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Analytical Modeling of Through Beam Connection Detail

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Synopsis: The "Through beam connection detail" has been identified as an ideal rigid connection for attaching steel beams to concrete filled tube (CFT) columns. A combination of analytical and experimental studies is being conducted to comprehend the behavior of this detail. The test specimen consisted of a CFT column and a steel beam passed through the column to represent an interior joint in a building. This paper presents a summary of the finite element analysis that was conducted to comprehend the force transfer mechanism and identify locations of potential stress concentration. The analytical results were verified by comparison with the experimental results. Both the experimental and analytical results showed the capability of the connection to develop the full plastic bending strength of the connected beam. The elements that contribute to the connection strength were identified as: the beam web, the steel tube, and the concrete core.

Keywords: composite structures; concrete filled tubes; connections; finite element

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INTRODUCTION

Composite construction consisting of concrete filled tube (CFT) columns with relatively thin-walled steel tubes, has been used in building construction in U.S. and far east. In general, in this type of construction, steel beams are framed to columns at each floor level. Detail and design criteria for connecting steel beams to CFT columns are almost non-existent. On the other hand the economy of this type of connection depends, to a large degree, on utilizing a suitable connection detail. The overall objectives of the investigations being conducted are to develop an economical connection detail for connecting steel beams to CFT columns and to provide accompanying design provisions.

A previous pilot study on a specific connection detail referred to as a through beam connection detail was conducted at University of Nebraska-Lincoln by Azizinamini and Parakash (2). Another study by Alostaz and Schneider (3) was recently completed at University of Illinois. This study examined the feasibility of using several connection details and concluded that the behavior of the through connection detail was the best.

A combination of analytical and experimental studies is being conducted to provide a thorough understanding of the behavior of the through beam connection detail. A three dimensional nonlinear finite element model was generated for the connection using ANSYS (1) finite element program. The experimental results were used to verify and fine tune the numerical model. The theoretical model was then used to identify the elements that control the joint behavior and evaluate their contributions to the connection strength.

TEST SPECIMEN

A cruciform shaped specimen representing an interior connection in a plane frame was tested to understand the force transfer mechanism. Details of the test specimen are shown in Figure 1. The specimen consisted of a concrete filled tube column and a steel beam passed through the column. The upper and lower portions of this cruciform type test specimen represent the distance from the floor to mid-heights of the lower and upper stories. The length of the beam represents the distance to the inflection point on each side, which is assumed to be half the span. Loads were applied at the beam ends to simulate member shears at inflection points which occur under lateral loading of the frame. Axial load was applied on the column to represent the reaction of upper stories due to gravity loads. The magnitude of the axial load was taken as 20% of the squash load, P_o, given by the following equation:

$$P_o = A_s F_y + A_c f_c'$$

where

A_s = cross sectional area of the steel tube
A_c = cross sectional area of the concrete core
f_c' = concrete compressive strength
F_y = yield stress of the steel tube

The specimen is approximately 2/3-scale compared to the member sizes needed for the prototype building. The specimen had a 16"ϕx1/4" pipe and a W18x35 beam. The measured yield stresses of the tube and the beam are 64 ksi and 52 ksi, respectively. The measured concrete compressive strength is 5.0 ksi. Full penetration weld was used to attach the beam to the pipe. The total height of the column was 9 ft. Lateral reaction points on the column were located 6 in. away from the column ends resulting in a distance between the lateral reaction points of 8 ft. The total beam length between the loading points for all the specimens was 13 ft-6 in. The loading points were located 1 ft away from the beam end. The total length of the beam was 15 ft-6 in.

The specimen had a column-to-beam bending capacity ratio of 1.2, which satisfies the code requirement that the column should be at least 20% stronger than the beam. Failure of the specimen was due to plastic hinging in the beam outside the joint.

GEOMETRICAL MODEL AND SOLUTION METHOD

Figure 2 shows the finite element mesh. The X-axis aligned with the beam longitudinal axis, the Z-axis aligned with the column longitudinal axis, and the Y-axis was perpendicular to both longitudinal axes of the column and girder. Due to symmetry in the in the X-Z plane, one half of the connection was modeled. Reaction points were located on the center lines perpendicular to the plane of loading at the top and the bottom of the column. Reaction points at the column top were restrained from translation in the X and Y directions, but the displacement along the Z-direction was not restrained to allow for the application of the column axial load. Reaction points at the bottom of the column were restrained from translation in the X, Y, and Z directions.

To ensure uniform distribution of axial loads at the column ends, the model included rigid steel cap plates. These steel cap plates were located at column ends using shell elements. The column axial load was applied in the form of uniform pressure on the rigid caps. The column axial load was applied first. Then, the girder was loaded monotonically in small increments. Full Newton-Raphson procedure with line search and automatic time stepping was used for the analysis. The frontal solver method was used to solve the simultaneous equations.

MATERIAL MODELS

Since concrete is a brittle material it tends to develop tensile fracture perpendicular to the direction of the largest tensile strain. Under uniaxial compression, tensile cracks develop on planes parallel to the direction of the compressive stresses. Under triaxial compressive stresses, the mode of failure is crushing of concrete.

The concrete material model in ANSYS accounts for both cracking and crushing. The cracking is modeled through an adjustment of the material properties which effectively treats the cracking as a smeared band of cracks rather than discrete cracks. The presence of a crack at an integration point is represented through modification of the stress-strain relations by introducing a plane of weakness in a direction normal to the crack face. Also, a shear transfer coefficient is introduced which represents a shear strength reduction factor for those subsequent loads which induce sliding (shear) across the crack face. If the crack closes, then all compressive stresses normal to the crack plane are transmitted across the crack. Since cracked concrete cannot transfer tensile stresses, tensile strength should drop suddenly after cracking. To avoid numerical problems, tensile strength right after cracking was assumed to drop to 99% of its original value. Tensile strength was then gradually decreased with crack widening until it reached a value of zero.

If the material at an integration point fails in uniaxial, biaxial, or triaxial compression, the material is assumed to crush at that point. Crushing is defined as the complete deterioration of the structural integrity of the material (e.g. material spalling). Under conditions where crushing has occurred, material strength is assumed to have degraded to an extent such that the contribution to the stiffness of an element at the integration point in question is ignored.

The tensile strength was assumed to be $7.5\sqrt{f_c'}$ psi. The ratio of the ultimate biaxial compressive stress to the ultimate uniaxial compressive stress was specified as 1.2.

A rate-independent plasticity model was used to simulate the inelastic behavior of the steel components. Von Mises' yield criteria was used to define the material yield surface, and an associated flow rule was used to determine the plastic deformation. The associated flow rule with the von Mises yield criterion is known as Prandtl-Reuss flow equation.

A bilinear kinematic hardening model was used to simulate the elastic and inelastic behavior of all steel components. Kinematic hardening assumes that the yield surface remains constant in size and the surface translates in stress space with progressive yielding. Material properties measured from testing coupons obtained from the test specimen were used in the finite element model.

ELEMENTS

Eight-node brick elements (SOLID65 of ANSYS) with three translational degrees of freedom at each node were used to model the concrete core. This element allows cracking in three orthogonal directions at each integration point. Four-node shell elements (SHELL43 of ANSYS) with six degrees of freedom at each node were used to model the steel tube and the girder. For both elements extra displacement shapes were excluded to help accelerate convergence.

The interface between the steel and concrete was modeled by using contact elements (CONTAC49 in ANSYS). The contact element can transfer compressive and frictional forces only. The coefficient of friction between steel and concrete was assumed to be 0.3. The weld was idealized by providing common nodes for the girder and the steel tube at the weld locations.

FINITE ELEMENT RESULTS

In order to verify the analytical model, the results from the finite element (FE) analysis are compared to the experimental data. Figure 3 shows the experimental and analytical load-deflection curves at the beam end. A good agreement between the experimental and analytical results is observed. This indicates the capability of the FE model to predict the behavior of the joint in both the linear and the nonlinear stages. Also shown in Figure 3 is the load at the cantilever tip (P_p) that is required to develop the plastic moment capacity of the beam. Both the analytical and experimental loads exceeded P_p, which indicates the capability of the connection to develop the plastic flexural capacity of the beam.

Figure 4 shows the Von Mises stresses in the steel tube and the steel beam. Stress concentrations are found at the intersection of the beam flange and the tube. The distribution of the hoop stress in the steel tube at the tension flange level is shown in Figure 5. This figure shows that high hoop stresses are transferred to the tube at the tip of the tension flange.

The variation of the axial stress in the flange along the beam longitudinal axis is shown in Figure 6. It is noticed that there is a significant stress drop in the beam flange once it enters the tube. Due to the curved shape of the tube the stress drop at the flange edges is higher than at the flange centerline. The axial stress in the tension flange just inside the tube is approximately 70% of the stress value just outside the tube. This indicates that a portion of axial tensile force in the flange is transferred to the tube as shear force through the weld. This behavior agrees with the experimental results which indicated an axial strain drop in the flange, once the beam enters the tube. At the compression side, the stress drops almost to zero once it enters the tube. This indicates that axial compression force in the flange is transferred to the concrete core by bearing of the tube wall against the concrete. The same behavior was also observed experimentally. The gradual reduction in the axial tensile stresses in the flange, inside the tube, is due to transfer of the flange force to the concrete core by friction.

The minimum principal stresses in the concrete core are shown in Figure 7. High vertical and horizontal bearing stresses are generated in the concrete panel near the compression flange. The resultant of these forces causes the formation of the diagonal compression strut which contributes to the joint shear capacity. The formation of the compression strut is confirmed by the distribution of the minimum principal stress in the concrete panel as shown in Figure 7.

The tensile and compressive forces in the beam flanges are transferred as horizontal shear forces through the connection zone. The connection zone is defined as the portion of the column bounded by the beam top and bottom flange levels. The concrete in the joint zone is divided into inner and outer concrete panels. The inner concrete panel consists of the concrete between the beam flanges while the outer concrete panel is the concrete outside the flanges. The joint shear is distributed between the beam web, the tube, the inner concrete panel and the outer concrete panel. Figure 8 shows the amount of shear resisted by each of the four elements. Figure 8 shows that all the elements are activated from the beginning of the loading. The web, however, seems to pick up more forces than other elements until it reaches its capacity. Figure 8 shows that the web yielded at a load of 47 kips. After web yielding, the increase in the shear force is distributed between the tube and the concrete. Figure 8 also indicates that the shear capacity of the tube and the concrete panel was not fully mobilized since the specimen failed by beam yielding outside the joint.

CONCLUSIONS

The through beam connection detail provides a good detail that is capable of developing the plastic bending capacity of the connected beam when the strong column-weak beam philosophy is followed. The tensile and compressive forces in the beam flanges are transferred as horizontal shear forces through the joint zone. The joint shear is distributed between the beam web, the tube, the inner concrete panel and the outer concrete panel. In order to design the connection, the shear strength of each element needs to be established. Experimental studies are being conducted on test specimens that were designed to fail in the joint zone in order to establish the ultimate strength of the connection.

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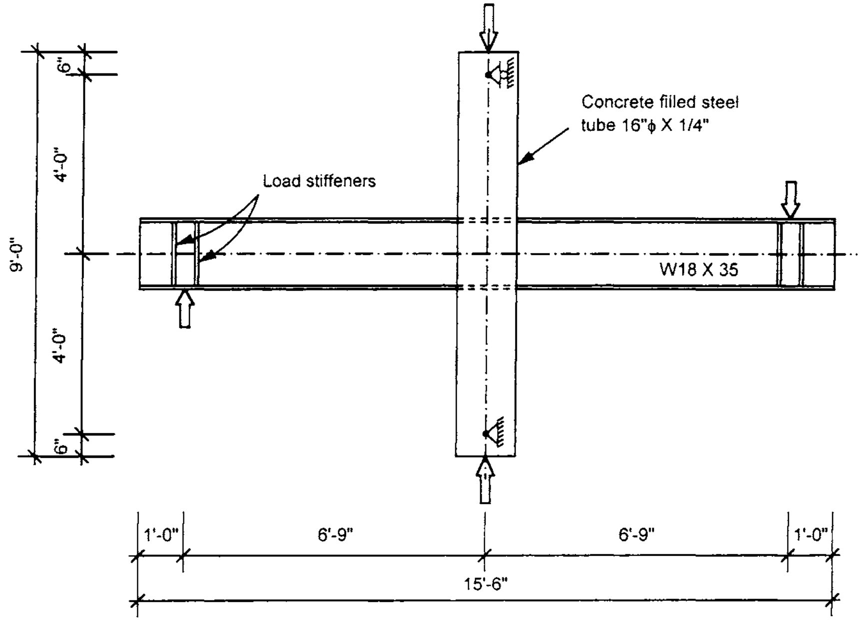

Figure 1 Test Specimen

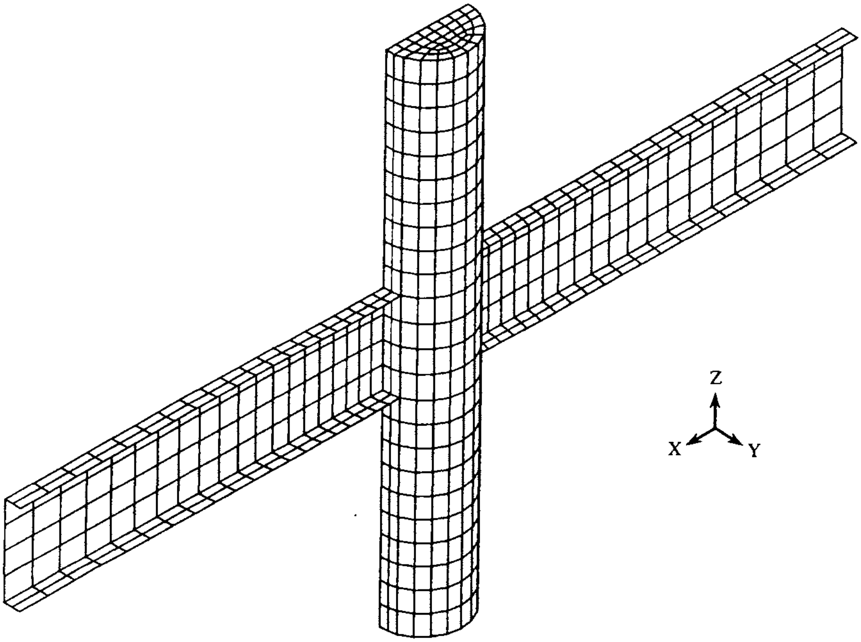

Figure 2 Finite Element Mesh

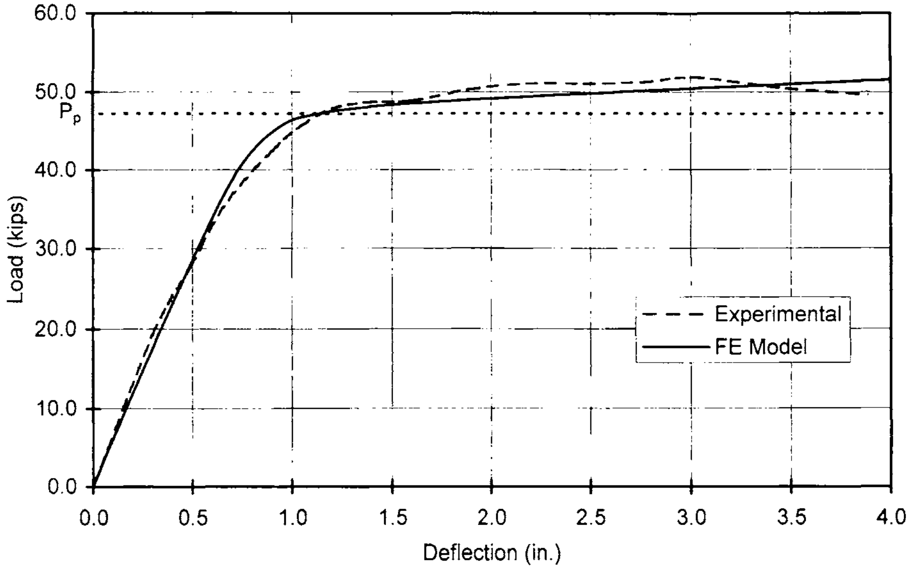

Figure 3 Load-Deflection Relationship at Beam End

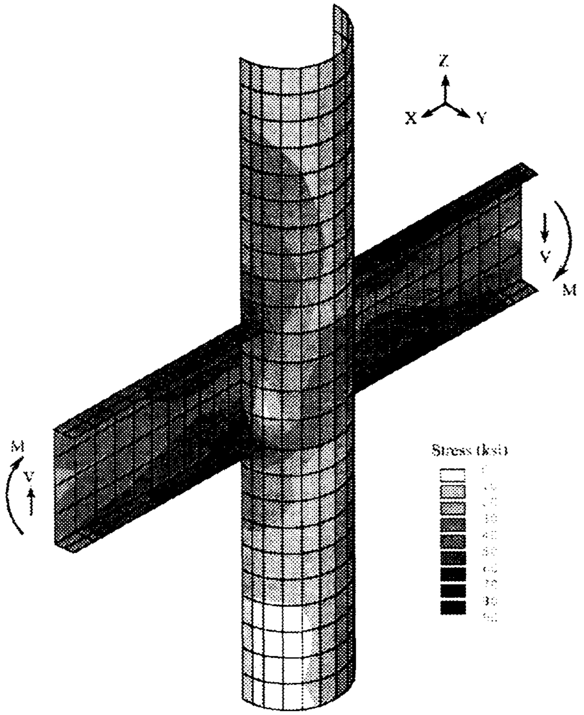

Figure 4 Von Mises Stress in the Steel Tube and Beam

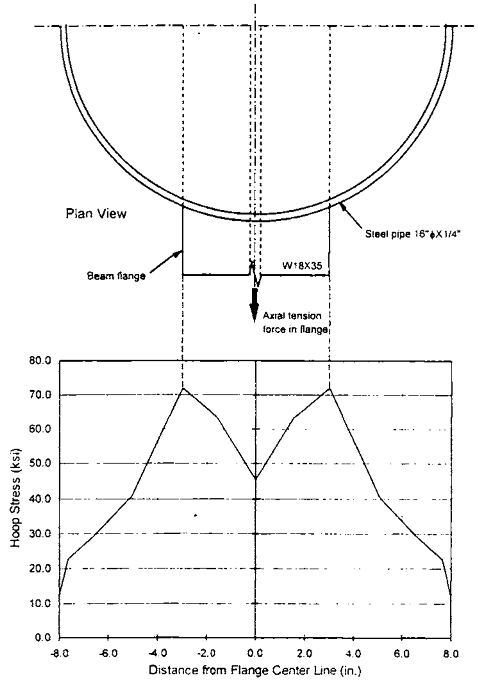

Figure 5 Hoop Stress Distribution at the Tension Flange Level

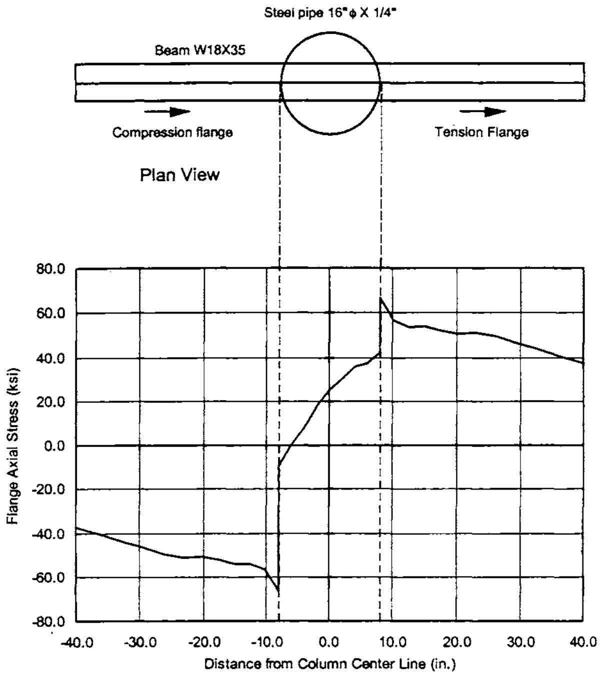

Figure 6 Variation of Axial Stress in Top Flange along the Beam Axis