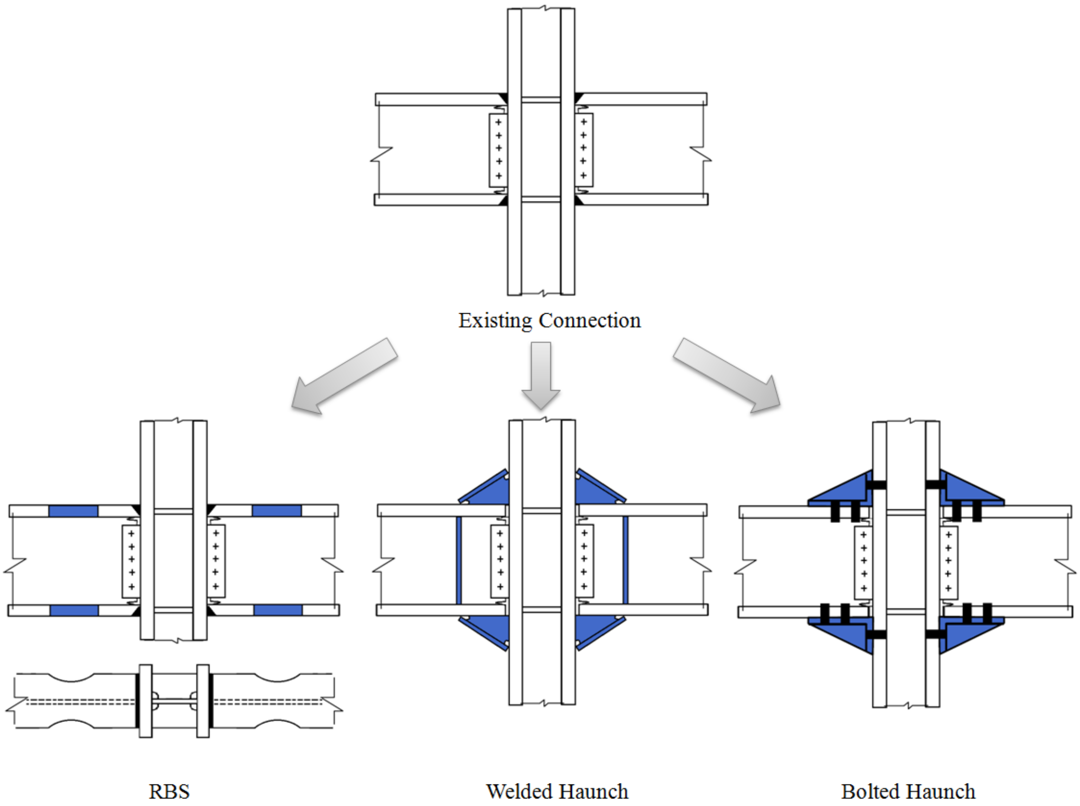

Figure 3. Retrofit options considered for beam-to-column connection repair.

The scope of the retrofit was completed with the stiffening and strengthening of the existing columns, and the strengthening of column foundations. The first was accomplished with the "boxing" of the wide flange columns with steel cover plates of variable thickness. For the second, strengthening of column foundations, after exhausting the potential use of enlarged footings, it was decided to use micropiles at the base of all main columns. The level of fixity achieved with the micropiles helped mitigate the excessive lateral displacements at the lower floors of the building, which could not be mitigated even after the column steel "boxing" described above. The final scope of retrofit is shown in Fig. 4.

The structure in its existing condition, and after retrofit, was modeled nonlinearly using Perform 3D. It was observed that the retrofit led to a significant enhancement of the system ductility in both main directions. The overall stiffness and strength of the lateral building was also increased by a factor of 1.5 to 2.0 approximately. Fig. 5 shows the pushover curves before and after the proposed retrofit program. The target displacement was reduced to 1% drift or less in both main directions, small enough to mitigate the potential pounding of the building against existing adjacent structures. The collapse mechanism of the building was shifted from a soft story to a beam sway mechanism, which combined with the enhanced ductility of the beams led to the enhancement of the overall system ductility mentioned above. Fig. 6 compares the collapse mechanism, before and after retrofit, in one of the main building directions.

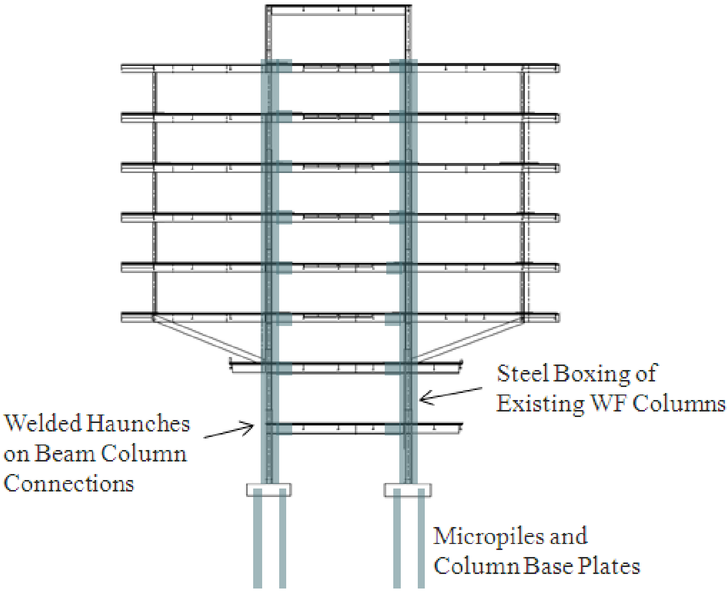

Figure 4. Retrofit scope using performance based approach.

The seismic performance of the retrofitted structure was thus found to satisfy the criteria for a life-safe performance level as described in ASCE 41-06. Project specific criteria such as limited drift and the shift of the undesirable collapse mechanism into a more adequate beam sway mechanism were also deemed satisfactory. The retrofit scope also included other supplementary tasks such as reattachment of perimeter concrete panels and new slab to beam connections.

The retrofit project is currently under construction and it is expected to be completed approximately on a 24-month schedule, during which the hospital will keep using the building for the functions that currently are in place. After the retrofit is complete and the building is deemed life-safe, it can remain in operation until the year 2030 under current California law.

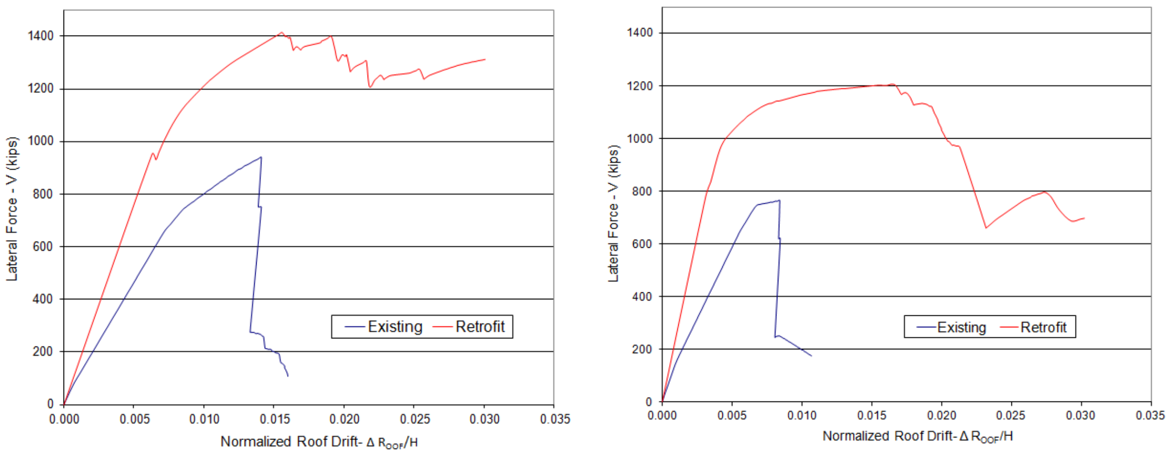

Figure 5. Comparison of pushover curves before and after retrofit in two main building directions

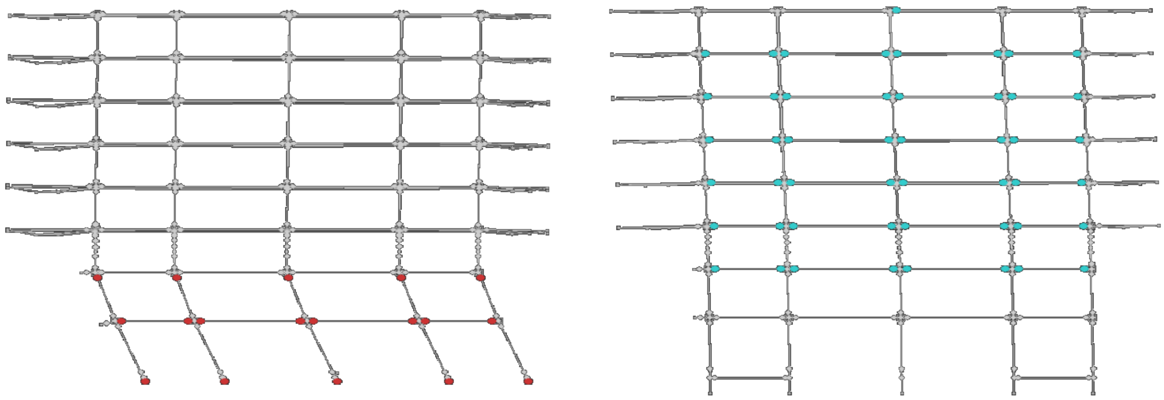

Figure 6. Comparison of deformed shapes before and after retrofit at target displacement

6. COMPONENT TESTING PROGRAM

Even though ASCE 41-06 has criteria for the nonlinear modeling of plastic hinges on steel beams with top and bottom haunches, the heavy wide flange beam and column sections used in the building, as well as the uncertainty on the weld used during the original construction, led to the implementation of a component testing program intended to verify the post-elastic performance of the proposed beam-to-column connection retrofit. Before the implementation of the testing program, a detailed finite element analysis of the proposed retrofit was conducted by Uang and Kim (2010) to observe the nonlinear behavior of the proposed specimens. Results of the finite element analysis showed that the presence of the haunches would force the formation of plastic hinges on the beam itself, while protecting the original beam-to-column brittle weld region. Fig. 5 shows some results of this supplementary finite element analysis.

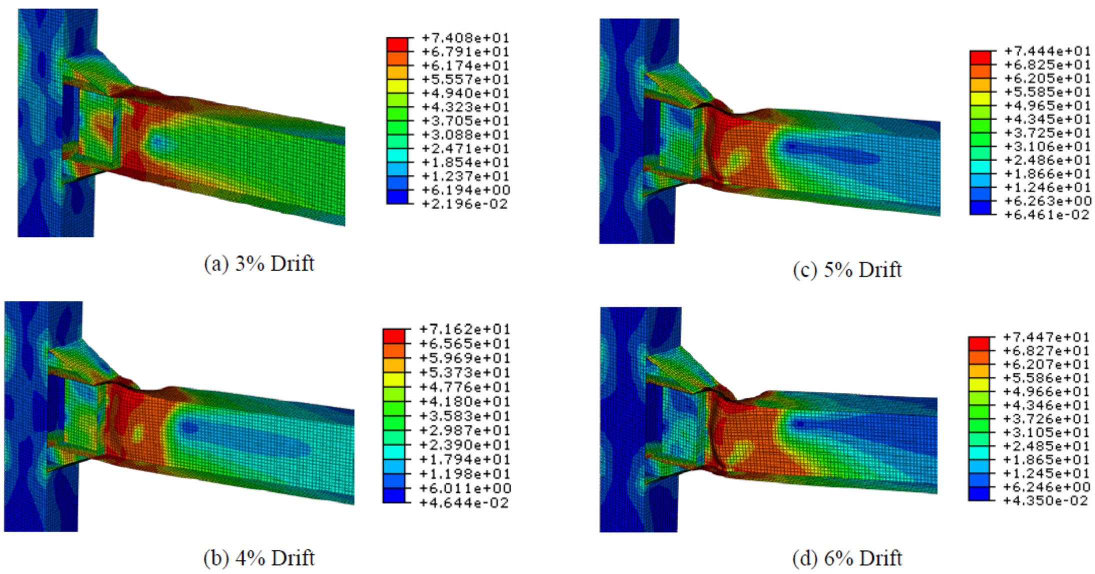

Figure 5. Non-linear finite element analysis for pre-testing evaluation of beam column connection repair (Uang and Kim, 2010)

Five beam-column specimens were built. The specimens were built in two phases. First, the beam-to-column connections, without any retrofit, were constructed. The welding during this first phase followed procedures expected to have been used during the early 1970’s, time when the original building was built. Steel used in this phase was also selected to match strength and strain properties of the original structure. The second phase was the construction of the proposed haunches. The top and bottom haunches would use the details and follow the procedures expected to be used during the actual retrofit of the building.

The five specimens were cyclically tested at UCSD’s structural laboratory. The typical beam-column specimen and the testing set up are shown in Fig. 6. Photos of the actual testing are presented in Fig. 7. The testing protocol followed the criteria prescribed in AISC 341-05 Seismic Provision for Structural Steel Buildings. The cyclic performance of all five specimens met the criteria of AISC 341-05 for beam-to-column joints and connections used in the seismic load resisting system of special moment frames, namely (1) the connection must be capable of sustaining an interstory drift angle of at least 0.04 radians, and (2) the required flexural strength of the connection, determined at the column face, must equal at least 80 percent of the nominal plastic moment of the connected beam at an interstory drift angle of 0.04 radians. The component testing results, therefore validated the expected performance of the proposed beam-to-column connection retrofit.

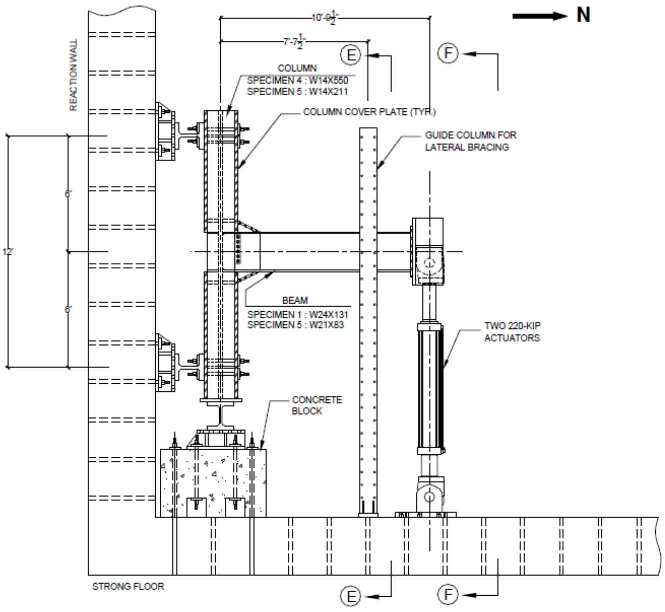

Figure 6. Typical specimen and testing set up

Figure 7. Pictures of two specimens during testing at UCSD structural laboratory.

7. CONCLUSIONS

The life-safe seismic retrofit of an eight story steel moment frame building with pre-Northridge beam-to-column connections and other systemic deficiencies has been developed using a performance based approach. The contrast of the retrofit scope required using prescriptive linear elastic procedures vs. that using a non-linear performance based approach shows that the later has the potential to deliver satisfactory seismic performance while limiting scope of retrofit. The implementation of the performance based approach however could be far-reaching, involving not only extensive analysis but also supplementary tasks such material and component testing, finite element modeling of particular conditions, and others.

ACKNOWLEDGEMENTS
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