DESIGNING AFTER NORTHRIDGE

A California engineering firm redesigned a project's connections to ensure adequate seismic performance

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IN LATE 1992, OVE ARUP & PARTNERS CALIFORNIA (OAPCAL) was commissioned to provide building engineering services—ranging from structural to acoustics—for new facilities at the City of Hope National Medical Center in Duarte, CA. The master plan included new buildings for patient care, research, medical offices, outpatient clinic and central plant. As was typical for buildings in California, a steel moment frame was selected for the majority of the structures.

Unfortunately, the January 17, 1994 Northridge earthquake caused widespread damage in steel moment-resisting frames, resulting in emergency code changes and necessitating a redesign of the buildings using either an alternate lateral force resisting system or an acceptable moment connection.

DESIGN OBJECTIVES

The following project-specific objectives were set at the beginning of the connection development phase and prior to concept selection:

- The new connection should not affect the previously designed steel moment frame system The change in beam stiffness and flexural demands imposed on the columns should be negligible.
- The onset of local flange/web buckling should essentially be delayed beyond the superstructure maximum plastic beam rotation demand, based on Upper Bound Earthquake (UBE), 950-year return period. Furthermore, the connection should have reserve capacity to resist future earthquakes following the UBE event.
- The through-thickness demands on column flanges should be reduced to about 75% of beam yield stress. The beam flanges and welds at the beam/column interface should behave essentially in an elastic mode.
- The new connection concept and configuration should be applicable to weak-axis connection configuration without major modifications.
- The new connection should be constructable in the same manner as the "Pre-Northridge" connection to keep the cost premium to a minimum. Total cost premium should be at or below the premium costs asso-

ciated with other available revised connections.

- The possible failure zone at ultimate connection capacity should be located away from the column and field welds to enable repairs, if required.
- The new connection should meet interim SAC performance criteria and be acceptable to OSHPD and other local agencies.

CONCEPT SELECTION

The study of all of the different options led to the selection of the beam flange area reduction concept with the following features:

- Reduce beam flange area by drilling holes or by flame cutting a tapered profile.
- Use welded vertical rib plates at beam/column interface.
- Use welded shear plate.

Design of MRFs is generally governed by drift limitations, that is, stiffness, not strength, dictates member sizes. Unfortunately, the over-strength (or excessive strength) of beam flanges at yield imposes large force demand to groove welds between column and beam flanges. Because of this over-strength, weakeng the beams is feasible for steel MRFs. Introducing a "fuse" by reducing beam flange width close to the beam-column connection has little effect on member or overall lateral stiffness; but the flange force that can be transmitted to the connection is controlled. For example, cutting 2⅝" on each edge of W36x150 beam flanges reduces the plastic moment capacity by about 30%. The only limitation is that the strength at the reduced section of the steel beam should still satisfy the code-prescribed strength requirement, which for steel MRF design is an easy task.

Reducing beam moment capacities not only reduces the force demand on flange connections but also provides the following advantages:

- The strength requirement for the through-thickness column flange stresses and horizontal stiffeners (i.e., continuity plates) is reduced.
- The strength requirement for the panel zone is reduced.
- And the weak beam-strong column requirement is easier to achieve.

Furthermore, the longer "predetermined" plasticity zone enables the achievement of larger plastic rotations at lower plastic strains, thereby delaying the onset of local buckling.

Specimen	Beam	Z_x	Z_x	$\frac{Z_x}{Z_x}$	$\frac{b_f}{b_f}$	$\frac{b_f}{2t_f}$	$\frac{b_f}{2t_f}$	$\frac{h_c}{t_w}$
Arup-1 (Strong-Axis)	W36x150	581	409	0.70	0.58	6.4	3.6	51.4
COH-1 & -2 (Strong-Axis)	W27x178	567	401	0.71	0.55	5.9	3.7	33.2
COH-3, 4 & 5 (Weak-Axis)	W33x152	559	388	0.69	0.63	5.5	3.1	46.8

Note: b_f and Z_x are the beam flange width and plastic sectional modulus at the narrowest section, respectively. h_c is the clear beam web depth.

Member	Specimen	Coupon	F_y (ksi)	F_u (ksi)	ε_u[3] (%)	$\frac{\Delta A}{A}$[5] (%)	Steel Supplier
Beam W36x150	Arup-1 (Strong-Axis)	Flange Web	55.5 62.5 (62.6)	73.0 77.0 (82.4)	29 25 (19)	68 66	British Steel
Beam W27x178	COH-1 & -2 (Strong-Axis)	Flange[1] Web[1]	44.0 48.0 (58)[6]	62.0 62.0 (72)	33 31 (29)	70 69	Nucor- Yamato
Beam W33x152	COH-3 & 4 (Weak-Axis)	Flange Web	57.6 62.0 (60.0)	78.5 84.5 (78.5)	26 23 (26)	65.0 60.0	U S Steel
Beam W33x152	COH-5 (Weak-Axis)	Flange Web	62.8 69.1 (66)	86.0 93.7 (98)	23 – (23)	63 57	U S Steel
Column W14x426	Arup-1	Web	(55)	(76.5)	(24)	---	British Steel
Column W14x455	COH-1 to -5	Flange[1] Web[1,4]	55.0 54.0 (51)	84.0 86.0 (68)	27 21 (26)	58 55	British Steel

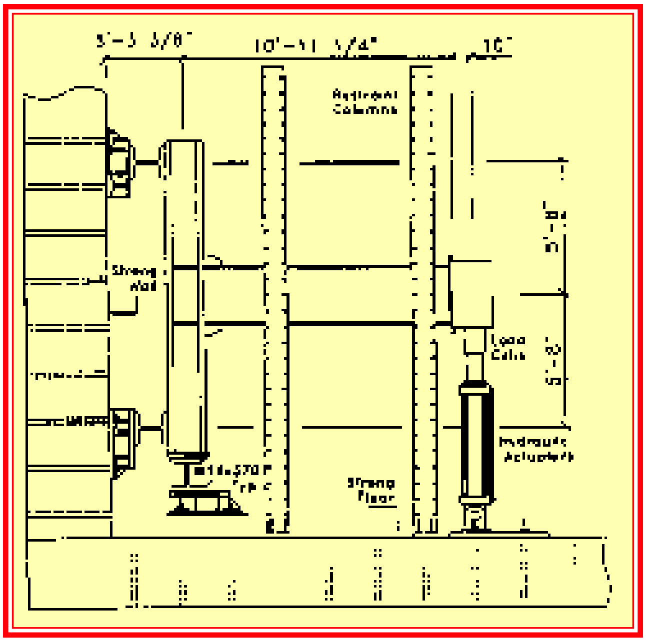
NOTES

1. Tabulated values are based on the average of two coupon tests.
2. Values in parentheses are from Mill Certificate Reports.
3. Elongation is based on 8 in. gage length.
4. F_y is based on 0.2% strain offset.
5. $\frac{\Delta A}{A}$ = reduction of area
6. The mill certificates identified all beam and column material as ASTM A572 Gr. 50.

Several issues also arise by using reduced flange sections: lateral bracing requirement as a result of reduced radius of gyration (r_y or r_T) about the weak axis of the beam; web buckling due to increased flange buckling stress ($b_f/2t_f$) and possibly due to reduced lateral restraint provided by the beam flanges; location, length and contour of the reduced section; and workmanship such as cutting method and edge grinding requirements.

Welding rib plates to beam flanges has been shown to be effective in reducing stresses in flange groove welds. By shifting the critical section from the column face to the tips of the rib plates, researchers have demonstrated that the cyclic performance of moment connections can be significantly improved.

Welded shear plate at beam web, as opposed to a standard bolted connection, was chosen to transfer the



Pictured above is the test setup for Ove Arup's design.

bending and shear stresses in the beam web directly to the column.

Non-Linear FEA

The non-linear finite element analysis (FEA) technique was used to investigate the proposed connection performance. Prior to the testing program, various configurations were analyzed to allow the evaluaiton and proportioning of different features as well as verification of the simplified connection design procedure. Among the many schemes analyzed were: constant size drilled holes at beam flanges; varying size drilled holes in beam flanges; and taper-cut beam flanges. The beam flange area reduction was based on the beam plastic moment gradient to ensure yielding in the "structure fuse" region. The initial FEA analyses were performed using the ANSYS program. Since material properties could not be estimated with reliable accuracy, the FEA work was based on "$Fy_{\text{-beam}}$ + 1.0 σ" as required by OSHPD interim guidelines.

The results indicated that the taper-cut flange scheme achieved the target beam plastic rotation at much lower strain levels. The plastic straining in the reduced beam flange region was uniform as opposed to clear strain concentration regions occurring at the beam flanges with drilled holes. Thus, it became evident that the taper-cut flange scheme performed better than the drilled-hole schemes. Further, the behavior of the model at a plastic rotation of 1.5 to 2% radians was without apparent local buckling. The other observation was that the average stress level at the beam-column interface was about 65% of the beam yield stress.

Considering the performance and cost effectiveness, the taper-cut flange scheme was selected over the others for full-scale connection testing.

Test Specimens

The scope of the full-scale moment connection testing, which was conducted at the Charles Lee Powell Structural Laboratories at the University of California (San Diego) was set as three strong-axis and three weak-axis connections. A572 Gr. 50 steel was specified for all beams, columns, shear plates, continuity plates and rib plates. The specimens were fabricated and "field" erected by AISC-member Herrick Corporation and other fabricators. The beam flanges were tapered by a computer-controlled flame cutting process to within 1⁄16" of the specified profile. The flame cut portion was then ground with a hand held grinder in a direction perpendicular to the mill rolling direction.

In accordance with the project specific welding specifications, the steel contractors qualified the Welding Procedure Specification by testing. Self-shielded fluxed core arc welding (FCAW) with E70TG-K2 electrode was used to make the beam flange groove welds in the flat position. For groove welds in other positions and fillet welds, FCAW with E71T-8 electrode was used. The back-up bar under the beam top flange was not removed; in such case the back-up bar was attached to the column and beam flanges by fillet welds along the complete bar length on the underside of the bar. All the groove welds were inspected per AWS section 9 (1994) and passed the ultrasonic testing.

The beam sectional properties for each sample are summarized in Table 1, while the mechanical characteristics are summarized in Table 2.

Test Results—ARUP-1

The deflection measurement was made at 123" from the face of the column. The panel zone rotation was negligible and it practically remained in the elastic region. The column response was also in the elastic range. The maximum beam plastic rotation was 4.1% radians based on a span (= 1.2") from the beam tip to the midpoint of the reduced section. The "M_p" computations, for the M/M_p axis, are based on the plastic modulus of the full "unreduced" section and F_y values from tension coupon tests.

The flange yielding at the reduced section was uniform and intensified with increased beam tip displacement. The web local buckling (WLB) developed prior to the flange local buckling (FLB). However, the initial WLB did not cause strength degradation.

The final failure of the specimen was by fracture at the narrowest section of the top flange in the second negative 5" cycle. The external lateral restraint adjacent to the reduced section did not possess sufficient stiffness. This allowed the beam flanges to laterally translate up to 3" in either direction. The lack of sufficient lateral restraint resulted in the lateral torsional buckling (LTB) to the beam.

No damage was observed at the vertical rib plate to column welds nor at the beam to column and beam to shear plate connections. Most of the yielding was concentrated within the reduced beam flange region.

Test Results—COH-1

The panel zone and column response were in the elastic range. The maximum beam plastic rotation was 6.4% and 3.5% radians, in the positive and negative directions respectively. This is based on a span (= 106") from the beam tip to the midpoint of the reduced section.

The flange yielding at the reduced section was uniform and intensified with increased beam tip displacement. Minor WLB developed prior to the FLB. However, the initial WLF did not cause strength degradation.

The final failure of the specimen was by fracture at the narrowest section of the top flange in the second negative 5" cycle. Subsequent to the top flange fracture, the beam tip displacement was reversed in the opposite direction in an attempt to rupture the bottom flange. The actuators reached their stroke limits before the bottom flange could be ruptured, and the test was stopped. The modified external lateral restraint adjacent to the reduced section effectively minimized LTB. The strength degradation, even at high plastic rotations, was minor.

No damage was observed at the vertical rib plate to column welds nor at the beam to column and beam to shear plate connections. Most of the yielding was concentrated within the reduced beam flange region.

Test Results—COH-2

The panel zone and column response were in the elastic range. The maximum beam plastic rotation was 4.5% and 5.6% radians, in the positive and negative directions respectively. This is based on a span (= 106") from the beam tip to the midpoint of the reduced section.

The observed performance was very similar to that of Specimen COH-1. The strength degradation, even at high plastic rotations, was minor. The final failure of the specimen was by fracture at the narrowest section of the top flange in the second negative 6" cycle. However, the displacement was incremented further by 1" to the actuators limit before the specimen was unloaded to stop the test.

No damage was observed at the vertical rib plate to column welds nor at the beam to column and beam to shear plate connections. Most of the yielding was concentrated within the reduced beam flange region.

Test Results—COH-3

The column response was in the elastic range. The maximum beam plastic rotation was 4.6% and 3.9% radians, in the positive and negative directions respectively. This is based on a span (= 103") from the beam tip to the midpoint of the reduced section.

The flange yielding at the reduced section was uniform and intensified with increased beam tip displacement. WLB developed prior to FLB. However, the initial WLB did not cause strength degradation.

The WLB, FLB and LTB became more apparent at a displacement amplitude of 4". A small crack in the narrowest section of the beam top flange was observed at the first cycle of negative 4" displacement. The final failure of the specimen was by fracture at the narrowest section of the top flange in the first negative 5" cycle. The test was stopped after the first positive 6" cycle.

As early as the 2" cycle, the beam bottom flange started to exert a lateral force on the lateral restraint guide column adjacent to the reduced section, indicating a tendency for LTB.

No damage was observed at the vertical rib plate to column welds nor at the beam to column and beam to shear plate connections. Most of the yielding was concentrated within the reduced beam flange region.

Test Results—COH-4

The column response was again in the elastic range. The maximum beam plastic rotation was 5.4% and 4.6% radians, in the positive and negative directions respectively. This is based on a span (= 103") from the beam tip to the midpoint of the reduced section.

The flange yielding at the reduced section was uniform and intensified with increased beam tip displacement. WLB developed prior to FLB. However, the initial WLB did not cause strength degradation.

The WLB, FLB and LTB became more apparent at a displacement amplitude of 4". A small crack in the narrowest section of the beam top flange was observed at the second cycle of negative 5". The LTB, FLB, WLB and the small crack contributed to the strength degradation. The final failure of the specimen was by fracture at the narrowest section of the top flange in the first negative 6" cycle. The test was stopped after the first positive 7" cycle.

As early as the 2" cycle, the beam bottom flange started to exert a lateral force on the lateral restraint guide column adjacent to the reduced section, indicating a tendency for LTB.

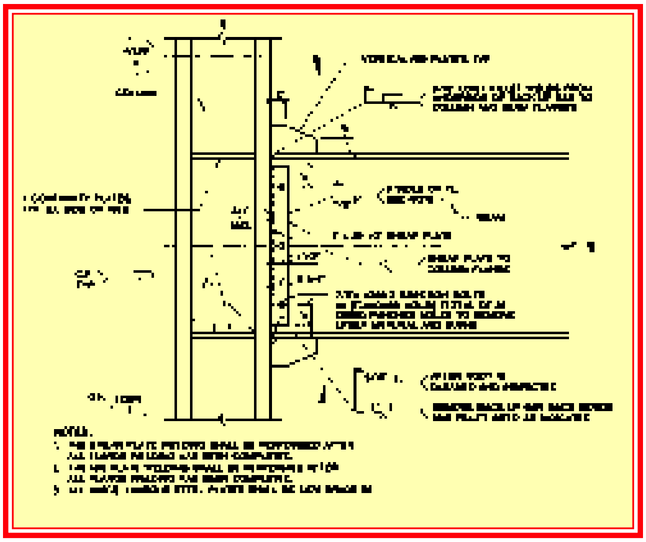
No damage was observed at the vertical rib plate to column welds nor at the beam to column and beam to shear plate connections. Most of the yielding was concentrated within the reduced beam flange region.

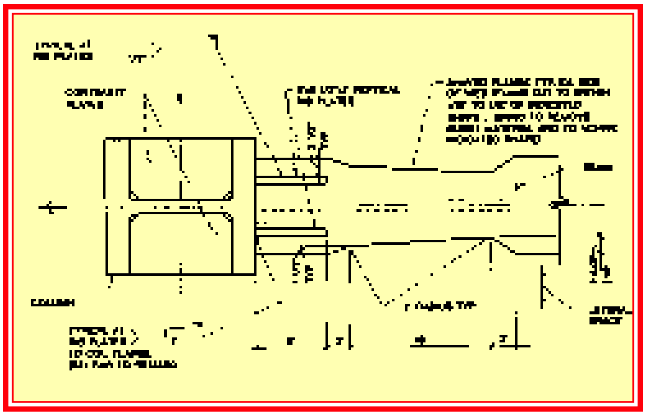
Test Specimen—COH-5

The column response was in the elastic range. The maximum beam plastic rotation was 2.1% and 2.2% radians, in the positive and negative directions respectively. This is based on a span (= 103") from the beam tip to the midpoint of the reduced section.

The flange yielding at the reduced section was uniform and intensified with increased beam tip displacement. WLB developed prior to FLB.

Minor FLB was observed during the 3" cycle. At the first cycle of positive 4", LTB was noticed as the beam top flange began to push against the guide column. During the 4" displacement cycles, yielding continued to increase in both the flanges and web; however, neither FLB nor WLB increased dramatically compared to previous cycles. The final failure of the specimen was a sudden fracture, which began at the narrowest section in the beam top flange and extended into the beam web during the second cycle of negative 4". It appears that the earlier failure is attributed to the very high beam yield strength. The yield and





Shown is the strong axis connection with a W27x178 beam and W14x455 column.

ultimate strengths of the beam used for specimen COH-5, based on tensile and coupon tests, was approximately 10% higher than the beam material used for specimens COH-3 and COH-4. Accordingly, this higher strength material had a lower ultimate strain capacity.

As was the case in all five previous specimens, no damage was observed at the vertical rib plate to column welds nor at the beam to column and beam to shear plate connections. Again, the yielding was concentrated within the reduced beam flange region.

Correlation & Comparison

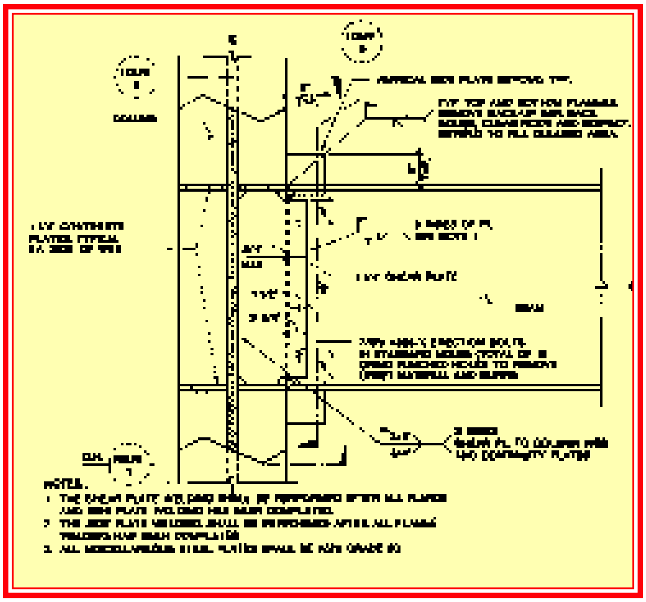
One of the ANSYS FEA models used for the initial development was translated into a DYNA3D model to perform inelastic cyclic analysis matching the displacement history imposed to the test specimen. The actual material properties, obtained from coupon tests, were used in this analysis. A close match was found between the computer simulation and the actual testing. More importantly, the close match of force vs. displacement relationship from the FEA and the test is a further verification that the new connection behavior is predictable by computer modeling.

The new connection was developed for the buildings under the jurisdiction of OSHPD. Currently, it is being used in the construction of the Ambulatory Care Clinic building at the City of Hope. This four-story building has a steel moment frame lateral system and was reviewed by the County of Los Angeles. The analysis of the superstructure, with the tapered beam properties, revealed that the impact of the building drift was negligible. The strength requirements also were not compromised. All moment frame beams have been provided with lateral bracing near the reduced flange zone.

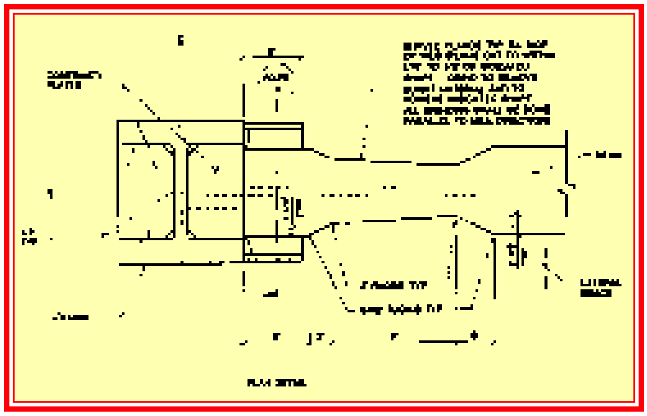
Conclusions & Recommendations

Based on the test results, the following conclusions and recommendations are made:

- The test results indicate high beam plastic rotation capacities while maintaining considerable flexural strength. It is important that a strength and/or energy dissipation capacity limit be introduced to allow for a more realistic comparison of test results from different connection concepts, present and future. Nevertheless, the application of large plastic rotation capacities, without considering the strength degradation or rotation demand range, to justify a particular moment frame system would not be a rational approach.
- Shaving beam flanges increases beam buckling stress due to reduced width/thickness ratio. As a result, WLB occurred prior to FLB during tests. The reduced lateral restraint provided by the shaved beam flanges may also have contributed to WLB. The WLB was followed shortly by FLB and finally by LTB of the beam. It was not until all of these buckling modes occurred that the strength degradation began.
- Lateral bracing at or near the reduced flange portion is very important to avoid rapid strength degradation. Bracing force as high as 3% to 6% of the beam flange force was observed in this testing program.
- Brittle failure of welded connections did not occur. Instead, stress concentration caused significant yielding of the beam at the narrowest section. Stress concentration at the re-entrant corner eventually led to fracture of the beam flange at the



Shown is the weak axis connection with a W33x152 beam and W14x455 column.



narrowest section.

- Vertical rib plates appeared to assist in reducing stresses of groove welds between the beam and column flanges.
- Beams with high yield and ultimate strengths appear to have achieved smaller plastic rotations. It appears desirable to set an upper limit on material strengths.
- Proper grinding of the reduced beam flanges, especially the re-entrant corners, should be done with care. The narrowest section, which had the sharpest corner and the smallest flange area, was the critical section for all specimens. In addition, sharp 90-degree edges of the cut flanges should be ground to a smooth radius.
- For the strong-axis specimens, the back-up bar under the beam bottom flange was removed whereas for the top flange it was left in place. In such cases, the back-up bar was fillet welded to the column and beam flanges along its complete length on the underside of the bar. The tests indicated this approach had no detrimental effects on the beam top flange groove weld.

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