SEISMIC BEHAVIOR OF STEEL BEAM-TO-COLUMN CONNECTION SUBASSEMBLAGES

by Vitelmo V. Bertero[I]

SYNOPSIS

The behavior of beam-column subassemblages under actions similar to those expected from severe earthquakes is reviewed in light of the data now available. From evaluation of these data it is concluded that such actions cause significant deformations in the panel zone of these subassemblages, which should be considered in estimating the stiffness of the subassemblages even in the linear elastic range. It is also found that as the number of cycles of repeated reversed lateral loads increases, the stiffness of the panel zone decreases. Since the gravity loads acting on the beams may significantly affect the behavior of these subassemblages, and as previous works have neglected them, a new comprehensive study including these loads is proposed.

INTRODUCTION

The design of multistory steel-framed structures which may be subjected to severe dynamic actions during their service life--such as those expected from strong ground motions--requires prediction of the inelastic behavior under repeated actions. This prediction, in turn, is based on knowing the behavior of the different structural components rather than that of the whole structural system. Although some data are available regarding behavior of beams under repeated loadings(1) there is little information about the behavior of columns under heavy axial forces when subjected to repeated lateral actions. Furthermore, to the best of the writer's knowledge, no comprehensive study is available regarding behavior of actual or proper model representation of beam-column subassemblages under realistic simulation of their loading conditions during an extreme earthquake. Without a thorough understanding of the behavior of such subassemblages, no rational prediction of the behavior of multistory buildings can be made.

A recent investigation carried out at the University of California(2,3) yielded valuable information regarding inelastic response of different types of beam-to-column connections subjected to repeated loading. However, because the specimens and the testing program were designed to study behavior of the beam and connecting media rather than the whole beam-column subassemblage, the question still remained whether the inelastic deformation that controls the behavior of actual joints occurs--in the beam or its connecting media, in the column, or in the "joint area" (panel zone).

Tests carried out in Japan on full scale models of the beam-to-column connections in a building(4) establish that the panel zone is a major locale for deformation. They show that: (1) even in the elastic range,

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the deformations taking place in the panel zone were responsible for a large percentage of the total relative story displacement; (2) inelastic deformation started in the panel zone; (3) when the load was reversed, plastic deformation occurred not only under a considerable lower load, but also at a higher rate than that observed during first yielding; and (4) the contribution of the panel zone deformation to the total displacement of the subassemblage varied from 42 to 68 percent. These tests clearly indicate the necessity of including panel zone deformation in any rational method for predicting the behavior of subassemblages and, therefore, of a structure in general.

Premature yielding or buckling of the panel zone can be avoided by stiffening this region, but adding stiffeners is usually very expensive. Present methods of design are concerned only with the strength of the panel zone, and these methods are usually based on results of tests in which specimens have been loaded directly to collapse. Therefore the direct use of these methods in the design of panel zones which belong to multistory buildings that may be subjected to severe lateral actions of alternating directions is questionable. In these cases, a knowledge of the panel zone's strength, stiffness, and ductility under repeated reversible loading is required before any rational assessment of structural behavior can be made. The present study is part of a general investigation to attain this knowledge.

The objective of this paper is to discuss the inelastic behavior of beam-to-column connection subassemblages--with emphasis on the behavior of the panel zone--under repeated actions similar to those expected from severe earthquakes. Accordingly, in this paper the author has reviewed, summarized, and evaluated the available experimental data and analytical studies concerning the behavior of this type of subassemblage. In the light of this evaluation the author proposes a comprehensive program of experimental and analytical studies of these subassemblages.

REVIEW OF EXPERIMENTAL AND THEORETICAL STUDIES

Introduction: This review is limited to typical office or apartment multistory buildings which may be subjected to severe lateral actions such as those imposed by strong earthquake ground motions. When these actions govern the structural design of the building, economy dictates that the design be based on the energy absorption capacity of the structure rather than on strength and in this case, of primary importance are the continuity of members based on rigidity of the connections, and the stiffness and ductility of these connections.

Types of Actions Considered: Although the type of building considered in this report may be subjected to a variety of actions, it is assumed that their design is controlled by the loading conditions imposed by: (1) Gravity Load: "Dead Load" (D.L.) and "Live Load" (L.L.) and (2) Lateral Load. Figure 1a shows a typical bending moment diagram under gravity load. While columns, especially the exterior ones, may be subjected to some bending stresses under dead load, these bending stresses are small. More severe column bending moments are caused by the "checkerboard disposition" of (L.L.), though the stresses developed at the beam-to-column connections are usually not critical. It should be noted that in tall buildings, uneven column shortenings can modify significantly the distribution of moment, especially in the outside girders, introducing some significant moments in the columns. The effect of uneven column shortenings is accentuated in the case of frames with unequal spans, particularly when the frame's outside spans are considerably shorter and when very stiff girders are selected for them. Assuming that the lateral load is acting alone, it produces the bending moment diagram illustrated in Fig. 1b. This type of load induces the most severe state of stress at the beam-to-column connections, creating serious problems for the strength, stiffness, and ductility of these connections, as well as for the whole structure.

Factors Controlling the Behavior of Beam-Column Subassemblages: The response of multistory buildings to actions producing severe lateral forces can be calculated if the real behavior of the beam-column subassemblage, which is illustrated in Fig. 2, can be predicted with sufficient accuracy. Neglecting uneven axial deformations of the columns, it can be assumed that the ends of the girders remain at the same level. Consequently, the lateral displacement of the buildings can be measured by predicting the relative lateral displacement of the inflection points of the column and of what will be referred to as column ends. Following a procedure similar to that indicated in reference[4] the author considers it convenient to break down the total displacement of the column ends, δ_C, in the components illustrated in Fig. 3. Then:

$$\delta_C = \delta_M + \delta_R + \delta_P + \delta_Q + \delta_J \qquad (1)$$

where: δ_M = displacement due to deformation of column as a cantilever;

δ_R = displacement caused by a rigid rotation of the panel zone equal to that resulting from the elastic deformation of the beam when considered as a continuous member;

δ_P: same as δ_R but due to inelastic deformation of the beams;

δ_Q = displacement caused by elastic and inelastic deformations of panel zone

δ_J = displacement due to deformations of connecting elements

While δ_M and δ_R can be calculated by any standard method of frame analysis, the others are not so easy to estimate, because these displacements are usually the result of local deformations which are present in the beam-to-column connection. These local deformations depend not only upon the type of connection but also upon the kind of connecting media that is used.

Types of Beam-to-Column Connections: The different types of beam-to-column connections depend upon the types of beam and column sections used. While beam sections are usually of the WF and I shapes, different ones are used for column sections. Because the most common shape is the rolled WF, or H, it is the basis for this review. The beam-to-column connections can be classified as: (1) Two-Way Connection and (2) Four-

Way Connection. Although two-way connection is uncommon in the type of building considered in this study, it appears to constitute the most critical case for the panel zone[5]. Regarding kinds of connecting media, there are several ways of connecting the beam to the column. Analysis of the results obtained in the University of California investigation[2,3] indicates that the connection shown in Fig. 4 gives the best performance when subjected to repeated reversed loading. As this connection also has less complicated details, it appears that this type of connecting media is the most suitable for use in the subsequent discussion.

Problems Introduced in the Beam-to-Column Connections: The most important problems created in a typical beam-to-column flange connection are illustrated in Fig. 5. These problems are aggravated by the fact that the lateral loads considered in this study are of a repeated reversible nature (particularly if seismic in origin). Repeated reversible loads seem to have considerable effect on the resistance to buckling[1].

Review of Experimental Results: Tests of steel moment connections, mostly conducted to examine behavior under gravity load alone, have been comprehensively summarized by Professor L. Beedle[6].

A series of experiments was done in Japan under the direction of Professor Umemura[7] to examine the strength, rigidity, and ductility of beam-to-column connections within the elastic and plastic ranges when subjected to the effects of lateral loading. Different types of connections were tested under the loading shown in Fig. 6a, which simulates the effect of lateral loads alone. Most of the specimens were tested under monotonically increasing load but a few were subjected to cyclic loading. Except for the conclusion that this type of loading had practically no effect on the ultimate strength of the connection, no other information is available.

Dr. Naka and associates[8] have reported the results of a series of tests on beam-to-column connections under repeated reversed loading. In addition, they have compared their experimental results with theoretical predictions based on an expression obtained by using a stress function and on a simplified equation. The agreement was satisfactory. Figure 6b shows the type of specimen and the manner of loading used in the tests.

Three full-scale models of the beam-to-column connections at the 25th story of the structure designed for the 29-story Nusantara Building were tested under repeated reversible lateral loads[4] using the test setup illustrated in Fig. 6c. This setup provides a state of stress similar to that induced by lateral loads, acting as if alone on the building.

From an analysis of the experimental results available to the writer, which are reviewed in detail in reference[19], it is concluded that: (1) Only the effects of lateral load acting alone have been studied. No consideration has been given to the gravity forces acting on the beams, yet these forces may significantly affect the subsequent behavior of the subassemblage when subjected to the effects of lateral loads; (2) Under lateral load very significant strains, hence deformations, occur in the panel zone. These deformations should be considered in estimating the stiffness of the subassemblage even in the linear elastic range; (3) As soon as yielding occurs in the panel zone, the stiffness of the region is considerably reduced. Therefore, it is recommended that a thickness of the column web be selected which would avoid yielding under the maximum expected load. Basler's expression[9] can be used to select the thickness of the panel zone that would avoid shear buckling. However, the validity of this, as well as other suggested expressions, is limited to only one load application; (4) Under repeated reversed lateral loads, the stiffness of the panel zone decreases as the number of cycles increases. Shear buckling strength appears to deteriorate also as the number of cycles of reversed loading increases; (5) Proper reinforcement of the panel zone increases the yielding and ultimate strengths of this region but does not effectively reduce the deterioration in stiffness under repeated reversible loading; and (6) The problem of the panel zone appears to be one of reduction of stiffness, which may lead to undesirable drift under repeated reversible loadings. The ductility and strength of properly designed panel zones appear to be satisfactory.

Review of Proposed Analytical Methods: Of the different methods that have been proposed in the United States for predicting the behavior of the panel zone under the effect of different loading combinations, most deal with the prediction of strength. Several simplified equations and procedures have been recommended for analyzing stiffener requirements in the tensile and compressive regions of connections[10,11,12]. The various design procedures that have been suggested use simplified formulae to check and determine the following parameters: (1) Required web thickness of column to prevent web crippling; (2) Required web thickness to prevent column web buckling; (3) Flange stiffener thickness (when required) for column web crippling; (4) Required column flange thickness (to avoid tensile distortion); (5) Required web thickness for shear; and (6) Diagonal web stiffeners for shear.

To the best of the writer's knowledge, no method is available in the American literature which allows for the reduction in the panel zone stiffness due to shear deformation and the effects of possible repeated reversible loadings. Japanese investigators have devoted considerable effort to develop analytical procedures for estimating not only the strength, but also the deformation, of the panel zone. These procedures can be classified in the following groups: (1) those based on the use of simplified equations; and (2) those based on the use of stress functions.

Procedures Based on the Use of Simplified Equations[8]: Let us consider the panel zone shown in Fig. 7.

(a) Shear Strength: The shearing stresses acting on the edges of the panel can be estimated from

$$\tau_B = \left[\frac{M_C^T + M_C^B}{h_C} - \frac{Q_B^R + Q_B^L}{2}\right]\frac{1}{h_B t_p} \qquad \tau_C = \left[\frac{M_B^L + M_B^R}{h_B} - \frac{Q_C^B + Q_C^T}{2}\right]\frac{1}{h_C t_p} \qquad (2)$$

From equilibrium of moment at the middle of the panel $\tau_C = \tau_B = \tau_p$. Therefore, to avoid shear yielding of the web, according to Mises' Criteria,

$$\tau_p \leq \frac{\sigma_y}{\sqrt{3}} \tag{3}$$

The maximum capacity in shear can be evaluated using a similar equation, but replacing σ_y by the tensile strength of the material σ_t.

(b) Shear deformation: This can be estimated from

$$\gamma = \frac{\tau}{G} \tag{4}$$

The displacement of the story due to shear panel deformation (δ_Q) can be obtained from

$$\delta_Q = h \cdot \gamma \tag{5}$$

where h is the story height. It should be noted that in general the panel zone is subjected to flexure and shear. Therefore, it is convenient to consider

$$\gamma = \gamma_1 + \gamma_2 \tag{6}$$

where γ_1 = distortion angle due to shear and γ_2 = distortion angle due to bending. These two distortion angles as well as the expressions for their approximate computation are given in Fig. 7. Usually $\gamma_1 > \gamma_2$.

Reference (4) suggested an approximate method to include the effects of the panel deformation in estimating the relative displacement of a story. The method consisted in using a modified slope deflection equation proposed by Dr. Muto. Other approximate methods were suggested in References (8), (13), and (14).

Procedures Based on the Use of Stress Functions: Basically all of these procedures consist in formulating a stress function ϕ which satisfies a certain assumed distribution of stresses along the boundary of the panel zone. Once ϕ has been selected, the expressions for σ_x, σ_y and σ_{xy} are derived. Substitution of the expressions for σ_x, σ_y, and σ_{xy} in the Von Mises' Yield Condition allows the determination of where yielding begins in the panel zone. Computations based on these procedures show that yielding starts near the center of the panel zone and propagates towards the outside boundaries as the lateral load is increased.

Strains ε_x, ε_y, and γ_{xy} can be obtained directly from the stresses using Hooke's Law. Displacements in the x and y directions can be obtained approximately by integrating the strains and by the use of proper boundary conditions.

All of the above procedures are based upon the assumption that the panel zone can deform as an independent member of the column-to-beam connection subassemblage, neglecting the restraints imposed by the adjacent parts of the columns and beams. In a recent publication, Naka and Associates have included the restraints imposed by the continuity of the column in a method using trigonometric series to form the stress functions(15).

Comparison of the results obtained using stress functions with those using the proposed simplified equations reveals that they are the same for all practical purposes. They also closely agree with test results.

It should be noted that all the proposed analytical methods concern the prediction of behavior in the elastic range. Except for an empirical rule suggested by Naka et. al.(15), no attempts have been made to predict the deformation in the plastic range. Moreover, the possibilities of deterioration of the initial yield strength and of the stiffness caused by repeated reversible loads have not been included in these methods.

Evaluation of Available Experimental Results and Proposed Analytical Methods: The significance of the information just reviewed can be better understood by analyzing the actual state of internal forces that can exist in a subassemblage of a typical multistory frame. Owing to gravity load (D.L. + L.L.) the resulting bending moment diagram will resemble the one illustrated in Fig. 1a. The bending moment diagram corresponding to lateral load acting alone is shown in Fig. 1b. If the response of the structure under the combined loads is limited to the linear elastic range, then the total bending moment can be obtained by direct superposition of the individual moment diagrams. Thus, the study of the effects of the lateral load can be carried out independent of the gravity forces. However, for the forces developed during extreme earthquake, design should permit large inelastic deformations, which eliminates the application of the principle of superposition. In the following discussion, it will be assumed that design is based upon plastic moment redistribution that takes place according to the presently accepted practice of designing with "strong" column and "weak" girders, which contemplates columns that are free from inelastic deformations.

Although the gravity loads may not control the design of the overall frame, they may govern the design of the beams. Assuming that the design is based on full redistribution of moments under ultimate gravity loads, [1.70 or 1.85 (D.L. + L.L.)], the moment at the ends of the beams under 1.30 or 1.40 (D.L. + L.L.) will be close or equal to its plastic moment of resistance. Therefore, when we consider the combination of gravity loads and lateral loads, it can be assumed that due to gravity loads, the bending moment at the ends of the beam reaches its plastic moment. If so, when the lateral forces are superimposed for the first time on the gravity forces, the change in bending moment will be that indicated in Fig. 8 rather than in Fig. 1b. Therefore, the rotation of the joint (assuming just rigid axis) is obtained directly by estimating the rotation at the left end of the beam which behaves as if it were simply supported. Then, as shown in Fig. 8,

$$\theta_A = \frac{H.h.l}{3.E.I_B} \tag{7}$$

On the other hand, if it is supposed that the change in bending moment takes place as shown in Fig. 1b, we will have

$$\theta_A = \frac{H.h.l}{12.E.I_B} \tag{8}$$

Comparison of (7) and (8) reveals that it is not possible to neglect the initial effect of gravity loads in estimating the lateral deformation of the subassemblage.

The internal stresses introduced in the panel zone by the gravity loads also affect its behavior when the frame is subjected to severe lateral forces. Since high concentrations of stresses are developed in the panel zone, some regions of this zone may yield under the effect of gravity loads alone. Accumulation of plastic deformation in this region may occur when the lateral load is superimposed on the gravity loads, and this may lead to a deformation pattern quite different from that which would be obtained by neglecting the effect of gravity load.

<u>Example</u>: To illustrate the importance of the gravity forces and the behavior of the panel zone in the overall behavior of beam-column subassemblages the results of the analysis of a lower floor connection subassemblage of a multi-story frame that has been designed by a plastic method using A36 steel (Frame B of Reference (11)) are summarized below. The nominal dimensions and sizes of the different members as well as the gravity loads are indicated in Fig. 9. The effects of lateral loads were simulated by the shear forces H_1 and H_2 which were assumed to be in the ratio $H_2/H_1 = 1.14$.

In the case where the gravity forces are neglected the beam starts yielding under a force $H_1 = 54K.$, and the corresponding rotation of the joint, assuming that the panel zone is rigid, amounts to 5.0×10^{-3} radians. On the other hand, if the gravity forces are assumed to be acting when the lateral forces are applied, yielding of the beam starts under $H_1 = 26.4K.$, first plastic hinges form at $H_1 = 34.1K.$ and the subassemblage is converted into a mechanism when $H_1 = 60.2K.$ These last two forces are defined in Fig. 10 as "Yielding" and "Ultimate" lateral loads respectively. The rotation of the joint under a load $H_1 = 54K.$, when gravity forces are considered, is about 11.0×10^{-3} radians, i.e., approximately 2.2 times the number estimated neglecting these forces. These results clearly point out the need for including the gravity forces in any comprehensive program of investigation.

To obtain an idea of how the behavior of the panel zone affects the overall behavior of the subassemblage a computer program based on the use of stress functions has been developed(19). The results obtained in the analysis of the panel zone corresponding to the subassemblage of Fig. 9 shows that yielding starts under a force H_1 of about 19.5K. The corresponding average shear distortion angle amounts to approximately 1.2×10^{-3} radians, which is more than 60% of the rotation that the joint will undergo under this same force if the panel zone is considered to be rigid. Furthermore, analyses of the panel zone under the internal forces induced by the "Yielding and Ultimate Lateral Loads", Fig. 11, indicate that keeping the behavior of this panel zone in the elastic range for these two loads requires an increase inits thickness to approximately 1.5 and 2.5 times respectively. These results clearly point out the need for including the deformation of the panel zone in the estimation of the story drifts of multistory frames and also the need for proper stiffening of such a zone.

Concluding Remarks: The above review and example indicate that behavior of the panel zone not only contributes significantly to the elastic drift of a story relative to its adjacent stories, but that it may also govern the behavior of the whole subassemblage in the inelastic range, especially in the case of repeated reversible loadings. The available information regarding behavior of subassemblages under repeated loadings comprises results obtained in a few tests carried out on a particular subassemblage subjected to very limited loading conditions. Clearly, a comprehensive program of testing is urgently needed to develop rational design criteria for the panel zone and methods for predicting how the behavior of this zone affects the lateral displacement of a story. The effects of gravity loads should be introduced in this program.

Since all the analytical methods of predicting behavior of the panel zone have been based upon linear elastic response and upon the assumption that the panel zone can deform as an independent member of the whole subassemblage, it will be necessary to develop a method which can predict the behavior of this zone in the inelastic range and where the actual boundary conditions of the whole subassemblage are considered. The finite element technique developed by Professor Clough and his associates(16) makes possible the more accurate prediction of the behavior of the whole subassemblage. Such a possibility is presently under investigation.

PROPOSED PROGRAM OF INVESTIGATION

Introduction: In the search for an efficient program of investigation, different approaches have been considered(19). The approach that has been selected can be divided in two phases. First Phase: Development of a testing program to study the behavior of the panel zone with the main objective of finding out the parameters that govern such behavior. As many of the parameters involved may have little significance in the overall behavior of the panel zone, the less influential parameters are first eliminated by preliminary theoretical studies. These studies are now in progress using the finite element technique.(16) Second Phase: To design and study experimentally the most promising beam-to-column connections, once the behavior of the panel zone is understood.

In the development of the testing program several problems have been encountered. One of the most important problems has been the selection of the test setup.

Test Setup: The selection of the testing arrangement has been controlled by the loading conditions and the constraints at the ends of the subassemblages that have to be reproduced.

Regarding the loading conditions, the question of whether the loading should be applied dynamically or quasi-statically had to be answered. Although, strictly speaking, the severe loads considered in this study are of dynamic nature, it is believed that the rate of strain developed in the response of the structural system to these loads is not high enough to introduce significant variations in the so-called "static characteristics" of the material[17]. Therefore, it is recommended that the loads be applied in a quasi-static manner. After analyzing the advantages of all the different test setups used in previous experiments that have been reviewed by the author (Fig. 6), as well as several new possible testing arrangements[19], the test setup illustrated in Fig. 12 was adopted. This is the only setup--of all those considered by the author--that allows the simultaneous introduction of axial force in the column and gravity loads on the beams. As illustrated in Fig. 12, all the relative movement between the column ends is achieved by allowing the bottom support to move horizontally.

Since the beams will move horizontally, the vertical loading of these elements should be done using gravity-load simulators[18]. Although one disadvantage of the proposed system is that it does not permit axial forces to be introduced in the beams, these forces usually have very little effect on the overall behavior of the subassemblage.

The loading frame, as well as the loading system and instrumentation, have been designed to permit great flexibility in selecting the test specimens and the type and sequence of forces to be applied to them. The test setup should permit prototypes as well as medium scale models of two- or four-way connections to be tested.

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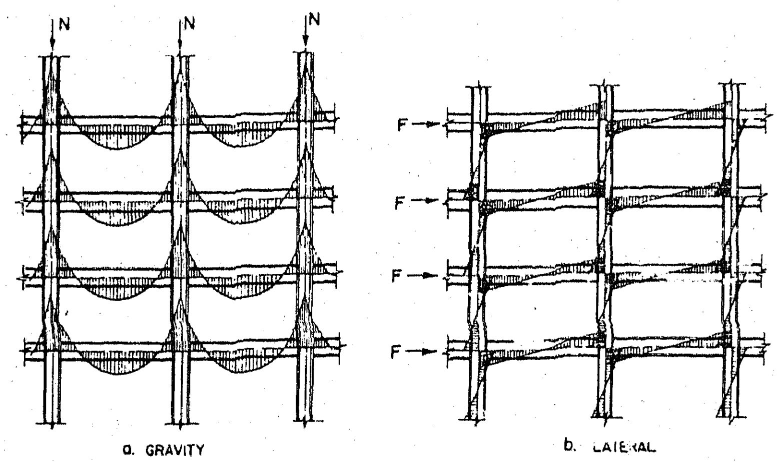

FIG. 1 BENDING MOMENT DIAGRAMS DUE TO GRAVITY AND LATERAL LOADINGS

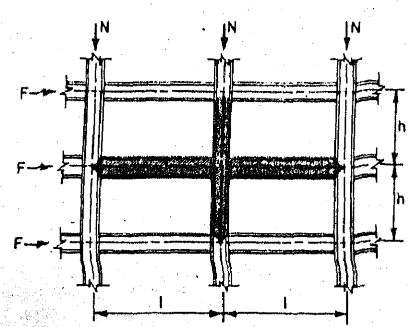

FIG. 2 BEAM-COLUMN SUBASSEMBLAGE

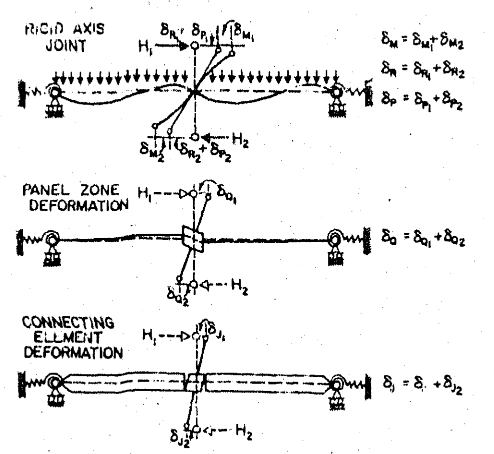

FIG. 3 DISPLACEMENT COMPONENTS OF COLUMN ENDS

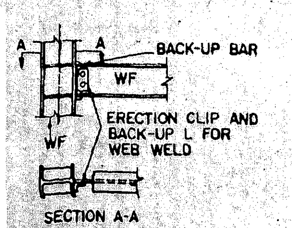

FIG. 4 WELDED CONNECTION

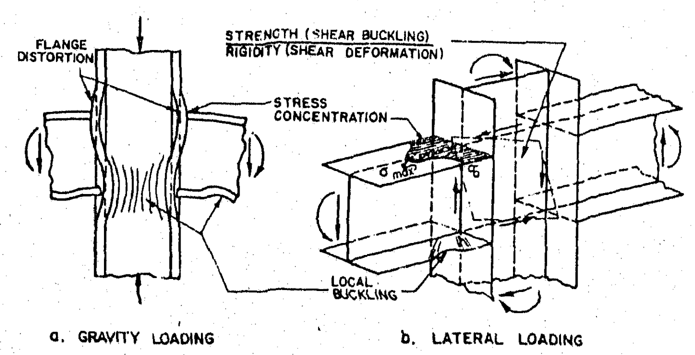

FIG. 5 PROBLEMS INTRODUCED AT BEAM-TO-COLUMN CONNECTION

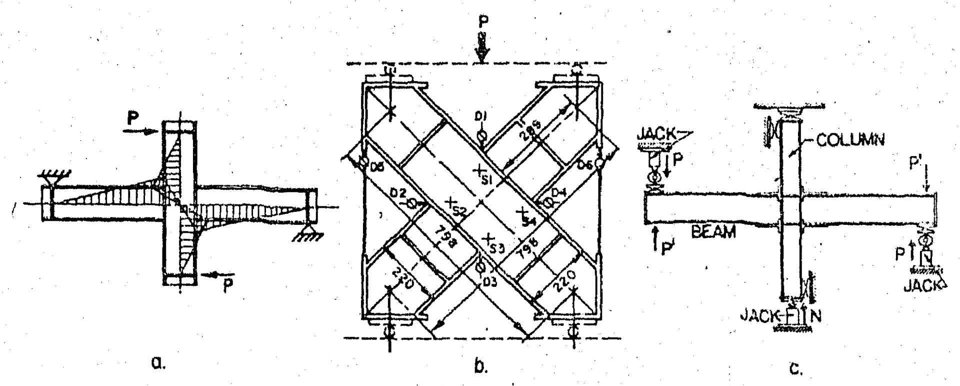

FIG. 6 TESTING ARRANGEMENTS USED IN PREVIOUS INVESTIGATIONS

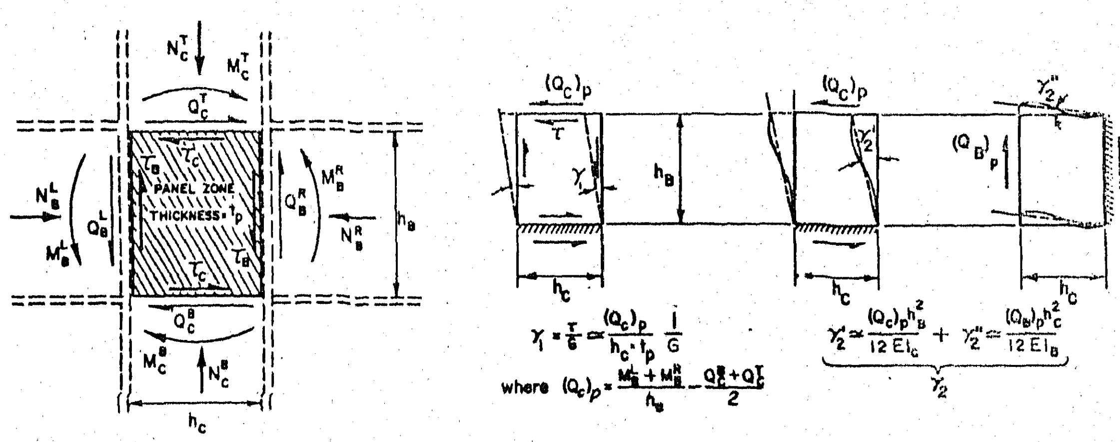

FIG. 7 APPROXIMATE CALCULATION OF PANEL ZONE DEFORMATIONS DUE TO SHEAR AND BENDING

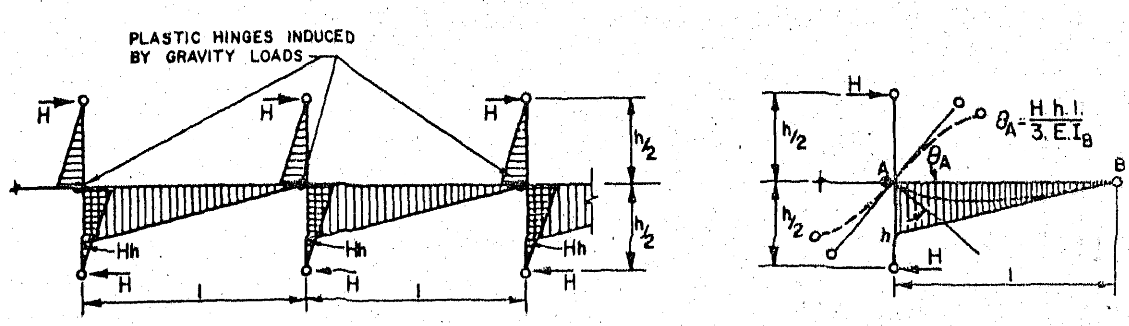

FIG. 8 CHANGE IN BENDING MOMENT AND JOINT ROTATION DUE TO FIRST APPLICATION OF LATERAL LOADING

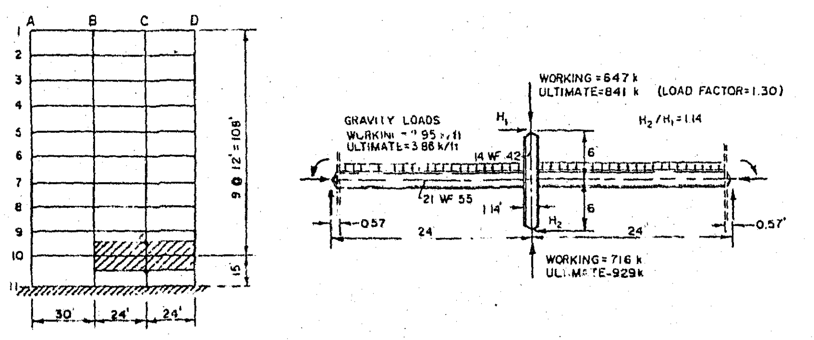

FIG 9 LOCATION, DIMENSIONS AND FORCES OF ANALYZED SUBASSEMBLAGE

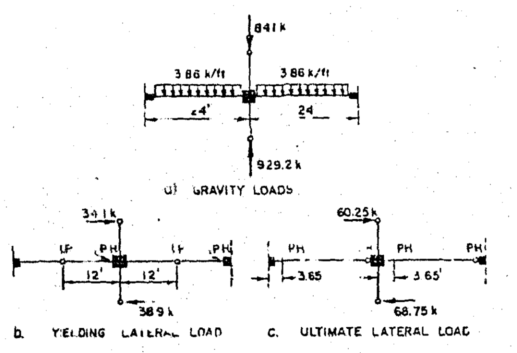

FIG 10 YIELDING AND ULTIMATE LATERAL LOADS

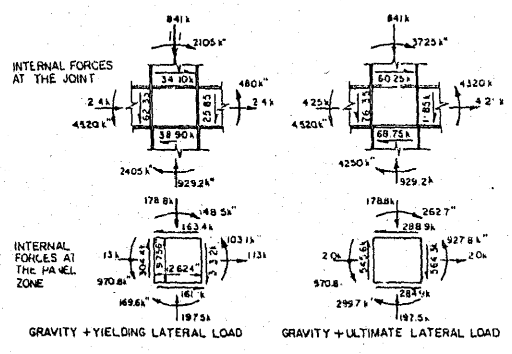

FIG 11 INTERNAL FORCES AT JOINT AND AT PANEL ZONE

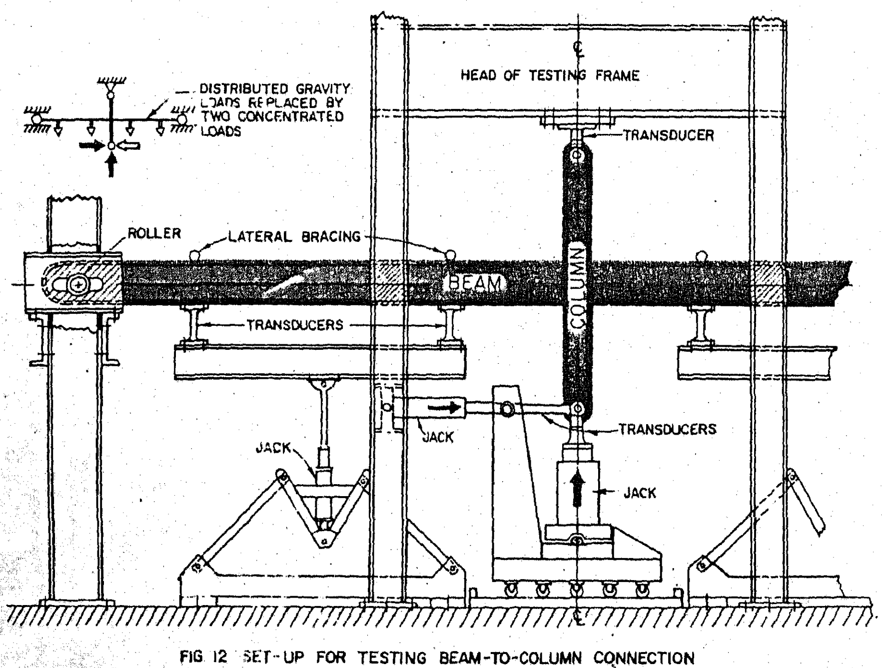

FIG 12 SET-UP FOR TESTING BEAM-TO-COLUMN CONNECTION