Cyclic Testing and Analysis of Retrofitted Pre-Northridge Steel Moment Connections Using Bolted Brackets

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Abstract

The 1994 Northridge earthquake exposed the vulnerability of both older and newer steel moment frame structures to seismic-related damage. This damage was generally in the form of brittle failures at the interface of the beam and column members and was found in buildings from 1 to 26 stories in height. Improvements to these existing steel moment frame connections (Pre-Northridge moment connections) are often required as part of an overall strengthening scheme particularly when higher performance levels are desired.

This paper presents a summary of the development of strengthening schemes for pre-Northridge moment connections and the full scale testing and analysis of the connections. This was one component in the overall evaluation and retrofit of an essential service facility located in northern California. Technical information regarding the evaluation and development of the final overall building retrofit can be found in a companion paper, "Seismic Evaluation and Rehabilitation of a Three Story Pre-Northridge Steel Frame Essential Service Facility" (Blaney, et. al., 2010) also published in these proceedings.

Having just completed a rehabilitation project utilizing expensive and somewhat intrusive welded haunches for connection strengthening, the design team decided to evaluate alternatives, particularly a scheme using the proprietary Kaiser Bolted Bracket (KBB).

To supplement previous testing data, three full-scale test specimens were subjected to laboratory and analytical investigation. The specimens were subjected to a project-specific cyclic loading sequence of increasing amplitude with all three tests exceeding the predefined acceptance criteria. Using a general finite element methodology, the analytical study provided additional insight into the performance of the connections both with and without the bolted brackets.

This paper summarizes factors associated with the development of the test specimen configurations. It also summarizes the results for each of the tests and the analytical studies. The tests and analysis support the use of the bolted brackets to improve and enhance the performance of pre-Northridge steel moment frame connections in the existing building.

Introduction

The 1994 Northridge earthquake caused significant damage to over one hundred steel moment frame buildings in the form of weld fracture in the beam–column connections. This damage caused considerable concern in the engineering community about the vulnerability of this structural system and prompted a major Federal Emergency Management Agency (FEMA) funded research effort. The subsequent research program resulted in the publication of a series of FEMA guideline documents for use by practicing engineers.

Over the past fifteen years, many owners have commissioned the assessment of the anticipated seismic performance of these pre-Northridge buildings and chosen to voluntarily strengthen them. The most common strengthening strategies are the addition of a secondary system that protects the existing moment frame connections, strengthening the connections themselves, or some combination of the two.

This paper focuses on the connection strengthening for an essential service facility located in Northern California. This was one aspect of the overall evaluation and retrofit of the structure that is a presented in a companion paper, “Seismic Evaluation and Rehabilitation of a Three Story Pre-Northridge Steel Frame Essential Service Facility” (Blaney, et. al., 2010) also published in these proceedings. This paper summarizes the procedures used to develop and evaluate the various possible connection strengthening schemes, the selection of the test member sizes, laboratory set-up, and the results of laboratory tests and analysis. Additional information on the path used to choose and develop the final overall building strengthening scheme as well as the analysis methods used can be found in the companion paper (Blaney, et. al., 2010).

Building Description

The subject building is an essential service facility located in Northern California adjacent to the Hayward Fault. It is three stories tall, with a large central atrium and a mechanical penthouse. The building is rectangular in plan, with increasing setbacks at the upper floors (Figures 1 and 2). The building was designed under the provisions of the 1991 Uniform Building Code (UBC) and employs steel special moment resisting frames for lateral resistance. The floors are concrete fill over metal deck, the roof is insulating concrete fill on metal deck, each spanning between structural steel beams and girders. The exterior facade is comprised of glass fiber reinforced concrete (GFRC) panels.

The majority of the moment frame columns are W14 sections, of ASTM A572 Grade 50 steel. Built-up steel box columns, of ASTM A572 Grade 50 plate material, exist at the intersection of orthogonal moment frames. All other structural steel members are ASTM A36.

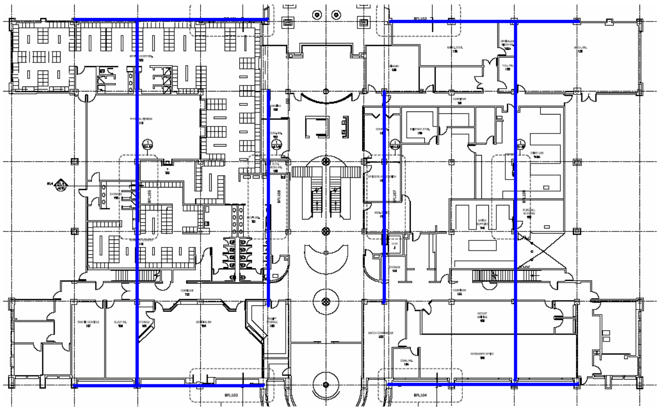


Figure 1 - Plan at First Level

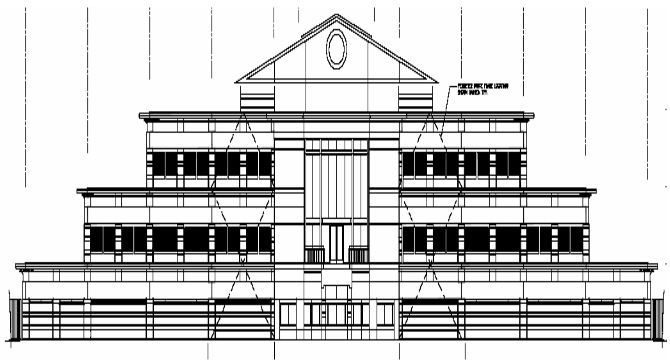


Figure 2 - Front Elevation

Selection of Beam-Column Connection Strengthening Schemes

Decision to Strengthen Connection

Early in the building evaluation process, it became apparent that a supplemental strengthening scheme alone, such as damping devices or Buckling Restrained Braces (BRBs), would not be sufficient to achieve the Immediate Occupancy (IO) structural performance goal desired by the owner for this facility. This was due to the low acceptable interstory drift limits for the pre-Northridge moment connections at a designated IO performance level. For example, given a 90% and 50% confidence level to prevent global and local collapse, FEMA 351, *Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings*, limits the interstory drift angle to 0.006 radians for pre-Northridge moment connections. With such a low drift limit, the choices that emerged were to either provide an essentially new lateral force resisting system capable of limiting the interstory drifts to very low levels, thereby protecting the existing connections, or to upgrade the pertinent connections where greater demands were computed. The following computation illustrates the method of calculating the allowable drift limit per FEMA 351.

$$D = \frac{\lambda \phi c}{\gamma \gamma a} = \frac{1.14\,(0.8)\,.001}{1.4\,(1.04)} = 0.006 \text{ radians}$$

Where:

D = Calculated demand for the structure (radians)
$Ø$ = Resistance factor for uncertainty and variability
c = Capacity of the structure or element
λ = Confidence index parameter
γ = Demand variability factor
γa = Uncertainty factor in analytical procedure

Selection of the Kaiser Bolted Bracket

FEMA 351 and AISC Design Guide 12, *Modification of Existing Welded Steel Moment Frame Connections for Seismic Resistance,* (Gross et. al., 1999) provide a number of pre-qualified connection modification schemes for the rehabilitation of pre-Northridge connections. A few of the non-propriety connections schemes include the Reduced Beam Section, Welded Bottom Haunch, Welded Top and Bottom Haunch, and Welded Cover Plate Flange. Proprietary connections include the Side Plate, Slotted Web, and Kaiser Bolted Bracket (KBB).

Having just completed a similar rehabilitation project utilizing expensive and somewhat intrusive welded haunches to strengthen pre-Northridge moment connections for the Caltrans District 4 Headquarters Building in Oakland, the project team decided to evaluate alternate schemes which might eliminate or reduce the amount of welding as well as the degree and length of potential occupant impact.

After lengthy deliberation, a proposed connection strengthening scheme using the KBB was selected. The choice of this product was complicated by the need for additional testing and the small risk that the rehabilitated connection might not perform adequately during testing. As a proprietary product, the specification of the KBB also posed some challenges in maintaining the competitiveness required by the public bidding process. However, the clear cost and schedule benefits were compelling enough to move the project in this direction.

Evaluating Previous Testing and Prequalification Data

Evaluation of connection modifications involving the KBB began with the compilation and review of all available tests. Fortunately, the design team and the peer review engineer were already familiar with the KBB as one specimen was tested on a previous project (Newell and Uang, 2006). Although exhibiting good ductility under laboratory conditions, the previous test specimen fell short of project goals as it experienced a brittle bolt fracture at the first positive excursion at 4% drift. Generally, two complete cycles are required at 4% drift to be classified as a Type 2 connection per FEMA 351 and AISC 341, *Seismic Provisions for Structural Steel Buildings*.

While the KBB was eventually deemed inappropriate for the previous project, there were a number of unique circumstances that prompted the design team to explore the use of the KBB for this project, including:

- Smaller existing moment frame beam sizes
- The lower target drift levels due to the proposed addition of BRBs
- The targeted IO structural performance level
- A modified connection configuration which reduced column bolt prying
- The recent publication of Supplement No. 1 to AISC 358, *Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications,* which provided additional guidance for the design and specification of the KBB

While AISC 358 was intended for prequalified connections in new construction, the document was especially helpful for establishing specification requirements for the fabrication and inspection of the test specimens. In general, however, two of the existing member characteristics fell outside of the AISC 358 prequalification parameters. These included the beam depth, which is limited to W33 and smaller members, and the beam weight, which is limited to 150 pounds per linear foot and less.

Bolted Bracket Modification

During the same time that connection strengthening schemes were being developed, extensive on-site field verification was also taking place. One early finding determined that the proposed modifications to the top flanges on the existing beam-to-column joints would impact the width of existing corridors and more significantly the existing GFRC system. Although impacts at corridors could be dealt with reasonably, it was discovered that the existing GRFC system could not be re-installed to fit around the top bolted bracket without substantial modifications. While removal and re-installation of the GFRC skin system was not ideal, modification of these panels would also present both water intrusion and aesthetic concerns which were not tolerable. This problem necessitated connections be developed incorporating the standard KBB at the bottom flange and an improved welded connection at the top flange.

Project Specific Testing

As previously discussed, the existing member sizes and the proposed strengthening schemes did not conform to established prequalification parameters. This necessitated connection testing to establish the expected performance of the strengthened connections and to confirm conformance with project goals. Project specific testing can be a lengthy process requiring significant time for a series of tasks. For this project, these included:

- Review of AISC 341 and AISC 358 for prequalification data and project specific testing guidelines
- Review of exiting beam column configurations and initial analytical calculations to determine possible beam-column combinations and bracket sizes
- Confirmation of existing field conditions and material properties
- Development of details and specifications for the construction of both the original pre-Northridge and the strengthened connections
- Solicitation and negotiations with several steel fabricators, testing labs, and special inspection firms
- Oversight of specimen fabrication
- Establishing test protocols
- The lab setup, specimen testing, and break down.
- Evaluation of laboratory results

Conveying the project intent to both fabricators and special inspectors took considerable effort including the preparation of full design details, specifications, and fabrication sequencing. Cost estimates received for these tasks varied widely, indicating the importance of soliciting multiple bids.

From start to finish, this process took approximately eight months and was a “critical path” element of the overall project schedule. At times, the process was daunting as there were high expectations of success, especially given the overall expenditures and tight design schedule.

Selection of Beam and Column Sizes for Test Specimens

Once the KBB was selected for strengthening the connections, the process of member selection for the test specimens began. Appendix S of AISC 341 was used as the guiding document. Section 5.2 of AISC 341 restricts the extrapolation of test data based on the following requirements:

- The depth of the test beam or link shall be no less than 90 percent of the depth of the prototype beam or link.
- The weight per foot of the test beam or link shall be no less than 75 percent of the weight per foot of the prototype beam or link.
- The depth of the column shall be no less than 90 percent of the prototype column.

Given these requirements, various parameters of the existing beam-column combinations were studied to determine the most appropriate combination for testing. While the final choice of beam-column combinations was based upon panel zone strength, many other parameters were evaluated and discussed with the peer review engineer, including frequency of occurrence, column-beam moment ratios (M_{pc}/M_{pb}), and panel zone demands. A brief summary of each of these follows:

Frequency of Occurrence

One of the first parameters considered for beam-column combination selection was the frequency of connection occurrence in the existing structure sorted as follows:

- Recurrence of beam-column connection
- Recurrence by beam
- Recurrence by column
- Recurrence by floor

Tables 1 through 3 indicate the total frequency sorted by the various parameters. Since the W33 beams are prequalified, most attention was directed at the deeper W36 beams and particularly beams weighing 150 pounds per linear foot and more. The highlighted items represent the members selected for the final test specimens.

Beam-Column Connections Connection Recurrence			
Beam	Column	Level	# of Connections
W33x118	W14x311	Roof	12
W36x150	W14x311	3rd	12
W36x170	W14x370	2nd	10
W33x118	W14x193	Roof	8
W33x118	W14x257	Roof	8
W36x135	W14x257	3rd	8
W36x194	W14x311	2nd	8
W36x160	W14x342	2nd	8
W36x194	W14x370	2nd	8
W36x135	W14x370	3rd	6
W36x135	W14x193	3rd	4
W36x150	**W14x193**	**3rd**	**4**
W36x160	W14x257	2nd	4
W21x147	W14x311	3rd	4
W21x147	W14x370	2nd	4
W33x141	W14x370	2nd	4
W36x170	W14x311	3rd	2
W36x135	W14x311	3rd	2

Table 1 - Connection Recurrence

Beam-Column Connections Beam Recurrence		
Beam	Level	# of Connections
W33x118	Roof	28
W36x135	3rd	20
W36x194	2nd	16
W36x150	**3rd**	**16**
W36x170	2nd/3rd	12
W36x160	2nd	12
W21x147	2nd/3rd	8
W33x141	2nd	4

Table 2 - Beam Recurrence

Beam-Column Connections Column Recurrence		
Column	Level	# of Connections
W14x311	2nd/3rd/Roof	40
W14x370	2nd/3rd	32
W14x257	2nd/3rd	20
W14x193	**3rd/Roof**	**16**
W14x342	2nd/3rd	8

Table 3 - Column Recurrence

Column-Beam Moment Ratio

Column-beam moment capacity ratios (M_{pc}/M_{pb}) were computed for all existing configurations and varied from approximately 0.6 to 2.4. Approximately 20 percent of the existing connections in the building have M_{pc}/M_{pb} ratios less than 1.0 when computed in accordance with ASIC 358. While the addition of the KBB will actually increase the moment demand to the face of the column due to the slight shift in the potential plastic hinge, it was considered acceptable for a number of reasons. These reasons included project specific tests for which the M_{pc}/M_{pb} ratio was approximately 0.75, and the weak panel zones as described below which protect the column from forming plastic hinges, and because of the low ultimate demands on the connections due to the planned addition of the BRBs into the building. Additionally, a large percentage of the strong-beam/weak-column conditions occur at the roof where there are lower expected drift demands. At the more critical lower floors, the ratios are nearly all 1.0 or greater.

Panel Zone Demands

As mentioned above, many of the existing beam/column configurations within the building are prone to significant panel zone yielding due to the higher grade steel strengths which are present within the building. This led to significant concern regarding the possible kinking within these probable weak column panel zones and the effect this might have on the performance of the KBB connection. Based upon this concern, a column with a weak expected panel zone was chosen for all three test specimens. The W14x193 section had the greatest panel zone demand for a single sided connection of 1.85 and the least desirable strong column-weak beam ratio of 0.75 a one sided connection.

Final Member Selection

The final choice for beam and column test member sizes took a bit longer than initially assumed and in the end was based upon both evaluation of the data and engineering judgment.

The W14x193 column and the W36x150 beam were ultimately chosen largely because of the depth of the beam (W36) and the very weak column panel zone which was expected to yield significantly. During the previous KBB test conducted by the same team, the observed premature bolt fracture was assumed to be directly related or at least accentuated by the observed panel zone distortion (Newell and Uang, 2006). Even as the cast bolted bracket was modified to decrease the gage of the bolts at the column flange and thus possible flange bending, it was still considered important to verify that the KBB would be reliable for the weak panel zone conditions prominent throughout this building.

Bolted Bracket Design

As reported by Adan and Gibb (2009), the patented KBB is a high strength steel casting developed in part for the rehabilitation of weak or damaged moment connections. The KBB is manufactured in a variety of sizes and is proportioned to develop the probable maximum moment capacity of the moment frame beam. The bracket's proportions are designed in accordance with the criteria outlined by Gross et. al. (1999). As shown in Figure 3, the B-series bracket is bolted to both the column and beam. Once installed, the design intent is to promote yielding and plastic hinge formation in the beam at the end of the connected bracket.

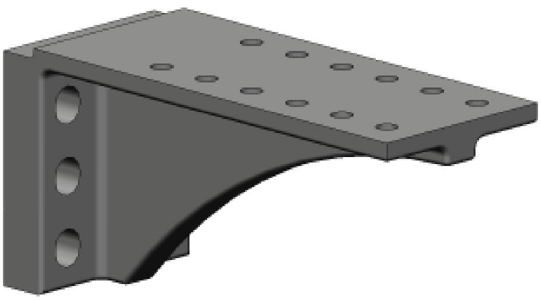


Figure 3 - B1.0/B1.0C Kaiser Bolted Bracket

Bolted bracket connection limitations, detailing requirements, and design procedures are outlined in Chapter 9 of AISC 358.

Testing Setup Configuration

The overall geometry of the test setup is shown in Figure 4. The column was supported by short W14x370 sections to simulate inflection points. Half of the beam span was included, and cyclic displacement was imposed by two actuators attached to the free end of the beam. One pair of bracing columns was used to provide beam lateral restraint for Specimen 1. After testing Specimen 1, it was decided to use two pairs of bracing columns in Specimens 2 and 3 to minimize the beam out-of-plane deformation.

The concrete over metal deck floor structure was not included in the test specimens due to cost and schedule constraints. Based on review of previous test results (Adan and Gibb, 2009, Newell and Uang, 2006) and engineering judgment it was agreed that deck would only improve the anticipated performance by providing additional restraint to the beam top flange.

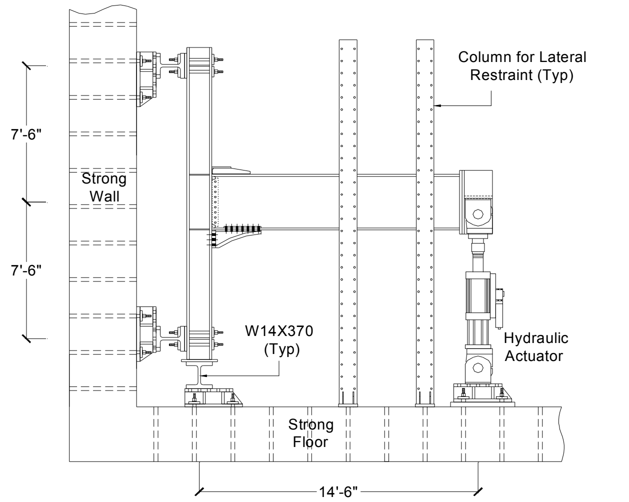


Figure 4 - Test Setup

Test Specimens

Specimen Configuration

Table 4 illustrates the basic configuration of each of the test specimens. More detailed descriptions and illustrations follow.

Specimen No.	Beam	Column	Rehabilitation Type
1	W36×150	W14×193	KBB at top and bottom flanges
2	W36×150	W14×193	KBB at bottom flange and modified CJP weld at top flange
3	W36×150	W14×193	KBB at bottom flange, modified CJP weld and double-tee bracket at top flange

Table 4 - Member Sizes and Rehabilitation Types

The condition of the existing pre-Northridge moment connections before modifications is shown in Figure 5.

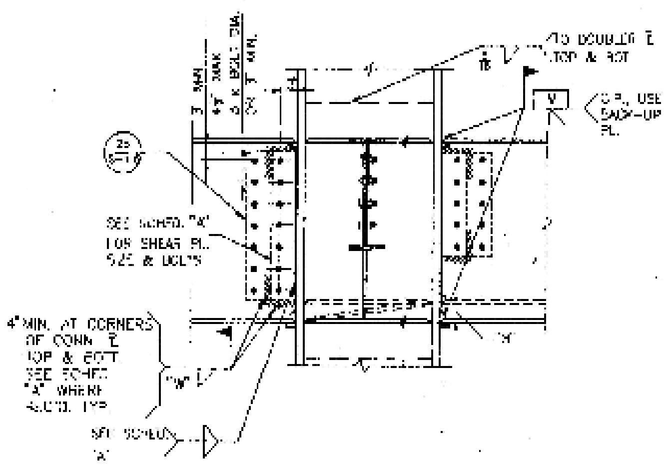


Figure 5 - Existing Moment Connection Detail

Specimen 1 was strengthened with KBB on the beam top and bottom flanges as illustrated in Figure 6. The existing pre-Northridge CJP welds, including the backing, were not modified.

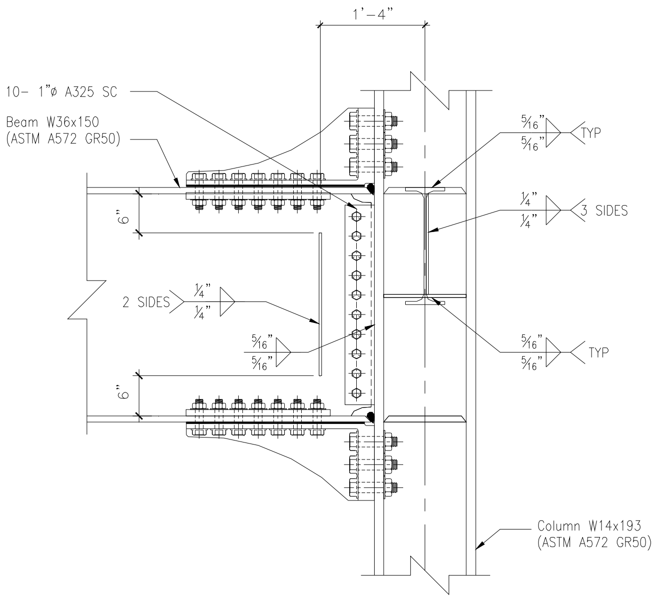


Figure 6 - Specimen 1 Connection Detail

Specimen 2 was strengthened with a KBB on the beam bottom flange and the beam top flange pre-Northridge weld was replaced by an improved CJP weld as illustrated in Figure 7.

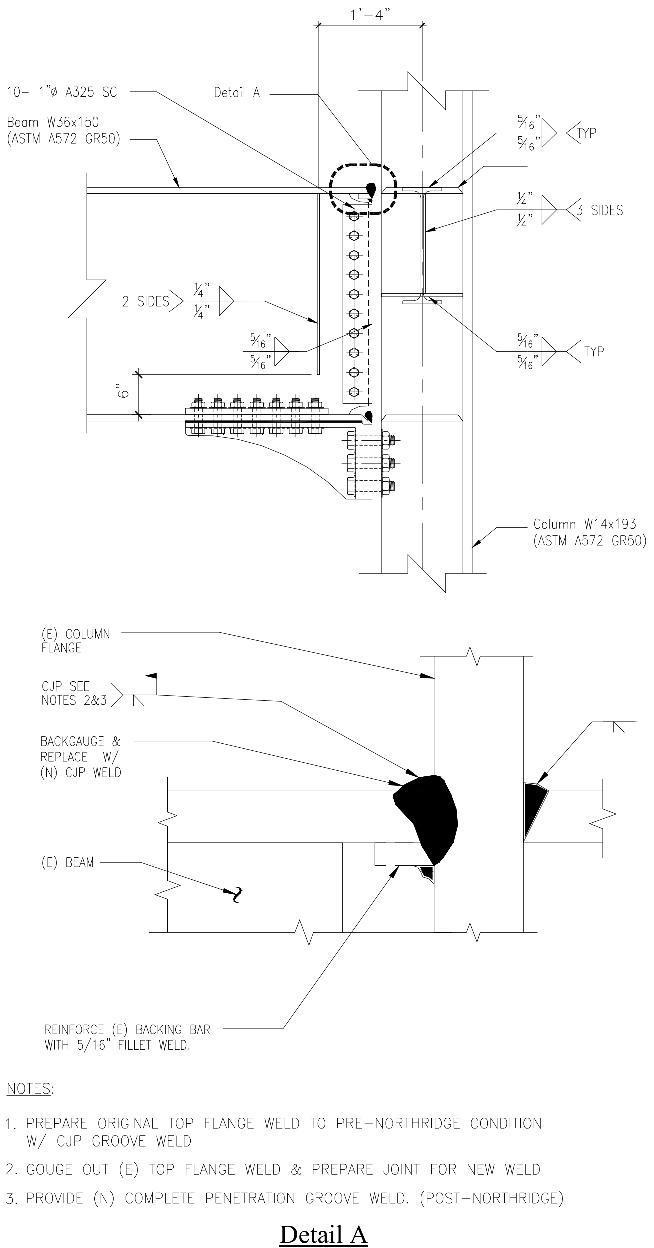
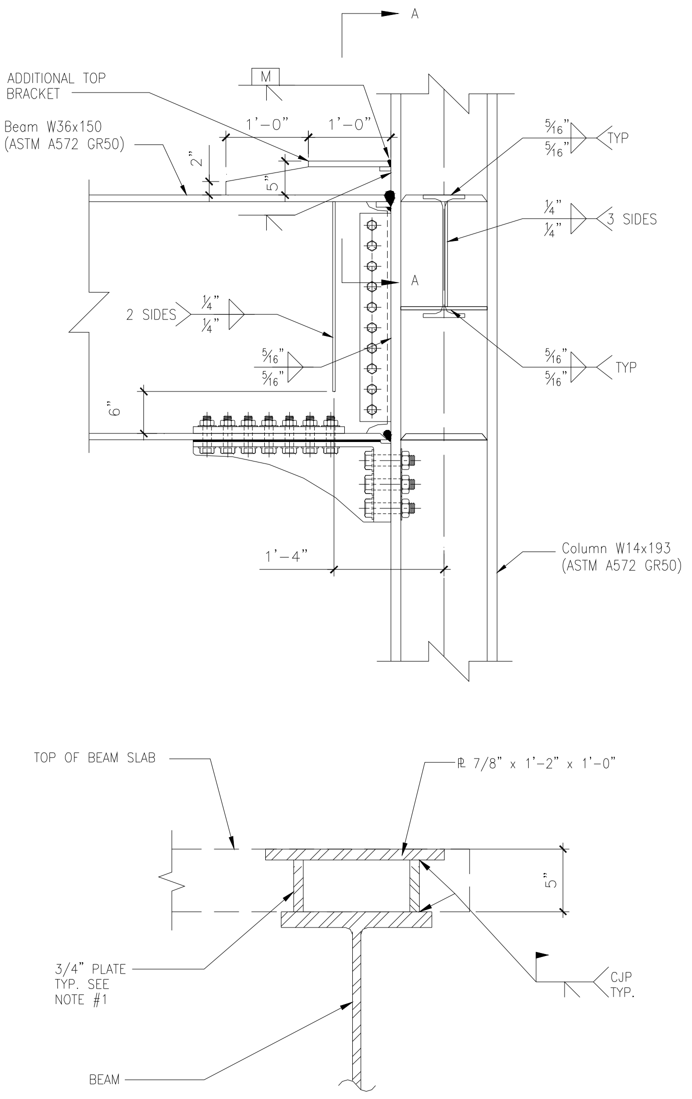


Figure 7 - Specimen 2 Connection Detail

Specimen 3 was strengthened with a KBB on the beam bottom flange. As with Specimen 2, the beam top flange weld was replaced by a new CJP weld. The top flange was then strengthened with a welded inverted double tee plate bracket. Specimen 3 is illustrated in Figure 8. This was done to provide additional reinforcing at the top flange and to create symmetry in the connection bending resistance and a more reliable expected plastic hinge location.

In Figures 6 through 8, the vertical plates on the beam web, located approximately 16 inches from the column centerline, correspond to the GFRC connection plates discovered in the existing structure.



Section A-A

Figure 8 - Specimen 3 Connection Detail

Specimen Fabrication

All specimen fabrication including the modifications was performed by a commercial steel fabricator. Specimens were first fabricated to the existing building conditions, see Figure 5, using typical pre-Northridge shop and field practices. The flux cored arc welding (FCAW) process with an E70T-4 electrode was used to make the CJP groove welds.

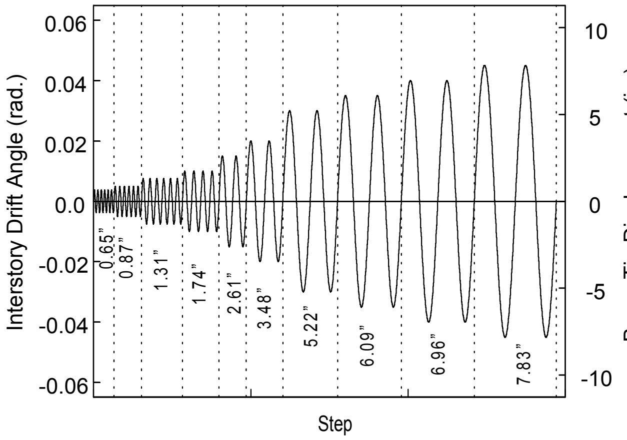
Simulated field rehabilitation welding for the top flange of Specimens 2 and 3 and top double tee bracket of Specimen 3 were done by the FCAW process using an E71T-8 electrode having a minimum Charpy V-Notch impact value of 20 ft-lbs at -20°F and 40 ft-lbs at 70°F. Using the bracket as a template, bolt holes in the column and beam for the KBB attachment were made using a magnetic-base drill. After the specimens were positioned in the laboratory test setup, the bolts were fully tensioned with a hydraulic torque wrench.

Specimen Material Properties

Although the original construction documents specified ATSM A36 material for the beams, based on original mill certificates, the steel was actually closer to that of A572 Grade 50. Therefore, ASTM A572 Grade 50 steel was used for the specimen beams and columns as well as for the double tee rehabilitation plate bracket material. The material for KBB high-strength castings was ASTM A958 Grade SC8620 class 90/60. This material has a specified nominal yield and tensile strength of 60 and 90 ksi, respectively. Bracket-to-column fasteners were 1-5/8 inch diameter ASTM A354 Grade BD high-strength bolts and bracket-to-beam fasteners were 1-1/4 inch diameter ASTM 490 high-strength bolts.

Loading Protocols

The loading sequence for beam-to-column moment connections defined in AISC 341 was followed with some minor modifications. The loading protocol used for the project is shown in Figure 9. Due to the planned addition of the secondary BRBs in the building, the target acceptable drift for connection acceptance was 3.5% drift or 0.035 radians. Accordingly, the loading protocol was modified to include an additional two complete cycles at 0.035 radians. Due to test setup limitations, the testing could not exceed 0.045 radians. Therefore, after the completion of two cycles at 0.04 radians the specimens were cyclically loaded at 0.045 radians until failure. Predetermined vertical displacements (Δ), based on interstory drift angles, were applied to the end of the beams. A combination of displacement transducers, inclinometers, strain gage rosettes, and uniaxial strain gages were placed on the specimens to measure global and local responses.



No.	θ (radians)	Δ for Testing (inches)
1	6 cycles at θ = 0.00375	Δ = 0.00375(174") = 0.65"
2	6 cycles at θ = 0.005	Δ = 0.005(174") = 0.87"
3	6 cycles at θ = 0.0075	Δ = 0.0075(174") = 1.31"
4	4 cycles at θ = 0.010	Δ = 0.010(174") = 1.74"
5	2 cycles at θ = 0.015	Δ = 0.015(174") = 2.61"
6	2 cycles at θ = 0.020	Δ = 0.020(174") = 3.48"
7	2 cycles at θ = 0.030	Δ = 0.030(174") = 5.22"
8	2 cycles at θ = 0.035	Δ = 0.035(174") = 6.09"
9	2 cycles at θ = 0.040	Δ = 0.040(174") = 6.96"
10	2 cycles at θ = 0.045	Δ = 0.045(174") = 7"

Figure 9 - AISC Cyclic Loading Sequence

Test Results

Specimen 1 - KBB at Top and Bottom Flanges

Testing stopped after completing two cycles at 4.5% story drift without fracture. The global deformed and yielding pattern at this drift level is shown in Figure 10. Yielding of the panel zone as indicated by flaking of the whitewash was observed at 1.5% story drift. Beam plastic hinge formation by yielding and local buckling outside the brackets was observed. The beam also experienced a significant out-of-plane deformation. A plot of the moment at the column face versus story drift ratio is shown in Figure 11, where M_{pn} is the nominal beam plastic moment.

Figure 10 - Deformed Shape of Specimen 1 at 4.5% Story Drift

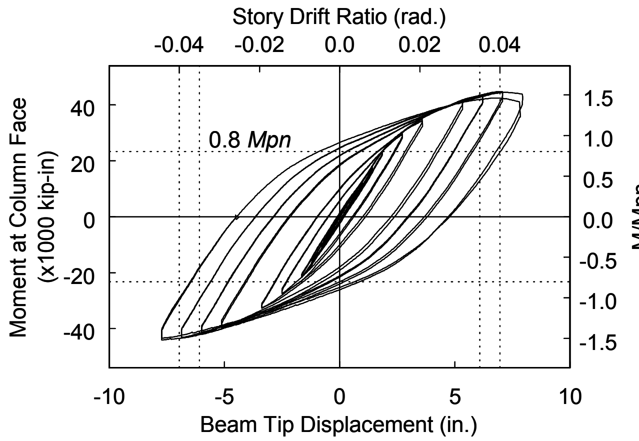


Figure 11 - Global Response of Specimen 1

Specimen 2 - KBB at Bottom Flange and Modified CJP Weld at Top Flange

Like Specimen 1, significant panel zone yielding was observed in this specimen. The global deformed and yielding pattern is shown in Figure 12. Brittle fracture of the rehabilitated beam top flange groove weld occurred during the second negative cycle at 4% story drift. As a consequence, fracture of the shear tab and the fillet weld connecting it the column was also observed. These fractures are presented in Figure 14. The measured cyclic response is shown in Figure 13.

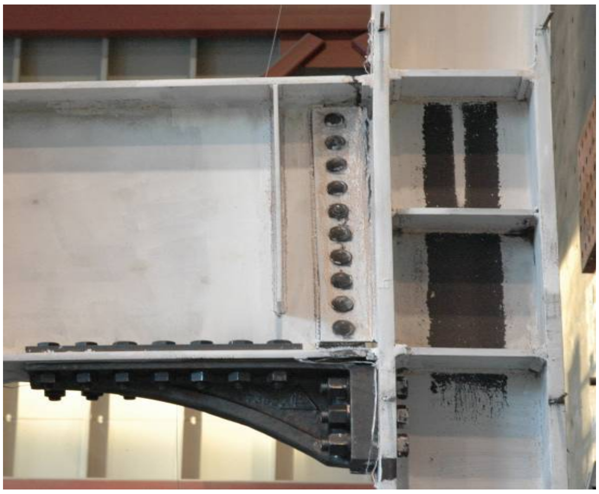


Figure 12 - Deformed Shape of Specimen 2 at 4% Story Drift

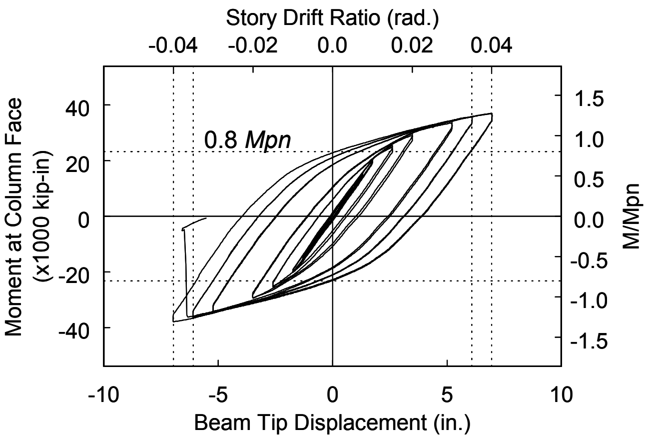


Figure 13 - Global Response of Specimen 2

(a) View 1 – Top Flange Fracture

(b) View 2 – Shear Tab Fracture

Figure 14 - Fracture of Specimen 2 at 4% Story Drift

Specimen 3 - KBB at Bottom Flange and Modified CJP Weld and Reinforced Top Flange

The specimen was successfully loaded up to two complete cycles at 4% story drift without fracture. Figure 15 shows the global deformed configuration, and Figure 16 shows the yielding pattern of the beam top flange. The specimen was then loaded repetitively at the 4.5% story drift level until fracture. Brittle fracture eventually occurred during the fifth negative cycle. The fracture occurred across the column flange and propagated into the column web as shown in Figure 17. A plot of the cyclic response is shown in Figure 18.

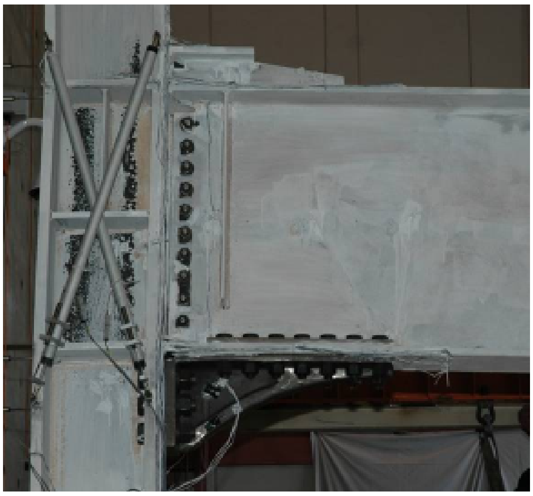


Figure 15 - Deformed Shape of Specimen 3 at 4% Story Drift

Figure 16 - Beam Top Flange Yielding Pattern of Specimen 3 at 4% Story Drift

Figure 17 - Fracture of Specimen 3 during 5th Negative Cycle at 4.5% Story Drift

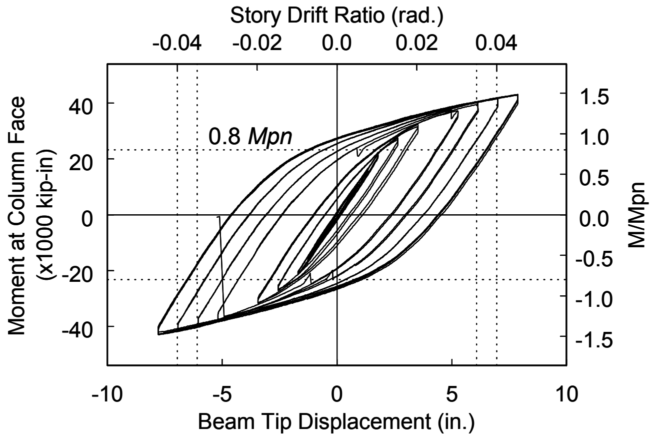


Figure 18 - Global Response of Specimen 3

Evaluation of Test Results

All of the test specimens passed the project specific interstory drift target of 0.035 radians and were considered acceptable. With identical beam and column sizes, all three specimens had very similar performance at least up to the 4% drift cycles and were dominated by weak panel zone behavior. Due to the increased panel zone height due to the KBB, Specimen 1 appeared to experience greater beam flange local buckling which was also desirable. The eventual fracture through the column in Specimen 3 was not considered significant as it occurred on the 5th cycle at 4.5% drift.

Specimens 2 and 3 were originally designed as an alternative approach where the presence of existing GFRC connections prevented the installation of the KBB at the top flange without significant modifications to the GFRC. Incorporating Specimen 2 was considered a bit more cost effective than Specimen 3 which would also require modification of the GFRC supports. The fracture at the top flange of Specimen 2 during the second cycle of 4% story drift, however, prompted the design team to abandon this concept and implement the concept of Specimen 3 at the exterior.

Analysis

In an effort to provide insight into the behavior of the test specimens both with and without the brackets, analysis was performed using nonlinear finite element models. The analysis utilized a general purpose finite element modeling program, ANSYS/Multiphysics (2008). Finite element models were created for a baseline unmodified connection and for first two tested connections, Specimens 1 and 2.

Modeling Parameters

The finite element models were constructed from a quadrilateral mesh of four node nonlinear shell finite elements as illustrated in Figure 19. Each shell node has six degrees of freedom, three translational and three rotational. The shell elements have plasticity, stress stiffening, large deflection, and large strain capabilities.

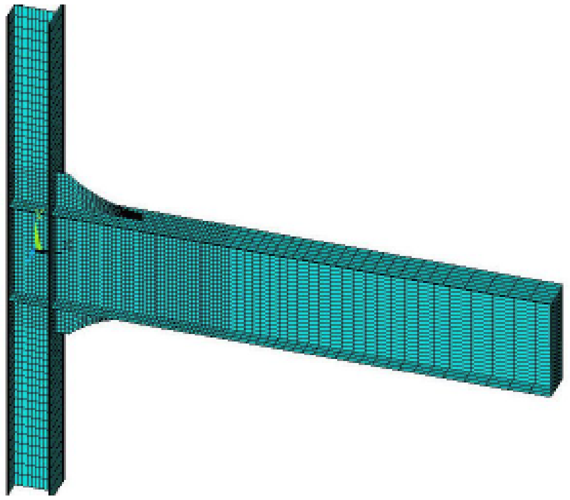


Figure 19 - Finite Element Mesh of Specimen 1 Model

To simplify the analysis, bracket and shear tab connections were modeled as rigid (no bolting). The analysis was concerned primarily with the inelastic performance of the overall connection and was not intended to address the localized bracket-to-column, bracket-to-beam, or shear tab bolting.

Model Boundary Conditions

To restrain the models, boundary conditions were applied to simulate that of the actual test setup. For example, the top and bottom of the columns were restrained against out-of-plane movement and a vertical reaction was provided at the column base. The top and bottom beam flanges were restrained against out-of-plane movement approximately 12 feet from the column centerline and at the beam end.

Model Material Properties

The modeled specimens were representative of ASTM A992 steel. The yield strength was defined at 53 ksi with an elastic modulus of 29,000 ksi and a Poisson's ratio of 0.3. Data from Lehigh University cyclic coupon testing (Kaufman et al. 2001) was used for the material plasticity parameters.

The selected shell elements use the associate flow and kinematic hardening rules to determine plastic straining direction and to describe changing yield surfaces. Deflection and strain geometric nonlinearities associated with buckling are accounted for through a small strain, large displacement formulation.

Model Incremental Cyclic Loading History

The models were subjected to the incremental loading history specified in AISC 341, up to a 4% story drift. In order to limit computational processing, the number of cycles in each load step was limited to one. To achieve the specified drifts, vertical displacements were imposed at the beam free end, similar to that of the test specimens.

Analytical Results

Baseline Unmodified Connection

In the baseline unmodified connection model, the analysis indicated initial yielding at 1% story drift. Yielding initialized in the column panel zone between continuity plates and locally at the intersection of the beam and column flanges.

With increasing levels of imposed drift, yielding and strain hardening enveloped the entire column panel zone and remained concentrated in this region. A plastic hinge subsequently formed in the column panel zone. The deformation and stress contours corresponding to 4% story drift in the unmodified connection model are shown in Figure 20. Deformation in the figure is representative of a downward beam deflection.

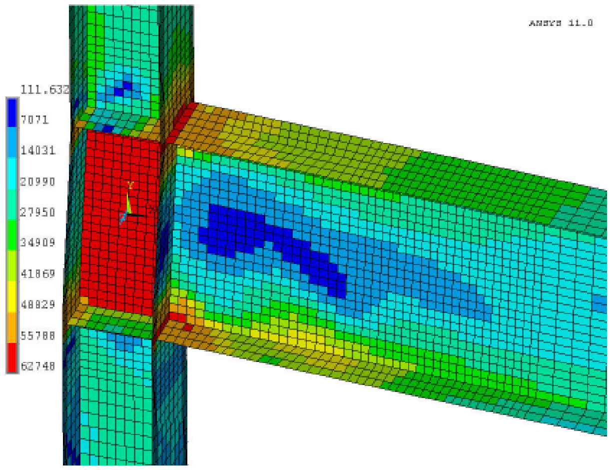


Figure 20 - Unmodified Connection Deformation with Von Mises Stress Contours at 4% Story Drift

Specimen 1 – KBB at Top and Bottom Flanges

As with the baseline connection model, yielding initialized in the panel zone of the Specimen 1 model. However, with increasing cycles of story drift, rather than being concentrated within the region bounded between the continuity plates, yielding of the column web was distributed beyond to match the depth profile of the brackets. In addition, yielding and local buckling deformations appeared in the beam web and flanges near the ends of the brackets. A plastic hinge subsequently formed in this same region. Following formation of the beam plastic hinge, stress levels in the column panel zone decreased notably.

The deformation and stress contours corresponding to 4% story drift in the Specimen 1 model are shown in Figure 21. As with the previous figure, deformation is representative of a downward beam deflection. At this level of story drift, large inelastic stresses and local buckling deformations are concentrated in the beam plastic hinge region. In comparison, panel zone stress concentrations are relatively moderate.

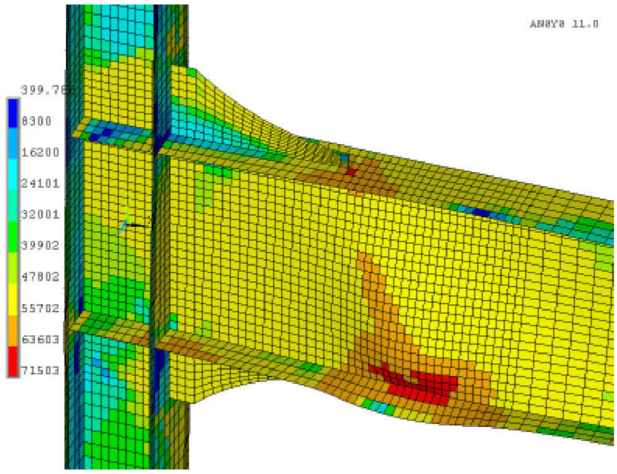


Figure 21 - Specimen 1 Deformation with Von Mises Stress Contours at 4% Story Drift

Specimen 2 - KBB at Bottom Flange and Modified CJP Weld at Top Flange

As with the previous models, yielding again initialized in the panel zone of the Specimen 2 model at 1% story drift. Localized yielding also occurred at the intersection of the upper beam and column flanges.

With increasing levels of imposed drift, yielding and strain hardening enveloped the entire column panel zone and extended below the lower continuity plate, matching the depth of the bottom bracket. Without a matching bracket on the top flange of this specimen, a plastic hinge was unable to form in the beam. The hinge subsequently formed in the column panel zone. The deformation and stress contours corresponding to 4% story drift in the Specimen 2 model are shown in Figure 22.

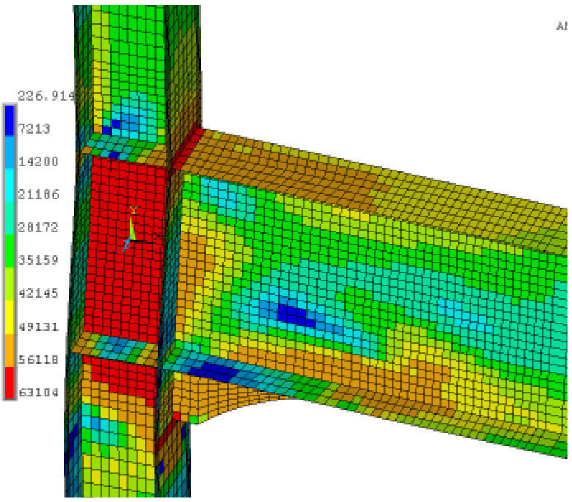


Figure 22 - Specimen 2 deformation with Von Mises Stress Contours at 4% Story Drift

Evaluation of Analytical Results

In both the baseline and Specimen 2 models, yielding in the panel zones and column flanges hindered the ability of the connection to force any significant flexural yielding into the beam. This condition allowed a plastic hinge to form in the panel zone. In the Specimen 1 model, the use of top and bottom brackets appears to have adequately strengthened the connection's ability to force flexural yielding and plastic hinge formation into the beam. This strengthening phenomenon was confirmed during testing when beam plastic hinge formation occurred in Specimen 1 (Figure 23).

Figure 23 - Specimen 1 Plastic Hinge Formation as Indicated by Paint Flaking and Local Buckling

While the analysis does not address the issue of fracture propagation, it can indicate regions that would facilitate fracture if a flaw or other irregularity were introduced. For example, in the Specimen 1 model shown in Figure 21, stress contours within the beam plastic hinge region approach the ultimate tensile strength of the material. This level of inelastic stress would be indicative of a potential for fracture to occur in this area. As with previous bolted bracket testing (Adan and Gibb, 2009), the material in this region would be susceptible to low cyclic fatigue and the initiation of a ductile fracture through the flange gross area.

In the Specimen 2 model, the top beam-to-column flange connection assumes a CJP weld. Considering the presence of pre-Northridge low notch toughness weld material, the analysis results, shown in Figure 22, indicated that the top flange weld would be subjected to considerable levels of inelastic stress. Hence, the Specimen 2 (bottom-only bracket) configuration would most likely result in a premature top flange brittle fracture in the presence of the pre-Northridge weld. This confirmed the requirement to remove and replace the existing top flange weld in conjunction with the bottom-only bracket retrofit configuration.

Although Specimen 3 was not modeled, based on the performance of the specimen in the laboratory, the results can be assumed to be similar to that of the Specimen 2 model.

Conclusions

Based upon the project specific testing and analysis, the following conclusions can be drawn regarding the use of the KBB to strengthen pre-Northridge moment connections:

1. The addition of bolted brackets on both the top and bottom beam flanges was an effective method of strengthening the pre-Northridge moment connections typically occurring with the subject building and was able to force flexural yielding and plastic hinge formation into the beam.

2. The bottom-only bracket configuration is not an effective retrofit alternative if the connecting top flange weld contains flaws or low notch toughness material.

3. Significant panel zone yielding did not adversely influence the performance of the KBB and specifically the bracket bolts to column connections.

Based on the teams experience on this project and their previous involvement with project specific testing, the following is a list of lessons learned from the process:

1. Project specific testing can be a very lengthy and expensive process but can save significant construction dollars where well applied.

2. Project planning to include adequate budget and schedule needs to be addressed if project specific testing is required.

3. Existing conditions should be verified to identify any parameters that could affect the proposed strengthening configuration prior to testing.
4. It is important to have well defined test specimen details and specifications as well as testing protocol in order to obtain competitive bids for fabrication and testing

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