

## Analytical and experimental studies on the seismic response of steel column bases

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### ABSTRACT

This paper focuses on the experimental and analytical investigations of the seismic response of a typical exposed moment resisting steel column base. Specimen design, experiment setup and instrumentation are discussed first. Development of a 3D nonlinear finite element model is explained, along with the element properties used to model the column, the base plate, the concrete foundation, and the anchor bolts. Results obtained from the experiment are compared to the ones obtained from the finite element analyses. The comparison is carried out by examining normalized force-displacement and normalized moment rotation responses, the yield sequence, and the state of stresses in the base plate. Agreement of analytical and experimental findings is good, showing that the analytical model can be used to complete the development of performance based design provisions for steel column bases.

### INTRODUCTION

The 1994 Northridge and the 1995 Kobe earthquakes revealed seismic vulnerability of steel moment resisting buildings. Since then, the engineering community has been focused on beam to column connection failures. Other elements of steel moment resisting frames received much less attention. Yet, the results of a detailed survey of 600 steel buildings in Kobe, conducted by the Building Research Institute of Japan (Midorikawa, 96), show that column bases were the most commonly damaged structural element. The studies on cyclic behavior of connections other than beam to column joints are needed to establish the conditions under which they can reliably deliver the required design strength, stiffness, ductility, and plastic rotations. To date, practically no cyclic testing of moment resisting steel column bases designed according to the current US practice were performed. US codes lack seismic design and detailing provisions for steel column bases. The goals of our research are to examine the seismic behavior of moment resisting steel column bases, to distinguish ductile versus brittle failure mechanisms, and to develop performance based design provisions for steel column bases.

This paper focuses on the experimental and analytical seismic response of a typical exposed moment resisting steel column base. The column base connection specimen has a W10x77 A572 GR 50 column and a story height of 14 ft (4.2m.). Specimen design, experiment setup and instrumentation are discussed first. Development of a 3D nonlinear finite element model is explained, along with the element properties used to model the column, the base plate, the concrete foundation, and the anchor bolts. Results obtained from the experiment are compared to the ones obtained from the analysis (Fahmy, 98). Comparison of the results is carried out by examining normalized force-displacement and normalized moment rotation response, the yield sequence, and the state of stresses in the base plate. Agreement of analytical and experimental findings is good, showing that the analytical model can be used to complete the development of performance based design provisions for steel column bases.

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## LITERATURE REVIEW

Design procedures for steel column bases used in the US design practice today are limited. Design for bending alone and for combined bending and axial loads is done following the AISC Steel Design Guide 1 (DeWolf, 90) and different procedures suggested in various textbooks. The design procedures suggested in the AISC Steel Design Guide 1 is based on the experimental results of small scale test from column base connection subjected to axial load only (Dewolf 78, Murray 83, Hawkins 68) and to combined monotonic bending and axial loads (Dewolf 80, Stephenson 81, Thambiratnam 91). Other monotonic lateral loading experiments on exposed column bases (Picard 85, Picard 87, Targowski 93, Carrato 91) provided more insight toward the real connection behavior, but have not been used to support the AISC design provisions.

The seismic response of exposed steel column bases designed according to the US design practice was examined by (Astaneh, 92). All specimens were small scale and were non-moment resisting column bases. Experimental database on the behavior of column bases constructed following the Japanese practice is much more developed (Nakashima 79, Masuda 80, Ohtani 96). Unfortunately, column shapes, bolt configurations, connection details, and material properties used in these experiments are substantially different from those used in current US practice. Therefore, it seems that the column base design produced according to the current US design practice have not been examined either experimentally or analytically in a strong seismic demand setting. Yet these designs are used in steel frame buildings exposed to the highest level of seismic risk.

European efforts to predict the beam column connection stiffness, strength and ductility have been focused on a component based approach. In this approach the connection is considered as an assembly of individual components. The mechanical properties of these components define the overall connection properties. By introducing normal force, the component-based concept used for beam column connection was extended to include the column base connection (Wald, 92). The components considered in this method include: 1) concrete block in compression; 2) base plate in bending; 3) anchor bolt in shear and tension; and 4) column web and flange in compression. A series of monotonic load tests on individual components and complete column base connections have been conducted (Wald 93, Vandegans 98, Colson 92). These analytical and experimental results are used to develop new Eurocode design provisions.

## TEST SETUP AND INSTRUMENTATION

Our test setup is shown in Figure 1. The seismic shear acting at the column inflection point was simulated by applying a horizontal displacement at the top of the column specimen, 7ft (2.1m) above the base plate level. Thus, the column base was exposed to uniaxial bending. Horizontal displacements were applied in a cyclic quasi-static manner. The cycle amplitude was incremented in steps, following the SAC test procedure (ATC-24, 92). The column base connection was detailed and designed using an LRFD equilibrium method to form a plastic hinge in the column above the base plate. The connection design and details were reviewed and approved by the AISC advisory committee for this project.

The test specimen consists of a W10x77 A572 GR.50 column welded to a 20x20x2 3/4 in. (500x500x70mm.) A36 (250 MPa) base plate. Partial joint penetration (PJP) weld connects the column flanges to the base plate. Fillet weld with 1/4 in. throat connects the column web to the base plate and reinforces from the back the PJP weld used to join the flanges to the plate. Notch tough E70 TG-K2 weld metal was used for all welds (Figure 1). The base plate was anchored using six 1 1/4 in. (31.25 mm.) A354BD anchor bolts. Each row of bolts was fully anchored to the concrete foundation by means of an embedded steel bearing plate so as to develop their tensile capacity. All anchor bolts were equipped with steel strain gages along their lengths (Figure 1). The dimensions of the footing (74x74x36 in.; 1850x1850x900mm.) were selected to accommodate attachments to the reaction floor. The mean compressive strength of the foundation concrete was 6000 psi. (42 MPa). Concrete strain distribution was measured by twelve concrete strain gages. Non-shrink construction grout with 8000 psi (56 MPa) strength was packed between the base plate and the concrete footing. The final grout thickness was about 2in.(50mm). Right angle strain rosettes together with linear strain gages were attached to the base plate and to the column at location where strains were expected to be critical. These locations were determined from finite element analysis. In addition to the strain gages, the column was equipped with LVDTs to measure the connection rotations.

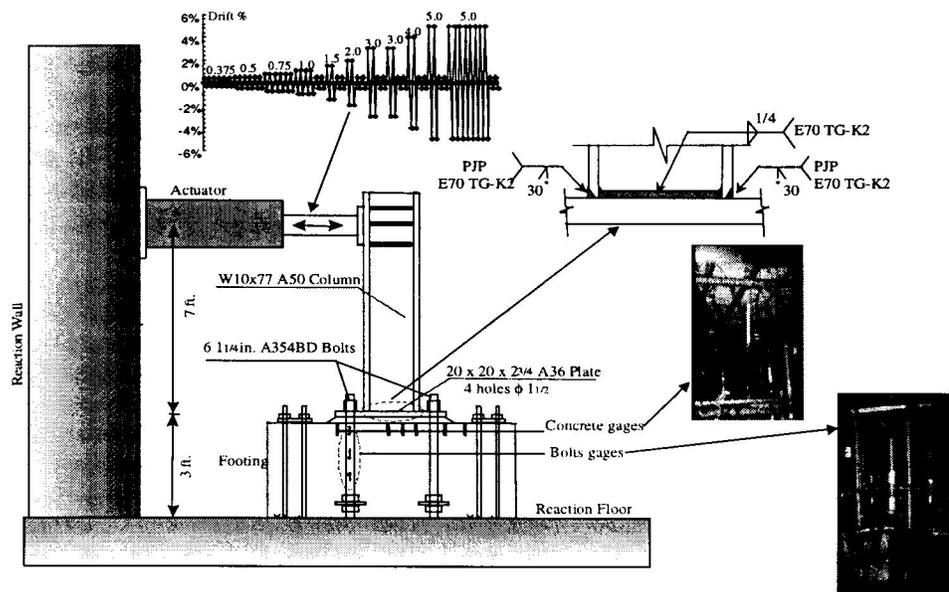


Figure 1. Test setup and instrumentation.

### ANALYTICAL MODELING

The finite element package ABAQUS was used to conduct the 3D nonlinear finite element analysis of the column base connection specimen before the test (Fahmy, 98). The brick element (ABAQUS, 97) was used to model the column and the base plate. Only full, reduced-integration, and incompatible mode elements (C3D8, C3D8R, C3D8I) with linear interpolation elements were suitable for both linear and nonlinear analyses. The numerical formulation of the fully integrated linear element C3D8 gives rise to "parasitic" shear. ABAQUS manual recommends avoiding it in bending-dominated problems and advises using a fine mesh of first-order, reduced-integration element (C3D8R). Most of the elements used in this study had an aspect ratio ranging from 1:1 near the bottom of the column to 1:2 away further. The choice of C3D8R reduced integration element and a regular mesh improves convergence and accuracy of results. The modified Riks method was chosen as the nonlinear solution method.

For the purpose of this numerical study, the material for all steel structural elements in the connection was assumed to be elastic perfectly plastic with an elastic modulus of 29000 ksi (200E3 MPa) and a Poisson's ratio of 0.3. The Modified Hognestad stress-strain model for normal concrete in compression was chosen as the material model for the foundation concrete. The effective strength of the foundation concrete used in the model was taken as its nominal strength of 6000 psi. (42 MPa).

The connection was modeled as a column attached to a plate resting on a spring foundation. This foundation was modeled using nonlinear compressible springs that act in a fixed direction between the plate node and the ground. The spring behavior was determined by specifying a non-linear force displacement relation, defined in compression by the Modified Hognestad stress strain curve. The strain in concrete was assumed to be uniformly distributed along the depth of the foundation. Thus, the spring displacement was proportional to the depth of the concrete foundation. The spring force is set to zero when the spring elongates. Because of an expected small base plate sliding, and anticipated numerical difficulties associated with the convergence of friction surfaces models, shear force was resisted by means of series of stiff lateral springs.

Anchor bolts were modeled as springs. The elongation of a bolt embedded in concrete was derived from a simplified expression (Sato, 87) as the elongation of an embedded long bar:

$$\Delta L = d_c \frac{T_{bolt}}{A_{bolt} E} \left( 2 + 3500 \frac{T_{bolt}}{A_{bolt} E} \right) / \sqrt[3]{(f_c / 20)^2} \quad (\text{mm.})$$

## ANALYTICAL AND EXPERIMENTAL RESULTS

The normalized force displacement curve obtained from the experiment is shown in Figure 2.A. The shear force  $V$ , was normalized with respect to the shear force  $V_p$  needed to form a plastic hinge in the column just above the base plate. The displacement was normalized with respect to the column height, thus mimicking the building drift.

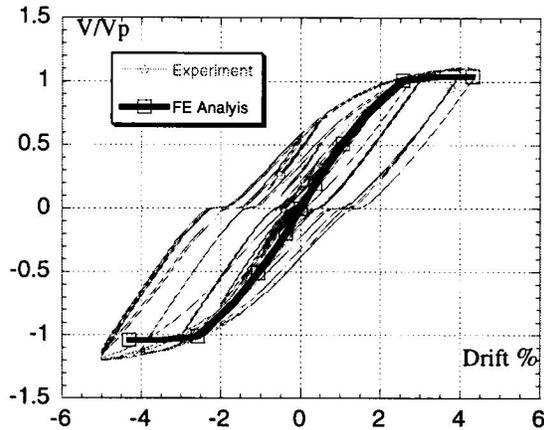


Figure 2. A- Normalized force displacement hysteretic response of the column base connection.

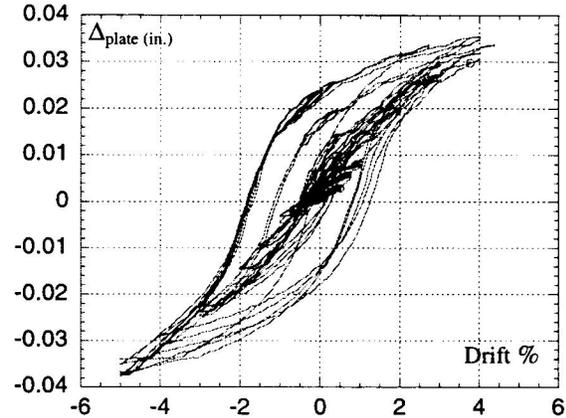


Figure 2. B- Sliding hysteretic response of the base plate. (1 in. = 2.54 cm).

Examination of the hysteretic loop shows that the column base behavior was ductile. The connection reached its ultimate capacity without fracture. The test was stopped when out of plane twisting of the column, caused by local flange buckling, threatened to damage the actuator. The hysteretic response of the specimen was stable, with sustained energy dissipation. Loop pinching at zero loads occurred because of column base sliding. Strength degradation in consecutive loops at the same displacement level was small. Included in Figure 2.A is the normalized force displacement relation obtained from the pushover finite element analysis performed before the test (Fahmy, 98). The graph shows that the finite element model predicted closely the stiffness, ductility, strength and the cyclic envelope of the column base specimen.

Plate sliding response is shown in Figure 2.B. At maximum drift, base plate sliding was small, even smaller than the distance between the edges of the bolt and of the oversized hole (5/16 in.; 8mm.). This fact indicates that friction between the base plate and the grout provides a significant source of shear resistance and that it should not be disregarded in design practice. Furthermore, small sliding values justify finite element modeling assumptions regarding shear resistance.

Normalized moment-rotation curves obtained from the finite element analysis and from the experimental envelopes are compared in Figure 3. The moment was normalized with respect to the column plastic bending strength,  $M_p$ , while the rotation was normalized with respect to the elastic rotation,  $\phi_p$ , corresponding to  $M_p$ . This normalized rotation is expressed as (EC 3, 1992):

$$\bar{\phi} = \frac{\phi}{\phi_p} = \frac{EI_c}{M_p L_c} \phi$$

where  $\bar{\phi}$  is the rotation of the column;  $M_p$  is the plastic bending strength of the column;  $I_c$  is the second moment of inertia of the column about its bending axis; and  $L_c$  is the column length. The envelope of the cyclic response and analytical push-over moment rotation curves agree well in all stages of loading except at the end. This difference is caused by strain hardening, which was not modeled in the finite element analysis. The initial slope of both response curves (Figure 3) shows that the connection initial stiffness is equal to  $1.9 EI_c / L_c$ . This value of initial stiffness demonstrates clearly that the connection can neither be considered fixed nor pinned. This fact is illustrated by considering the stiffness limits for "fixed" ( $S = 25 EI_c / L_c$ ) and "pinned" ( $S = 0.5 EI_c / L_c$ ) behavior adopted in Eurocode 3 and shown in Figure 3 by dashed lines. The ultimate strength of the column base is somewhat higher than the plastic capacity of the column. Thus, the column base behaves in a semi-rigid manner. The effect of the column base semi-rigidity on the global seismic behavior of a buildings was studied in (Stojadinović, 98).

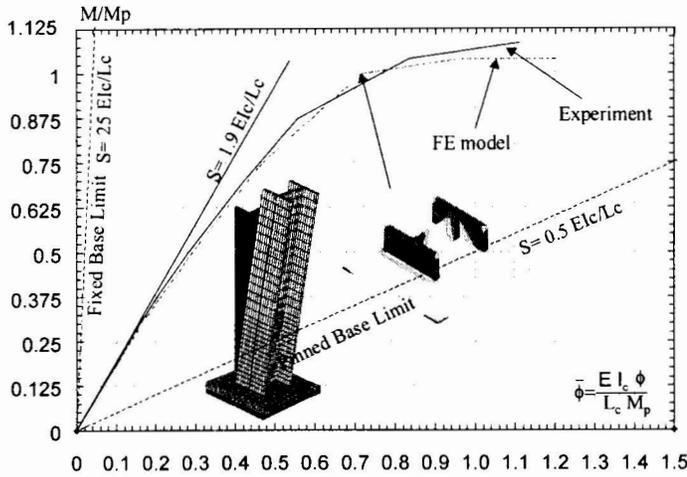


Figure 3. Normalized moment rotation envelopes obtained from the test and the finite element analysis.

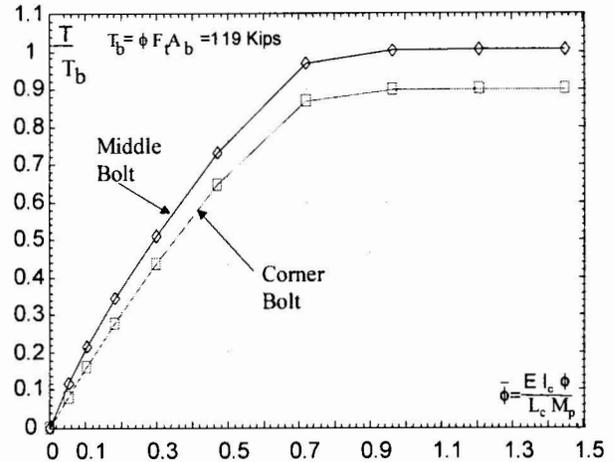


Figure 4. Normalized bolt tension versus normalized rotation.

The first signs of yielding were observed throughout the central area of the column flanges, the outside edges of the web near the flanges, and the base plate at the column flange junction. These signs of yielding agree well with the ones predicted by the finite element model. The finite element model further suggests that the base plate yielded in the region between the middle bolt and the column flange. This was verified by strain gage measurements. At the end of the test a plastic hinge formed in the column. Minor grout deterioration was observed. Finite element analysis indicates that the middle bolt was among the first elements to yield and remained the most heavily loaded bolt throughout the analysis (Figure 4). Overloading of the middle bolt was verified in the test during the early stages of loading only, because the bolt strain gages failed after the 1% drift cycles. Nevertheless, the position and tightness of the nuts at the end of the test showed that the middle bolt was stretched more than the other bolts. This fact is important since it is not accounted for in the design procedure.

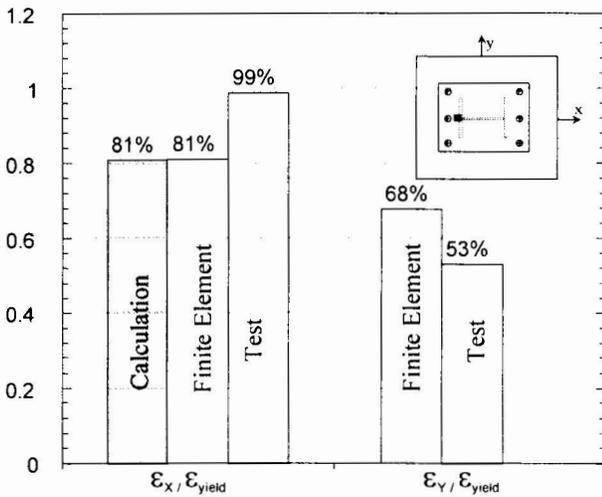


Figure 5. A- Normalized base plate stresses at maximum load near the web & tension flange junction.

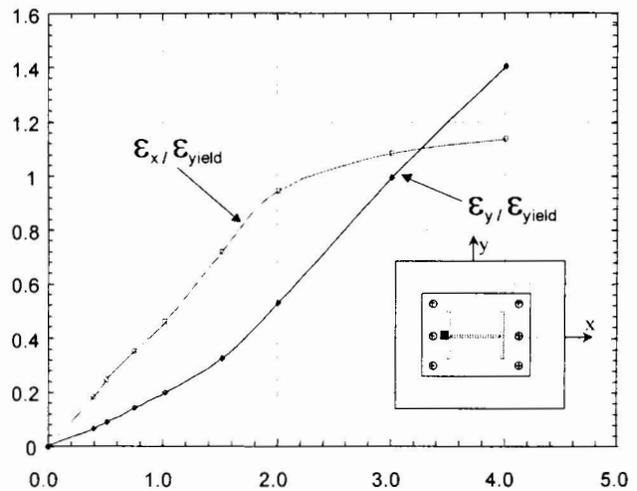


Figure 5. B- Normalized base plate hysteretic envelope near the web & tension flange junction.

At ultimate load, the experiment shows that the maximum longitudinal and lateral bending strains in the plate occurred near the web and tension flange junction. These strains were higher than predicted by both design procedure and finite element analysis (Figure 5.A). Both the experiment and the finite element analyses show that lateral bending strains increase with further loading and exceeds the longitudinal bending strains at drift levels higher than 3% (Figure 5.B). Beside these two bending strains, the base plate, right under the column flange, is subjected to direct vertical strains caused by the tension force in the column flange. These strains create a triaxial tension stress. Such stress state can cause brittle failures,

failures, particularly if the column flanges are fillet welded to the base plate. Yet, this stress state is not considered in the design process.

### CONCLUSION

Our results show that the finite element analysis conducted before the test (Fahmy, 98) predicted the experimental results very well. Therefore, finite element models are being used in the ongoing development of performance-based seismic design procedure for moment resisting steel column bases. Further research work is focusing on the triaxial tensile stress state in the column base plate and the overloading of the middle bolt.

### ACKNOWLEDGMENT

The authors gratefully acknowledge the support of the American Institute for Steel Construction and specimen donation by MBM Fabricators and Erectors of Romulus, Michigan.

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