The Dogbone Connection: Part II

Selectively trimming a portion of a beam allows connection strength to exceed beam strength without the need to develop a stronger connection

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SINCE THE 1994 NORTHRIDGE EARTHQUAKE, INTENSIVE RESEARCH AND TESTING EFFORTS have been underway to find better methods to design and construct seismic resistant steel moment connections. A wide variety of solutions have been proposed over the last two years, many of which have shown greatly improved performance in the laboratory compared to the previous "Pre-Northridge" connection. The overall goal in the development of new connections is to provide highly ductile response, reliable performance, and economy.

Many of the new moment connections combine improvements in welding with some type of reinforcement at the connection (cover plates, ribs, haunches, side plates, etc.). The purpose of the reinforcement, in the most general terms, is to provide a connection that is stronger than the beam. A strong earthquake would be expected to develop plastic hinges at the beam ends in a traditional fully restrained (FR) moment frame. The reinforcement is intended to force the plastic hinge away from the face of the column, where premature fractures can occur due to potential weld defects, stress concentrations at weld access holes, stress concentrations due to column flange bending, high levels of restraint and the associated states of triaxial tension, etc. The reinforcement reduces stress levels within this vulnerable region near the column face, and forces the large stresses and inelastic strains further into the beam. Reinforcing the connection, however, increases its cost. Further, if excessive reinforcement is used, new problems can be created resulting from the need for very large welds with high shrinkage, and higher degrees of restraint and triaxial tension.

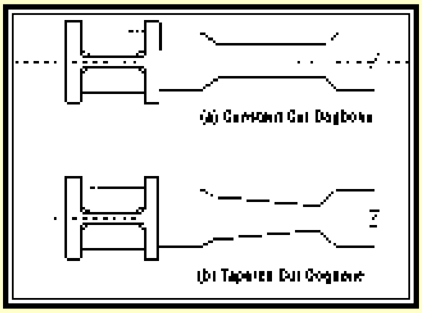


Figure 1: Typical "dogbones"

An alternative to reinforcing a moment connection that provides benefits similar to reinforcement, but may avoid some of the disadvantages, is the "dogbone" moment connection. A distinguishing feature of the dogbone, also known as the Reduced Beam Section (RBS) connection, is that portions of the beam flange are trimmed away in the region adjacent to the beam-to-column connection (Figure 1). Various shape cutouts are possible. The result is similar to reinforcement, i.e., the connection is stronger than the beam. In this case, however, the connection is made stronger than the beam not by increasing the strength of the connection, but rather by decreasing the strength of the beam. The dogbone can be viewed as a ductile fuse. It forces yielding to occur within the reduced section of the beam, an area that can sustain large inelastic strains. At the same time, the dogbone acts as a fuse, limiting stress at the less ductile region near the face of the column. Previous work has been conducted on the constant cut and tapered cut dogbone shapes shown in Figure 1. In the case of the tapered cut, the taper is intended to follow the moment diagram in order to promote uniform yielding over the full length of the dogbone.

At first glance, it may seem counterintuitive that removing a portion of a structure actually improves the performance of the structure. In the case of the dogbone, however, this is exactly the case! The dogbone results in only a small reduction in lateral strength and stiffness of a frame but can provide a large increase in ductility, the key to survival of a structure in a strong earthquake. Trading a small amount of strength in return for a large amount of ductility represents an excellent bargain for earthquake resistant design.

The size of moment frame beams is normally controlled by code-mandated drift limitations rather than by code strength requirements. The dogbone generally results in only a slight

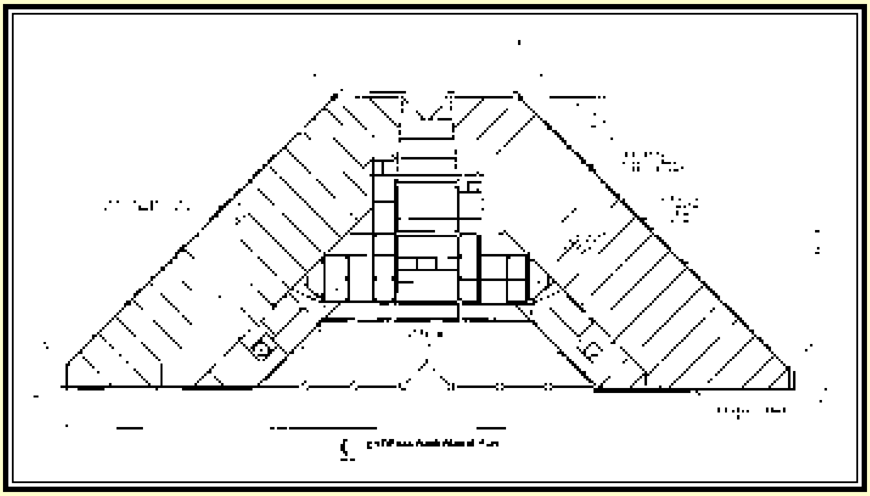


Figure 2 (above): Typical floor framing plan

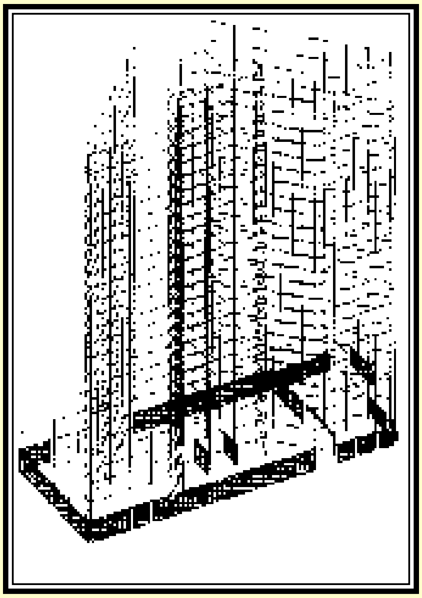


Figure 3 (left): 3D view of lateral framing members

decrease in frame stiffness. Consequently, in many practical cases, the use of the dogbone will likely not necessitate a change of beam size in order to meet code strength and stiffness requirements.

A significant amount of research and full scale testing has already been done on the dogbone concept. An article by Nestor R. Iwankiw, P.E., and Charles J. Carter in the April 1996 Modern Steel Construction summarized some of the recent testing. Early work was conducted by S.J. Chen and C.H. Yeh in Taiwan, and by A. Plumier in Belgium. More recently, additional work has been conducted by AISC at Smith-Emery Company in Los Angeles, and by Ove-Arup & Partners working with the University of California at San Diego. This previous research, conducted on either constant cut or tapered cut dogbones (Figure 1) has shown very promising performance. Typically, large plastic rotations have been obtained. In a few cases, fractures developed during testing, either at the beam flange to column connection or within the dogbone, but only after large levels of plastic rotation were achieved.

Based on the earlier laboratory successes of the dogbone, this concept was recently applied to a 25-story steel office building in Salt Lake City. In applying this concept, some further refinements were made to the dogbone approach and verified by full scale laboratory testing. The remainder of this article describes this building project, and the associated connection testing program

American Stores Company Headquarters

When the American Stores Company and Howa Construction Company decided to build the new American Stores Headquarters Building in Salt Lake City, HKS Inc. of Dallas designed a 25 story, 650,000-sq.-ft. structure with steel moment frames as its primary lateral force resisting system. W&W Steel Company of Oklahoma City was responsible for detailing, fabricating and erecting the building's 7,900 tons of structural steel.

The typical floor of the building is triangular shaped in plan (Figure 2). The plan measures 302-ft. along the long face and 121-ft. along the width. A prominent feature of the plan is an enclosed atrium on the west side of the building. The atrium consists of a series of individual four-story and two-story open spaces that stack above each other throughout the height of the building. Surrounding the atriums are the elevator cores and lobbies, conference rooms and typical office space. The office space is designed for maximum flexibility and is programmed to be an open space plan layout, requiring 43-ft.-4-in. spans for the floor framing.

The floor framing was designed using composite steel beams with 3¼-in. lightweight concrete slabs over 3-in. composite deck. The AISC Load and Resistance Factor Design Specification (LRFD) was utilized in the design of the beams and columns. ASTM A572 Gr. 50 steel was used for all members.

The primary frames are located around the building perimeter and around the elevator cores. A three dimensional perspective illustration of the lateral framing members is shown in Figure

Table 1: Description of Test Specimens & Results

Spec.	Column	Beam	Dogbone Type	Plastic Rotation	Comments
DB1	W14x426	W36x160	Constant Cut	0.020 rad.	Fracture at dogbone
DB2	W14x426	W36x150	Radius Cut	0.030 rad.	No conn. failure
DB3	W14x426	W36x170	Radius Cut	0.038 rad.	No conn. failure
DB4	W14x426	W36x194	Radius Cut	0.037 rad.	No. conn. failure
DB5	W14x257	W30x148	Radius Cut	0.040 rad.	No conn. failure; signif. column panel zone participation

Table 2: Beam Tensile Properties

Spec.	Beam	Yield Stress (ksi)		Tensile Strength (ksi)	
		Flange	Web	Flange	Web
DB1	W36x160	54.7	53.5	75.6	79.2
DB2	W36x150	41.4	47.1	58.7	61.8
DB3	W36x170	58.0	58.5	73.0	76.7
DB4	W36x194	38.5	43.6	58.6	59.8
DB5	W30x148	46.6	48.5	64.5	65.4

3. W36 and W40 frame column sizes were utilized to control drift. The building design called for 1,000 moment resisting frame beams, ranging in size from W27x114 to W36x210.

Located in UBC Seismic Zone 3, earthquake safety was an important design concern for this building. The design and construction team was faced with the challenge of developing a moment resisting connection that would perform as intended by the UBC for a Special Moment Resisting Frame. Working together with The University of Texas at Austin, HKS and W&W Steel explored several options, including reinforced connections, bolted connections, and the dogbone connection. Connection design was somewhat complicated by the very short 17-ft. clear spans on many of the building's moment frame girders since connection concepts that move the plastic hinge away from the face of the column are more difficult to implement on short spans. The high moment gradients associated with short spans cause a large amplification of moment from the point of plastic hinge formation back to the face of the column. This, in turn, increases stress levels at the face of the column, and negates some the beneficial effects of forcing the plastic hinge away from the column.

Considering the options, the dogbone emerged as the most cost effective solution for this project. Although the dogbone already had a good track record in earlier testing, Andy Zekany of HKS requested additional testing using representative W36 beams. These tests would permit evaluation of details specific to this project, and help build confidence in the connection. Remarkably, the decision was made to embark upon this test program after construction on the American Stores Company Headquarters had already begun on an extremely fast track schedule. Stuart King with American Stores and Rick Howa of Howa Construction required the most effective and economical moment connection available for seismic-resistant steel framing. Their consistent direction in meeting this goal and their support of the engineers' recommendations led to the test program.

Test Program

A test program with five large-scale specimens was undertaken to evaluate suitability of the dogbone for use in the 2,000 moment connections of the American Stores Company headquarters building. The specimens, designated DB1 to DB5 are outlined in Table 1. The concept adopted for these specimens was to combine the dogbone cut in the beam, together with a high quality welded beam-to-column connection at the face of the column. Both welding and design improvements were incorporated into the beam-to-column connection to provide significantly improved capabilities compared to the Pre-Northridge type connection. With these improved connection capabilities, together with a reduced demand on the connection due to the presence of the dogbone, the hope was for a seismic resistant moment connection system capable of developing large plastic rotations under severe cyclic loading conditions.

For the beam-to-column connection at the face of the column, each of the five specimens used essentially the same detail. A typical detail is illustrated in Figure 4. An all-welded connection was used, in which the beam flanges and beam web were connected to the column flange using complete joint penetration groove welds. It was believed that the welded web connection, although somewhat more costly than a more conventional bolted web connection, would provide a higher level of ductility. The welded web connection can transfer significant moment from the beam to the column, thereby reducing demands on the beam flange welds. The relatively thin shear tab provided at the beam web connection serves to support the beam during erection and also acts as backing for the beam web groove weld.

Testing done since the Northridge Earthquake has raised questions on previous design criteria for continuity plates in seismic resistant moment connections. For this project, the decision was made to provide continuity plates at all connections, with the thickness chosen to be comparable to the beam flange thickness. For the test specimens, beam flange thickness ranged from 0.94 to 1.26 in. One-in. thick continuity plates were used for all specimens.

Welding at the beam-to-column connection was accomplished by self-shielded flux-cored arc welding process using an electrode with a minimum specified Charpy V-Notch toughness of 20 ft.-lbs. at -20 degrees F. For the beam flange groove welds, properly oriented weld runoff tabs were used, and then removed after completion of the weld. At the bottom flange groove weld, the backing bar was removed, and a reinforcing fillet weld was placed at the root of the weld joint. Removing the backing bar permits visual inspection of the weld root, minimizing the chance of undetected root defects, and eliminating any possible notch effects of the backing bar.

The top flange backing bar was left in place. There were several reasons for this. First, root defects are less likely at the top flange since neither the groove weld nor the ultrasonic testing of the groove weld is interrupted by the beam web, as they are at the bottom flange. Further, removal of the top flange backing bar is more difficult and costly than at the bottom flange, since the arc-gouging must be done through the weld access hole. Consequently, for this project the top backing bar was left in place. However, a continuous fillet weld was provided between the backing bar and the column flange (see Figure 4). From a theoretical perspective, this fillet weld reduces the potential notch effect of a left in place backing bar.

For welding quality control, both in-process inspection and ultrasonic testing were specified. Inspection and testing were based on AWS D1.1-94, *Structural Welding Code - Steel*. Items emphasized for in process inspection included checking the joint fit-up, preheat, and enforcing the Welding Procedure Specification (WPS) for the project. Ultrasonic testing was done for the complete joint penetration groove welds, with acceptance criteria based on Table 8.2 of AWS D1.1-94.

The member sizes used for the test specimens are listed in Table 1. Each of the test specimen columns were A572 Gr. 50 steel. Four of the specimens used W14x426 columns, taken from three different heats of steel. For the beam sections used in the tests, mill certificates were not available. Tensile coupon tests were run on the beam sections, and the results are shown in Table 2. Actual beam flange yield stresses varied from 38.5 ksi to 58 ksi. Consequently, a wide range of beam strengths were represented in the test specimens. Member sizes in Specimens DB1 to DB4 were selected to force virtually all of the inelastic deformations into the beam. Specimen DB5 was sized to encourage shear yielding of the column panel zone, in order to evaluate its effects on the dogbone connection.

Test Results

Testing was conducted at The University of Texas Ferguson Laboratory. All tests were conducted on single cantilever type specimens, with cyclic loads applied to the tip of the beam following the loading protocol of ATC-24. The goal of the test program was to develop plastic rotations of at least 0.03 radian, as suggested by the *SAC Interim Guidelines*.

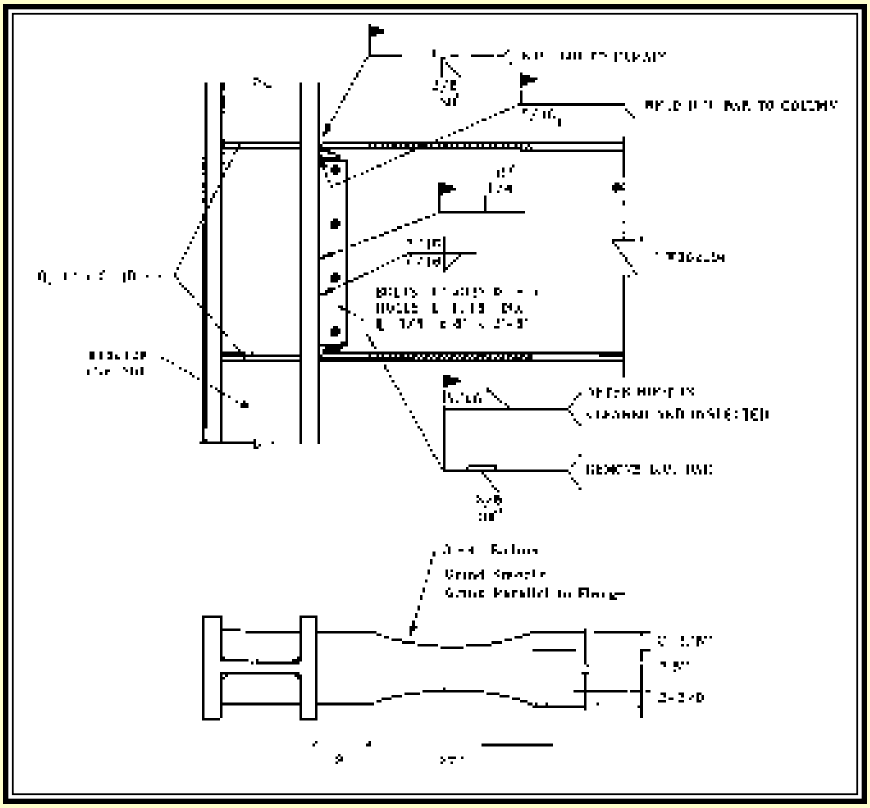
The first specimen, DB1, combined the all-welded connection described above with a constant cut dogbone (Figure 1(a)). Previous successful tests had been run on both constant cut and tapered cut dogbones. The constant cut was elected for this first test. During testing, Specimen DB1 showed excellent performance in the initial inelastic cycles. However, a fracture developed within the dogbone at the end of the flat portion of the cutout that was closest to the face of the column. A stress concentration resulting from the change of cross-section at this point apparently contributed to this fracture. Despite this fracture, the specimen still sustained 0.02 radian of plastic rotation. Further, there were no signs of distress anywhere within the beam-to-column connection at the face of the column.

Although the first test did not meet full performance expectations, the results were still encouraging. It appeared that achievement of the higher performance goal might be possible by changing the shape or dimensions of the dogbone cutout to reduce stress concentrations within this region. In order to provide a cutout region with minimum stress concentrations and one that would still be economical to fabricate, the concept of a circular radius-cut dogbone was suggested by Ted Winneberger of W&W Steel Co. This ultimately proved successful in the testing program.

The remaining four test specimens were all constructed with a radius-cut dogbone. Figure 5 shows the shape and dimensions. The same all-welded beam-to-column connection described earlier was used. A typical specimen, DB4, is shown in Figure 4.

Each of the four radius-cut dogbone specimens (DB2 to DB5) showed excellent performance. The plastic rotations developed by each specimen at the end of the test are listed in Table 1. In each case, testing was stopped as a result of limitations in the test equipment, rather than because of specimen failure. Consequently, the specimens would have likely developed larger plastic rotations had the testing been continued.

At large plastic rotations, the beams of the test specimens typically exhibited considerable



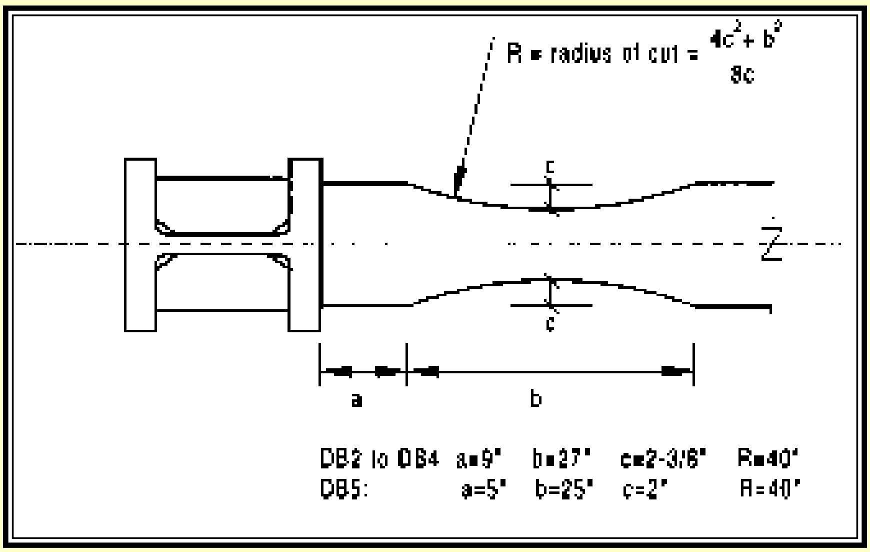


Figure 4 (top): Test specimen DB4

Figure 5 (above): Radius cut dogbones

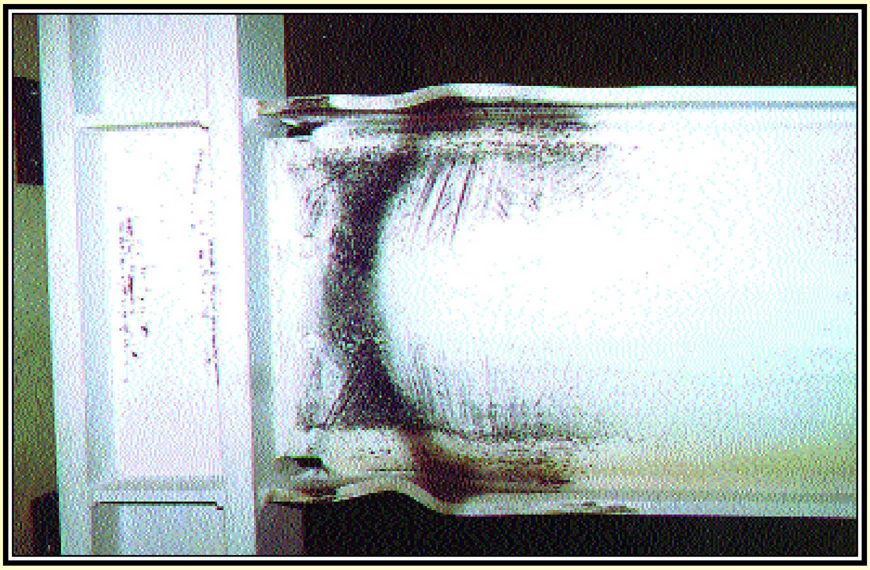
twist that accompanied local flange and web buckling in the beam. This resulted in a gradual reduction in flexural strength. Testing was stopped in each case to avoid damage to the test equipment from this twisting. It should be noted however that the twisting experienced by these dogbone specimens was no more severe, and perhaps somewhat less severe, than reinforced connections previously tested at The University of Texas.

At the point at which testing was stopped, no fractures had developed within the dogbone region for any of the radius-cut specimens. Further no failures occurred at the beam-to-column connections. The connection on Specimen DB3 showed small cracks near the weld access holes that developed during the cycles at 0.02 radian. These small cracks, however, did not grow during the remainder of the test and had no impact on the response of the specimen. For the remaining specimens, the weld access holes were enlarged and were provided with a smoother transition to the beam flange. The small cracks observed in DB3 were not observed in the remaining specimens.

Figure 6 shows photos of Specimen DB4 during testing. Figure 6(a) is a view looking down at the top beam flange at a plastic rotation of 0.012 radian. The darkened regions are areas where the whitewash coating has flaked off, providing a qualitative indication of where yielding has occurred. This photo shows yielding concentrated within the dogbone region of the beam, and indicates the connection was performing as intended. Figure 6(b) shows the specimen at plastic rotation of 0.022 radian. Finally, Figure 6(c) shows the specimen at a plastic rotation of 0.037 radian, the point at which testing was stopped.

As noted earlier, at large plastic rotations the specimens exhibited flange and web local buckling within the plastic hinge

Table 3: Cost Comparisons	
Connection Type	Relative Cost
Pre-Northridge welded flange-bolted web	1.00
Radius-cut dogbone	1.71
Constant or tapered cut dogbone	2.18
Cover plate	2.50
Flange ribs	2.91
Bottom haunch	3.01
Proprietary side plates	3.76
Side plates	3.96
Top & bottom haunch	4.00
Column tree	6.38



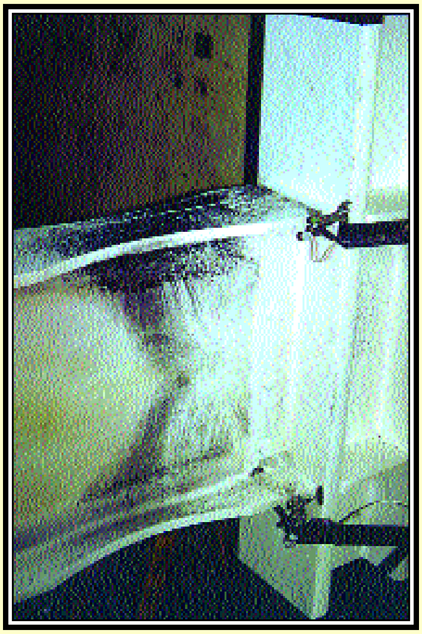


Figure 6a (above): View of beam flange of specimen DB4 at 0.012 rad. plastic rotation (column is at left)

Figure 6b (left): Specimen at 0.022 rad. plastic rotation

Figure 6c (top): Specimen at end of test; plastic rotation equals 0.037 rad.

region. Flange buckling is visible in Figure 6(c). Such local buckling has been observed in many tests on reinforced or other modified connection concepts. However, this severe local buckling typically occurs at very large plastic rotations. Figure 6(b) shows Specimen DB4 at a plastic rotation of 0.022 radian. At this level of demand, the degree of local buckling is still very slight, and in fact, is hardly noticeable. Analytical studies of steel moment frame buildings affected by the Northridge Earthquake suggest that most buildings experienced maximum plastic joint rotations on the order of 0.005 to 0.00 radian. Under this level of demand, these test specimens had no visible damage, other than yielding patterns indicated by whitewash flaking. Such connections would likely require no repair after experiencing this level of demand.

For all test specimens, with the exception of DB5, plastic rotations were developed by inelastic deformations in the beams. Specimen DB5 was designed to permit significant shear yielding of the column panel zone. During testing, a great deal of yielding was in fact observed in the panel zone. Analysis of test data indicates that of the total plastic rotation of 0.040 radian developed by DB5, approximately 25% was contributed by the column panel zone. For this specimen, inelastic deformations of the panel zone had no apparent detrimental effect on the connection, and contributed to the ductility of the test specimen.

Based on the successful results of this testing program, the overall cyclic loading performance of the radius-cut dogbone connection was judged as excellent.

Cost Comparisons

The test program indicated that the radius-cut dogbone connection was capable of providing a high level of performance. However, because of the large number of moment connections in this building, the cost of the dogbone connection relative to other options was also a critical concern.

To address the issue of cost, W&W Steel developed cost estimates for a number of moment connection options for this project. The results are shown in Table 3. Relative costs are provided with respect to the Pre-Northridge welded flange-bolted web moment connection. Cost comparisons were based on a W36x194 girder of A572 Gr. 50 steel with weld metal toughness requirements for all cases except

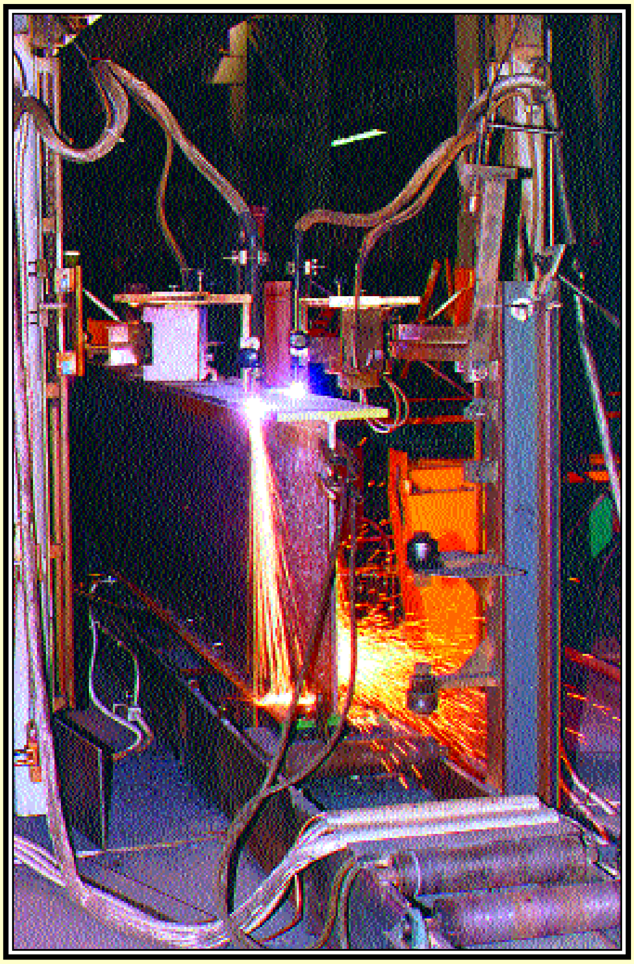


Figure 7: Mechanized plasma arc cutting of dogbones in fabrication shop

the Pre-Northridge connection. The various connection types listed in Table 3, other than the radius-cut dogbone, are connection options illustrated in the *SAC Interim Guidelines*. The cost data presented in Table 3 were developed for this project, and will vary for other projects depending upon location, labor factors, and connection design variations.

The cost comparisons for this project indicated that the radius-cut dogbone was the most economical alternative of all the options considered. Based on the successful testing program, combined with the favorable economics, the radius-cut dogbone moment connection was accepted for use on the American Stores Company Headquarters Building.

Design and Fabrication Considerations

In designing the radius-cut dogbone test specimens, it was necessary to establish dimensions of the dogbone. Key dimensions are shown in Figure 5, and were established based on the judgment of the writers. This judgment relied on these test results, results of previous dogbone tests, and experience with other connection test programs over the last several years. Based on this judgment, the distance "a" from the face of the column to the start of the dogbone cut was chosen to be approximately 50 to 75 percent of the beam flange width. The length of the cut, dimension "b" in Figure 5, was chosen in the range of about 65 to 85 percent of the beam depth. In general, it is preferable to keep the "a" and "b" dimensions as small as possible, in order to minimize the amplification of moment from the hinge location in the dogbone to the face of the column. The dimension "a" must be large enough, however, to permit stress in the reduced beam flange to spread uniformly over the flange width at the face of the column. Similarly, dimension "b" should be large enough to avoid excessive inelastic strains in the dogbone. Consequently, the suggested dimensions above are a compromise between different requirements.

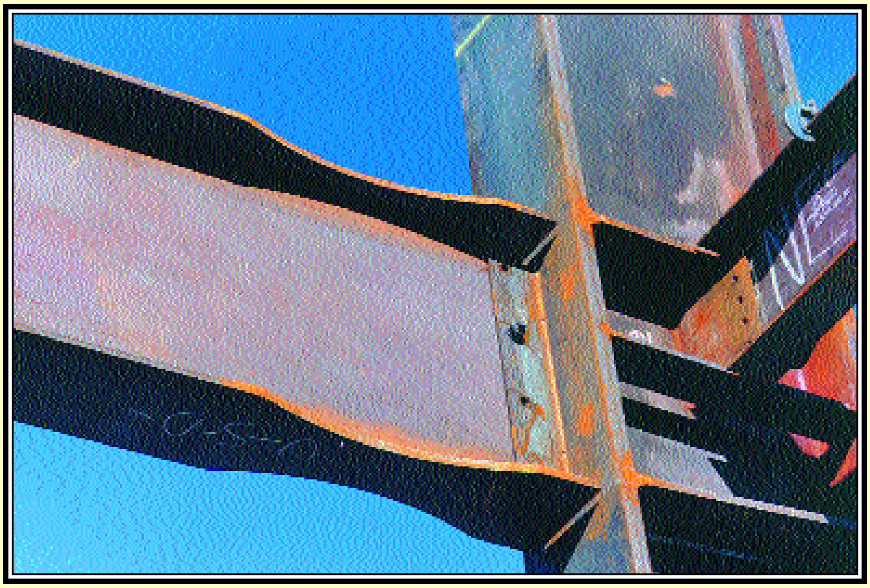
The depth of the cut, dimension "c" in Figure 5 is a critical dimension for the dogbone. The following approach was used to determine this dimension. First, it was assumed that the maximum moment that can develop at midlength of the dogbone was equal to the plastic moment of this reduced section, increased by 10 to 15 percent to allow for strain hardening. This moment was then projected to the face of the column using a moment gradient based on an assumed point of inflection at midspan of the beam. The maximum moment at the face of the column was then limited to approximately 90 to 100 percent of the plastic moment of the full cross section of the beam. It was believed that the all-welded connection combined with the welding details used in these tests was capable of sustaining such levels of bending moment. The reduced beam sections were also checked for conformance with flexural strength requirements of the LRFD Specification for all factored gravity and lateral load combinations. Following this basic approach, the depth of cut, "c," was chosen. For all of the radius-cut specimens, the depth of cut resulted in removing approximately 40 percent of the beam flange width.

The maximum moment developed at the face of the column can be reduced by increasing the depth of cut in the dogbone. The writers view removal of about 50 percent of the beam flange width as a maximum practical cut. Note that for a given depth of cut, a greater reduction in moment at the face of the column is possible in longer span beams with smaller moment gradients. If further reduction of stress at the face of the column is desired, some reinforcement could be added to the connection, for example, thin cover plates or ribs. In these tests, the actual maximum moments developed at the face of the column ranged from 92 percent to 113 percent of the actual plastic moment of the beam. These moments were sustained without connection failure. Consequently, no additional reinforcement was provided at the connection. It was felt that such additional reinforcement would largely negate the economy of this connection and was unnecessary.

Once the length of the dogbone "b" and depth of cut "c" have been chosen, the radius of the cut follows simply from the geometry of a circular arc, as indicated by the formula in Figure 5.

Once the radius-cut dogbone was accepted for use on the American Stores Company building, W&W Steel was faced with the task of cutting dogbones into both flanges of both ends of 1,000 moment frame girders. This required making 8,000 cuts. In order to make these cuts as economically as possible, W&W devised a mechanized process employing plasma arc cutting. Figure 7 shows two cuts being made simultaneously in the fabrication shop. The mechanized plasma arc cut left a very high quality surface. The cutting was followed by the use of a 3M paddle wheel grinder. Grinding was done parallel to the beam flange to minimize stress concentrations due to grinding marks.

Figure 8 shows views of moment frame girders with radius-cut dogbone moment connections during erection at the construction site in Salt Lake City. The structural steel framing is expected to be completed in November of 1996.



Figures 8a & 8b: Moment frame during erection

Conclusions

The test program conducted on the radius-cut dogbone connection showed excellent performance. Together with a number of other connection types devised and tested since the Northridge Earthquake, the dogbone connection promises to deliver a much higher level of ductility and safety as compared to the "Pre-Northridge" moment connection.

Structural engineers can now choose from a fairly wide variety of seismic resistant moment connection concepts that have shown good performance in laboratory cyclic loading tests. Engineers must carefully consider currently available test data combined with the economics of a project when choosing the most appropriate moment connection detail. This represents a rapidly changing picture in the current environment of intensive research and testing. Based on the experiences from this project, however, the dogbone connection appears to be one of the more promising moment connection concepts capable of delivering both performance and economy.

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