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GENERAL LOADING OF SEMI-RIGID CONNECTIONS

BY

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This is a report of the author's 1985 AISC National Education Foundation Fellowship Award Project in fulfillment of the requirements of the award. The author has, however, completed, and continues to work on, further research on this subject and is preparing manuscripts for publication. As these manuscripts are completed and when appropriate, the author intends to provide copies to the AISC as follow-up reports on this project.

ABSTRACT

GENERAL LOADING OF SEMI-RIGID CONNECTIONS

by Deryl Lee Earsom

This paper presents the development of a model for the behavior of semi-rigid connections under general loading, including moment reversals. The model is based upon experimental results of tests representing combined gravity and cyclically varying wind loads reported here and elsewhere. The model is implemented into a computer stiffness analysis of a simple frame and results are compared with approximate methods. Experimental data and analysis program listing are given in the appendices.

The conclusions of the paper suggest that semi-rigid connections are aviable structural performance option for moment transfer control. It is also suggested that by using a general connection model, more rational design and analysis methods could be developed

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GENERAL LOADING OF SEMI-RIGID CONNECTIONS

1. INTRODUCTION

Semi-rigid connections are becoming more attractive as designers search for new structural performance options. Additionally, these type of connections tend to be economical with respect to fabrication and erection costs. However,the lack of general knowledge regarding their behavior and specific guidelines regarding required performance limits their use.

1.1 STEEL CONSTRUCTION TYPES AND CONNECTIONS

There are three types of construction permitted under the provisions of Section 1.2 of the Specification for Design, Fabrication and Erection of Structural Steel Buildings issued by the American Institute of Steel Construction (AISC).

"Type 1, commonly designated as 'rigid frame' assumes that the beam-to-column connections have sufficient rigidity to hold virtually unchanged the original angles between intersecting members.

Type 2, commonly designated as 'simple framing' assumes that, insofar as gravity loading is concerned, the ends of the beams and girders are connected for shear only, and are free to rotate under gravity load.

Type 3, commonly designated as 'semi-rigid' framing assumes that the connections of the beams and girders possess a dependable and known moment capacity intermediate in degree between the rigidity of Type 1 and the flexibility of Type 2." (1)*. These three types of connections are shown diagrammatically in Figure 1.

Consider a moment vs. rotation diagram of a connection shown in Figure 2. According to the previous definitions and standard treatment, Type 1 (fixed-end) and Type 2 (pinned-end) connections represent the moment and rotation axes respectively. Type 3 (semi-rigid) connections represent anything between these axes.

In actuality, however, most of the connections commonly considered as either fixed or pinned have some finite moment-rotation relationship (Figure 3). Indeed, the boundaries between each type of connection are undefined and subjective. A schematic representation of some common connection types and their relative moment-rotation relationships, representing varying degrees of rigidity, is shown in Figure 4 (2).

* The number in parentheses in the text indicate references in the Bibliography.

Nevertheless, in most conventional analyses and designs of structural frameworks, idealized models of connections as either fixed or pinned continues as the only alternate cases to be considered by the engineer. This, despite the experimental evidence which shows that rarely are either models valid. In fact, "experiments carried out over the past decades have shown that actual joint behavior always falls between the two cases" (3).

In defense of the idealized models of connection behavior, the fact remains that they are easy to use and may represent behavior sufficiently well for most analyses. Implementation of actual connection behavior may be very complicated, depending upon the degree of accuracy desired. Full consideration of a connection's partial rigidity in analysis requires treatment of each connection as an individual structural unit, each with its own stiffness properties or matrix. To further complicate the matter, connection's behavior is highly nonlinear, as can be seen in Figure 4. The lack of specific guidelines regarding the connections and related servicability requirements, such as drift indices, makes the interactive decisions of frames and connections even more complicated (4). The extra consideration required of semi-rigid connections may lead to a considerable amount of additional and demanding work for the designer.

Despite the drawbacks of the semi-rigid connection treatment, there is need for their study and the inclusion of the semi-rigid nature of most connections in analysis. In all structural systems, and specifically building frames and bridge deck systems, the connections will be subjected to shear and bending moments resulting from "continuous structural action." The amount of continuity depends upon the ability of the connections to resist moment (2). Thus, the entire structural system is affected by the connections just as if they were standard structural members, and the fact that "the use of idealized connection behavior may, in many cases, lead to erroneous results" (5), perhaps they should be treated as such.

This argument becomes more compelling when one considers the trends of modern construction practices. In the past, walls and cladding, though often neglected in design, served as extra stiffening factors, which in general, produced larger factors of safety and better servicability. Modern construction, using thinner walls and cladding of virtually negligible rigidity, coupled with modern connections which along with being less labor intensive are also more flexible, results in structures which are more flexible than their predecessors (5).

These realizations have been recognized by the authors of the U.S. Limit States Design Code, referred to as the Load and Resistance Factor Design (LRFD) Specifications. In the

provisions of the LFRD Specifications, only two types of construction are designated (as opposed to the AISC Specifications which provide for three); Type FR, fully restrained connections, equivalent to Type 1 of the AISC, and Type PR, partially restrained connections, equivalent to Type 3, but also encompassing Type 2 of the AISC (3). Furthermore, and of a more subjective argument, it seems necessary that structural designers understand the properties of the materials and the techniques they use in their profession, whether or not these properties need be incorporated in all cases.

All reasoning behind consideration of semi-rigid connection behavior is not to avoid the possibly dangerous over-simplification of a structural system. The most significant aspect of semi-rigid connection behavior and analysis is that it provides for a new structural performance option and design refinement to the designer. Semi-rigid connections can be considered as an additional tuning element of a structure's behavior.

Consider Figure 5, which shows a simple portal frame under gravity loading. In the case of a pinned-end beam, the extreme moment in the beam is at a maximum, and the frame has its least resistance to lateral forces. In the case of a fixed-end beam, the moments at the center of the beam are reduced to their minimum at the expense of larger moments, their maximum, at the ends of the beam. This frame

condition has greatest resistance to lateral forces, but also the greatest amount of moment transfer to the columns.

Traditionally, these two cases represent the only options available to the designer. In the case of the fixed-end beam, any adjustments in relative moments requires changing the relative moments of inertia or spans of the beam and columns.

The use of semi-rigid connections results in a case intermediate between the pinned and fixed-end cases. The moments transferred to the columns can be reduced in proportion to the increase at the center of the beam. Thus, the relative connection stiffness, as well as the relative stiffnesses of the beam and columns, can be adjusted to change the moment distribution. As well, the frame still has an intermediate resistance to lateral forces.

Clearly, this structural option could have an impact on structural optimization and economy in many situations. For example, use of the moment resisting capacity in connections traditionally modeled as pinned has been shown to provide significant economy in the case of roof pounding (6). Nevertheless, the additional and sometimes extensive design work discussed earlier and the savings of steel, fabrication, and erection costs, "do not always have clear guidelines as to the economic consequences of such design precision" (4).

There is, however, much current work being done in providing more general guidelines for the design, uses, and costs of semi-rigidly connected systems. The increasing use of computers in analysis makes the inclusion of semi-rigid joint behavior more attractive by reducing the computational demands of the designer, while providing a better understanding of the response characteristics of real frames.

1.2 HISTORICAL DEVELOPMENT OF SEMI-RIGID CONNECTION TREATMENT

While it may have always been intuitive that connections would undergo nonlinear moment-rotation deformation at some, presumably high, load level due to yielding of connection components, the first attempts to experimentally analyze the moment-rotation characteristics of commonly used connections did not occur until the early 1930's. These early results showed the highly nonlinear semi-rigid nature of these connections. One of the first publications was by J.C. Rathburn in 1935 (7). He experimentally determined the monotonic (static) moment vs. rotation curves for several connection configurations in common use. In the discussions of his publication, his work is referred to as necessary and of unquestionable contribution to structural engineering but also a "bitter dose" to the notion of the current accuracy in rigid frame analysis and formed further complications to problems

already so complicated "that few engineers other than college professors bother to solve". Also reported in the discussions of this paper, is a method of analyzing these connections that is still in use today, the Beam-Line Method.

The Beam-Line Interaction Diagram, developed by C. Batho, is a direct method of determining the end conditions of a statically loaded member and connection by using the connection's monotonic moment-rotation curve together with the member loading, span, and moment of inertia. Its use is straightforward and illustrative of semi-rigid connection behavior.

The basis of the Beam-Line concept is that the moment and rotation at the end of a beam (or any member) must be compatible with the moment and rotation achievable in a connection. For the connection, the moment-rotation interaction is most directly expressed by the moment vs. rotation curve of the connection (see Figure 4). This curve may be experimentally obtained, or, as is the current research emphasis, analytically generated. For the beam, there is a relationship between the end moment and the end rotation which can be expressed in terms of load, span, and moment of inertia. For example, a uniform beam with a moment of inertia I, length L, under a uniformly distributed load w, and equal restraining moments at each end, has an end

moment and rotation relationship given by the beam equation.

$$\theta = \frac{1}{24}\frac{wL^3}{EI} - \frac{1}{2}\frac{ML}{EI}$$

This equation represents the beam-line on a moment vs. rotation plot and in the case of a single beam structure, can be easily plotted using the familiar points

Fixed-End Moment (Rotation=0) $\quad M_F = \frac{WL^2}{12}$

Pinned-End Rotation (Moment=0) $\quad \theta_0 = \frac{WL^3}{24EI}$

The intersection of the Beam-Line and the monotonic moment-rotation curve of the connection represents the only condition compatible for both the beam and connection under the given static loading and structural conditions (Figure 6).

The Beam-Line concept illustrates not only the principle of semi-rigid connection behavior but also the importance of having a reliable moment-rotation curve for the connection in question. The AISC Specifications state that "Type 3 (semi-rigid) construction will be permitted only upon evidence that the connections used are capable of furnishing, as a minimum, a predictable proportion of full end restraint". Implying that a connection's moment-rotation curve be available, this requirement has

contributed to the limited use of semi-rigid design and prompted considerable research into analytical generation of moment-rotation curves in addition to actual connection testing. A.V. Goverdhan has collected much of the experimental work and used it to evaluate many of the analytical predicting equations for monotonic connection moment-rotation curves (5). The need for the availability of connection moment-rotation curves for semi-rigid design has prompted some researchers to propose the establishment of experimental data banks (8).

While the Beam-Line and the connection's monotonic moment-rotation curve form the basis of semi-rigid connection response to gravity loading, methods were developed which sought to simplify the design and analysis of semi-rigid construction and use them in resisting lateral wind forces. The earliest method was "Flexible Wind Connection Design". This apparently contradicting description was, and continues to be, the most widely used method of semi-rigid design. The method is typically used for the Top and Seat Angle Connection with Web Shear Angles (Connection Type C, Figure 4).

The "Flexible Wind" connection design method is a design superposition of the connection components. The girders, together with the individually flexible web shear angles, are designed as a simple pinned-end assembly for gravity loads only. The relatively stiff flange angles are

designed for wind moments only. The columns are designed for wind moments and axial loads. Several notable buildings have been designed using this method, including The Empire State Building, The Chrysler Building, and The United Nations Security Building (9).

Though the connections are Type 3 (semi-rigid), this method has been designated as Type 2 with Wind Connections. This, apparently, to exempt this long and widely used design method from the AISC's requirements for Type 3 (semi-rigid) construction.

The first paper which rationalized parts of this method in terms of actual semi-rigid connection behavior was, "Directional Moment Connections - A Proposed Design Method", published by R.O. Disque in 1975 (10), at that time, Chief Engineer of the AISC. He based his analysis on the observed loading and unloading behavior of steel beam-to-column connections. Though his paper seems to deal primarily with what are considered as fully-continuous connections, it also seems applicable to the semi-rigid connection types considered here.

He asserted that the moment-rotation curve of a properly designed steel connection has considerable ductility at some plastic moment. However, if the lateral loading rotates a beam-column connection in the opposite direction that produced the plastic hinge, the connection acts elastically. A moment connection, therefore, can

result in a plastic hinge in one direction but an elastic hinge in the opposite direction (Figure 7).

He continued that if the connection was designed for a 1/3 increase in allowable stress for wind, then the connection should possess a plastic moment less than that of the girders and therefore plastic moments could develop in the connections under gravity load alone (Figure 8). Then, under wind loads, the leeward ends of the beams act as plastic hinges (additional wind moment in same sense as gravity moment) but the windward ends act as elastic moment connections (wind moment in opposite sense of gravity moment). Therefore, only the windward ends are available to resist moment (Figure 9). In his method, however, he proposes a more conservative column design than the traditional "Flexible Wind" connection approach. He proposes that the columns be designed to include the possibility of gravity moments equal to the design connection moments as well as the possibility of combined wind and gravity in the directional moment structure.

In 1976, the American Iron and Steel Institute (AISI) published a paper promoting the "Flexible Wind" connection design method because of economy in design, fabrication, and erection (9). The paper also expanded the rationalization of the separation of gravity and wind moments and provided further insight into semi-rigid connection behavior.

The paper's description of semi-rigid behavior begins with a connection moment-rotation curve and a beam-line providing the initial conditions of connection gravity moment and rotation at each end of a beam. The sign convention is that both end moments are positive under gravity loading (Figure 10).

When the wind blows positive from left to right, shown in Figure 11, the additional wind moment on the right connection is in the same sign sense as the gravity moment and thus the moment-rotation movement is up the connection moment-rotation curve from point 0 to point 1. The moment at the left end of the beam is reduced by the directional sense of the wind moment and unloads elastically to point 1'. It is assumed that both ends of the beam rotate the same amount until the entire wind moment is developed. Since it is stiffer, the left connection provides more of the resistance to the wind moment.

If the wind ceases, as shown in Figure 12, the left connection, unreduced by the wind moment, reloads elastically from point 1' to 2'. Relieved of the additional wind moment, the right connection unloads elastically from point 1 to 2. Both connections now having elastic stiffness, assumed equal, unload such that both resulting end moments are equal.

If the wind now blows in the negative direction (right to left), shown in Figure 13, an opposite sense of wind

moment is applied to each of the connections. The left connection now receives wind moment in the same sense as the gravity moment and loads elastically until it intersects the moment-rotation curve and then moves up the curve from point 2' to 3'. The right connection continues to unload elastically from point 2 to 3. Again, each connection is assumed to rotate through the same angle until the total wind moment is developed. The right connection, having a slightly greater net stiffness, resists slightly more of the wind moment.

When the wind stops again, Figure 14, both connections have presumably equal elastic stiffnesses and unload equally from point 3' to 4' and from point 3 to 4. The only loads at this point are gravity moments, which are assumed to have reduced significantly. They are said to have "shaken down". Figure 15 shows what would happen in this analysis if the wind were to again blow in either direction. The connection is now entirely elastic. No matter which direction the wind blows the maximum moment in the connections will not exceed the previous maximum moment, point 3'. Thus, shakedown of the connections is complete. The result is an elastic wind connection with reduced gravity moment.

In considering stability and servicability, the paper states that the softness of the connection does not significantly affect performance. It suggests that the rest of the frame analysis be conducted as if the connections

were Type 1 (fixed-end). This includes the effective lengths, K, of the members. However, the paper does reference a method to adjust the K factors of the AISC Type 1 Alignment Chart using the initial stiffness of the conncetion as a stiffness reduction factor, increasing the effective length of a member (11). Furthermore, the paper asserts that the lateral drift of the structure can be computed considering the structure as a rigid frame.

While each of these papers provide insight into the performance of semi-rigid connections and rationale for the "Flexible Wind Connection" design method, there are approximations and omissions, in both the publications and the design philosophy, which should be highlighted and form the basis of this work's objective.

Description of connection behavior in both papers is based upon the monotonic moment-rotation curve loading and elastic unloading stiffness equal to the initial connection stiffness. At the time of publication, this behavior was consistent with most of the current experimental data in which connections were monotonically loaded and unloaded for a single cycle. This is also true of much of the loadings in current experimental research. As will be discussed later, however, there is evidence that the unloading stiffness can vary depending upon the magnitude of the moments and decay after several loading and unloading cycles. Even so, in some respect, both papers and the design method, disregard

the effect of the connection softness, that is, less than full rigidity, in the consideration of continuous frame action in response to lateral loads. While both papers provide some manner of reducing the K-factor of the columns, they do not consider the connection stiffness in determining lateral drift and the secondary effects of Axial load-Deflection (P-Delta) in the columns although both papers seem to imply methods based upon the design assumptions. This omission may be especially significant in case of columns subjected to high axial loads.

The AISI's paper includes the assumption that both ends of the beam rotate equally in response to wind moments. Clearly, this assumption is not true if the beam has any flexibility, though the deviation may be insignificant (Figure 16). Additionally, the AISI paper does not consider the effect of the residual difference in rotations at each end of the beam which results from the analysis (see Figure 15). In accordance with the previous assumption of equal end rotation and beam equilibrium, the residual difference in rotations would require a residual drift of the frame in the direction of first wind loading (Figure 17).

The foremost feature of the "Flexible Wind" connection design method is the superpositon of the separate gravity and wind loadings. The validity of the assumption that the superimposed moments will produce a reasonable representation of the actual moment distribution after

shakedown, seems to require favorable loading and geometry of the structural system. The rationale seems to rely on a connection, designed by standard allowable stress methods, which results in favorable initial gravity end conditions for the particular structure without the explicit use of either beam-line or connection moment-rotation curve. The initial conditions can have significant effects and may, perhaps, warrant more attention.

The initial condition of high gravity end moments was rationalized in the previously discussed paper by Disque, provided that a single connection can resist the entire wind moment for both ends of a beam in one direction (Figure 7). This is also implied in the AISI's paper (Figure 15). However, these initial gravity end moments may be significantly higher than the connection design moment capacity, and a plastic moment capacity for these connections has yet to be well defined. High initial gravity moments would also result in an increase of the residual end rotation difference and greater residual drift (Figure 15). Additionally, a higher initial gravity moment would presumably result in a larger "shaken down" gravity end moment. These larger end moments would decrease the midspan moments of the beam and decrease the efficiency of the beam, designed for pinned-end conditions.

Lower initial gravity end moments would increase the likelihood of moment reversals in the connection, not

considered in either rationalization of this design method. If this behavior is acceptable, however, the result may be an excessively conservative design. Not considering moment reversals not only makes additional demands on the rationalization of the design method, as discussed, but as well, may not account for additional connection restraint potential. The use of only one direction of moment represents only one-half the plastic moment range of a symmetric connection (Figure 18). At present, even those engineers who regularly use semi-rigid design and perform full structural analyses of the frame and connections advise against use of these connections in resisting cyclic moment reversals because of the lack knowledge regarding their response.

Despite the assumptions, "Flexible Wind" connection design seems to be the most common method of using semi-rigid construction. Evasion of the AISC provisions for Type 3 (semi-rigid) connections and eliminating the use of connection moment-rotation curves makes this method quite simple to use and reduces design time significantly. The design method does not, however, provide for a complete description of semi-rigid connection behavior under general loading (Figure 19), and despite the recommended use of certain design assumptions in calculating frame action, does not represent a true structural analysis, such as that which could be used in a limit state analysis.

1.3 SCOPE AND OBJECTIVE

1) Test standard configuration semi-rigid connection assemblies subjected general loadings representing varying gravity and cyclically varying wind moments.
2) Using the test results, develop a model to describe semi-rigid connection response to general loadings.
3) Develop a computer program which incorporates the connection model for the analysis of a simple frame, designed by the "Flexible Wind" connection method, subjected to various loading combinations and geometry variations.
4) Using the computer program and the hysteresis model, investigate the acceptability of designing under the hypothesis of pinned connections for gravity loads, neglecting continuous frame action.

While there are methods for the design of semi-rigid connections, they may not adequately account for variations in connection stiffness and those of connecting members. Also, initial conditions, ultimate conditions, or secondary effects may be inaccurately modelled. The purpose here is to determine if assumptions in normal design of semi-rigid connections are reasonable, or if they are "over-simplified".

2. EXPERIMENTAL

The following tests were conducted at the Washington University Structural Laboratory (WUSL) in 1985 and '86 by the author.

2.1 INTRODUCTION

While there is limited test data available for certain connection configurations, including semi-rigid connections, under monotonic loading and unloading through a single cycle (for example 5, 7, 8), there is even less for cyclic moment reversals (5, 12, 13, 14). Most of these cyclic tests are for continuously welded connections (12, 13). None of the reports found included testing under combined static gravity loads and cyclic wind loads.

The objective of the tests on semi-rigid connection assemblies was to determine their behavior to general loadings representative of typical building conditions by subjecting them to loads representing varying gravity moments and cyclically varying wind moments, including moment reversals.

2.2 DESCRIPTION OF TEST ASSEMBLY

The test assembly, designed to simulate the effects of static gravity moments and cyclically varying wind moments on semi-rigid steel beam-to-column connections, is shown in Figure 20. It consists of a column stub and two beam stubs connected by semi-rigid connections creating a T-Frame assembly. The ends of the beams are positioned in pinned

"frictionless" sleeves and a transverse fulcrum reaction is supplied near the top of the column stub. Relative angular rotations between the beam stubs and columns are determined by measuring the relative displacements between the beam stubs and a level bar attached to the column stub.

This assembly produces a condition where the external loads, applied axially to the top of the column and transversly at the bottom of the column, produce determinant reactions and moments in the connections.

The pinned "frictionless" sleeves at the ends of the beam reduce the axial load through the connection. This is important in that the connections resist much of the applied moment by tension and compression coupling of the flanges. In actuality, however, the opposing reaction near the connections was measured at approximately 70% of the externally applied tranverse load, in the absence of the perpendicular reactions of the column axial load. The presence of the axial load reactions may very well reduce the effectiveness of the sleeves. Even so, the tensile and compressive forces in the couples required to resist the applied moments at the connections are considerably greater than the maximum possible axial load reducing the significance of its effect.

The test assembly T-Frame represents the intersection of two beam spans to a column. The axial load, P_g, applied at the top of the column, produces moments, M_g, which

represent those produced by gravity loads on the beams. The transverse load, P_w, produces moments, M_w, which represent those produced by lateral or wind loads. This produces an additional moment in the same sign sense as the gravity moment on one connection, increasing the total moment applied at the connection, M_c. On the other connection, the wind moment opposes the gravity moment, reducing the total moment (Figure 21). Thus, loading of the connections can be produced which is similar to that used in the AISI's rationalization of the "Flexible Wind" connection design method previously outlined and more representative of actual structural conditions than single cycle static or pure cyclic loading. However, the assembly requires that each connection resist an equal moment.

2.3 TEST CONNECTIONS

The connection tested is of a typical semi-rigid configuration consisting of A36 steel web and flange angles (Figure 22). The connection was designed so that the angles would yield prior to any yielding in the beam and column stubs, and bolts. It has a "Flexible Wind" design moment capacity of approximately 1.6 kip-feet (9). The connections have Grade 8, 3/8" diameter bolts, nuts, and washers in 7/16" drilled holes.

2.4 DESCRIPTION OF TEST LOADINGS

Four types of loadings representing the extreme conditions to which a connection would be subjected were

tested including two gravity moment conditions. An outline of the test loadings is presented in Appendix 1, Table 1. The two gravity moment conditions represent the bounds of actual structural conditions.

In the first case, a series of gravity moments were applied and held constant by the axial column load, P_g, while the lateral load moments, produced by P_w, were cyclically increased. This condition represents the case where the initial gravity moment, once applied, is not allowed to "shake down" (see Figure 15).

In the second case, an initial gravity moment was applied, producing a gravity rotation and deflection (see Figure 21). This deflection and net gravity rotation was then held constant by adjusting the gravity load while the lateral load moments were again cyclically increased. This loading represents the condition where gravity moment "shake down" is unimpeded. It also represents the case where the beam deformation inflection point is held at a constant location (see Figure 16).

Equilibrium of the assembly does, however, require that both connections resist the same moment. As this situation is not always required in actual structural conditions, it represents a deviation from general loading behavior. This condition is, however, a lower bound of performance in that the greater proportion of moment is not resisted by the stiffer connection. Therefore, the test results for the net

joint moment-rotation behavior, that is the lateral wind moment vs. column rotation, will indicate greater rotation, thus a lower bound of structural performance. The connections were also tested under a monotonic load and pure cyclic loadings.

2.5 DESCRIPTION OF TESTS

The following sections give brief descriptions of each of the experimental tests. The experimental data is given in Appendix 1.

In all the tests except 1 and 7, the fasteners were tightened to a torque of 30 foot-pounds. This value was obtained by tightening several fasteners by the turn-of-the-nut method and measuring them with a torque indicator wrench. The rest of the fasteners were then tightened to the selected value using such a wrench.

2.5.1 Test 1: Trial

Test 1 was a trial test run for the test assembly. As such, the connections were not carefully tightened, resulting in significant bolt slip producing considerable rotation in the connections. The data for Test 1 is not included here.

2.5.2 Test 2: Constant Gravity Moment

In Test 2, an initial gravity moment of 1 kip-ft was applied and held constant while a lateral wind moment was applied and cyclically increased to produce a maximum joint (both connections) moment of 14 kip-ft. The design moment

for the joint being twice that of each connection or roughly 3.2 kip-ft. Each lateral load increment was applied in two complete cycles. Deflections were recorded at each load increment.

Figure 23 shows the net joint moment vs. joint rotation for Test 1, representing the lateral wind moment vs. column rotation. The sign convention for the joint moment-rotation being counter-clockwise positive, as indicated in Figure 20.

Figure 24 shows the indivdual connection moment vs. rotation results for Test 1. The sign convention for the individual connection moment-rotation graphs being that the directions of gravity moment and rotation are positive, similar to that of the AISI's rationalization analysis of "Flexible Wind" connections (see Figures 10-15).

2.5.3 Test 3: Constant Gravity Moment

In Test 3, an initial gravity moment of 3.2 kip-ft was applied to the connections. This gravity moment was held constant while a lateral load joint moment was cyclically increased to 12 kip-ft, with two complete cycles for each load increment. The joint moment-rotation results are shown in Figure 25 and the connection moment-rotation diagrams are shown in Figure 26.

2.5.4 Test 4: Monotonic Loading

Test 4 is a monotonic loading of the connections and joint. With no gravity load applied, a lateral load moment

was incrementally increased in one direction to a total joint moment of 14 kip-ft.

Figure 27 shows each of the connection moment-rotation curves and the average symmetric moment-rotation of the connection. The joint moment-rotation curve is the first quadrant of Figure 27 with double the moment values.

2.5.5 Test 5: Constant Gravity Angle

In Test 5, an initial gravity moment of 3.2 kip-ft was applied to the connection and the deflections measured. Assuming equal deformation, the vertical deflection of the column represents the gravity rotation of the connections. By maintaining this vertical deflection, the gravity angle is also held constant. The lateral wind moments were then incrementally cycled, two cycles per increment, to a maximum joint moment of 4 kip-ft while adjusting the gravity load to maintain the deflection until the gravity moment "shook out" completely, that is, until both the gravity and wind moments were zero.

Figure 30 shows the joint moment-rotation diagram for each incremental cycle. The connection moment-rotation curves are shown in Figure 31.

2.5.6 Test 6: Cyclic Wind Moments

For Test 6, no gravity moment was applied. The connections were subjected to cyclic lateral load moments only. The moments were incrementally increased with two cycles per load step to a maximum of 12 kip-ft.

The joint moment-rotation results for each cycle are shown in Figure 34. The connection moment-rotation results are shown in Figure 35.

2.5.7 Test 7: Cyclic Wind Moments

In Test 7, the fastener torque was increased to 50 foot-pounds to examine the effect of slip in the connection. The loads were applied similar to those in Test 6. No gravity moment was applied and the lateral wind moments were cyclically increased to a maximum joint moment of 14 kip-ft.

Figure 37 shows the joint moment-rotation behavior and Figure 38 shows that of the individual connections.

2.6 TEST RESULTS

The following sections give brief discussions of results observable in the moment-rotation hysteresis loops for each of the tests. A more general analysis of connection behavior is contained in the next section which deals with development of a hysteresis model based upon these and other test results.

2.6.1 Test 1 Results

While the complete results of Test 1 are not included here it is worth noting that the effect of the bolt slippage in these connections produces an exaggerated pinching of the moment-rotation hysteresis. If extreme enough, the bolt slippage is directly observable through a horizontal (zero stiffness) portion of the moment-rotation response.

2.6.2 Test 2 Results

The effect of a low gravity moment means that the initial end conditions of a beam, determined by the beam-line and connection curve, are at a location nearer the origin of the connections moment-rotation curve. As such, each of the connections have greater ranges of relatively stiff behavior as can be seen in Figure 24. This results in a greater range of stiff behavior for the joint. Figure 23 shows that the joint has a relatively tight moment-rotation hysteresis up to a moment approximately + or - 8 kip-ft, roughly 2.5 times the design capacity of both connections or 5 times that of one. It should be reiterated here, however, that the test conditions represent the lower bound of performance by requiring that both connections resist equivalent moments, rather than moment distribution relative to stiffness.

Figure 24 shows that the connection behavior corresponds to an initial loading along a curve whose profile is similar to that of a monotonic moment-rotation curve with relativly elastic unloading and that the joint hysteresis widening corresponds to peak loadings along the low stiffness upper region of this curve profile, not the occurrence of reverse moments. At low loads, the connection response exhibits stiff behavior in what is shown as negative moment reversals as would be expected in a symmetric connection with a low initial gravity moment. The

reverse bending stiffness does not decrease significantly until the occurence of larger peak rotations.

2.6.3 Test 3 Results

The effect of a high gravity moment means that the initial end conditions of the beam are high on the connection moment-rotation curve, nearer the region of "soft" connection stiffness, and initial gravity angles are greater. Thus, as wind load is applied, one connection loads into the increasingly softer range while the other has a large range of elastic unloading stiffness. Therefore, as would be expected, the joint moment-rotation hysteresis is slightly more open at lower joint moments as can be seen in Figure 25, though still relatively tight to the same moment as in the low gravity moment condition, + or - 8 kip-ft. However, the connection moment-rotation loop remains tighter at higher moments than in the lower gravity moment Test 2.

The individual connection behavior, Figure 26, again shows a behavior that corresponds to loading along a curve similar in profile to a monotonic moment-rotation curve and stiffer, relatively elastic unloading. Figure 26 also shows that the stiffness under lower net moment reversals, though softer, does not drastrically diminish to zero. The behavior which produces the stiffer joint moment-rotation performance at higher loads is indicated in Figure 26. The

high initial moments have established large ranges of elastic behavior in the direction of the gravity moment.

2.6.4 Test 4 Results

The average symmetric monotonic loading curve for the connection is shown in Figure 27. One-half of this curve can also be used to represent the monotonic moment-rotation curve of the joint assembly by doubling the moment values.

Comparison of the connection monotonic moment-rotation obtained in Test 4 with the average initial loading peak profiles of the connections in Tests 2 and 3 are shown in Figures 28 and 29 respectively. These results indicate that the behavior of the connections in cyclic loading is related to the monotonic moment-rotation curve of the connection, even when the loading includes net moment reversals.

2.6.5 Test 5 Results

By holding the gravity angle constant, the net result is a much tighter and stiffer joint moment-rotation hysteresis, as is evident in Figure 30. Figure 31 shows the connection moment-rotation curves.

The stiffer behavior of the joint assembly is a result of the gravity moment shakedown and the restraints imposed upon the connection behavior by the test conditions. The gravity moment shakedown is implicitly evident in the connection moment-rotation curves. Maintenance of the initial gravity angle required reducing the gravity moment such that the connections maintained equal moment and angle

differences. This reduction is evident when comparing the connection moment-rotations curves of this test (Figure 31) and Test 3 (Figure 26). Though both tests began with the same conditions, the moment in either connection in Test 5 never excceds 5.5 kip-ft, even though the joint wind moment is 6 kip-ft, while at the same wind loading in Test 3, the moment in either connection has reached almost 6.5 kip-ft.

Analysis of the data shows that the gravity angle did begin to increase significantly during the last series of cycles and may explain the small instances of apparent negative stiffness of the connections near zero moment (Figure 31).

Figure 32 shows that again the profile of the initial peak loading curve for the connections in Test 5 is roughly that of the monotonic connection moment-rotation curve.

The shakedown of the gravity moments is explicitly shown in Figure 33 which shows the applied wind connection moment versus the gravity moment at each load step.

2.6.6 Test 6 Results

With no gravity moment applied, the result is a more flexible joint moment-rotation behavior. Figure 34 shows that the appreciable widening of the joint moment-rotation hysteresis occures at a lateral moment of +/- 8 kip-ft.

The individual conncection moment rotation curves (Figure 35) show once again that the connection behavior corresponds to loading along a moment-rotation curve profile

similar to that of a monotonic curve and relatively elastic unloading. However, without the initial gravity deformation, neither of the connections will be elastically unloading during initial application of a lateral load moment, which accounts for the wider joint hysteresis. These results also show that moment reversals begin to result in low stiffness as the unloading rotation intersect (zero moment) gets larger.

Figure 36 shows the comparison between the monotonic moment-rotation curve of Test 4 with the average initial peaks of the connection moment-rotation curves of Test 6. The softer curve of Test 6 believed to be attributed to slip in the connection fasteners. This slip is indicated in both the joint and connection moment-rotation curves (Figures 34 and 35 respectively) as sections of low, linear stiffnesses at higher moments corresponding to a rotation range of approximately 0.02 radians. This value is equivalent to the maximum bolt slip from the 1/16" oversized holes, top and bottem, over the 6" beam depth.

2.6.7 Test 7 Results

By increasing the fastener tightening to reduce the occurence of bolt slippage, the result is significanlty stiffer joint. Figure 37 shows that the joint moment-rotation behavior is nearly linear up to a joint moment range of + or - 10 kip-ft. As well, Figure 38 shows that the individual connections exhibit a similar increase in

stiffness. Figure 39 shows the average initial connection moment-rotation peaks for Test 7 and the monotonic curve from Test 4.

2.7 DISCUSSION OF TEST RESULTS

The most evident feature of these tests regarding the behavior of these semi-rigid connections to general loading is that the individual connection loading curves are closely related to the monotonic moment-rotation curve of the connection (Figures 27-29, 32, 36, and 39) despite the apparent deviation in Tests 6 and 7 (Figures 36 and 39 respectively). If this deviation can be attributed to bolt slippage, as the results suggest, then the significance of this effect will be less in a full-size beam and connection.

Another result is that the connections unload from this curve to zero moment with relatively elastic stiffness, which is, at lower rotations, roughly equal to the initial loading stiffness of the connection. This elastic unloading stiffness does seem to reduce somewhat in proportion to the extent of rotation (Figures 24, 26, 31, 35, and 38).

When reloading without a net moment reversal, the connections maintain this stiffness to the point of unloading from the initial loading curve profile, then follow this curve profile under further loading (Figures 24, 26, 31, 35, and 38).

At the occurrence of net moment reversals, the stiffness of a connection is reduced in proportion to the amount of rotation and does not load along a displaced initial loading curve. Rather, the connection seems to load along a path from the rotation at moment reversal towards a point on the symmetric initial loading curve profile at the origin. If large enough, the reverse moment path will eventually intersect and load along this profile (Figures 24, 26, 31, 35, and 38).

From the moment reversal loading path, the connection again unloads elastically and after zero moment, the connection loads with a relatively linear elasticity towards the maximum point attained on the initial loading curve (Figures 24, 26, 31, 35, and 38).

The behavior outlined here for the connections in one direction of net moment is, with the exception of reduced unloading stiffness in proportion to rotation, equivalent to that used in the AISI's rationalization of the "Flexible Wind" (9) and Disque's "Directional Moment" (10) connection design methods, even after the occurrence of net moment reversals. By including the behavior of the connections in reversed moment conditions, a model can developed to represent the simplified response of semi-rigid connections to general loading.

3. MODEL DEVELOPMENT

By applying the results of the tests of a standard semi-rigid connection configuration to loading conditions representing varying gravity and cyclically varying wind loads, a simplified model for semi-rigid connections of similar configuration to general loading will be developed. This model will then be used to generate comparitive results to experimental data reported here and elsewhere.

3.1 GENERAL CONNECTION BEHAVIOR

By examining the deformation of a standard semi-rigid connection in response to moment, an attempt can be made to qualify the major features of observable experimental behavior. To begin, consider a standard semi-rigid connection configuration consisting of web and flange angles and its symmetric monotonic moment-rotation curve shown in Figure 40.

If loading occurs in one direction, defined as positive, the connection loads up along its moment-rotation curve and deforms to a position shown in Figure 41. The standard assumptions regarding the resisting forces in the connection (2), can be visualized in the deformation pattern. In this configuration, the beam rotates about a point near the bottom flange angle which is closed and in compression. The top flange angle, along with the web shear angles, are in tension and have been pulled away from the column face. If the moment is large enough, plastification

will have occurred at some points in the tension elements and nonlinear rotation will have occurred, as is indicated in the figure. The moment at the connection is resisted by the tension and compression couple developed in the connection components. If the web angles are small enough and the gauge length of the column fasteners large enough, as is encouraged in design, then the major moment resisting couple is developed in the flange angles.

As the moment is reduced, the connection unloads to zero moment with relatively elastic stiffness equal to the initial stiffness. However, there exists residual deflection in the top flange and web angles due to the plastification and hinging which occurred during the initial loading. Thus, a residual rotation exists in the connection, Figure 42. If a positive moment were reapplied, the connection would load elastically with the same stiffness to the point of initial unloading. Upon further increase of positive moment, the connection would resume its path along the monotonic curve.

If, after positive unloading, a negative moment is applied to the connection, the top flange angle will go into compression and the bottom flange angle into tension. Recognizing the difference in geometry at this point, the top flange not yet in bearing with the column face, compared with the initial positive loading condition where the bottom flange was in closed bearing, it is evident that the negative

moment reversal loading path will not be as stiff as the elastic positive unloading nor will it follow the path of the negative monotonic curve displaced to the residual rotation. Rather, as the connection assumes a transitional deformation where both the top and bottom flange angles are open, its stiffness will be reduced and its moment-rotation path will also be transitional, Figure 43.

With further application of negative moment, the compression in the top flange will be sufficient to close the angle into full bearing against the column face. This situation is comparible with some point of the initial positve loading curve or initial negative loading instead of positive, one flange angle in full bearing against the column face and the other angles in tension, i.e. pulled away form the column face. Thus it seems reasonable that the negative loading path will intersect and begin loading along the negative monotonic curve at some point, Figure 44. The initial point of intersection seems to roughly correspond to the elastic limit of the connection's negative monotonic moment-rotation curve. This could be rationalized by noting that the moment at the elastic limit of the monotonic curve corresponds to forces in the tension flange which produce yieding. When reversed, the negative elastic moment produces compression forces in the displaced flange capable of yielding and thus sufficient to close the flange against the column face.

If the negative moment is relieved, the connection would again unload elastically to zero moment with residual deflection in the bottom flange. With the next positive loading, the situation is again a case of moment reversal. Now, however, the elastic limit of the connection in positive moment is the maximum point of unloading from the positive monotonic curve.

This rationale of observable experimental behavior forms the basis for the development of the model for semi-rigid connection response to general loading. A somewhat similar description of experimental behavior by Azizinamini, et. al. at the University of South Carolina (14) includes some high cycle fatique effects which will be discussed in a later section.

3.2 BASIC MODEL

The basic model, without fatigue effects, for the general behavior of a standard semi-connection configuration consisting of flange and web angles is an empirical model based upon observable experimental behavior and the rationale of this behavior, in terms of connection deformation, was outlined in the previous section. The model can be simply stated as follows: The moment-rotation path of the connection linearly intersects the monotonic curve at the elastic limit in that direction. After intersection, the load path is along the monotonic curve.

The connection unloads elastically with stiffness equal to the initial elastic stiffness of the monotonic curve.

Thus, the basic model behavior is equivalent to the previously outlined connection behavior except the transitional paths have been linearized. The elastic limit is either the initial monotonic elastic limit or the point of maximum excursion on the monotonic curve for that loading direction. This basic model for the general loading behavior is defined entirely by the connection's monotonic moment-rotation curve in both loading directions.

To demonstrate the model's application, consider a symmetric connection with a monotonic moment-rotation curve shown in Figure 45a. If loading occurs in a direction defined as positive, the connection loads along the monotonic curve to some point M1+ (Figure 45b). If the moment is removed, the connection unloads with the initial connection stiffness, Ke, to some residual rotation, R1+. The connection is now elastic for positve moment loading up to M1+.

If the connection is reloaded with a positive moment it will load from R1+ to the positive elastic limit, which is now M1+, and with further moment will load along the monotonic curve to some point M2+ (Figure 45c).

Once again, if the moment is removed, the connection unloads elastically with stiffness Ke to a new residual

rotation, R2+ (Figure 45d). The connection is now elastic for positive moment loading up to M2+.

If negative moment is applied, the linear loading path is from the residual rotation, R2+, to the initial negative elastic limit, $-Me_1$, then along the negative monotonic curve to some point M1- (Figure 45e).

Removal of the negative moment produces elastic unloading with stiffness Ke to a residual rotation, R1- (Figure 45f). The connection is now elastic for negative loading to M1-.

If a positive moment is applied once again, the loading path will be linear from R1- towards the positive elastic limit, M2+ (Figure 45g). If, however, the positve moment is removed prior to M2+ at some intermediate moment, M3+, the connection will again unload elastically, although the positive elastic limit remains M2+.

Now, if the moment is cycled from M3+ to M1-, the connection moment-rotation hysteresis is defined by the positive and negative elastic limits, points M3+ and M1-, together with the elastic stiffness, Ke, and the residual rotations R3+ and R1- (Figure 45h).

3.3 BASIC MODEL GENERATION OF EXPERIMENTAL RESULTS

The basic model can be applied to the test loadings reported previously to generate comparative results with the experimental data.

3.3.1 Model Generation of Test 2 Results Using Test 4 Monotonic Connection Curve

The model generated results for Test 2 uses the monotonic curve obtained in Test 4. This curve is reproduced in Figure 46 with the extreme ranges projected as the final tangent.

The model connection curves for the Test 2 loadings are shown in Figure 47, beginning with load cycle 3. The first two cycles would be straight lines along the initial loading path shown as dashed lines. Although they are linear approximations of much of the connection behavior, they agree quite well with the experimental results which were shown in Figure 24.

Superimposing the results (not shown here), shows that the model generated curves are bounds of the actual test curves. The model and actual results do deviate at some points. First, and most notably, although the model generated curves are quite close to the actual curves at the loading peaks, both in location and stiffness, the linear approximations of the unloading and transitional loading regions do not show the double curvature nature of the actual connection curves. As well, the models residual rotations at zero moment tend to be larger than the actual data.

The individual connection curves generated by the model can be used to generate joint moment-rotation response to lateral loading. The model generated joint moment-rotation

hystereses for Test 2 are shown in Figure 48. Again, these results compare very well with the actual data that was shown in Figure 23. The model generated joint hystereses, as with the model connections moment-rotation curves, produce reasonably good behavioral bounds and general trend reproduction. The locations of peaks and near peak stiffnesses coincide well. The double curvature of the lower ranges, however, are not reproduced by the linear paths of the model and correspondingly, the residual rotations and widths of the hystereses are somewhat exaggerated.

3.3.2 Model Generation of Test 3 Results Using Test 4 Monotonic Connection Curve

Again using the monotonic curve from the results of Test 4, the model can be applied to generate results of the Test 3 loadings. The effect of the difference between the initial peak connection loading curve of Test 3 and the monotonic curve of Test 4 (Figure 29) on the model generated results becomes apparent in the difference between the model generated connection curves, shown in Figure 49, and the actual data, shown in Figure 26. The softer monotonic curve used in the model results provides for significantly greater initial rotations. This also has the effect of imposing greater residual rotations from positve unloadings which, in turn, produce much less negative connection moment stiffness than the actual results. Thus, while many of the general

trends of the data are reproduced in the model, they are exaggerated.

These same comments are true when comparing the generated joint results, shown in Figure 50, with the actual results, shown in Figure 25. That is, while much of the general behavior is well represented by the model generated results, there is some exaggeration of the hystereses widths. While some of this effect is due to the linear approximations of connection behavior, much is due to the relative softness of the monotonic curve used in the model with respect to the peak curve of Test 3.

3.3.3 Model Generation of Test 3 Results Using Test 3 Initial Loading Peaks Connection Curve

If, instead of using the monotonic curve of Test 4, the initial peak loading connection curve of Test 3 is used as the monotonic curve for the model (See Figure 29), then the model generated results for the connection curves (Figure 51) better approximate the actual curves (Figure 26). This was rationalized in the discussion of the test results by saying, in effect, that the peak curves and the monotonic curve of Test 4 deviate because of connection slip and that in larger beams this deviation would not be as significant.

While these model generated results are closer to the actual results than those given previously, they still only approximate the actual behavior in the same manner as discussed for the Test 2 model results. Additionally, the

effect of the models softer approximation of the behavior near zero moment and moment reversals is especially noticable in these connection curves as well as generating wider and more skewed joint hystereses (Figure 52).

3.3.4 Model Generation of Test 5 Gravity Moment Shakedown Using Test 4 Monotonic Connection Curve

Using the basic model with the Test 4 monotonic curve, the gravity moment "shakedown" of the Test 5 loadings is generated in Figure 53. While the gravity moment has not yet "shakendown" to zero, the model results do show good agreement with the experimental trends.

3.3.5 Model Generation of Test 5 Gravity Moment Shakedown Using Test 5 Initial Peaks Connection Curve

The gravity moment "shakedown" of the Test 5 loadings applied to the basic model using the Test 5 peak loading curve (see Figure 32) is shown in Figure 54. The generated results do not differ significantly from those above, indicating that the model and/or these particular loadings and results are not so sensitive to the exact profile of the monotonic curve.

3.4 DISCUSSION AND LIMITITATIONS OF BASIC MODEL

Comparison of the model generated results and the actual results shows that while the model does approximate the general trends and features of the actual behavior, it has the tendency to exaggerate them. The major characteristics of the relationship between the actual and

modeled behavior can be highlighted in terms of particular model assumptions of the various regions of the load paths or stages of loading.

The most fundamental component of the model is the connection's monotonic moment-rotation curve. The model is sensitive to different curves, especially in the regions of maximum curvature or maximum stiffness changes. Because, as discussed earlier, the model's bounding hysteresis is basically defined by the points of maximum excursion on the "monotonic" curve used and its initial stiffness, the rest of the model's approximating behavior is also sensitive to the features of this curve.

It seems then, that much of the model's accuracy in predicting connection behavior, is dependent upon the assertion that the initial loading peak curves (Figures 28, 29, and 32) are equivalent to the monotonic curves of that particular test connection. Thus, the earlier assumption to this effect, that the deviation of these curves with the monotonic curve of Test 4 was due to the effect of bolt slippage, which is magnified by the small beam depth of the test assembly, becomes a more important point of possible contention regarding the veracity of this basic model assumption. However, even with the use of the monotonic connection curve of Test 4, the model does represent to some degree, the general behavior of the connection test data.

A feature of the data which is not represented at all by the basic model is the slight "walking" or cyclic increases of the moment controlled rotations of the load cycle peaks, Although not especially large or significant in the test data reported here, it will be incorporated into model adjustments presented in the next section which considers other test data.

The next major feature of the model is that, with the exception of loading along the monotonic curve, the various loading and unloading paths are linear. While this approximation is fairly accurate of the lower moment cycles and the more extreme moment regions of the higher moment cycles, the actual behavior near the zero moment regions of both the unloadings and moment reversals is somewhat different. The result is that the model tends to overestimate the rotations or rotation changes in these regions. If these regions correspond to the extremes of the connection load cycles, as in much of Test 3, the result is wider and more skewed net joint moment-rotation hystereses.

In spite of these apparent short-comings, there is much positive to be said of this basic model. First is that the model is so basic. Though there are deviations from the linear paths used, the fact remains that they are easily applied to the monotonic curve to generate both individual connection and combined joint modelling for any loadings. Additionally, the need for only the connection's monotonic

curve takes advantage of the extensive effort on analytical methods of producing these curves. Finally, the model does reproduce much of the connection's general behavior, even though some of the behavior tends to be exaggerated.

If the effects of the model's deviations from actual behavior are examined, it may be that they are not entirely undesirable, at least in the sense of their deviations. As mentioned, the model tends to produce bounds of the connections actual behavior. These deviations are least at the lower moment cycles. Therefore, the model seems to be representative of connection behavior at lower working load levels.

At higher load cycles, the model produces fairly good approximations of stiffness and rotations near the extreme peaks, the areas which would seem more critical. If there is deviation, it tends to be an overestimation of rotations and a corresponding underestimation of stiffness, a conservative deviation for stability and servicablitity requirements.

Thus, this basic model seems to provide some degree of representation of general connection behavior. Its tendency to oversimplify and exaggerate certain behavior is, for the most part, conservative. An exception to this is in the case of using this model to calculate hysteresis areas, where the linear simplifications do not produce very accurate estimates.

3.5 MODEL WITH DEGENERATIVE EFFECTS

The fact that the moment-rotation path of the semi-rigid connection does change somewhat during constant magnitude load cycling, as was mentioned above, indicates that the connection behavior is subject to fatigue effects. A study conducted at the University of South Carolina of similar semi-rigid connection configurations subjected to static and cyclic loads considered some of these fatigue effects (14). Included in the conclusions was that in cyclic loading the connections exhibit ductile behavior with the eventual establishment of generally stable moment-rotation hysteresis loops. The cyclic tests were displacement controlled and incrementally cycled through ten reversals after establishment of a stable hysteresis loop. This generally resulted in a total of 12 to 15 cycles at each displacement amplitude.

Since both static (monotonic) and cyclic tests were performed on many of the connection configurations, the first comparison to be made between the model assumptions and the South Carolina data (14) is the coincidence of cyclic moment peaks and the monotonic curves. Figure 55 shows the monotonic curves of the static tests compared with the average moment peaks or final peaks from the corresponding equivalent cyclic test specimen. The codes used are those of the original authors and are such that the numerical prefix denotes the beam depth, the S or C refers to static or

cyclic, and the numerical suffix is a specimen and test designation.

Examination of Figure 55 shows that while the points of the static and cyclic test peaks are not always entirely coincident, they are generally consistent in shape and, though less so, in magnitude. It is uncertain how much of the deviation is due to statistical variance. Certain features relating to expected fatigue effects can be seen from these figures, however. In the figures where more than the final peaks are shown, the trend at each successive displacement peak is towards less moment. This feature is consistent with the general increase in the displacement of the moment controlled tests reported earlier. Figure 55 also seems to indicate that the higher cycling produces less moment capacity and stiffness at the higher moment ranges.

Thus, it can be concluded that using the monotonic curve in the model assumption is again reasonable although some adjustment near the higher moment range could be made. In examining the cyclic peak profiles and the monotonic curves (Figure 55), two possible means of adjustment were considered, though both with somewhat similar results. The cyclic peak moment-rotation profiles seem to flatten out at a point somewhat coincident with the incidence of the constant linear stiffness which occurs in the upper regions of most semi-rigid connection monotonic curves. In these connection configurations, it is assumed that much of this

upper linear stiffness is the result of plate diaphragm action in the much deformed connection components. Generally nearby on the monotonic curve and just as good an approximation of the flattening of the cyclic peak curve in these results, is the moment on the monotonic curve corresponding to the intersection of the initial and final linear stiffnesses. This is the method that will be used here to adjust the monontic curve of the model to include degenerative effects and which will be used to generate comparitve model results with the South Carolina data. While a rationilzation for this adjustment and method is not being offered here, the adjustment does seem consistent with the notion of degenerative effects. The adjusted monotonic curves of the South Carolina test results for degenerative model application of tests 14C1-3 and 8C1 and 2 are shown in Figures 56 through 60.

By applying the basic model to the adjusted monotonic curves and applied test deflections, comparative model results can be generated. The stable hystereses of the test results and model generated results for Test 14C1 are shown in Figure 61. While there is representation of the test results at the peaks, one feature that is evident in comparing the test and model results in Figure 61 is the deviation of the unloading stiffnesses and residual rotations. The deviations here being even more pronounced than in the basic model application to the tests reported

earlier. As well, the test results indicate that linear unloading stiffness approximations decrease in some proportion to the magnitude of displacements and moments. This, too, does not seem inconsistent with expected fatigue effects.

In an attempt to represent the degrading unloading stiffness mentioned above, a method is proposed where the unloading stiffness is reduced by reducing the residual deflection produced by the initial elastic unloading from the truncated monotonic curve. An arbitrary elastic residual rotation reduction factor of 1/3 applied to the model seems to produce results which best balance the need for representation of most of the experimental behavior. Model generated results using this unloading stiffness reduction method with a factor of 1/3 are shown along with test results in Figures 62 through 66 for Tests 14C1-3 and 8C1-2.

3.6 MODEL GENERATED RESULTS

Comparing the degenerative model results and test data in Figures 62 through 66 for Tests 14C1-3 and 8C1-2 indicates that the model does represent much of the general behavior of these connections. There are still discrepancies which result from the model and its linear approximations. Most of the deviation between the model and test results is still of the stiffness near zero moment and overestimation of the residual rotations.

Some of the coincidence between model and test results is, however, masked by the nonsymmetry of the South Carolina test results. A possible explanation of this nonsymmetry is offered by the authors of these results as being due to the method in which the connections were assembled. They suggest that initial alignment and tightening of one flange angle at the expense of force fitting the other may have produced this nonsymmetrical behavior. This is, though, still the standard field erection procedure and its result should not be entirely disregarded since it represents behavior which could be expected.

The representation of actual behavior at and near the extreme peaks is much better, both in terms of stiffness and peak location. As can be seen in comparing these figures, the adjusted model does represent much of the general connection behavior.

3.7 DISCUSSION AND LIMITATIONS OF DEGENERATIVE MODEL

Much of what was discussed previously regarding the performance of the basic model can also be said of the degenerative model. The model does seem to represent much of the actual connection behavior considering its simplicity. Obviously, better behavior representation could be obtained by including more linear segments, especially at the transitional region around zero moment. Again, however, this seems less warranted when one considers that the representation of this model near peaks, seemingly

more critical points, is fairly accurate. As well, where the model does deviate, these deviations would appear conservative for the same reasons outlined in the discussion of the basic model. Also, though, the model does not provide for accurate estimation of the hysteresis area.

Though seemingly consistent with some of the expected effects, the adjustments made in the basic model to include these effects are empirical and somewhat arbitrary. The truncation of the monotonic moment rotation curve, though of no real consequence in applying deflection control test values, would present a problem in applying moment control. This feature does provide for some sort of plastic moment representation, however.

The method of reducing unloading stiffness by reducing the residual rotation of initial elastic unloading by some factor is, again, empirically based. The selection of a 1/3 reduction factor represents a value which best seemed to produce representative behavior from this simple model at extreme peaks and residual rotations and is arbitrary.

It will be assumed here that unloading from any point less than the elastic limit will occur with equal stiffness. That is, the unloading stiffness from any point is always determined by the elastic limit in the same direction. Thus, all positive (or negative) unloading stiffness is equal to the unloading stiffness from the maximum positve (or negative) moment (Figure 67).

An issue to be considered is under what circumstances are the adjustments of the degenerative model applicable. These adjustments are empirically based upon pure cyclic tests with several cycles of full moment reversal. It seems possible that much of the degrading properties result from this type of loading. It is not likely that this type of loading is representative of standard building semi-rigid beam to column connection loading. However, the degenerative model does represent more conservative stiffness properties. Thus, though more conservative, the degree to which these cyclic adjustments in the connection model represent the actual behavior in typical loadings is uncertain and as such, the increased complexity of the model because of these adjustments may or may not be justifed. Also, it must be emphasized that these adjustments are based upon a relatively few number of cycles and are not considered to account for all possible cyclic effects.

4. COMPUTER ANALYSIS OF PORTAL FRAME USING SEMI-RIGID CONNECTION MODEL

In keeping with the justification of developing a model for standard configuration semi-rigid connections under general loading conditions, the connection model developed can now be used in a structural analysis of a simple pinned-base column frame under the effect of constant gravity and various wind loadings. Using results like these, the design and analysis assumptions of these types of connections can be assessed.

4.1 DESIGN FRAME AND CONNECTIONS

For this analysis, the frame design was based upon the use of a specific connection along with frame span and height. The design frame is shown in Figure 68. The connection chosen was one tested by the South Carolina group (14), 14S and C2, and modelled in the previous section. It is shown in Figure 69. This connection was selected because of the availability of test data including its monotonic curve. There are analytical methods to generate the connection monotonic curve (for example Reference 5). The initial elastic stiffness of the connection, Ke_i in Figure 68, is based upon the experimental data. The beam and column selection is somewhat arbitrary but thought to be reasonable.

Given the spans, members, and connections, the allowable loadings could be determined. The beam has an

allowable pinned-end load capacity of 1.167 kips/ft determined by allowable stress design assuming adequate lateral bracing.

The connection and frame capacities were determined by the "Flexible Wind" design method. The connection components are separated and the shear web angles are found to have a shear capacity of 42.5 kips. This value is much higher than that required by the distributed gravity loading determined by the simple beam capacity. However, this effect will not be considered here. The moment capacity of the connection is determined by the allowable stress design of hangar type connections and has an allowable moment capacity of 14 kip-ft or 168 kip-in including a 1/3 increase for wind stress.

The corresponding allowable lateral load at the beam height is then determined by finding the lateral load which, along with axial load-lateral deflection effects, produces a fixed-frame moment of 14 kip-ft in the now assumed fixed connections. The effect of the gravity loading, other than providing a column axial load, is not considered. The allowable lateral load is determined to be 2.0 kips.

Because the column selection was arbitrary, the columns have additional axial load capacity. Using the method developed by DeFalco and Marino (11), the allowable axial load capacity of the columns in this frame is 80 kips, or an additional 66 kips in each column. This capacity uses

design values, as determined according to Reference 11, of C_m=0.85, G_a=1.164, G_b=infinity (for consistency with computer matrix analysis), and $Z=1/Ke_i$ from data (14) and a resulting effective column length factor of K=2.4.

4.2 LOADING

Using the values of allowable gravity and lateral loads, representative load factors of 1.7 for gravity and 1.3 for lateral are applied to obtain analysis load values of

Gravity Load = 2.0 kips/ft

Lateral Load = 2.6 kips.

Two methods of loading will be applied and are shown diagrammatically in Figure 70. In both loading cases, the gravity load is applied in its entirety first, before any lateral loading is applied. The two cases represent different lateral loading schemes. One case is were the lateral load represents an increasing direct lateral load. In the other case, the lateral load is cyclically increased. Both loading cases are applied until frame failure. The cyclic loads are not dynamic.

4.3 COMPUTER ANALYSIS PROGRAM

A computer program was written to perform the analysis of this frame and loading including the connection performance. A listing of this program is given in the appendix. The program obtains the balanced member end actions and displacements of the pinned-base portal frame with semi-rigid connections whose behavior is represented by

the degenerative model developed previously, under the effect of gravity and lateral loadings. The program uses an Newton-Raphson linear tangent stiffness technique for incremental loadings to incorporate the connection nonlinearities and the moment magnification approximation to the axial-column deflection (axial P-Delta) effect. The loading continues until an incremental load application does not produce balanced member-end-action convergence after 10 iterations.

The program uses a subroutine to apply the degenerative connection model to the connection response of the loadings. The truncated monotonic curve from the South Carolina data (11) is shown in Figure 71. Other model parameters, also determined from the experimental data are

Initial Elastic Stiffness,	Ke_i=3.67E05 kip-in/rad
Elastic Limit,	Me=242 kip-in
Plastic Moment,	Mp=720 kip-in

4.4 ANALYSIS RESULTS

The lateral load vs. lateral frame deflection for the cyclic loading peaks is shown in Figure 72. The frame failed after reaching a lateral load of -5.72 kips for the first time. This is 220% greater than the factored lateral load, almost 290% greater than the design lateral load, 2 kips. However, deflection greatly increases after surpassing the factored design lateral load of 2.6 kips. The loading was done with lateral load increments, Pinc, of

0.052 kips, and cyclic peak increments, Padd, of 0.520 kips. At the design load, the frame drift (lateral deflection over story height) is 0.0033, at the factored load it is .0045.

The connection moment-rotation behavior is given in Figure 73. The initial gravity moment in the connection is 583 kip-in or 48.5 kip-ft, 350% greater than the design moment capacity of the connection. Another feature from this analysis is that the rotation in the connection windward to the direction of first lateral loading (the left connection in Figure 68) continually increases and stiffness decreases, while the leeward connection remains relatively stiff and elastic.

The gravity moment shakedown for this case, as approximated by the average moment at each end of the beam, is shown in Figure 74. The figure gives the average beam end moment vs. the applied lateral load, P. At the design lateral load the average end moment is 93% of the initial gravity moment, 583 kip-in, at the factored lateral load it is 90%.

Figure 75 shows the "Flexible Wind" design moment distribution at design loads after "shakedown" has occurred. The corresponding moment distribution from the analysis at design loads is shown in Figure 76. The design and analysis moment distributions after shakedown of the factored loads are shown in Figures 77 and 78 respectively. These figures show that actual and design load distributions can vary

considerably, underestimating negative gravity moments at the beam ends or unbalanced gravity moments into the columns.

Figure 79 shows the lateral load vs. lateral frame deflection for the same beam and cyclic loading with the additional allowable axial column loads of 66 kips. The frame failed to converge after a lateral load of 4.29 kips, 165% greater than the factored lateral load, 2.6 kips, and 215% greater than the design lateral load, 2.0 kips. The lateral drift at design is 0.004 and 0.009 at the factored lateral load.

Also shown in Figure 79 is the lateral load vs. lateral frame deflection for direct (i.e. monotonic) wind loading. Under direct loading, the frame stiffness was diminished but its failing lateral load was significantly higher, 8.45 kips, 425% and 325% greater than the design and factored lateral loads, respectively.

The lateral load vs. lateral frame deflections for the three common connection stiffness analysis approximations are shown in Figure 80 superimposed on the values in Figure 79. The three cases represent three conditions of connection stiffness in response to lateral loads; the fixed-end (infinite connection stiffness) and the initial elastic stiffness for both connections (Ei) suggested in the AISI "Flexible Wind" connection design method (Reference 9), and the one-half (1/2) initial connection stiffness for both

connections (Ei/2) approximating the elastic-hinge end conditions implied in Disque's "Directional Moment" connection method (Reference 10). The connection stiffness value of Ei/2 gives the best approximation through low lateral load values up to even the factored lateral load. The other stiffness approximations are somewhat unconservative for low lateral loads and considerably more so at higher loads.

Figure 81 shows the peaks of the lateral load vs. lateral frame deflection for varying beam moments of inertia. The design beam, W14x30, has a moment of inertia of 291 inches^4. The initial connection gravity moments are also given.

4.5 ANALYSIS DISCUSSION AND CONCLUSIONS

The results of the analysis are dependent on the veracity of the connection model. This is especially true at the higher load cycles and direct loads where the truncation of the monotonic curve and/or the unloading stiffness decay features of the model become more significant factors in the frame response.

From the analyses presented here, the "Flexible Wind" connection design method seems to produce a structural system capable of withstanding the required loads, although the design assumptions are not well represented. The conclusions from this particular frame analysis are as follows:

4.5.1 Servicability

In so far as calculating lateral deflections or drift due to wind loads only, treating semi-rigid connections as fully fixed is clearly unconservative at all but the lowest load values. Using the initial elastic stiffness of the connections is representative at low load levels but becomes unconservative at higher loads. Accounting for only the windward connection stiffness, or here, approximating this condition by making each connection stiffness one-half the initial elastic connection stiffness best represents the lateral behavior, even at factored loads. However, this approximation can also become unconservative at higher loads.

4.5.2 Practical Considerations

The design method seems to produce acceptably conservative systems. However, the assumed design load distribution is not well representeted, especially in the case of initial gravity moments which can be much higher than even the design moment capacity of the connection. The method becomes less conservative as the beam becomes more flexible. In the case of very stiff beams, however, it can be excessively conservative. It must be reiterated that fatigue or dynamic effects may greatly change behavior.

4.5.3 Gravity and Wind Moment Separation

The design concept of separation of gravity and wind moment is not well represented by the actual behavior or load

distribution. The existence of gravity moment does reduce the wind failure load, not only because of buckling but because of connection behavior and deterioration, as well. The initial gravity moment conditions may be much greater than even the design moment capacity of the connection. Additionally, these results are very dependent upon the relative stiffness of the connecting members, especially the beam. Unless the beam is very flexible, the "Flexible Wind" method seems to produce an acceptably conservative structural system. In some cases, a very conservative system. However, high axial loads in the columns and secondary P-Delta effects may significantly change results.

4.5.4 Using the Benefits of Continuity

The design assumption of "shakedown" of the beam end negative gravity moments reduces the required unbalanced gravity moment capacity of the columns but increases the required midspan capacity of the beam. This analysis shows that "shakedown" of the gravity moment is not occurring to a degree which is compatible with these design assumptions. This indicates that the required beam capacity could be reduced to account for the gravity restraining moments at its ends while the columns, however, may need increased capacity for unbalanced gravity moment transfer.

5. DISCUSSION AND CONCLUSIONS

A simple model has been developed for the general moment-rotation behavior of a standard semi-rigid connection configuration consisting of web and flange angles. These connnections are widely used and represent an inexpensive connecting system. These connections also have the potential to represent a new structural performance option in their capacity to control the amount of moment transfer to connecting members.

It is believed that such a model is needed to develop new and more rational design and analysis provisions of these connections by conducting analyses like those presented here. Current methods of design exist (9) which place these connections under AISC Type 2 provisions (1) and seem to produce acceptably conservative systems in most cases. However, the implied design assumptions do not represent actual conditions because of unrealistic load separations and neglect of load and stiffness interaction. A design method may very well be possible which could tune the connection performance to optimize distribution of moments to the beams and columns and incorporate servicability provisions while allowing for

a) the separation of wind and gravity moments in the analysis procedure
b) the use of continuity
c) a conservative design.

Further work is needed to develop more rational design and analysis provisions for these connections. Because of the dependency of accurate connection modelling in any analysis, more experimental work of connections and analytical refinements are needed to make the characteristics of these connections readily accessible and representative. As well, work is needed in determining the effect of these connections on various structural systems and loadings. Because of their complexity, additional study is also needed in determining the economic consequences of their use.

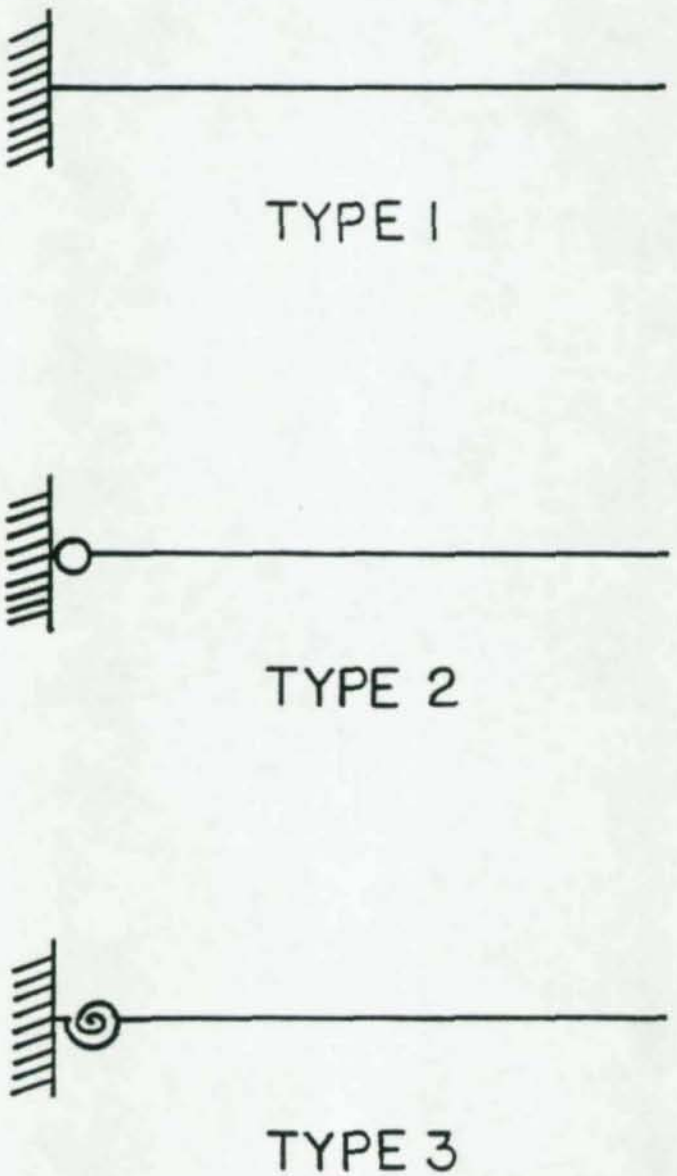


Figure 1. Diagrammatical Representation of AISC Connection Types.

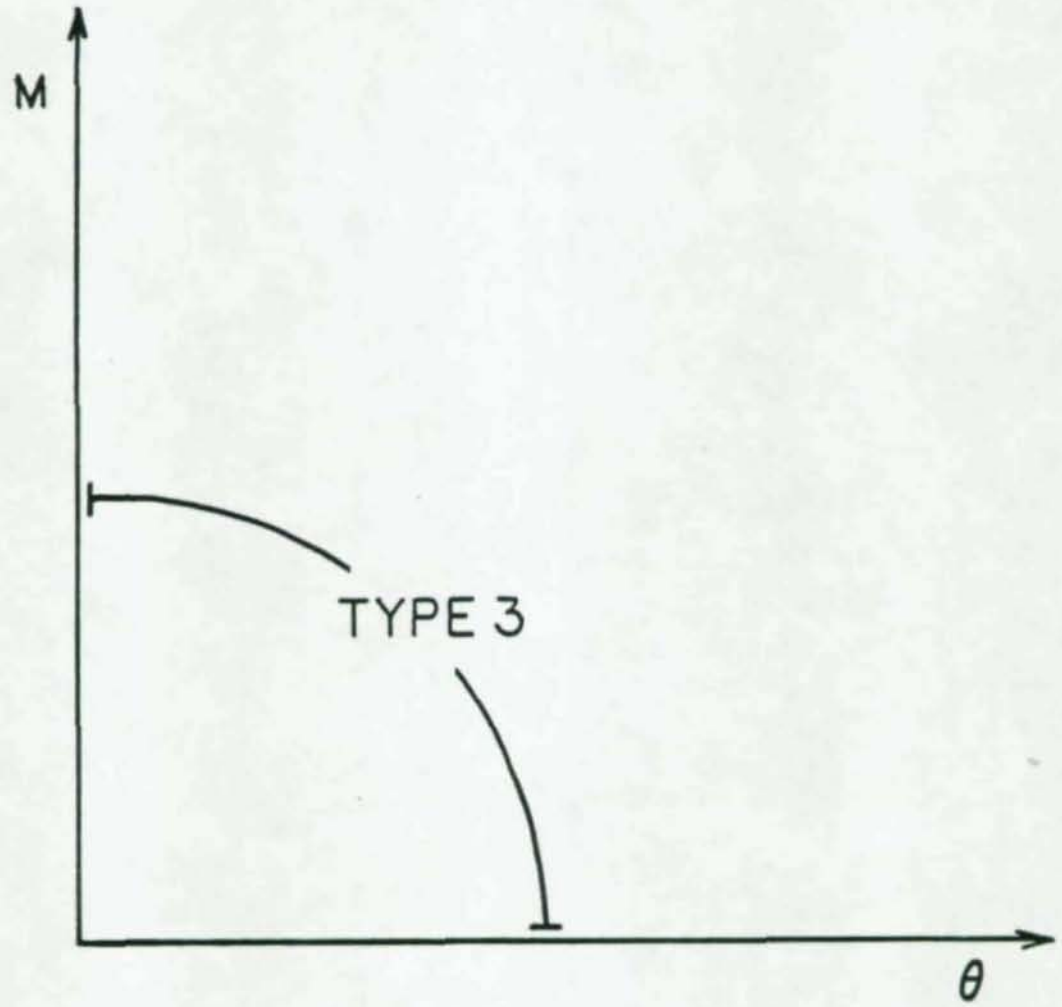


Figure 2. Moment vs. Rotation of Idealized Connection Types.

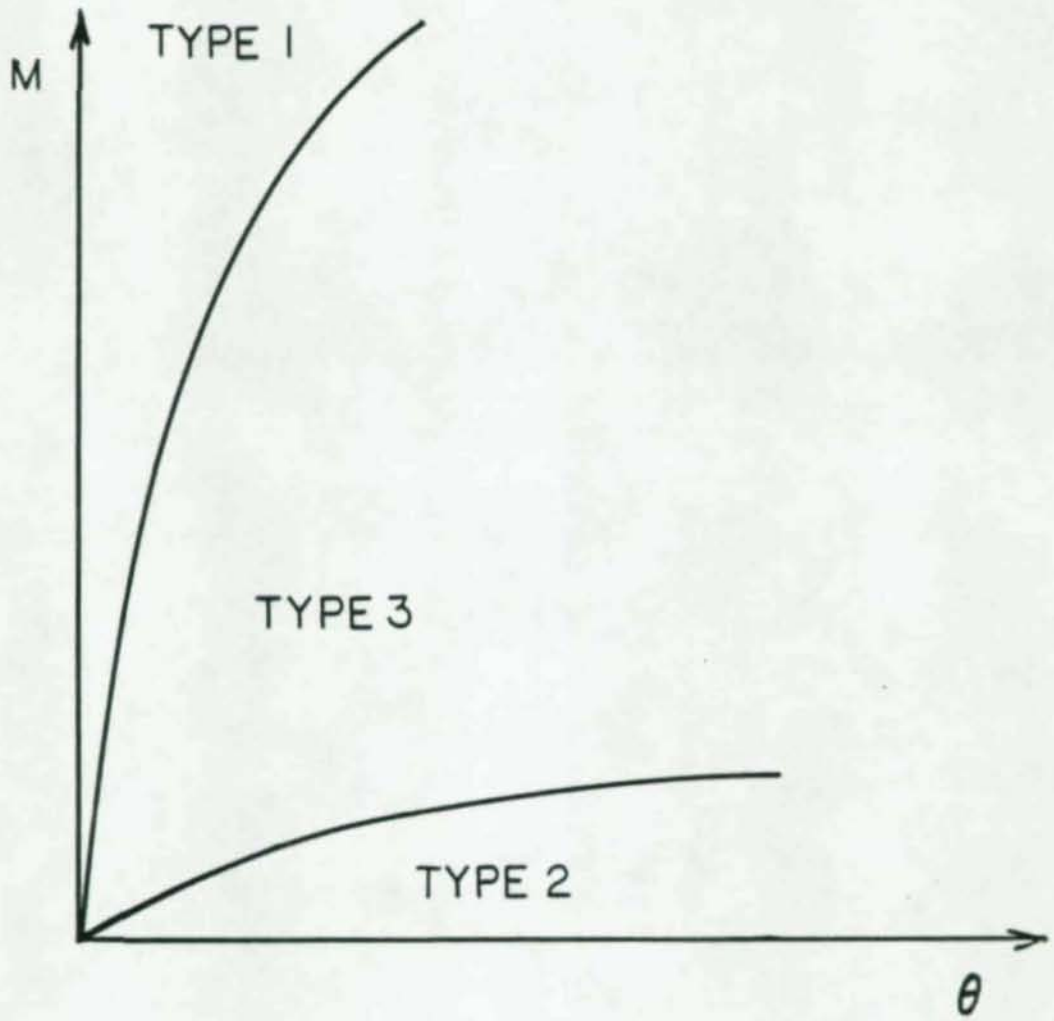


Figure 3. Moment vs. Rotation of Actual Connection Types.

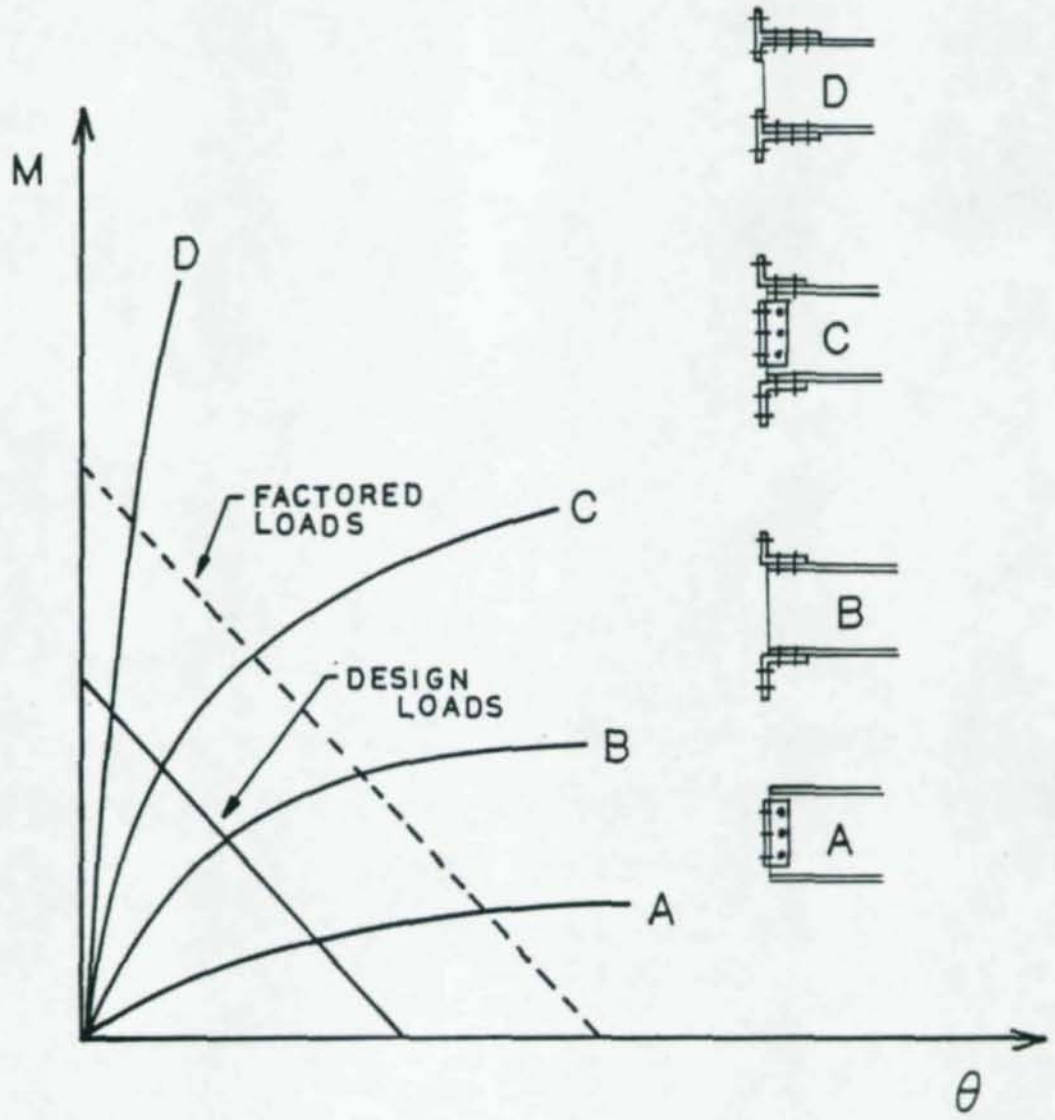


Figure 4. Relative Moment vs. Rotation Relationships of Typical Steel Beam-Column Connections.

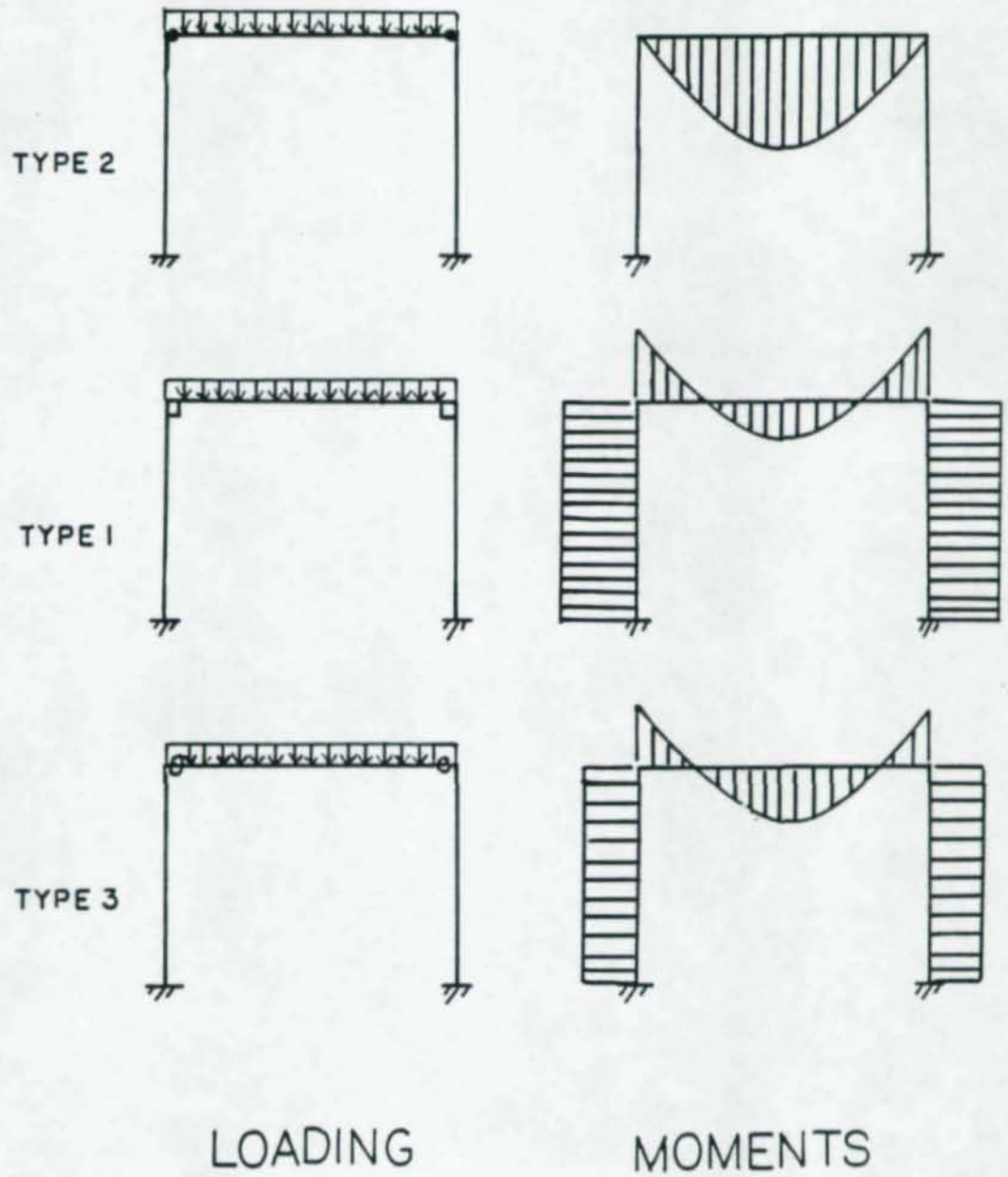


Figure 5. Portal Frame Moment Distributions with Different Connection Types.

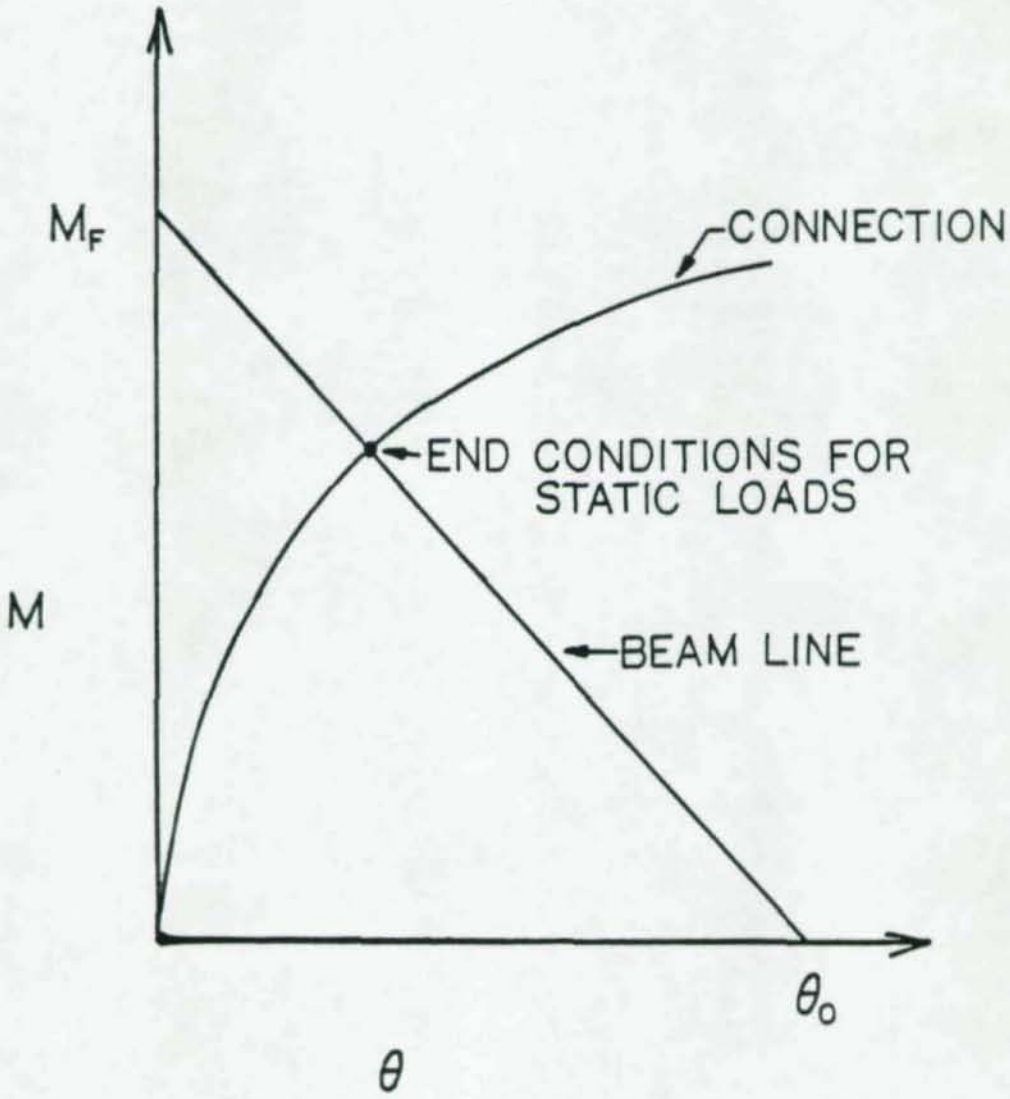


Figure 6. Beam-Line Interaction Diagram.

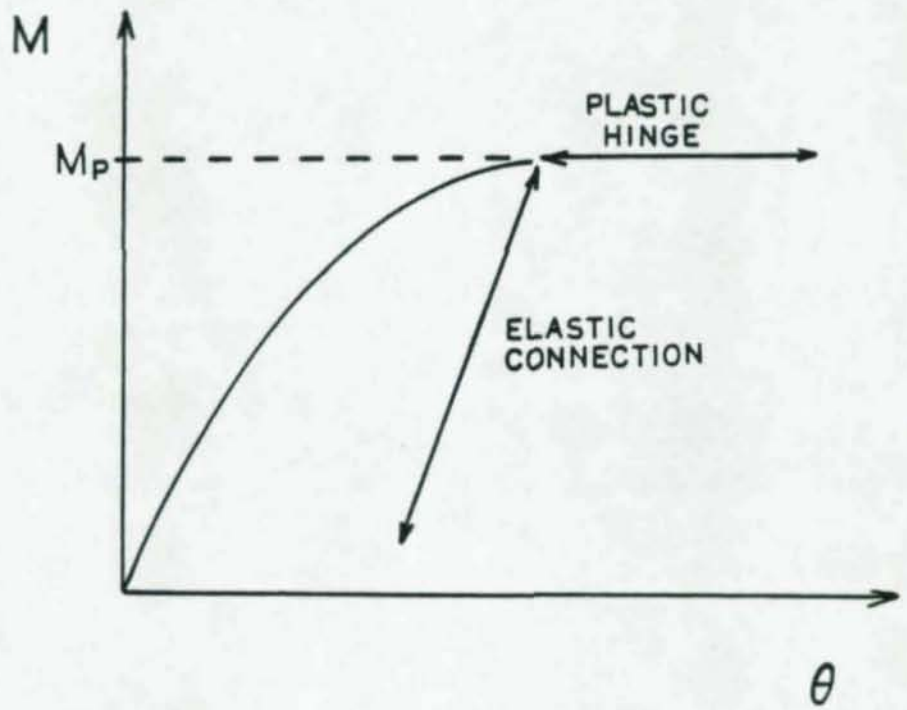


Figure 7. Loading & Unloading Model for "Directional Moment Connection" (from Reference 10).

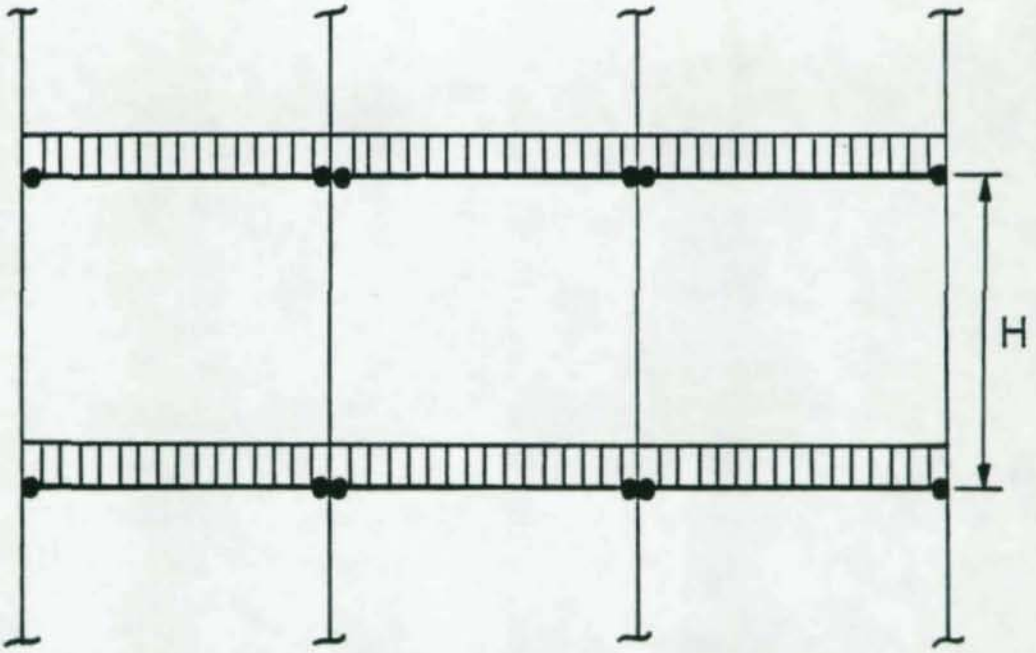


Figure 8. Gravity Loading with Plastic Hinges for "Directional Moment Connection" (from Reference 10).

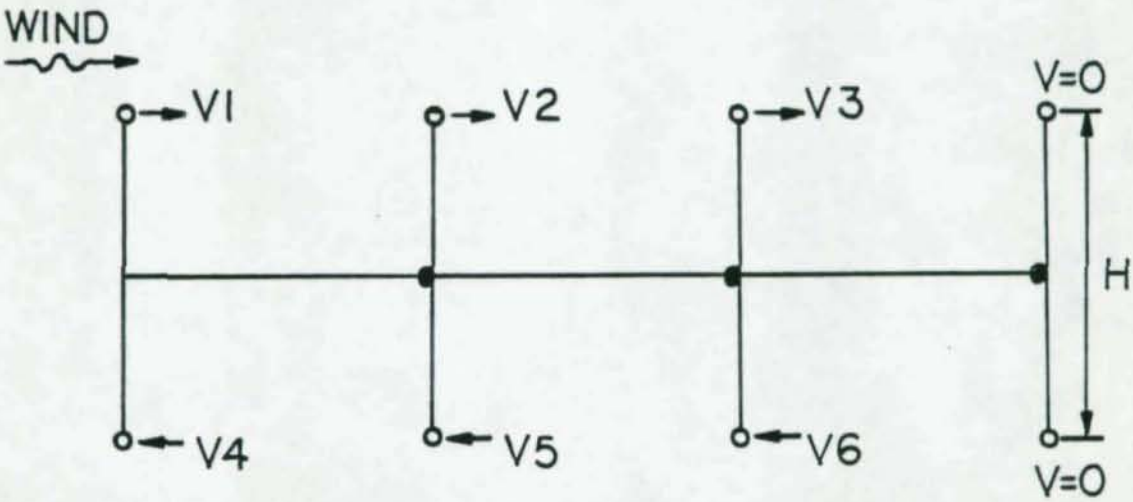


Figure 9. Wind Loading with Plastic Hinges for "Directional Moment Connection" (from Reference 10).

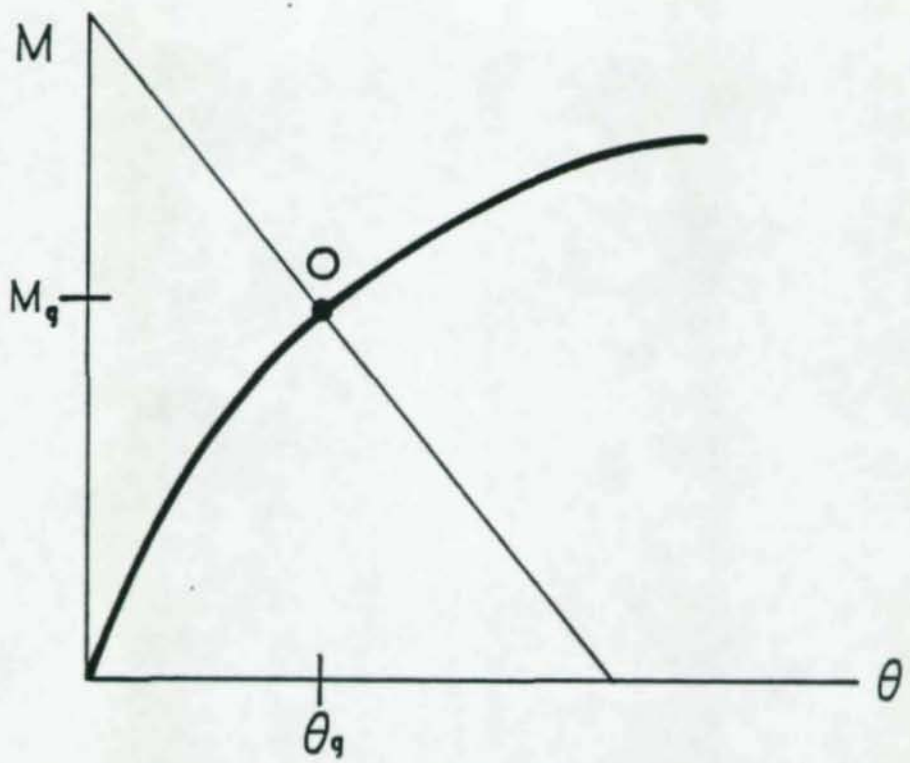


Figure 10. Semi-Rigid Connection End Conditions Under Initial Gravity Loading Conditions (from Reference 9).

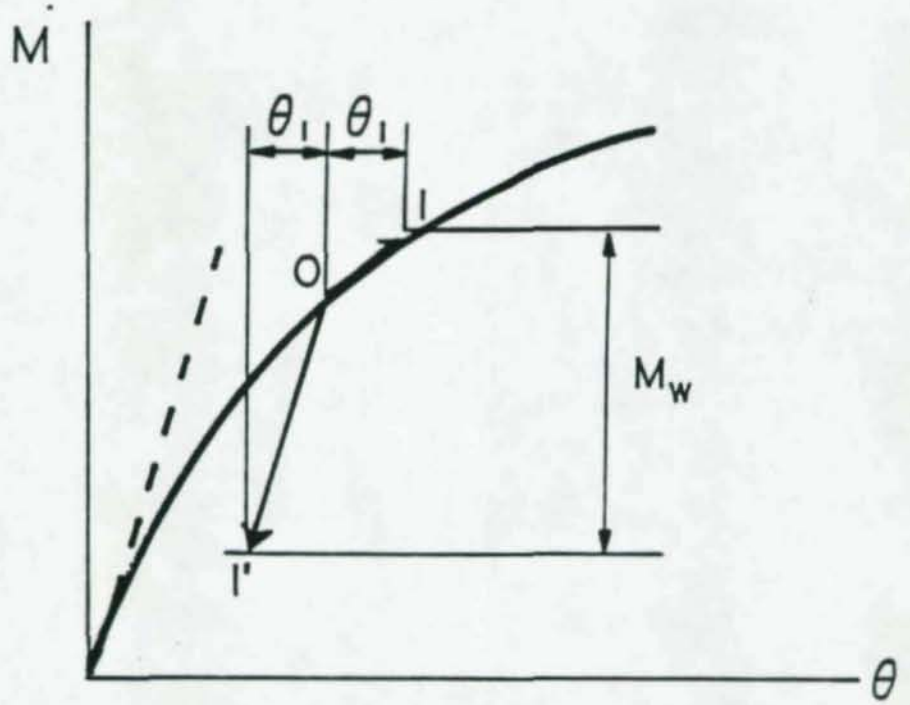


Figure 11. Semi-Rigid Connection End Conditions: First Wind Load in Positive Direction (from Reference 9).

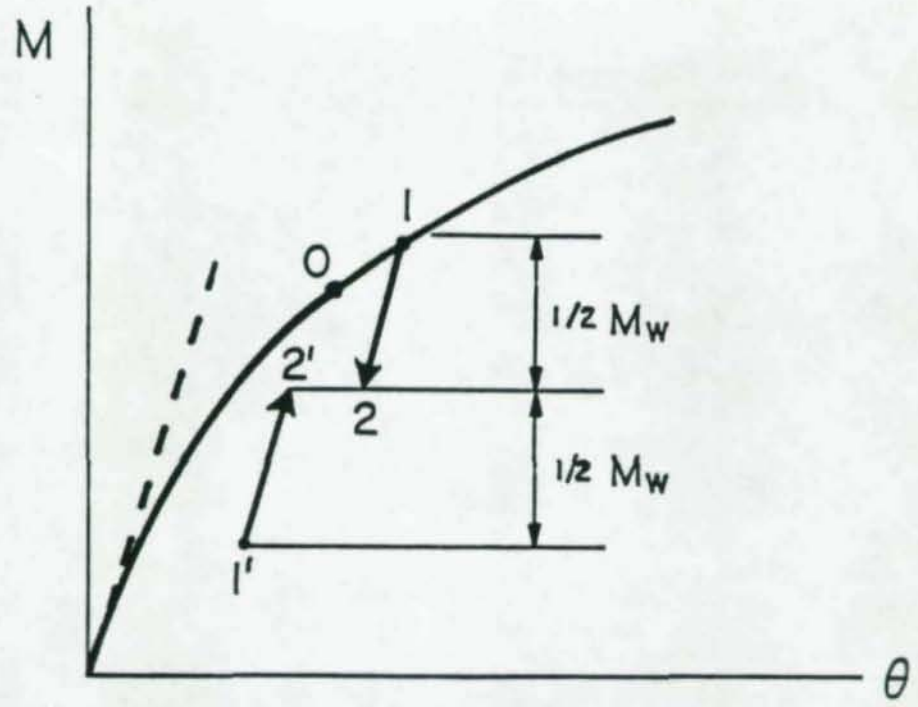


Figure 12. Semi-Rigid Connection End Conditions: First Wind Unloading (from Reference 9).

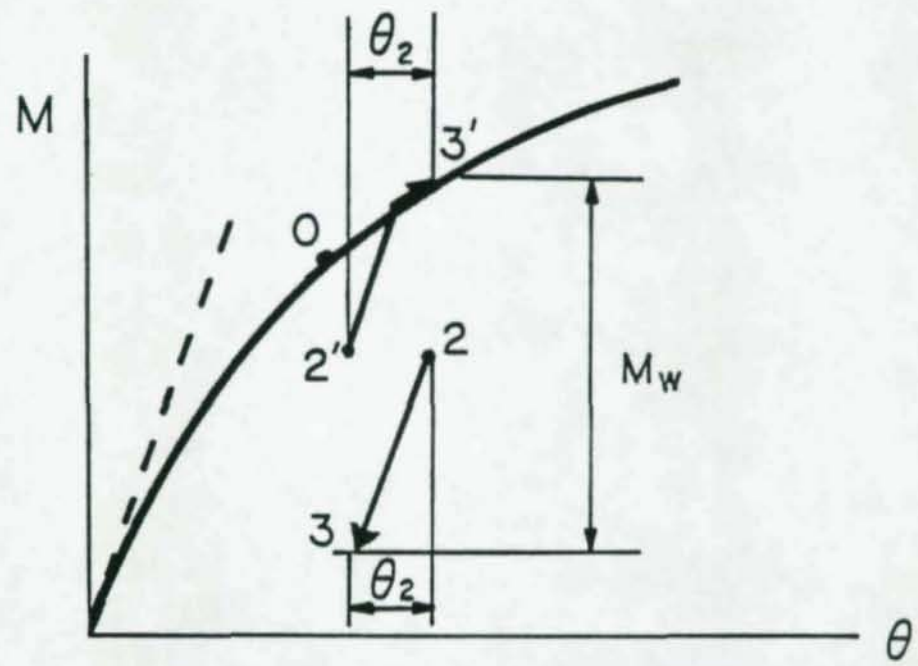


Figure 13. Semi-Rigid Connection End Conditions: Wind Load in Negative Direction (from Reference 9).

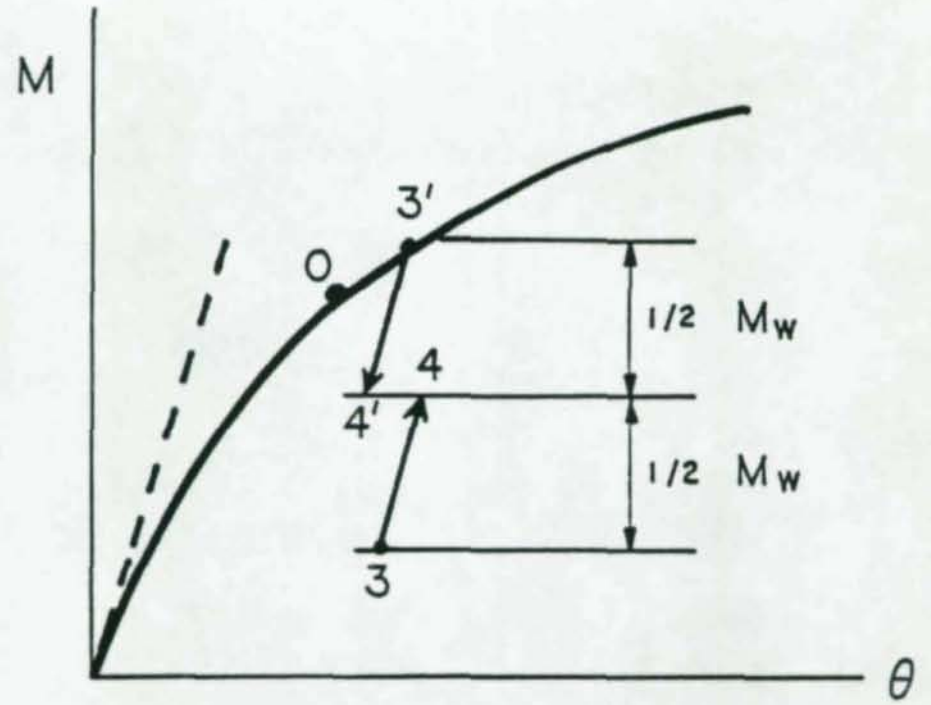


Figure 14. Semi-Rigid Connection End Conditions: Wind Unloading (from Reference 9).

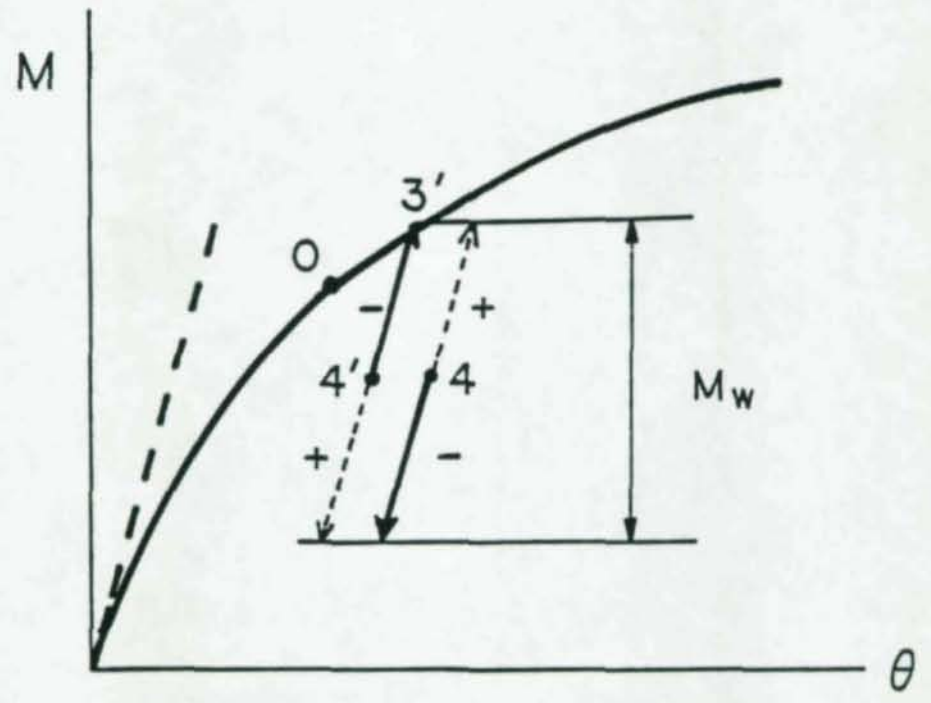


Figure 15. Semi-Rigid Connection End Conditions: Elastic Wind Connection with Gravity Moment "Shake Down" (from Reference 9).

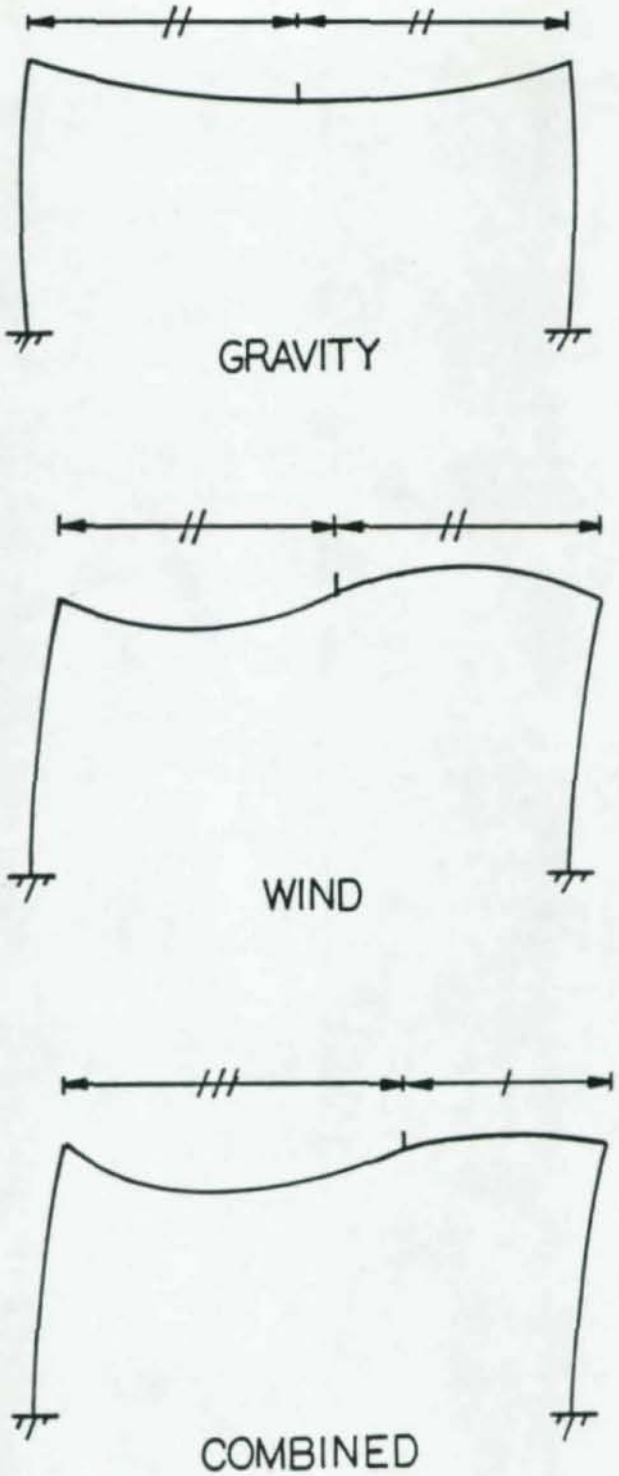


Figure 16. Inflection Points of Beam Deformation.

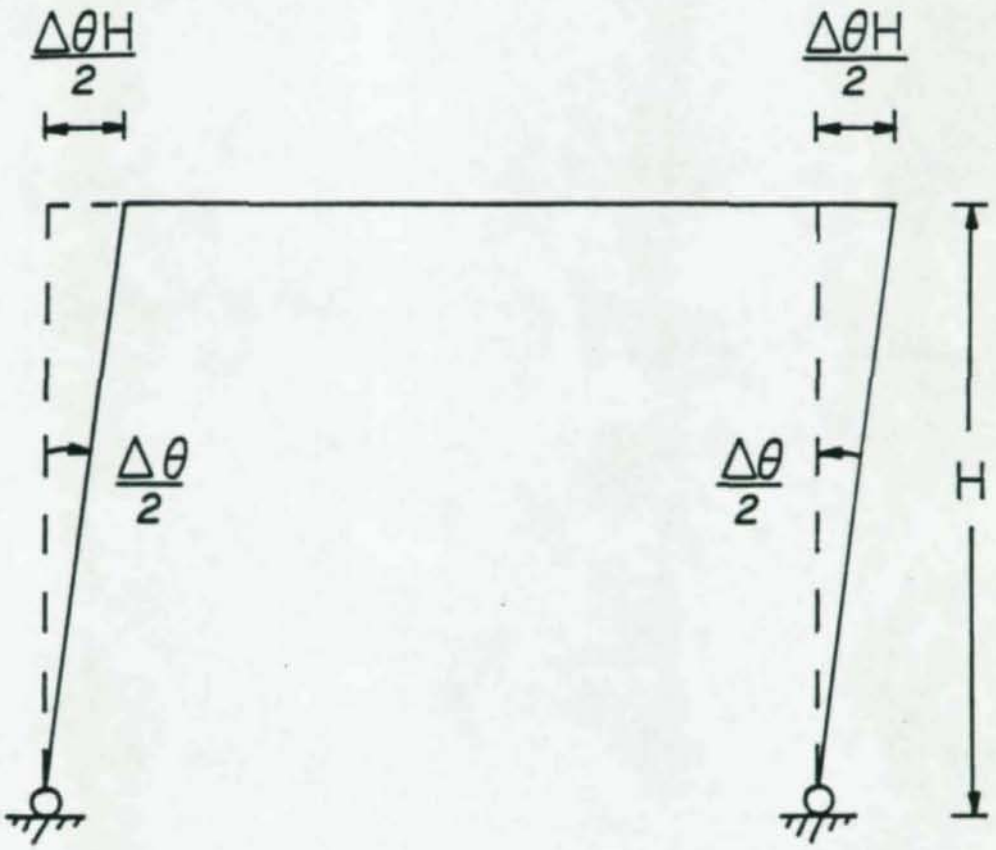


Figure 17. Residual Drift of Frame.

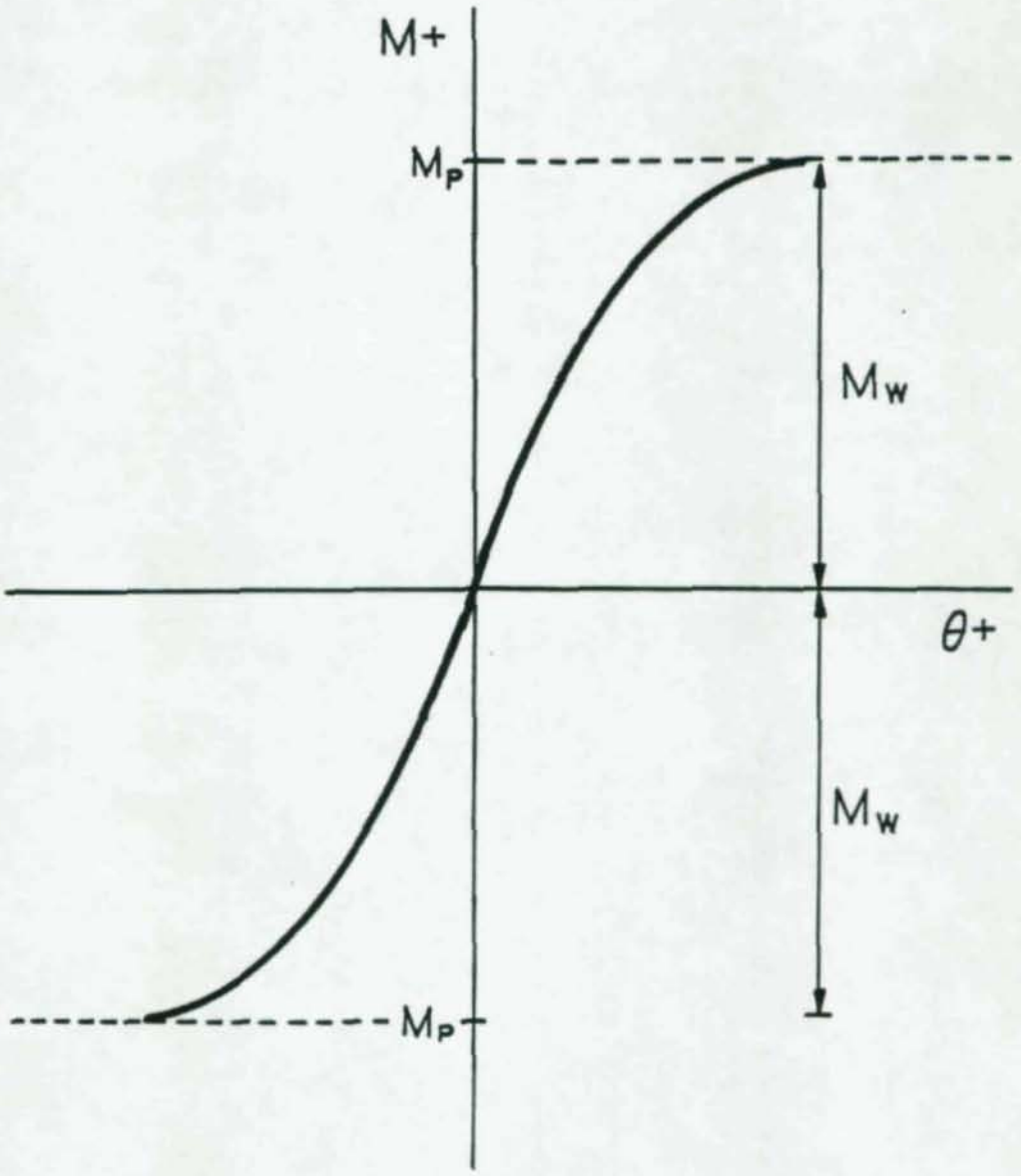


Figure 18. Symmetric Moment-Rotation Curve for Semi-Rigid Connection.

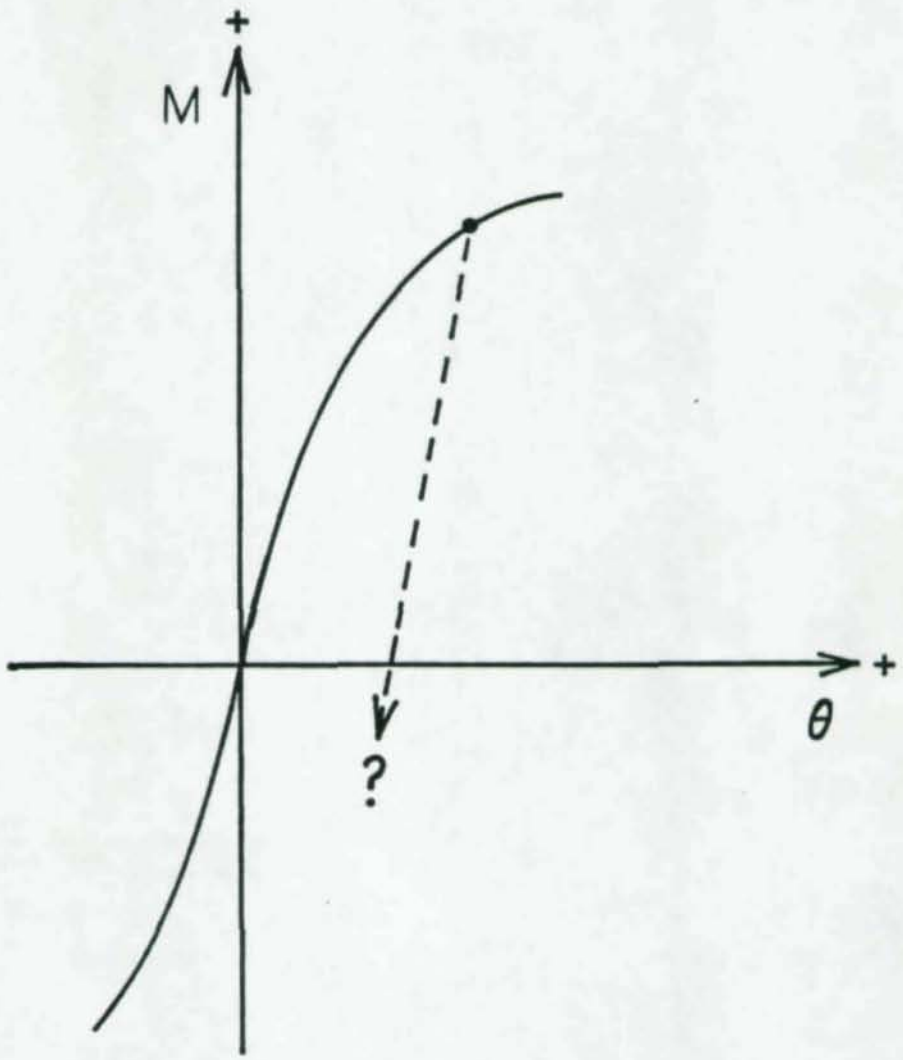


Figure 19. Moment Reversal of Semi-Rigid Connection.

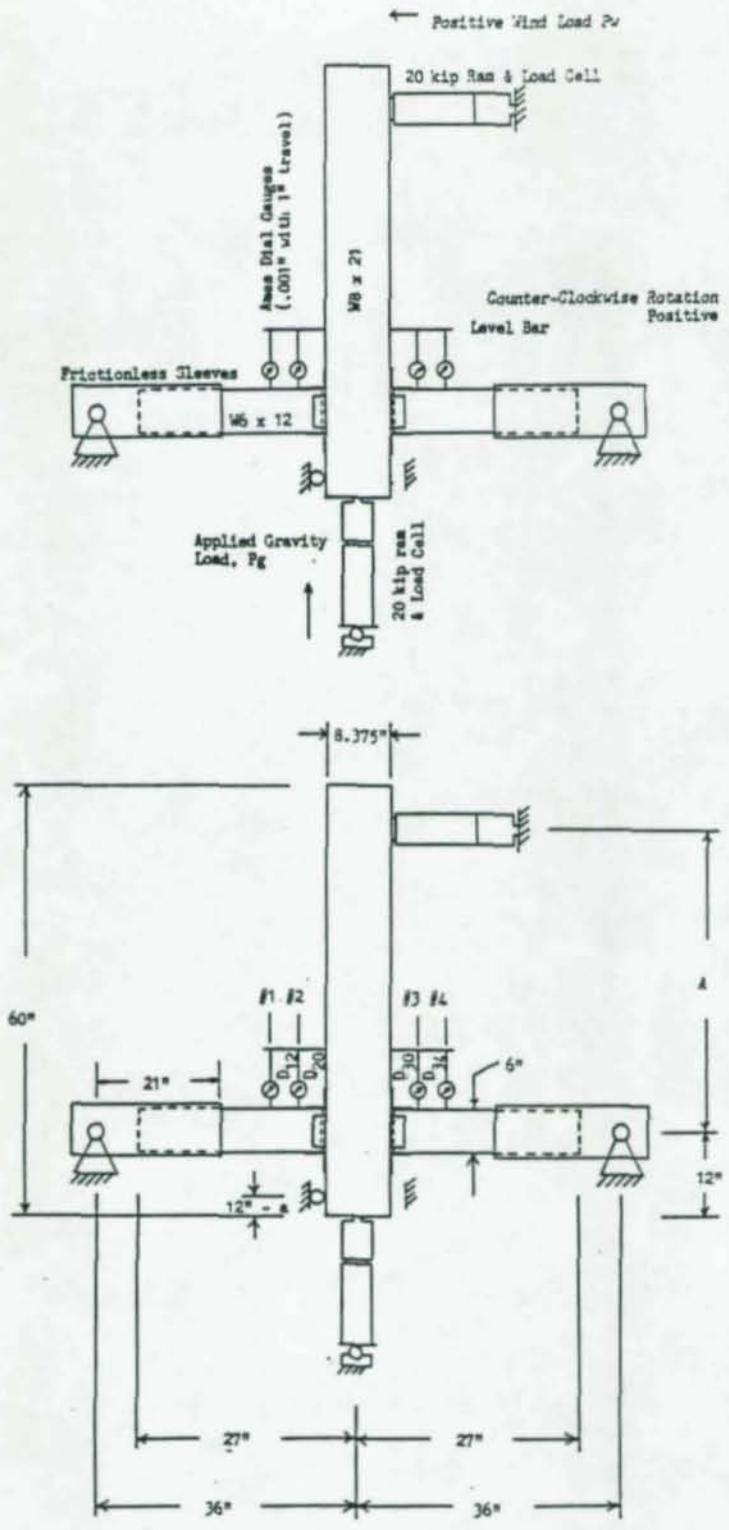


Figure 20. Semi-Rigid Connection Test Assembly.

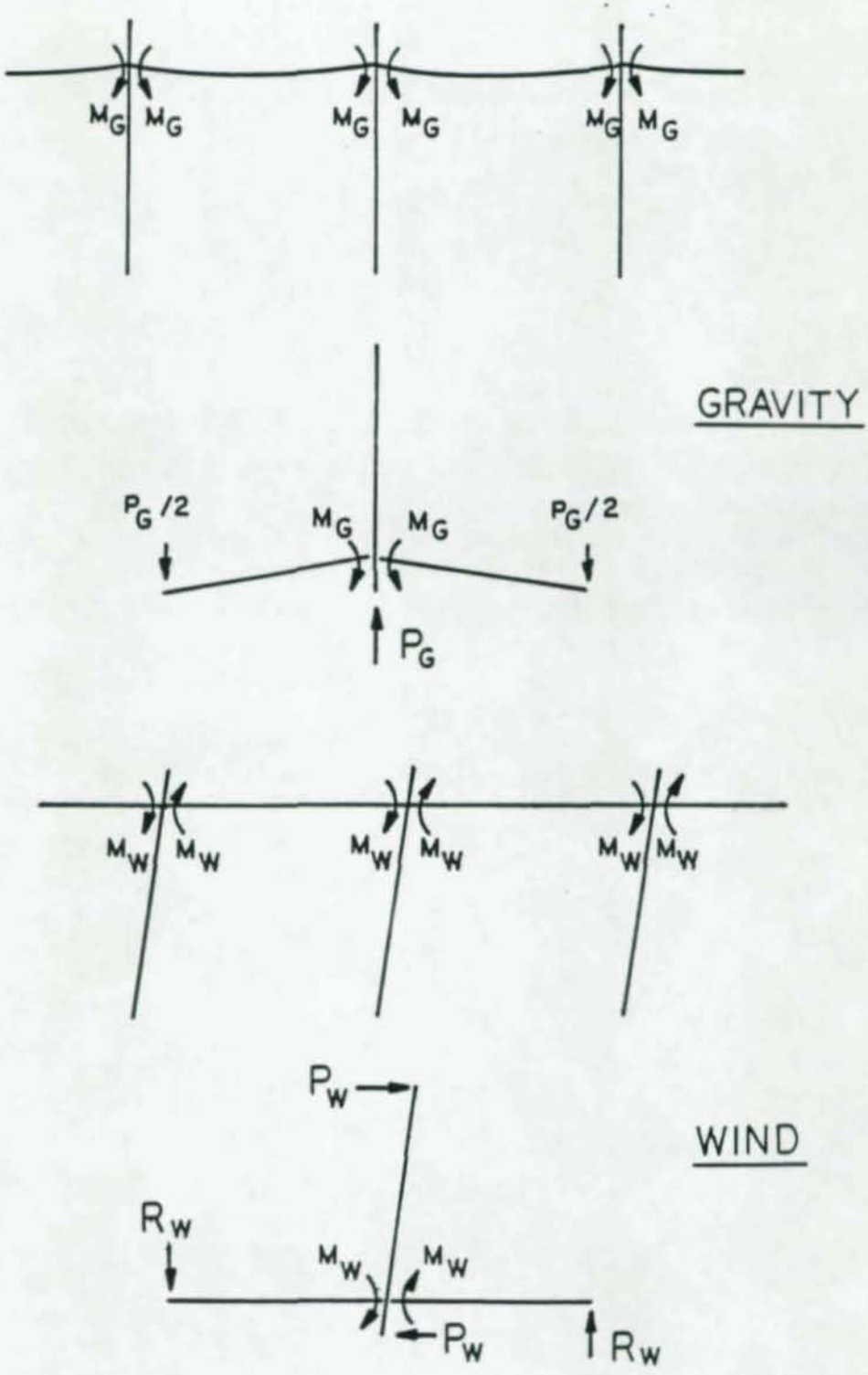


Figure 21. Assembly Loading Representation.

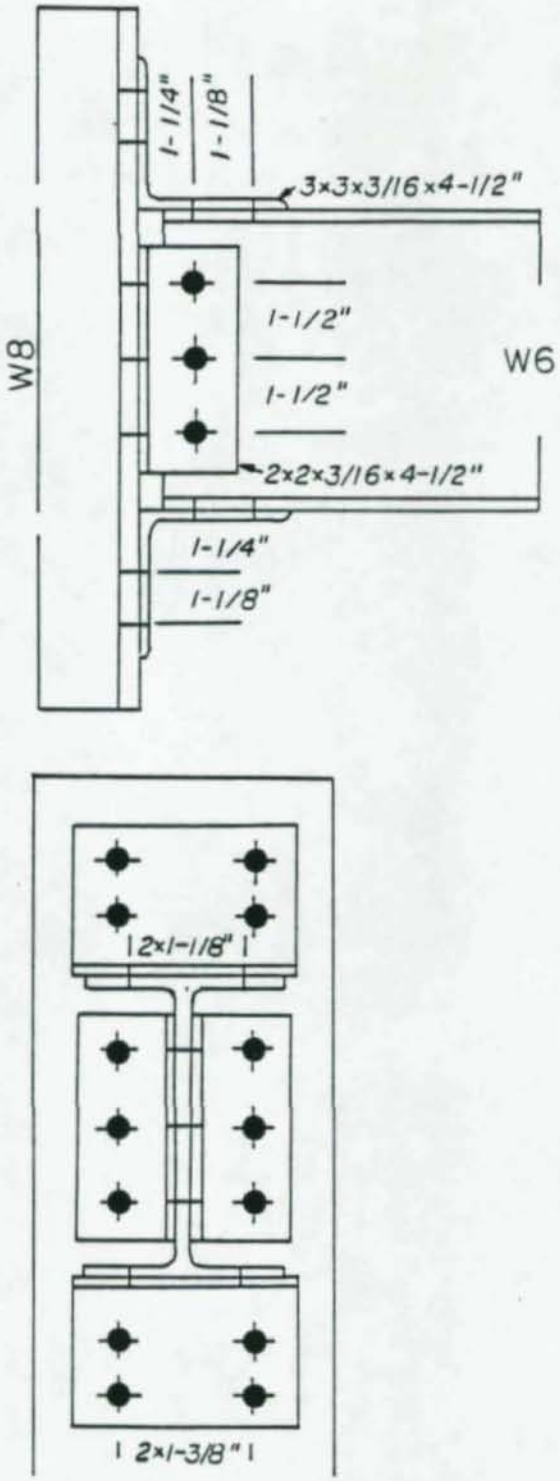
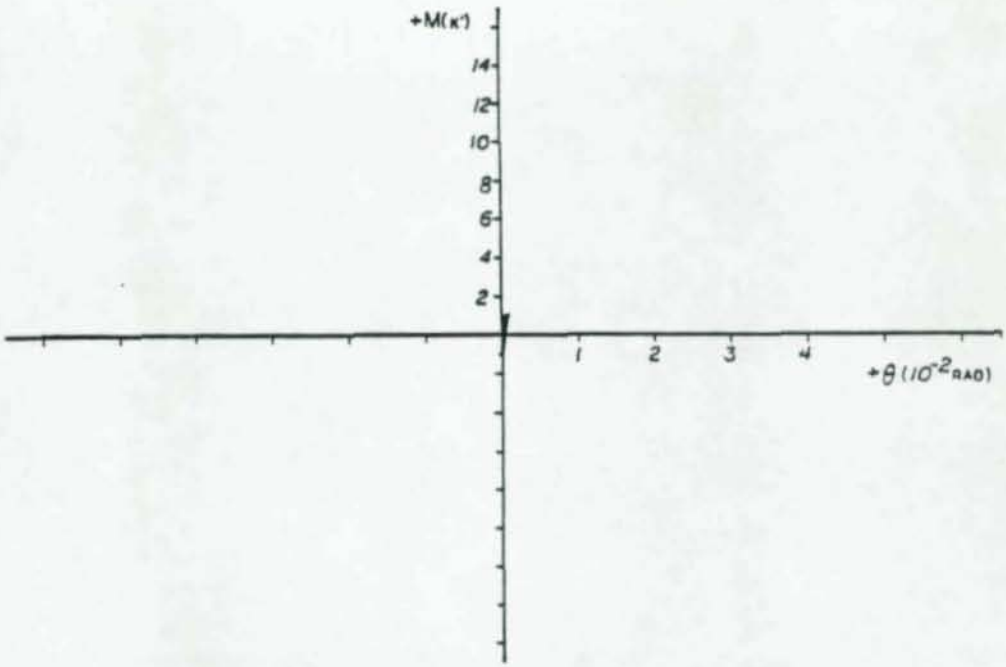


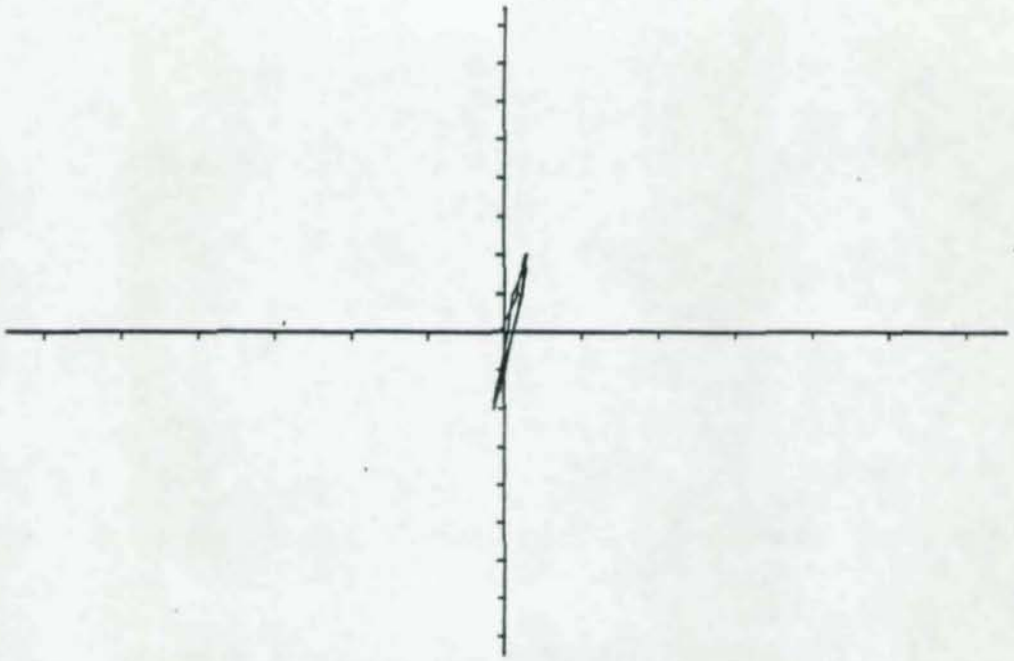
Figure 22. Test Connection Detail.



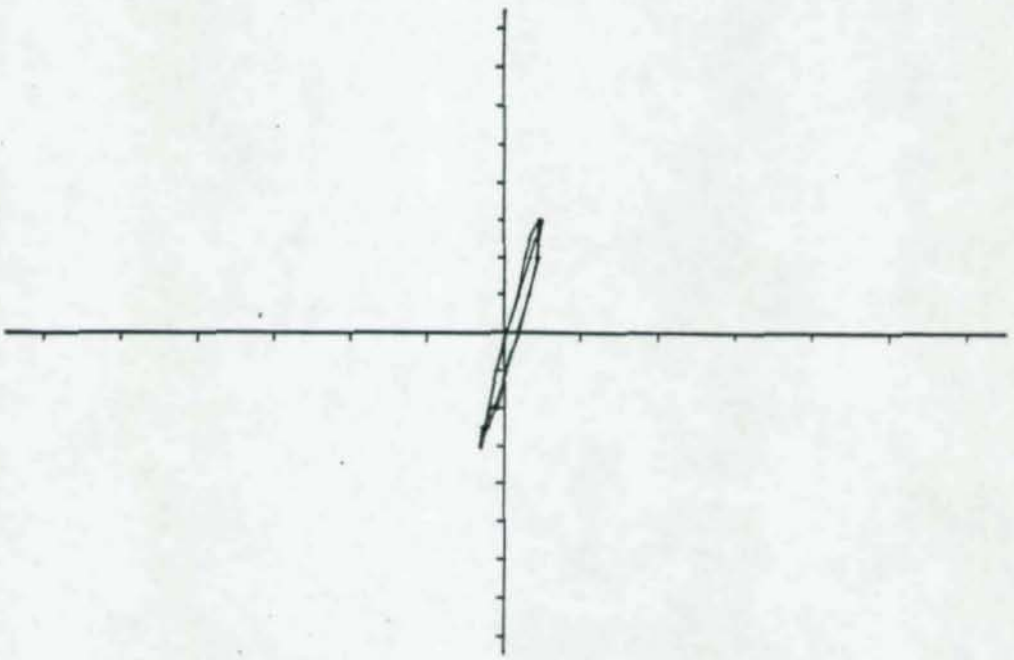
a) Test 2 : Load Cycle 1
Joint Moment-Rotation.

b) Test 2 : Load Cycle 2
Joint Moment-Rotation.

Figure 23. Test 2: Joint Moment vs. Joint Rotation

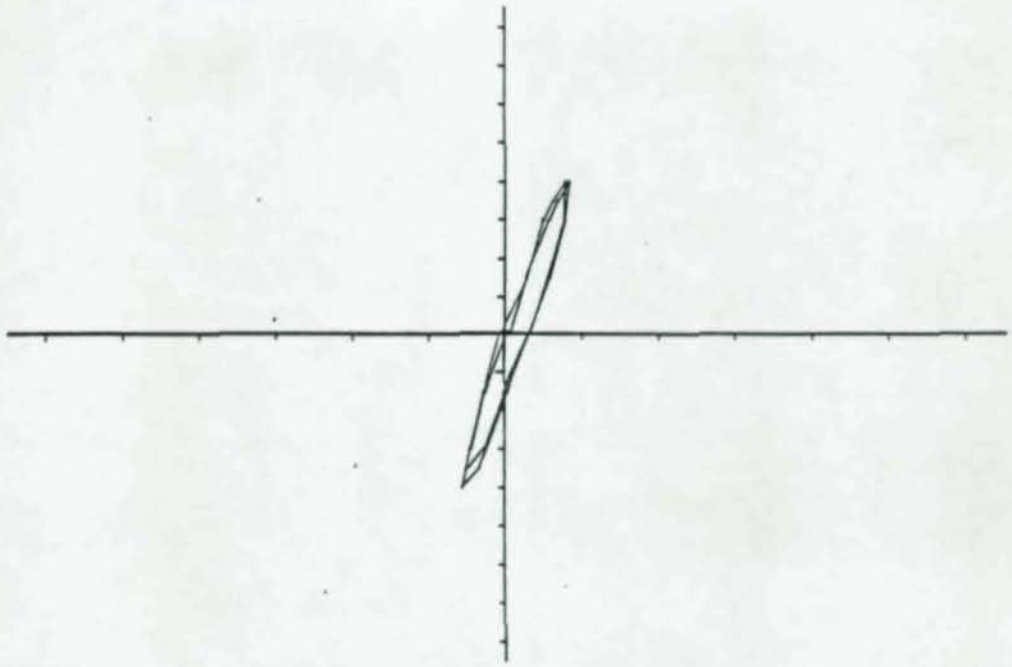


c) Test 2 : Load Cycle 3
Joint Moment-Rotation.

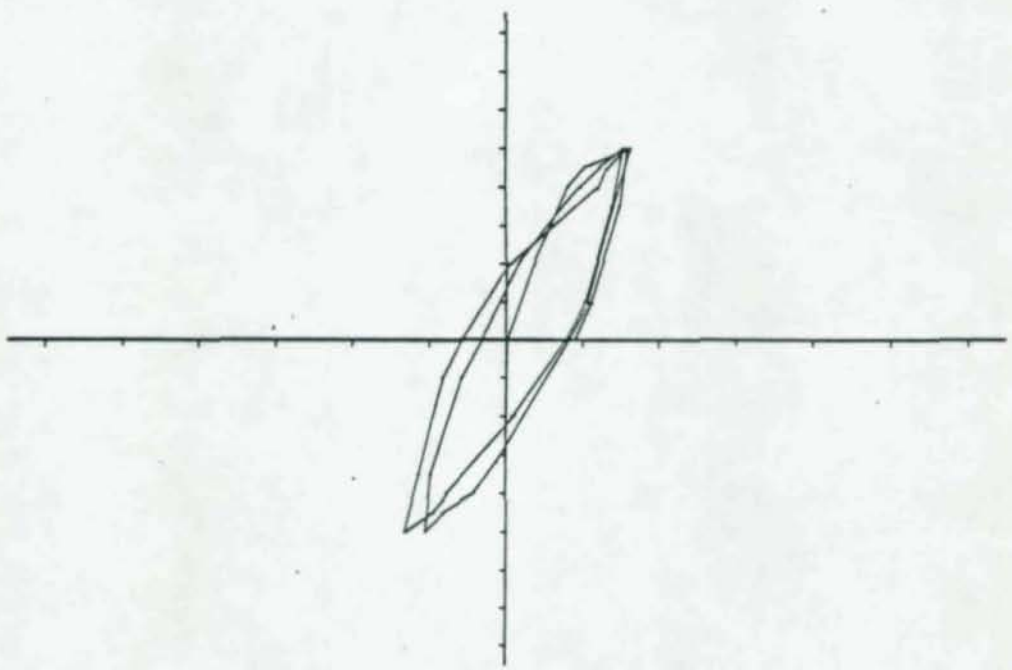


d) Test 2 : Load Cycle 4
Joint Moment-Rotation.

Figure 23 continued. Test 2: Joint Moment vs. Joint Rotation

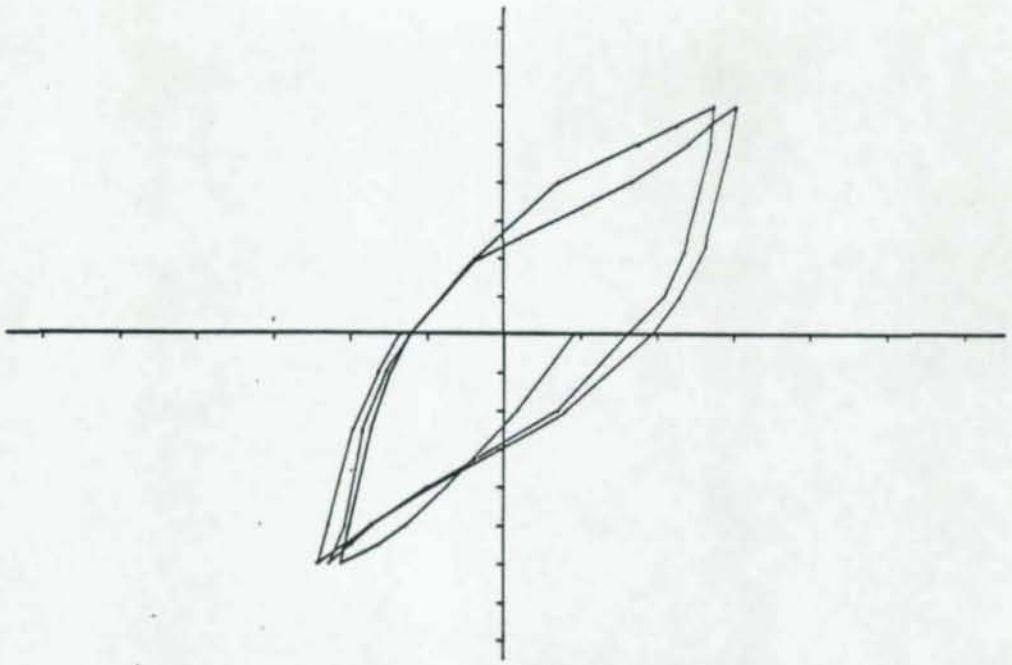


e) Test 2 : Load Cycle 5
Joint Moment-Rotation.

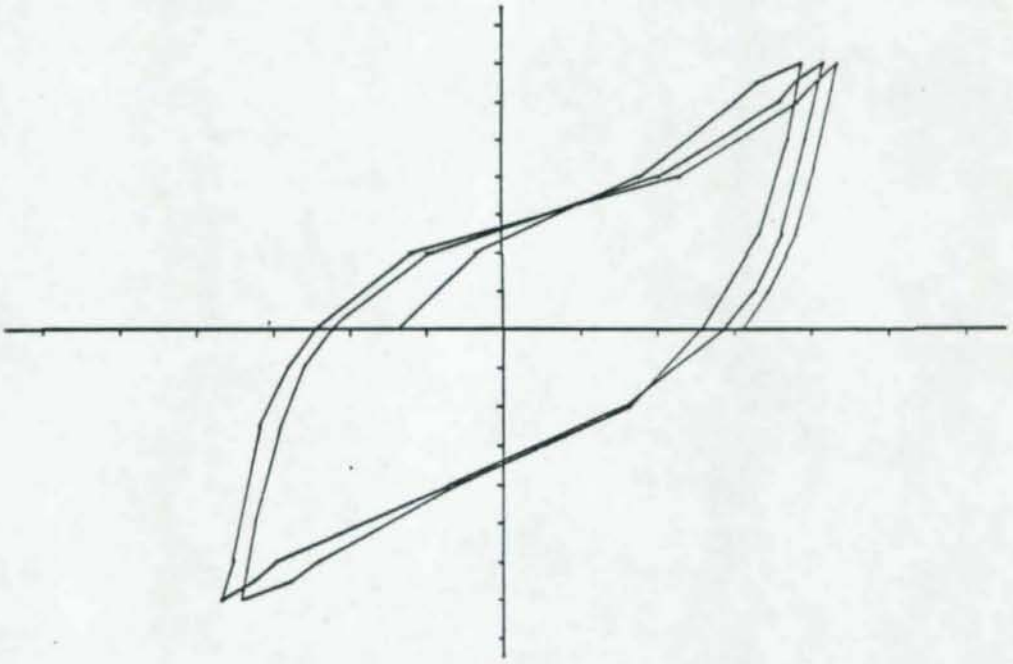


f) Test 2 : Load Cycle 6
Joint Moment-Rotation.

Figure 23 continued. Test 2: Joint Moment vs. Joint Rotation

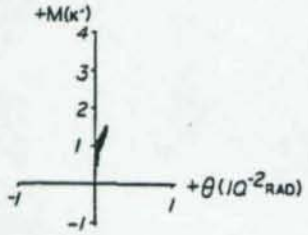


g) Test 2 : Load Cycle 7
Joint Moment-Rotation.



h) Test 2 : Load Cycle 8
Joint Moment-Rotation.

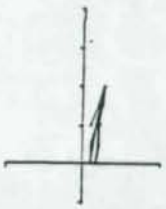
Figure 23 continued. Test 2: Joint Moment vs. Joint Rotation



a) Test 2 : Load Cycle 1
Connection 1-2.

b) Test 2 : Load Cycle 1
Connection 3-4.

c) Test 2 : Load Cycle 2
Connection 1-2.

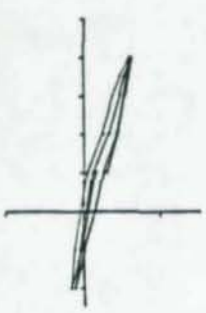


d) Test 2 : Load Cycle 2
Connection 3-4.

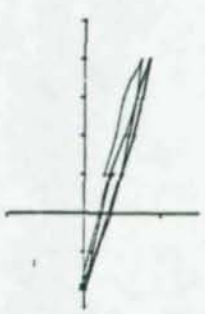
Figure 24. Test 2: Connection Moments vs. Rotations.

e) Test 2 : Load Cycle 3
Connection 1-2.

f) Test 2 : Load Cycle 3
Connection 3-4.

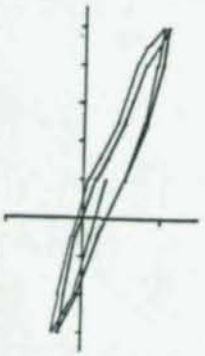


g) Test 2 : Load Cycle 4
Connection 1-2.

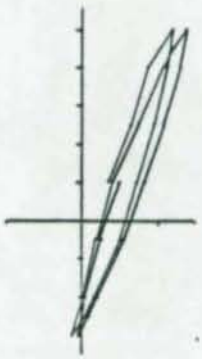


h) Test 2 : Load Cycle 4
Connection 3-4.

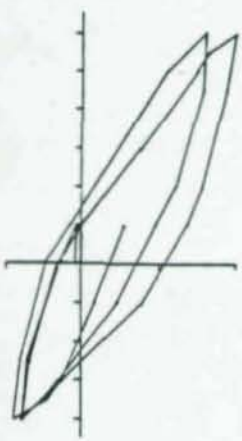
Figure 24 continued. Test 2: Connection Moments vs. Rotations.



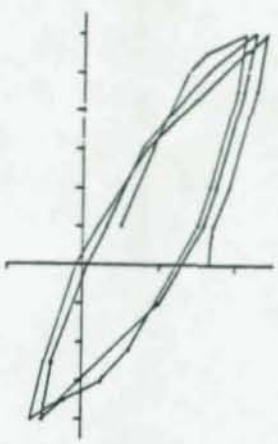
i) Test 2 : Load Cycle 5
Connection 1-2.



j) Test 2 : Load Cycle 5
Connection 3-4.

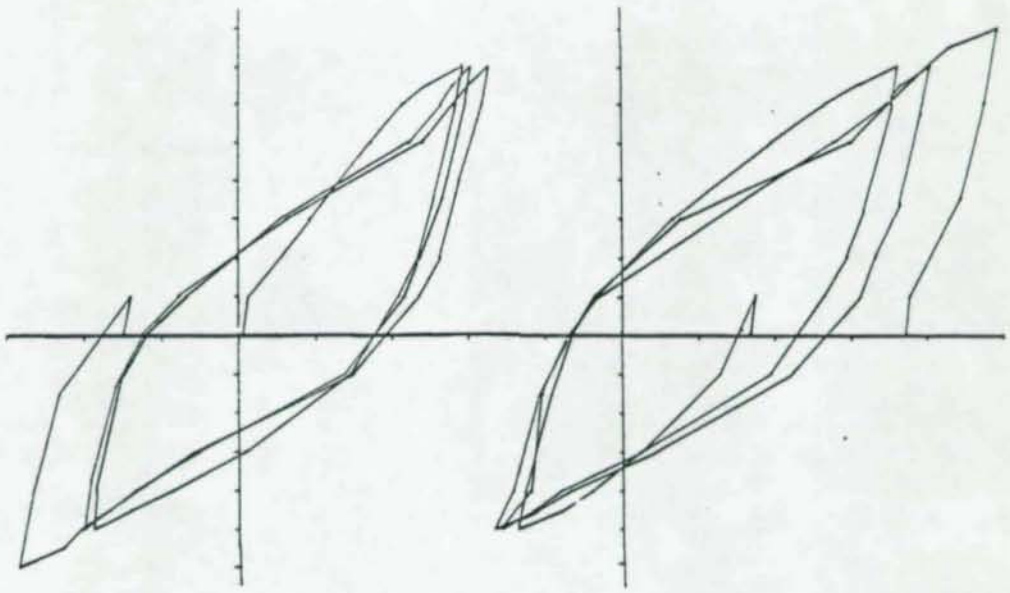


k) Test 2 : Load Cycle 6
Connection 1-2.



l) Test 2 : Load Cycle 6
Connection 3-4.

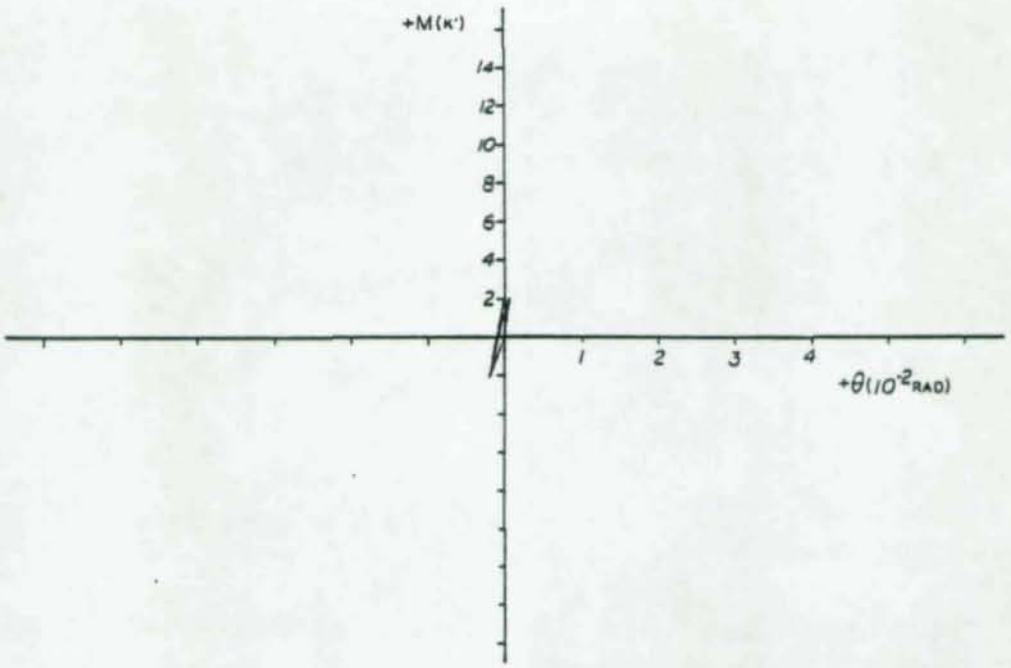
Figure 24 continued. Test 2: Connection Moments vs. Rotations.



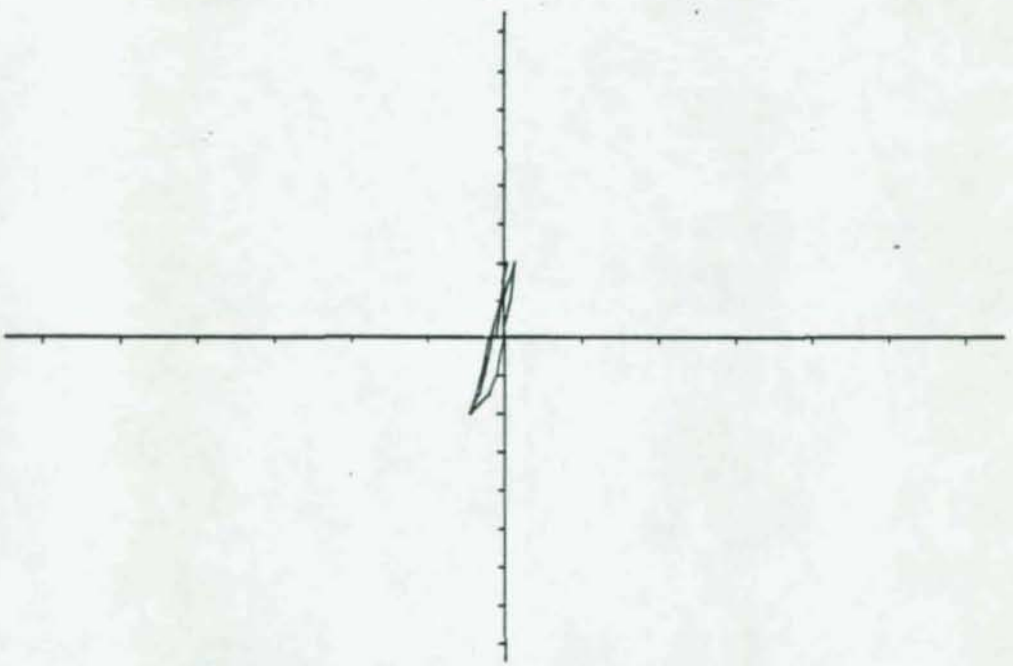
m) Test 2 : Load Cycles 7 & 8. Connection 1-2.

n) Test 2 : Load Cycles 7 & 8. Connection 3-4.

Figure 24 continued. Test 2: Connection Moments vs. Rotations.

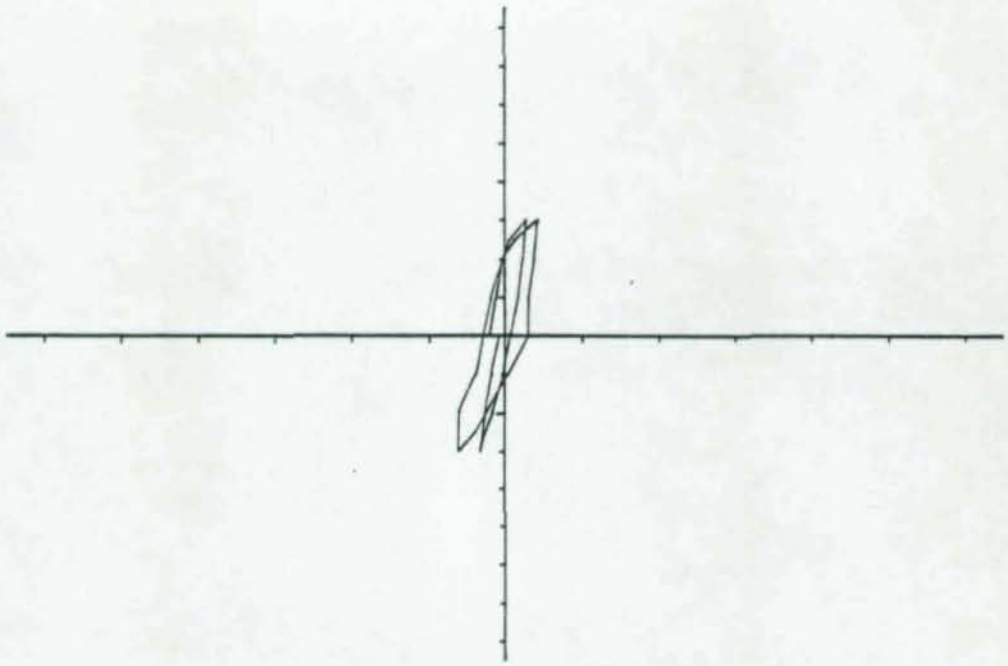


a) Test 3 : Load Cycle 1
Joint Moment-Rotation.

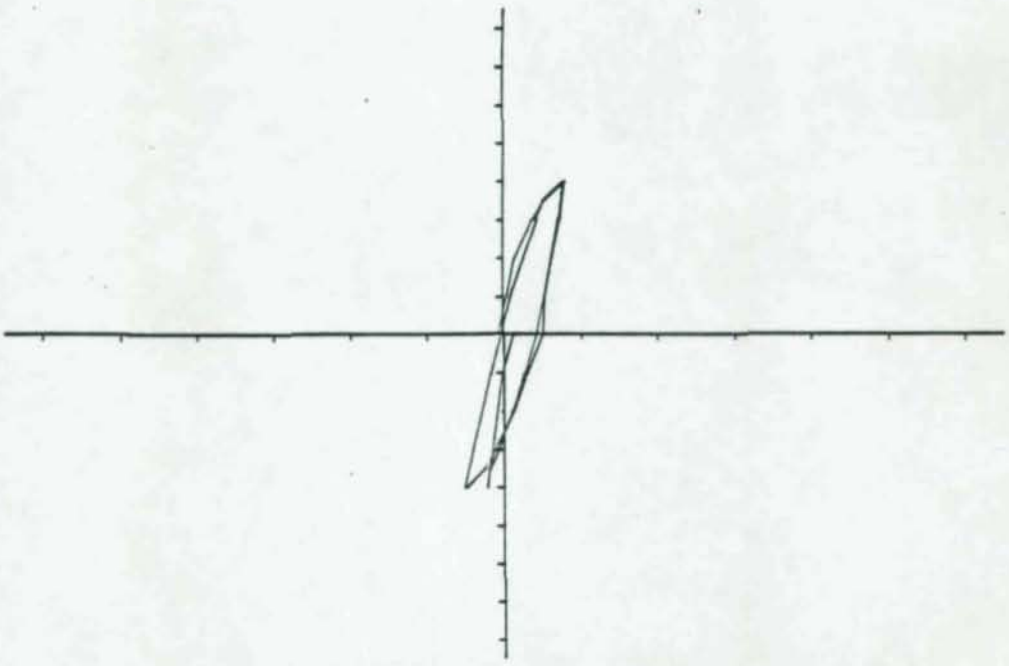


b) Test 3 : Load Cycle 2
Joint Moment-Rotation.

Figure 25. Test 3: Joint Moment vs. Joint Rotation

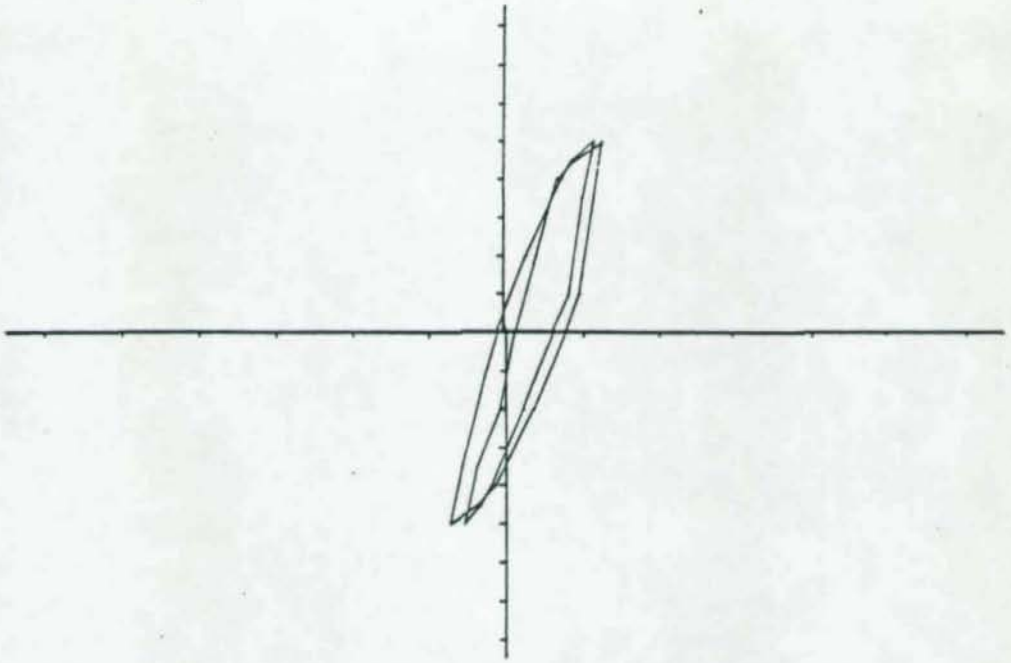


c) Test 3 : Load Cycle 3
Joint Moment-Rotation.

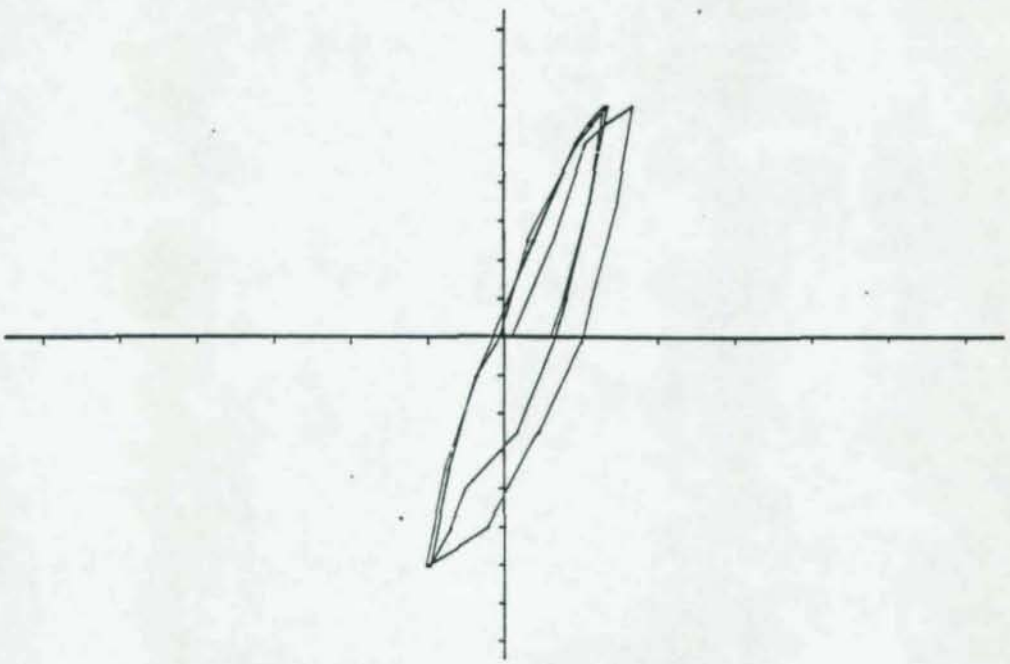


d) Test 3 : Load Cycle 4
Joint Moment-Rotation.

Figure 25 continued. Test 3: Joint Moment vs. Joint Rotation

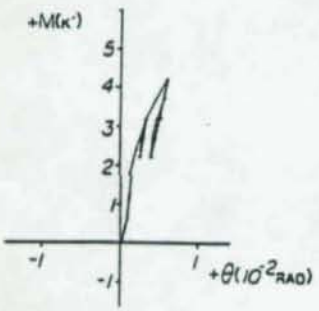


e) Test 3 : Load Cycle 5
Joint Moment-Rotation.

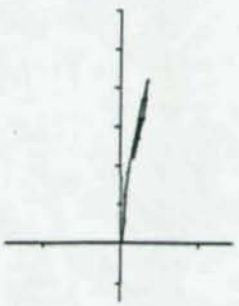


f) Test 3 : Load Cycle 6
Joint Moment-Rotation.

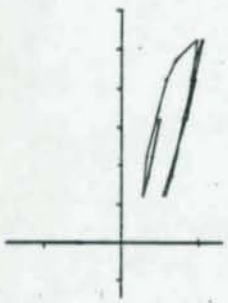
Figure 25 continued. Test 3: Joint Moment vs. Joint Rotation



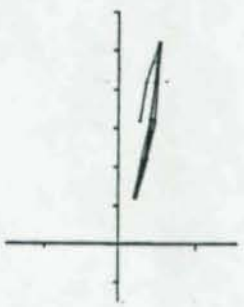
a) Test 3 : Load Cycle 1
Connection 1-2.



b) Test 3 : Load Cycle 1
Connection 3-4.

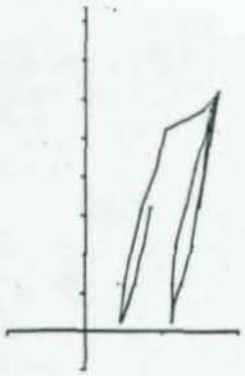


c) Test 3 : Load Cycle 2
Connection 1-2.

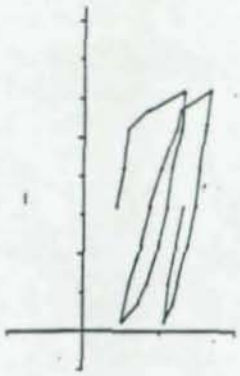


d) Test 3 : Load Cycle 2
Connection 3-4.

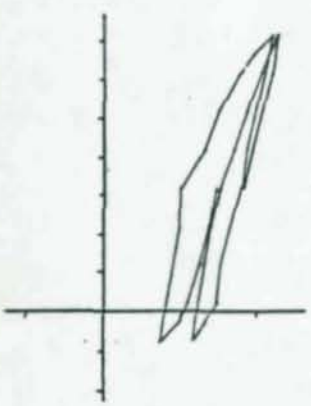
Figure 26. Test 3: Connection Moments vs. Rotations.



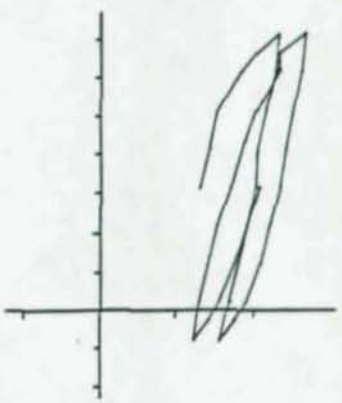
e) Test 3 : Load Cycle 3
Connection 1-2.



f) Test 3 : Load Cycle 3
Connection 3-4.



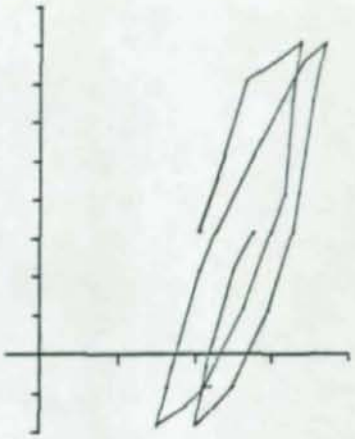
g) Test 3 : Load Cycle 4
Connection 1-2.



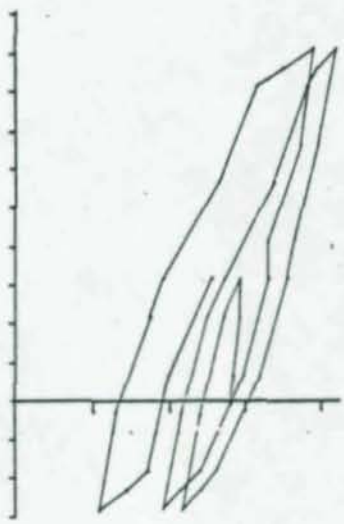
h) Test 3 : Load Cycle 4
Connection 3-4.

Figure 26 continued. Test 3: Connection Moments vs. Rotations.

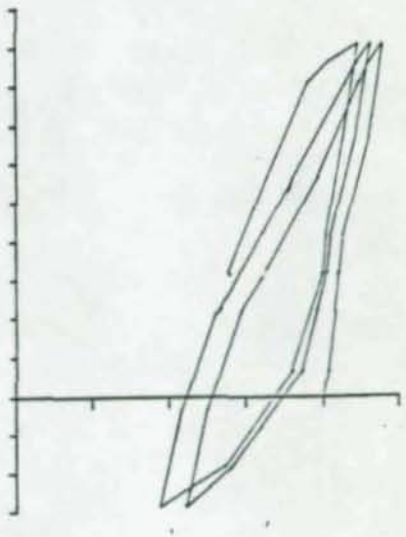
i) Test 3 : Load Cycle 5
Connection 1-2.



j) Test 3 : Load Cycle 5
Connection 3-4.



k) Test 3 : Load Cycle 6
Connection 1-2.



l) Test 3 : Load Cycle 6
Connection 3-4.

Figure 26 continued. Test 3: Connection Moments vs. Rotations.

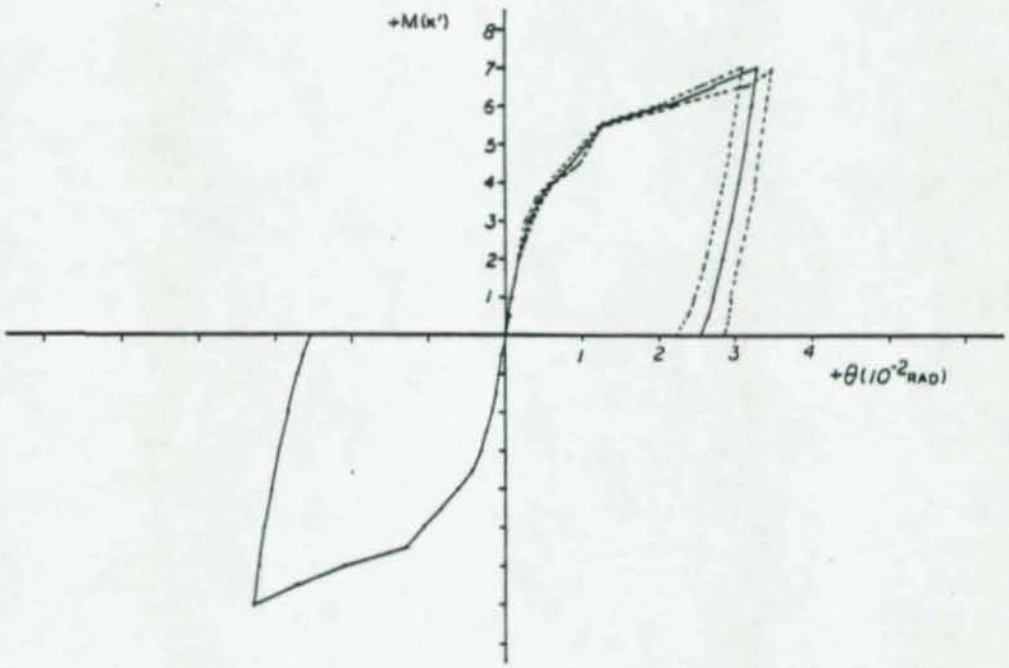


Figure 27. Test 4 : Monotonic Moment-Rotation Curve.

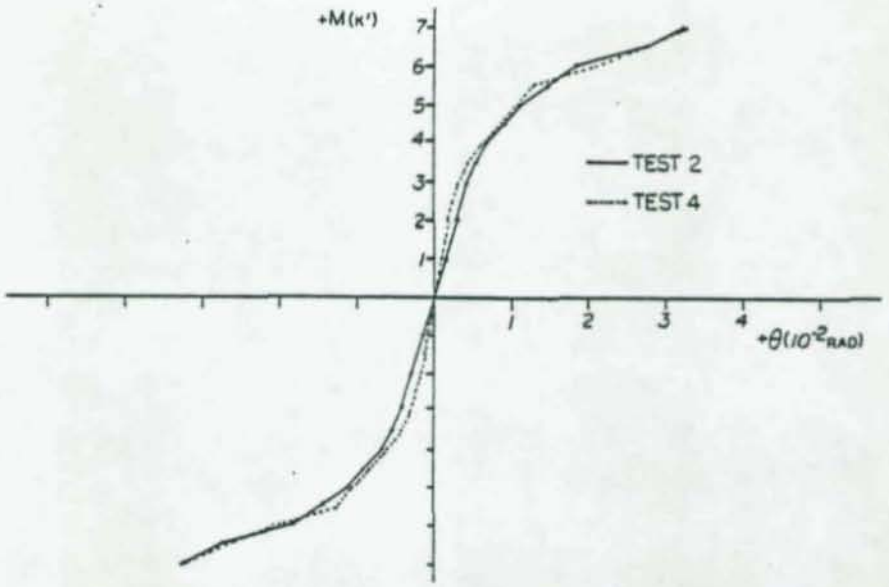


Figure 28. Test 2 : Initial Connection Loading Peaks Curve and Test 4 : Monotonic Moment-Rotation Curve.

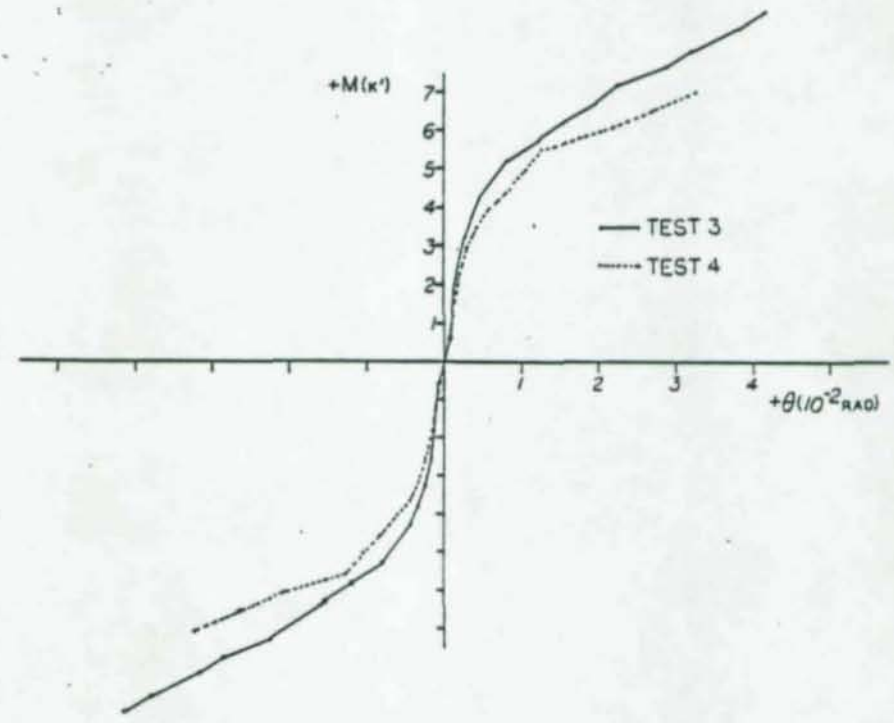


Figure 29. Test 3 : Initial Connection Loading Peaks Curve and Test 4 : Monotonic Moment-Rotation Curve.

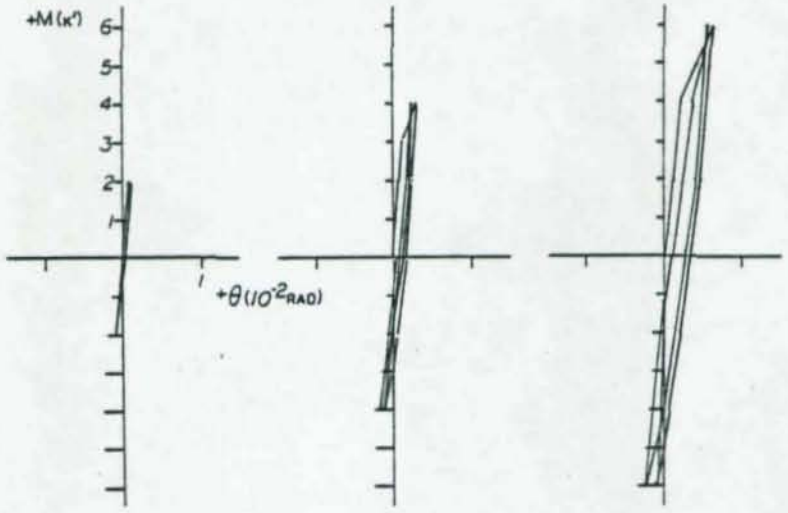
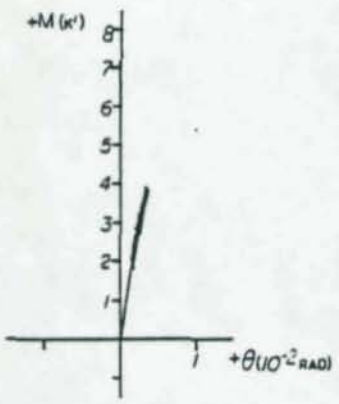
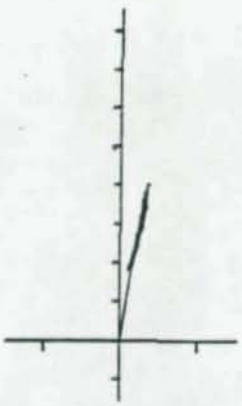


Figure 30. Test 5 : Joint Moment vs. Joint Rotation
Load Cycles 1 through 3.

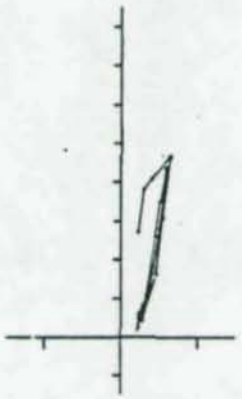


a) Test 5 : Load Cycle 1
Connection 1-2.



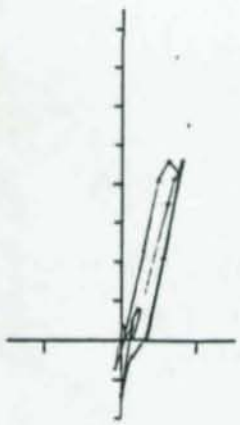
b) Test 5 : Load Cycle 1
Connection 3-4.

c) Test 5 : Load Cycle 2
Connection 1-2.

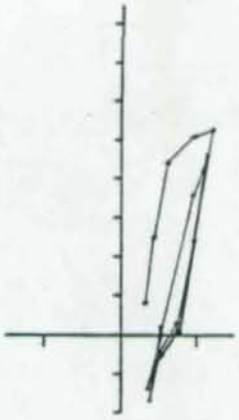


d) Test 5 : Load Cycle 2
Connection 3-4.

Figure 31. Test 5 : Connection Moments vs. Rotations.



e) Test 5 : Load Cycle 3
Connection 1-2.



f) Test 5 : Load Cycle 3
Connection 3-4.

Figure 31 continued. Test 5 : Connection Moments vs. Rotations.

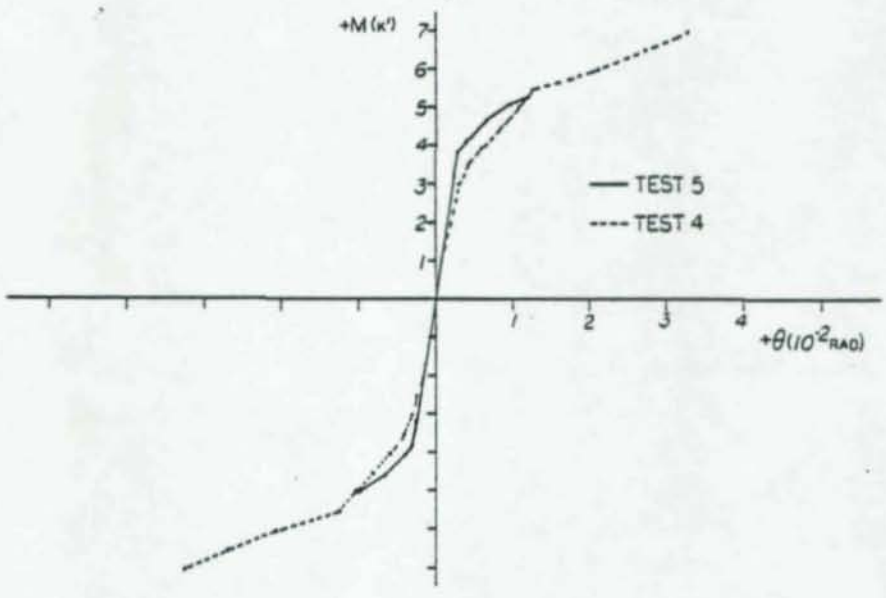


Figure 32. Test 5 : Initial Connection Loading Peaks Curve and Test 4 : Monotonic Moment-Rotation Curve.

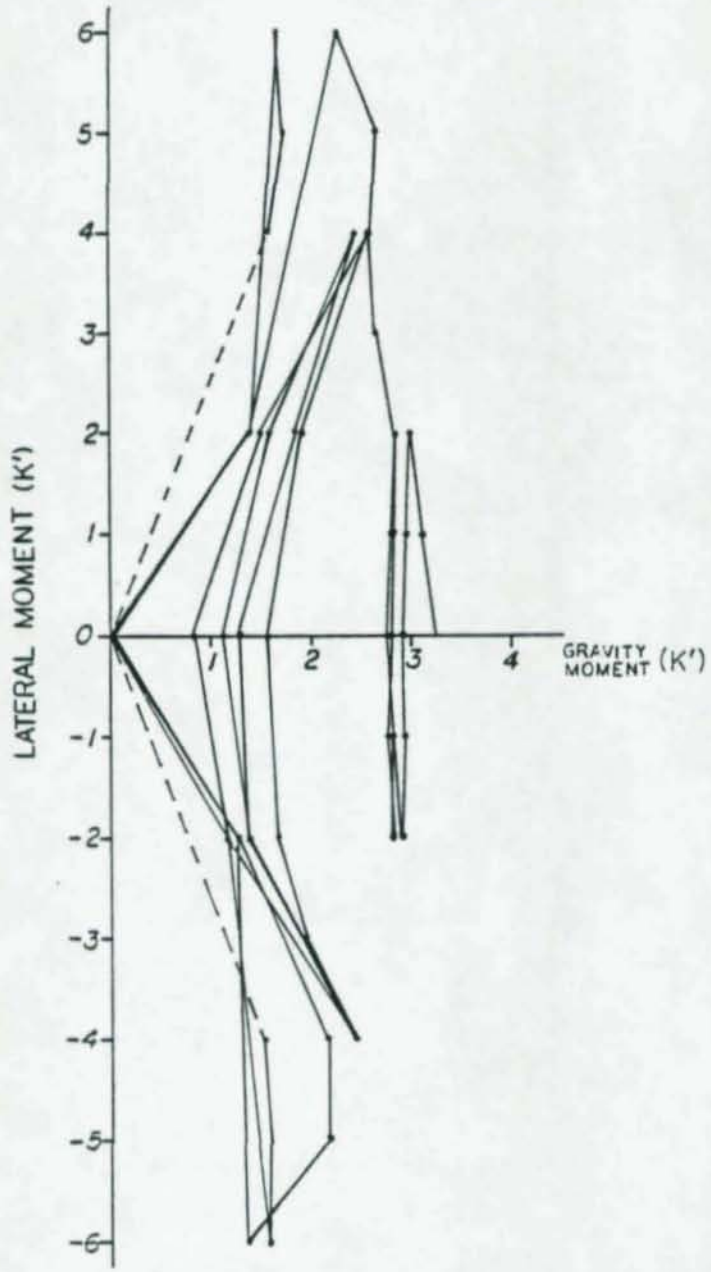
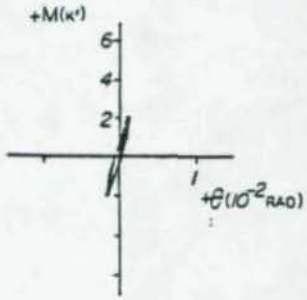
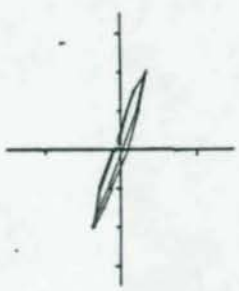


Figure 33. Test 5 : Applied Lateral Connection Moment vs. Gravity Moment. Gravity Moment Shakedown.

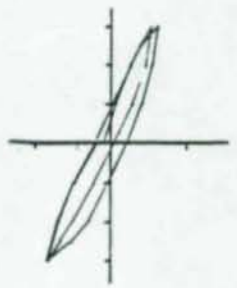


a) Test 6 : Load Cycle 1
Joint Moment-Rotation.

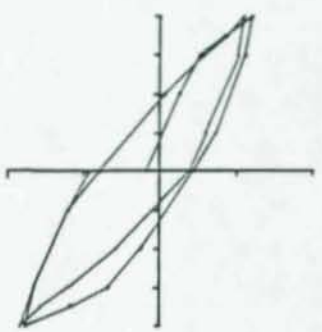


b) Test 6 : Load Cycle 2
Joint Moment-Rotation.

Figure 34. Test 6 : Joint Moment vs. Joint Rotation

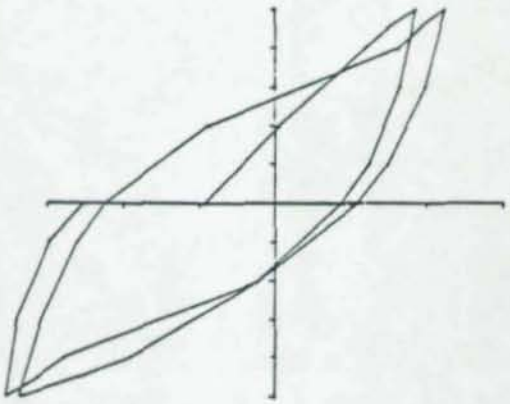


c) Test 6 : Load Cycle 3
Joint Moment-Rotation.

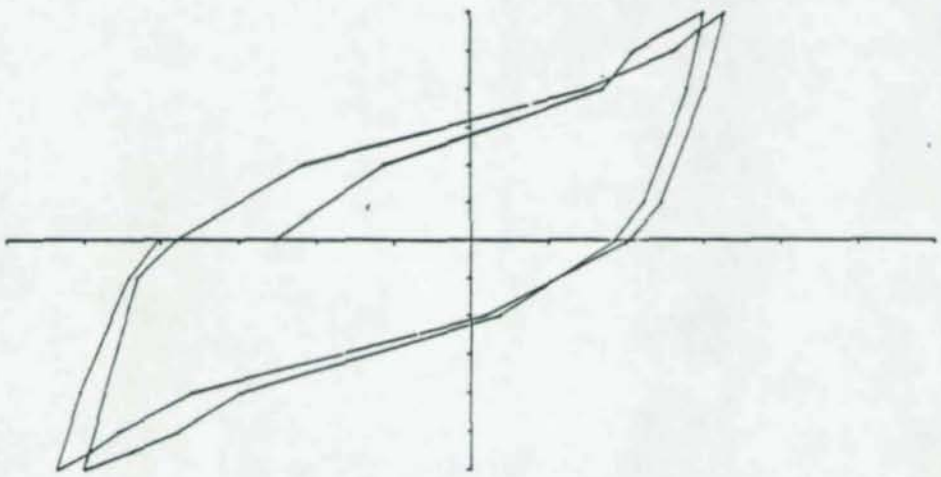


d) Test 6 : Load Cycle 4
Joint Moment-Rotation.

Figure 34 continued. Test 6 : Joint Moment vs. Joint Rotation

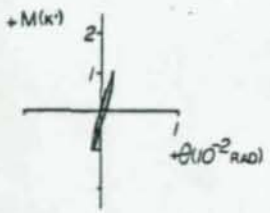


e) Test 6 : Load Cycle 5
Joint Moment-Rotation.



f) Test 6 : Load Cycle 6
Joint Moment-Rotation.

Figure 34 continued. Test 6 : Joint Moment vs. Joint Rotation



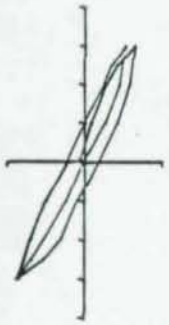
a) Test 6 : Load Cycle 1
Connection 1-2.

b) Test 6 : Load Cycle 1
Connection 3-4.

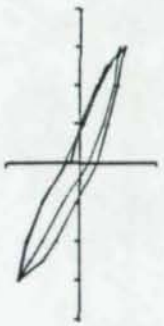
c) Test 6 : Load Cycle 2
Connection 1-2.

d) Test 6 : Load Cycle 2
Connection 3-4.

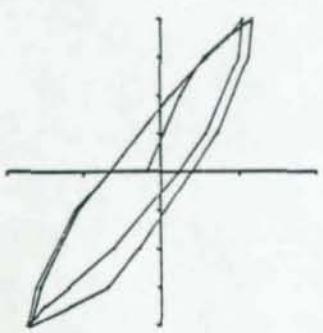
Figure 35. Test 6: Connection Moments vs. Rotations.



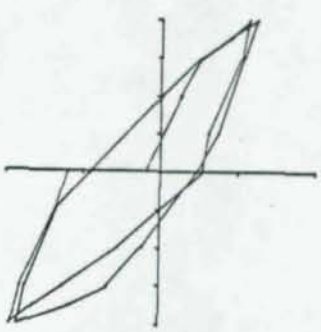
e) Test 6 : Load Cycle 3
Connection 1-2.



f) Test 6 : Load Cycle 3
Connection 3-4.

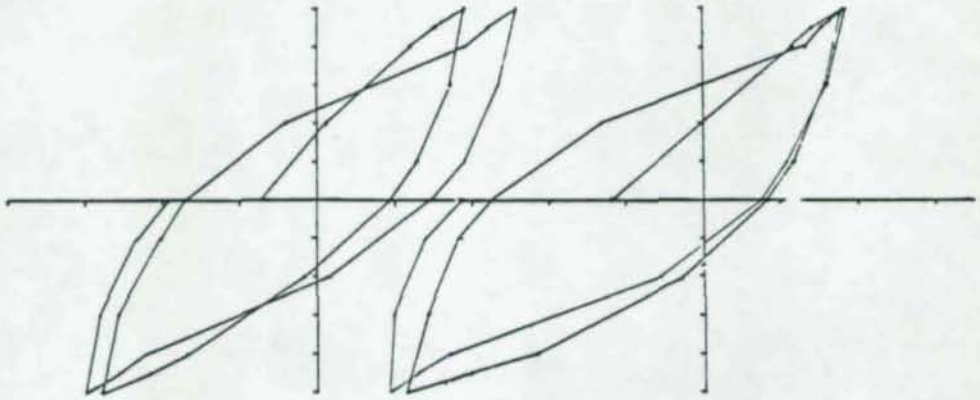


g) Test 6 : Load Cycle 4
Connection 1-2.



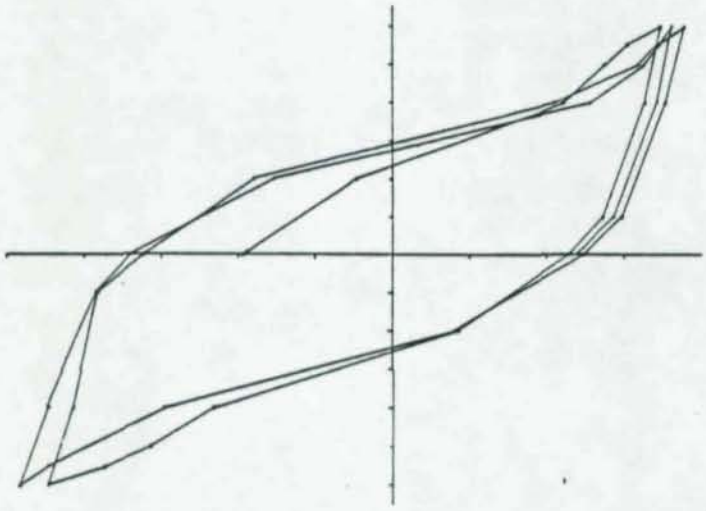
h) Test 6 : Load Cycle 4
Connection 3-4.

Figure 35 continued. Test 6: Connection Moments vs. Rotations.



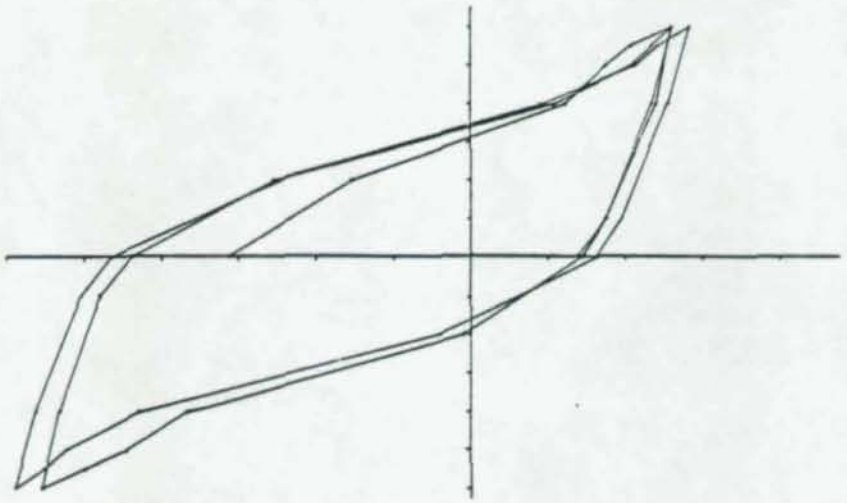
i) Test 6 : Load Cycle 5
Connection 1-2.

j) Test 6 : Load Cycle 5
Connection 3-4.



k) Test 6 : Load Cycle 6
Connection 1-2.

Figure 35 continued. Test 6: Connection Moments vs. Rotations.



1) Test 6 : Load Cycle 6
Connection 3-4.

Figure 35 continued. Test 6: Connection Moments vs. Rotations.

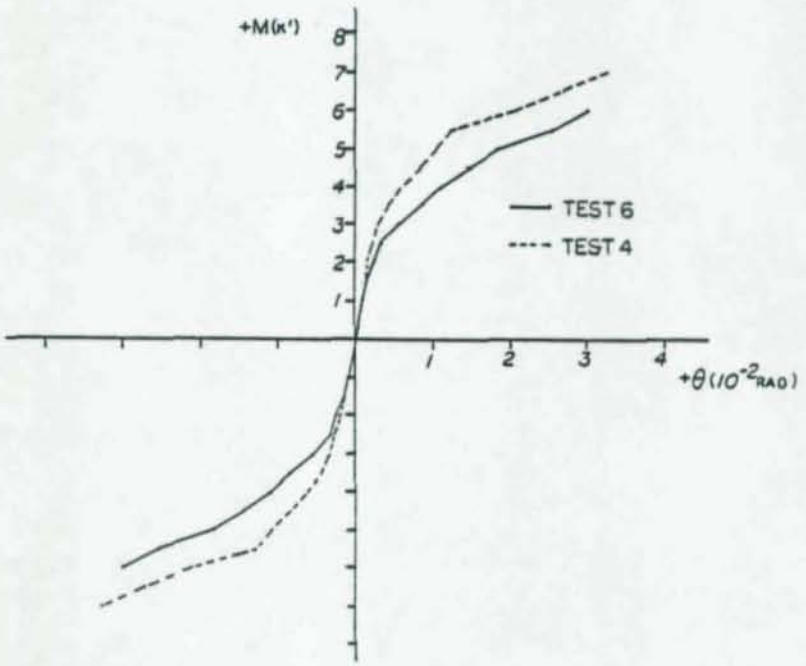
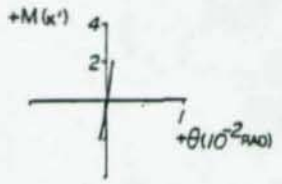


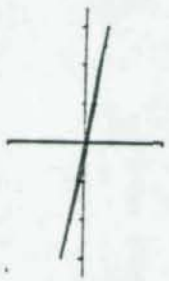
Figure 36. Test 6 : Initial Connection Loading Peaks Curve and Test 4 : Monotonic Moment-Rotation Curve.



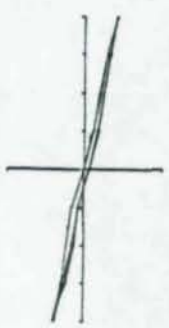
a) Test 7 : Load Cycle 1
Joint Moment-Rotation.

b) Test 7 : Load Cycle 2
Joint Moment-Rotation.

Figure 37. Test 7: Joint Moment vs. Joint Rotation

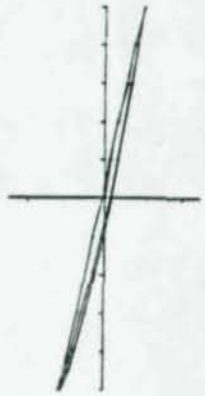


c) Test 7 : Load Cycle 3
Joint Moment-Rotation.

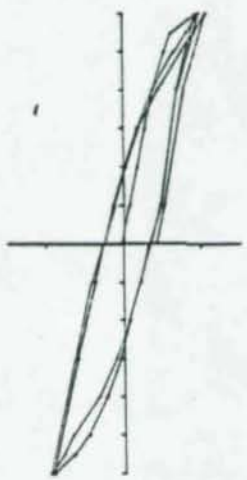


d) Test 7 : Load Cycle 4
Joint Moment-Rotation.

Figure 37 continued. Test 7: Joint Moment vs. Joint Rotation

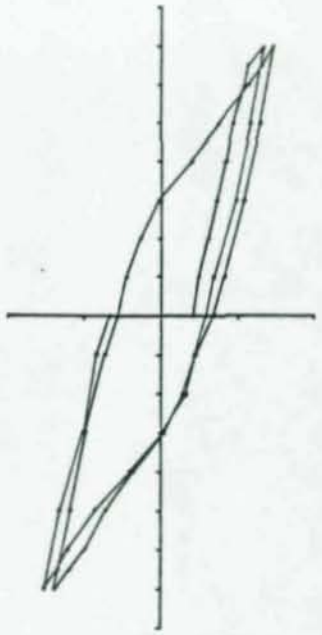


e) Test 7 : Load Cycle 5
Joint Moment-Rotation.



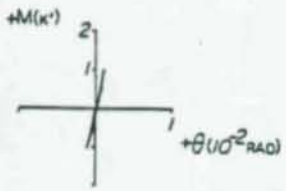
f) Test 7 : Load Cycle 6
Joint Moment-Rotation.

Figure 37 continued. Test 7: Joint Moment vs. Joint Rotation



g) Test 7 : Load Cycle 7
Joint Moment-Rotation.

Figure 37 continued. Test 7: Joint Moment vs. Joint Rotation



a) Test 7 : Load Cycle 1
Connection 1-2.

b) Test 7 : Load Cycle 1
Connection 3-4.

Figure 38. Test 7: Connection Moments vs. Rotations.

c) Test 7 : Load Cycle 2
Connection 1-2.

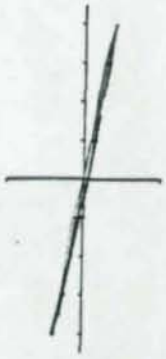
d) Test 7 : Load Cycle 2
Connection 3-4.

e) Test 7 : Load Cycle 3
Connection 1-2.

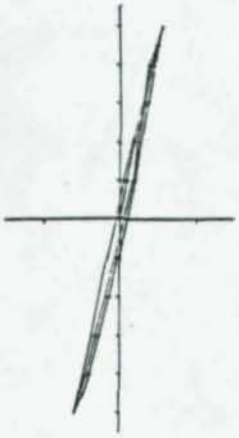
f) Test 7 : Load Cycle 3
Connection 3-4.

Figure 38 continued. Test 7: Connection Moments vs. Rotations.

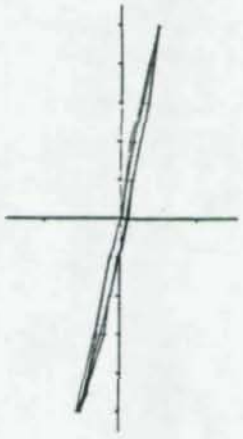
g) Test 7 : Load Cycle 4
Connection 1-2.



h) Test 7 : Load Cycle 4
Connection 3-4.

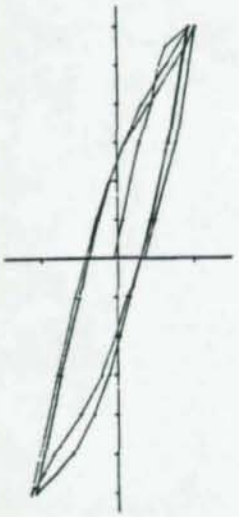


i) Test 7 : Load Cycle 5
Connection 1-2.

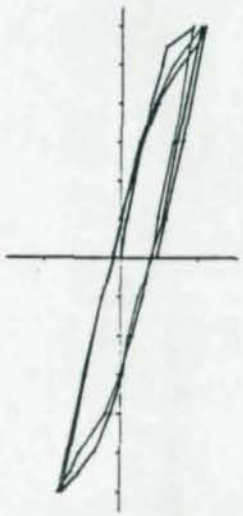


j) Test 7 : Load Cycle 5
Connection 3-4.

Figure 38 continued. Test 7: Connection Moments vs. Rotations.



k) Test 7 : Load Cycle 6
Connection 1-2.



l) Test 7 : Load Cycle 6
Connection 3-4.

Figure 38 continued. Test 7: Connection Moments vs. Rotations.

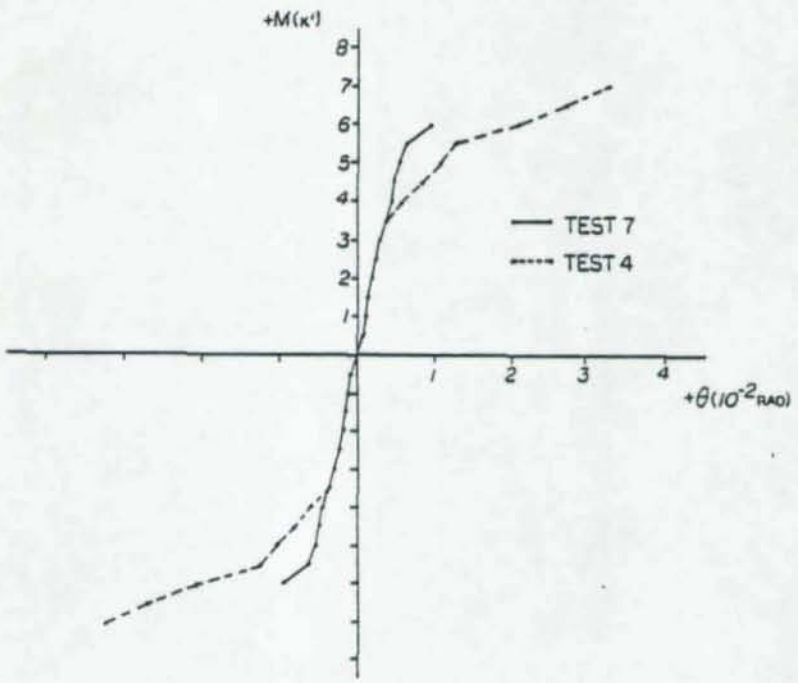


Figure 39. Test 7 : Initial Connection Loading Peaks Curve and Test 4 : Monotonic Moment-Rotation Curve.

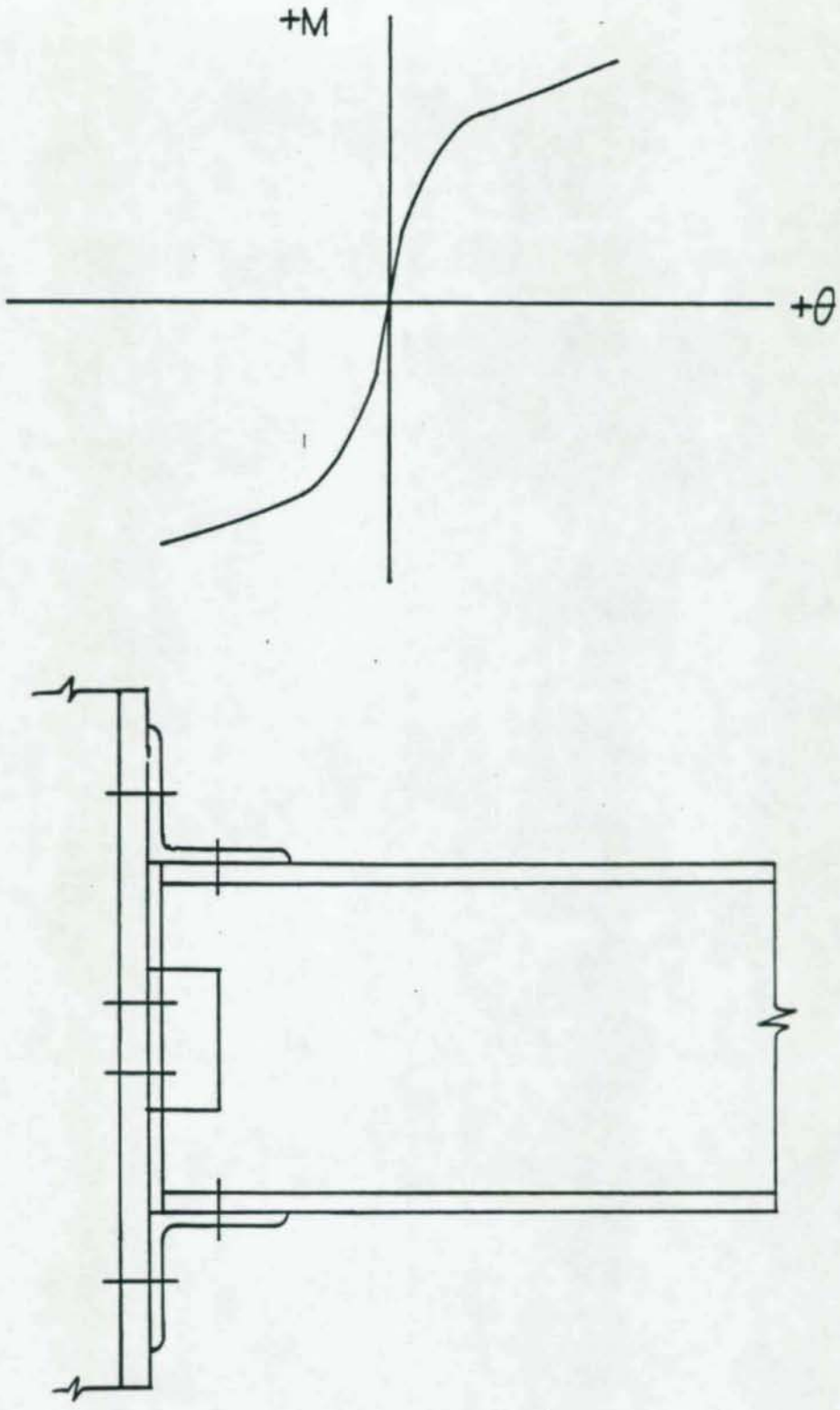
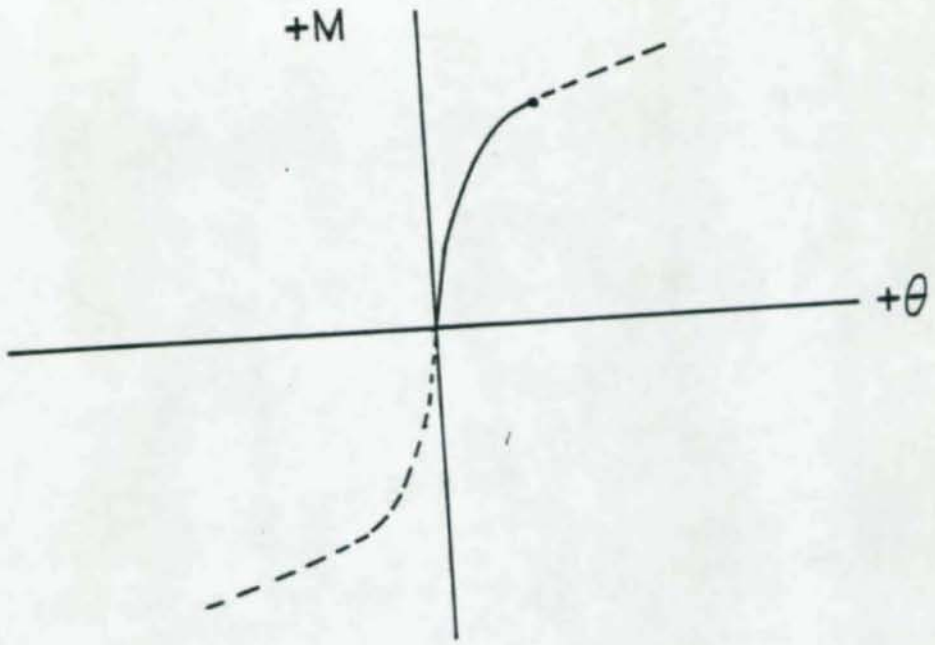


Figure 40. Moment-Rotation & Connection Deformation. Initial Conditions.



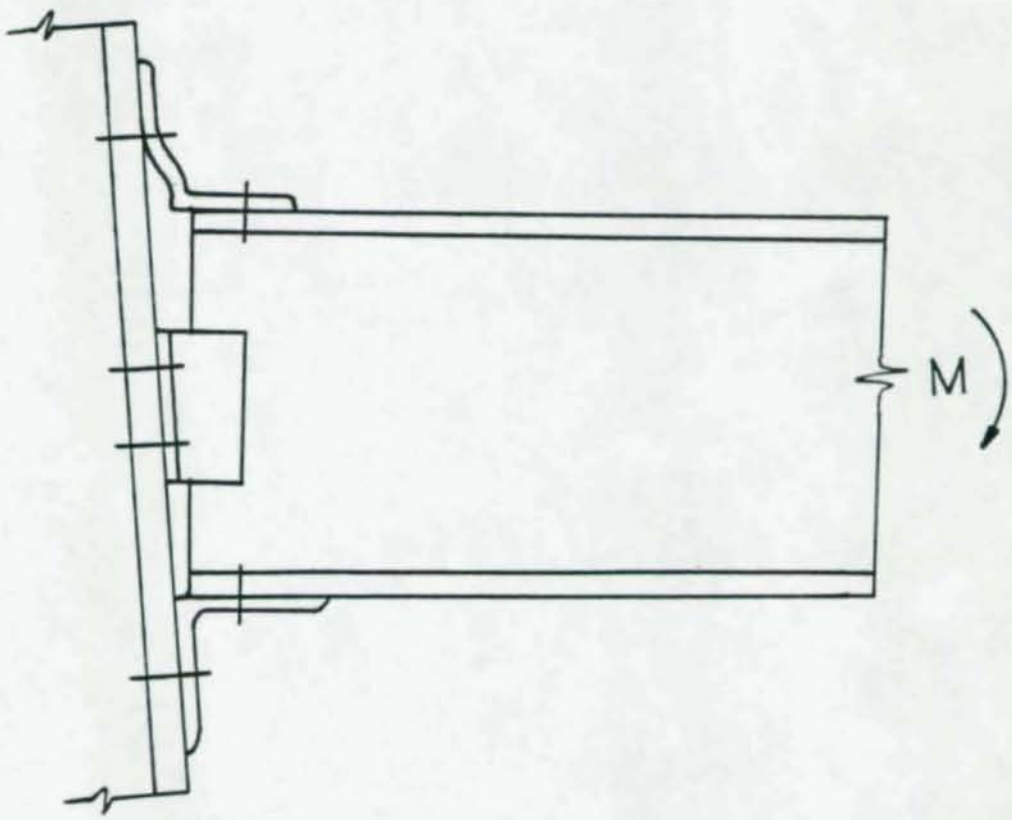


Figure 41. Moment-Rotation & Connection Deformation. Positive Loading.

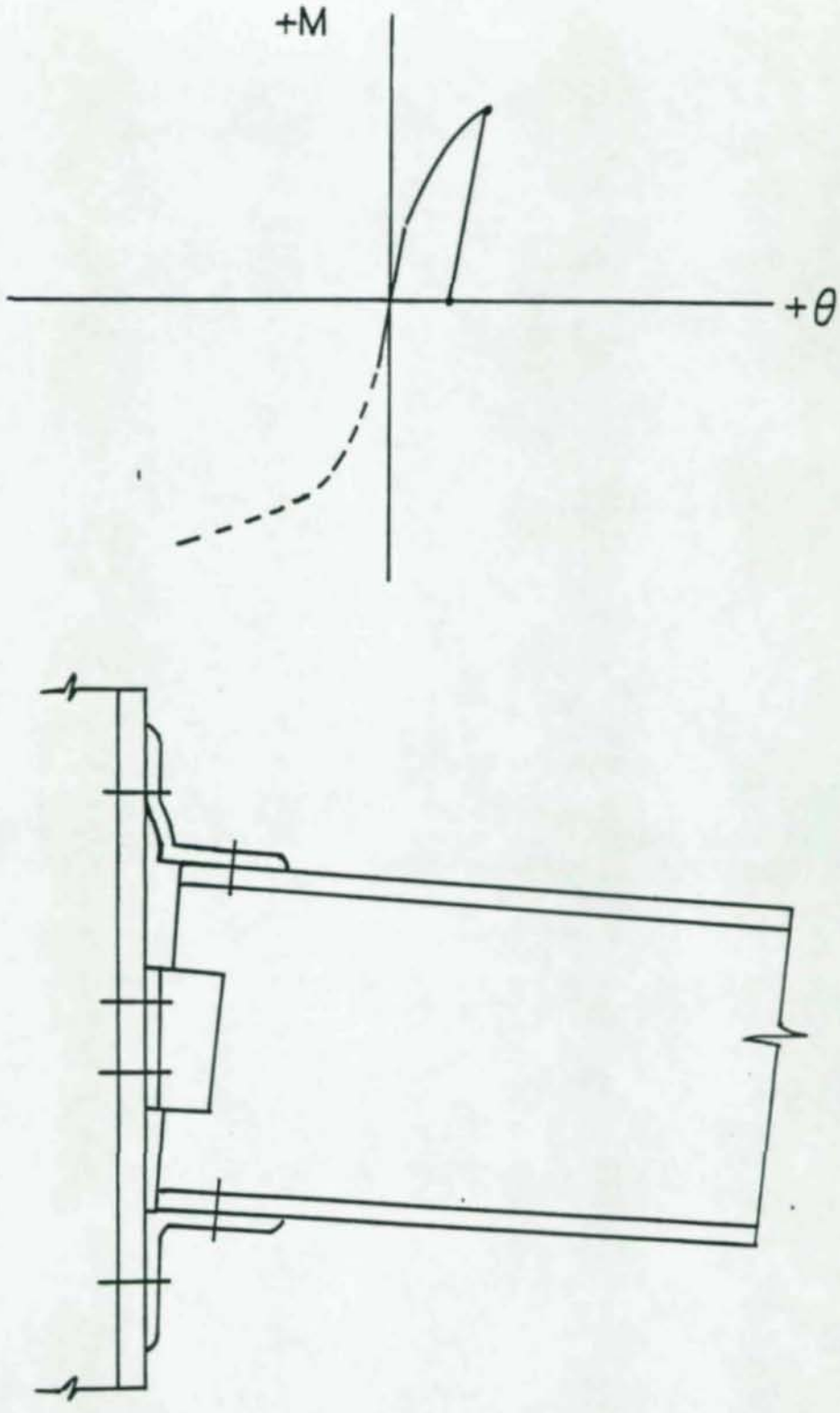


Figure 42. Moment-Rotation & Connection Deformation. Positive Unloading.

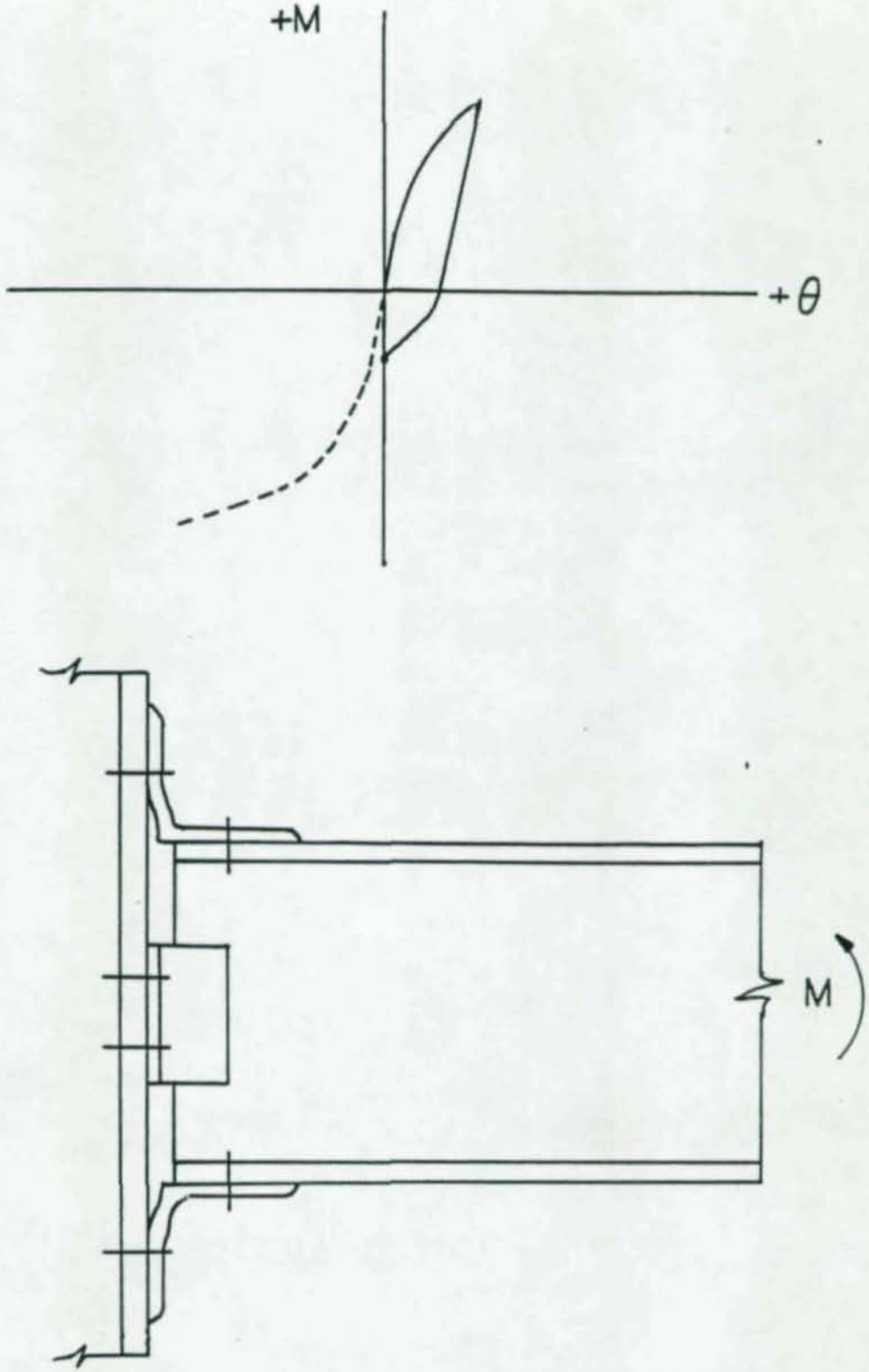


Figure 43. Moment-Rotation & Connection Deformation. Moment Reversal to Negative Loading.

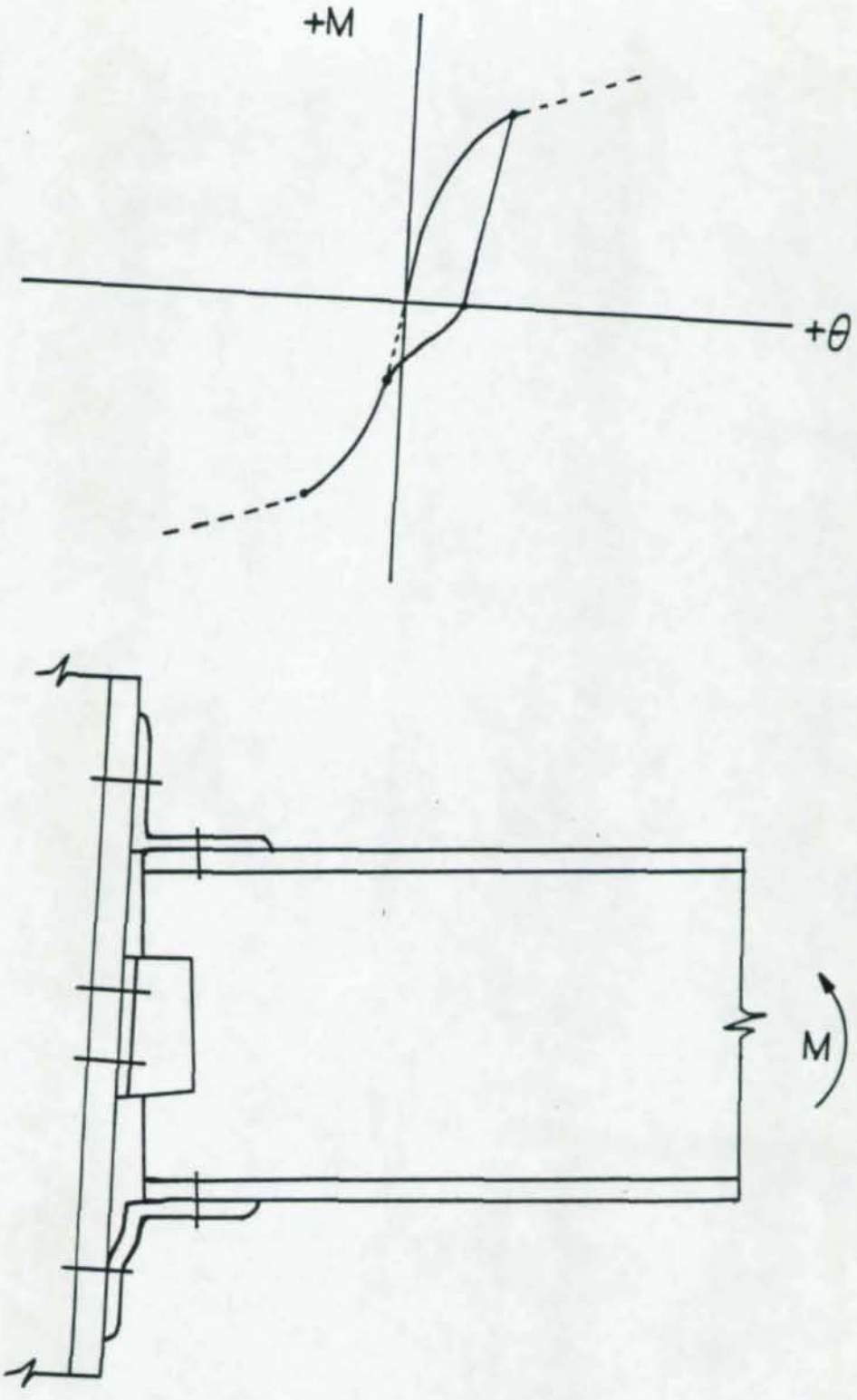
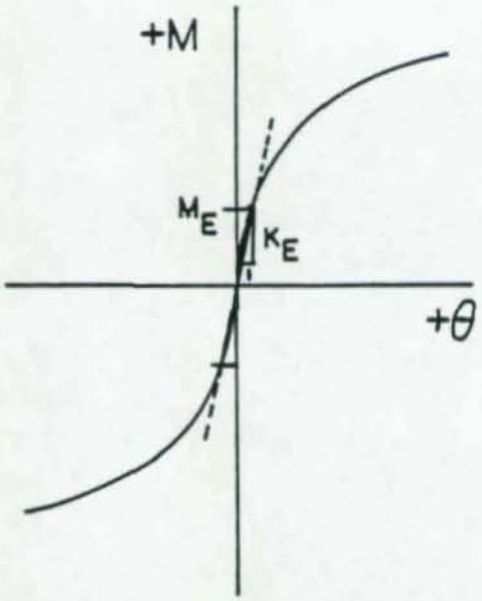
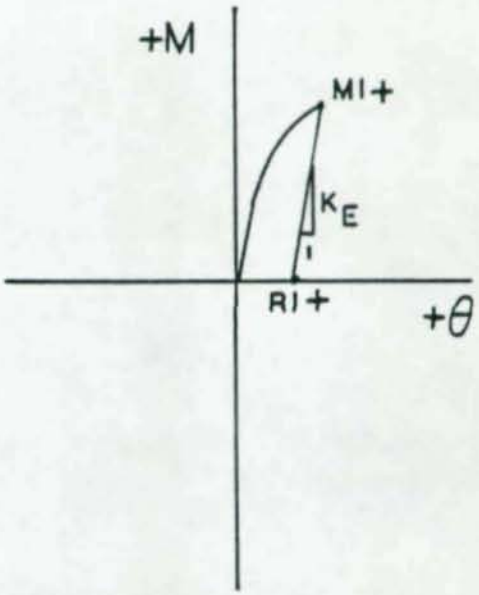


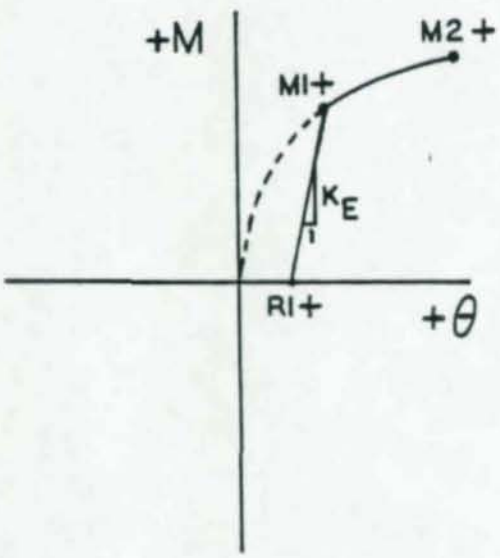
Figure 44. Moment-Rotation & Connection Deformation. Continued Negative Loading.



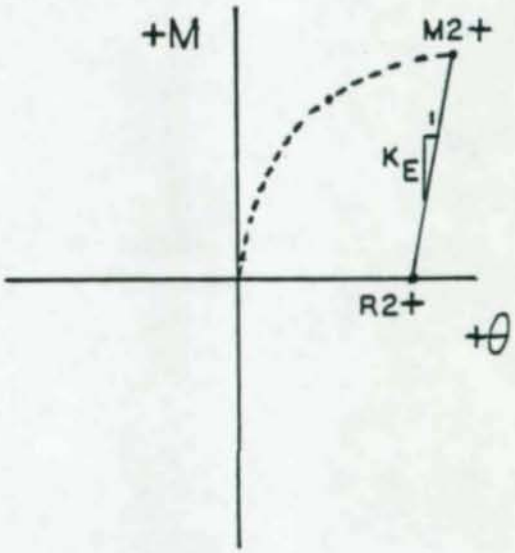
a) Initial Conditions.



b) Positive Loading to M1+ & Unloading to Zero at R1+.

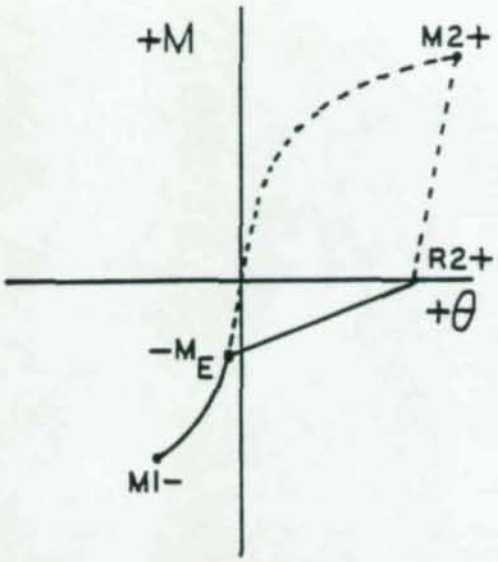


c) Positve Loading from Zero to M1+ and on to M2+.

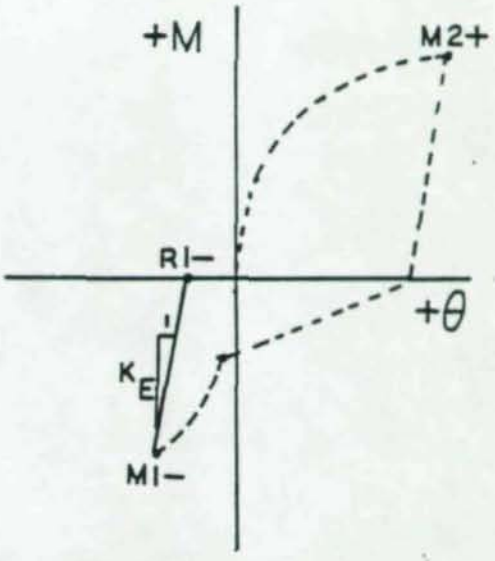


d) Positive Unloading from M2+ to Zero at R2+.

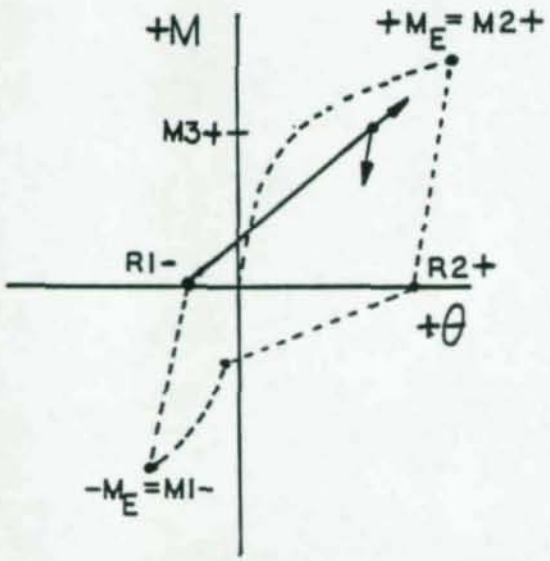
Figure 45. Basic Model Example.



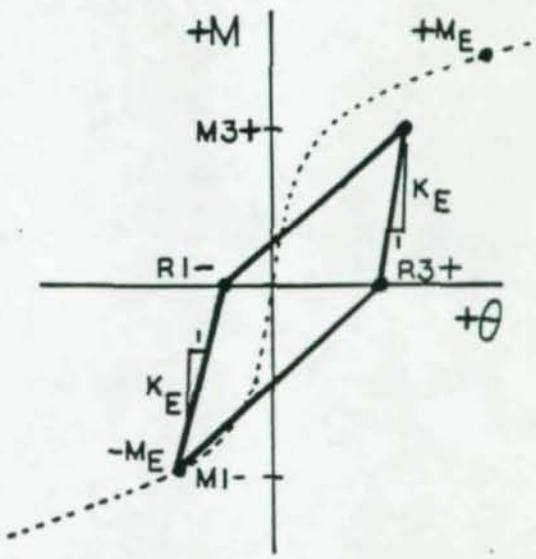
e) Negative Loading from Zero at R2+ to -Mei & on to M1-.



f) Negative Unloading from M1- to Zero at R1-.



g) Positive Reloading from Zero at R1- to M3+.



h) Positve Unloading from M3+ to Zero at R3+ & Moment- Rotation Hysteresis for M3+ to M1- Cycles.

Figure 45 continued. - Basic Model Example.

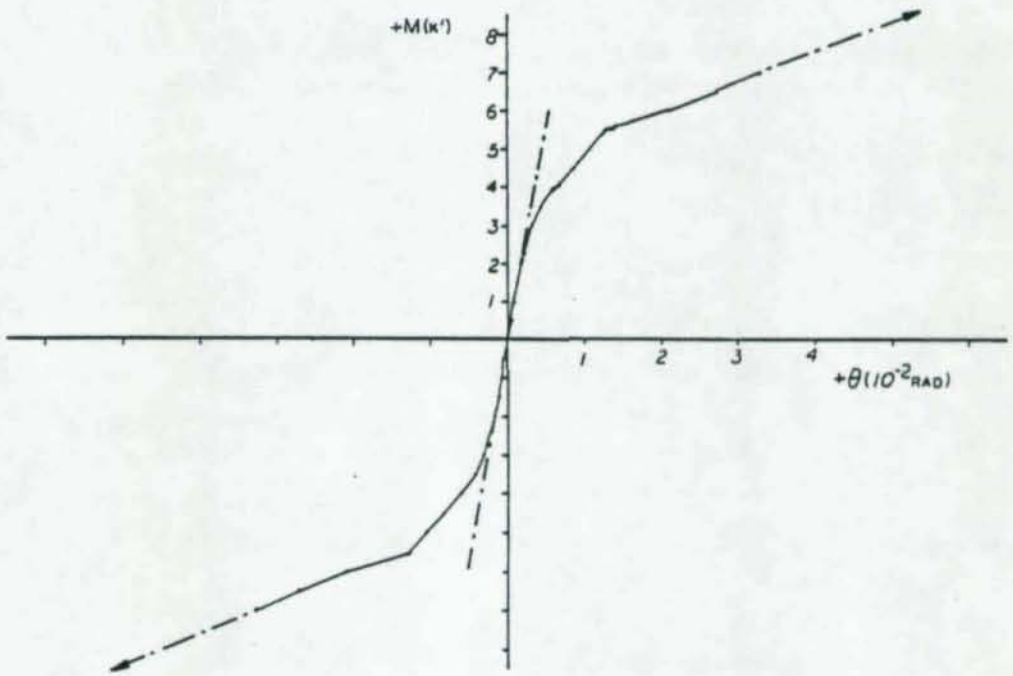
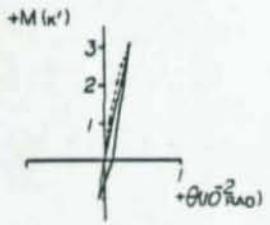
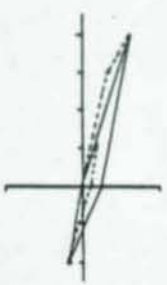


Figure 46. Projected Test 4 Monotonic Curve for Model Application.

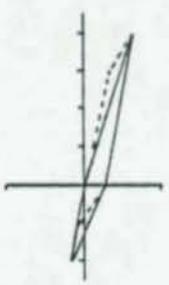


a) Test 2 : Load Cycle 3
Connection 1-2.

b) Test 2 : Load Cycle 3
Connection 3-4.

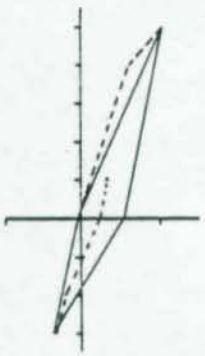


c) Test 2 : Load Cycle 4
Connection 1-2.

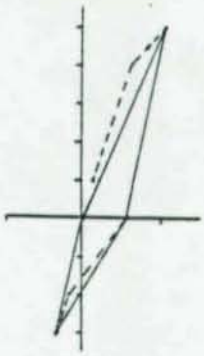


d) Test 2 : Load Cycle 4
Connection 3-4.

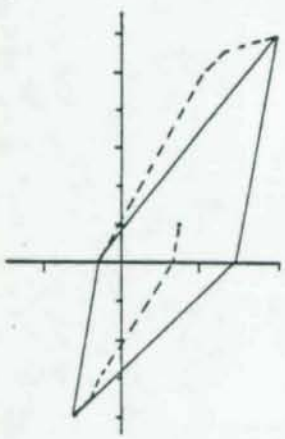
Figure 47. Model Generation of Test 2 Connection Curves Using Test 4 Monotonic Curve.



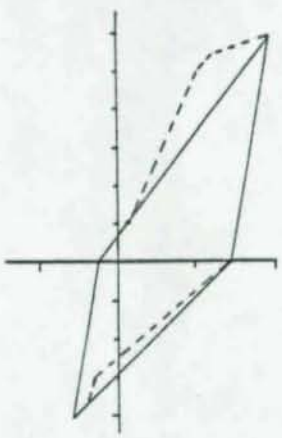
e) Test 2 : Load Cycle 5
Connection 1-2.



f) Test 2 : Load Cycle 5
Connection 3-4.

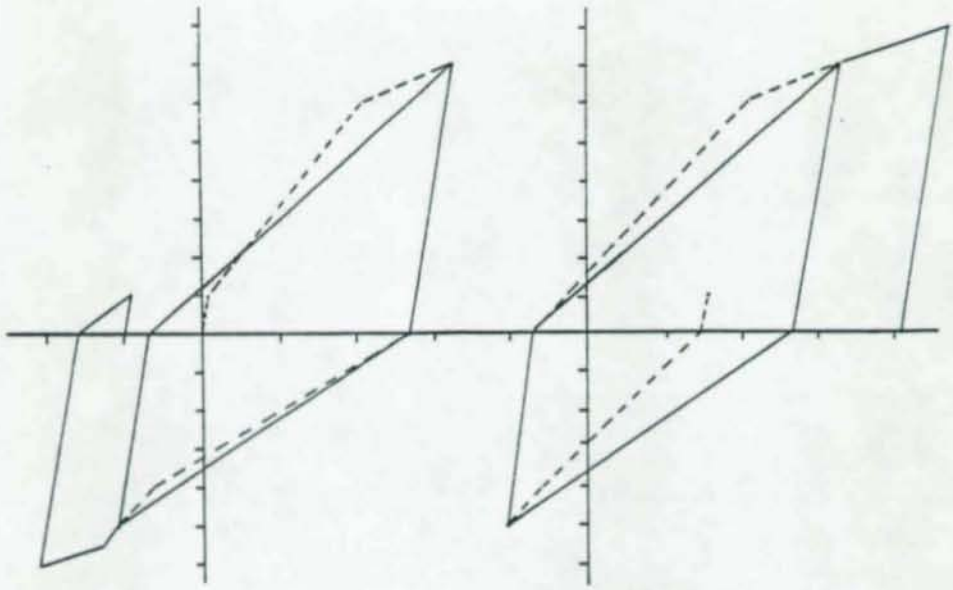


g) Test 2 : Load Cycle 6
Connection 1-2.



h) Test 2 : Load Cycle 6
Connection 3-4.

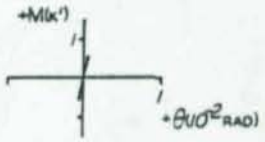
Figure 47 continued. Model Generation of Test 2 Connection Curves using Test 4 Monotonic Curve.



i) Test 2 : Load Cycles 7 & 8 Connection 1-2.

j) Test 2 : Load Cycles 7 & 8 Connection 3-4.

Figure 47 continued. Model Generation of Test 2 Connection Curves using Test 4 Monotonic Curve.

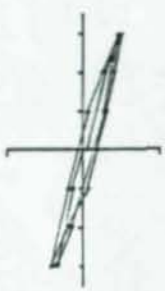


a) Test 2 : Load Cycle 1
Joint Moment-Rotation.

b) Test 2 : Load Cycle 2
Joint Moment-Rotation.

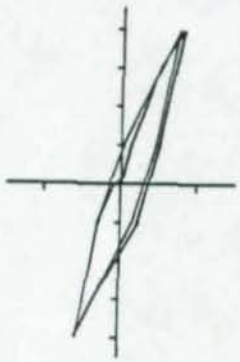
Figure 48. Model Generation of Test 2 Joint Moment-Rotation Hystereses using Test Monotonic Curve.

c) Test 2 : Load Cycle 3
Joint Moment-Rotation.

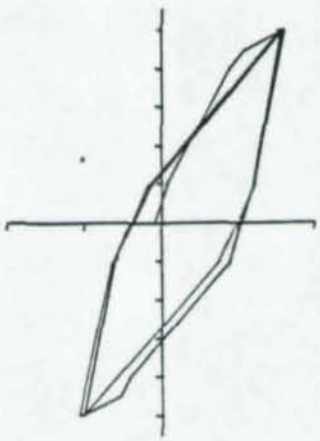


d) Test 2 : Load Cycle 4
Joint Moment-Rotation.

Figure 48 continued. Model Generation of Test 2 Joint Moment-Rotation Hystereses using Test Monotonic Curve.

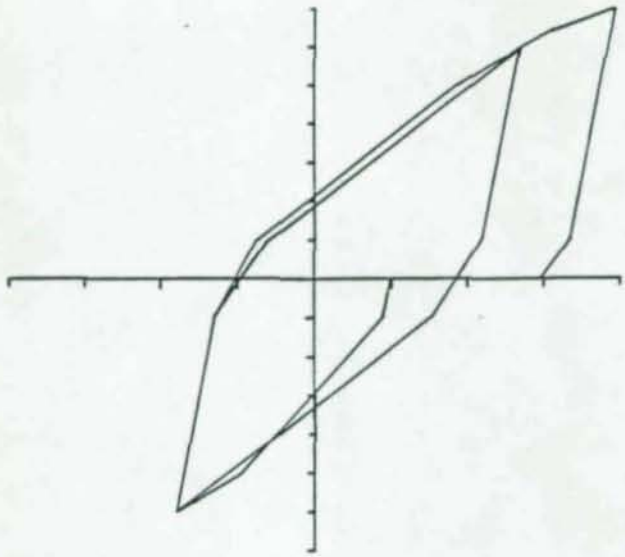


e) Test 2 : Load Cycle 5
Joint Moment-Rotation.



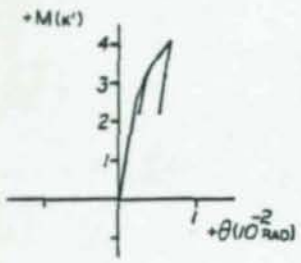
f) Test 2 : Load Cycle 6
Joint Moment-Rotation.

Figure 48 continued. Model Generation of Test 2 Joint Moment-Rotation Hystereses using Test Monotonic Curve.

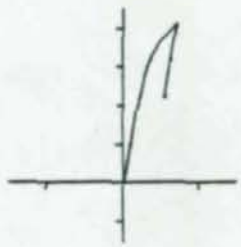


g) Test 2 : Load Cycles 7 & 8 Joint Moment-Rotation.

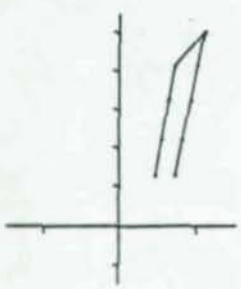
Figure 48 continued. Model Generation of Test 2 Joint Moment-Rotation Hystereses using Test Monotonic Curve.



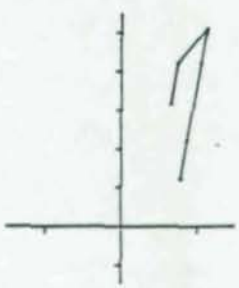
a) Test 3 : Load Cycle 1
Connection 1-2.



b) Test 3 : Load Cycle 1
Connection 3-4.

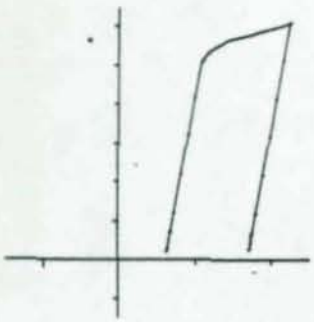


c) Test 3 : Load Cycle 2
Connection 1-2.

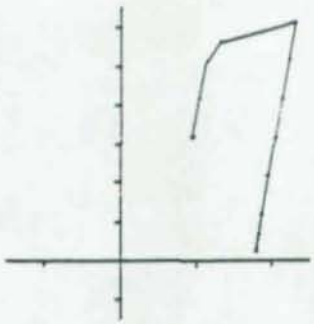


d) Test 3 : Load Cycle 2
Connection 3-4.

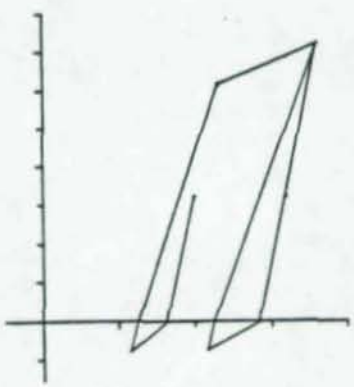
Figure 49. Model Generation of Test 3 Connection Curves using Test 4 Monotonic Curve.



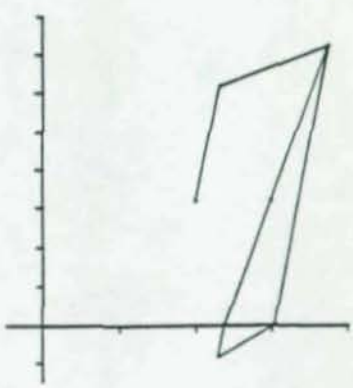
e) Test 3 : Load Cycle 3
Connection 1-2.



f) Test 3 : Load Cycle 3
Connection 3-4.

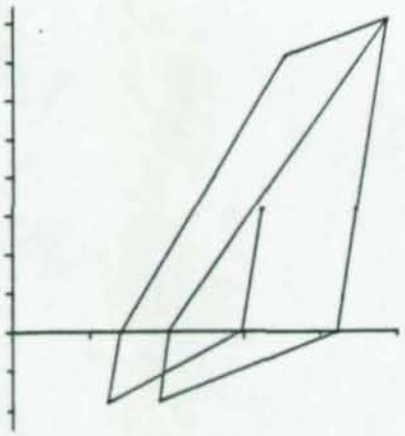


g) Test 3 : Load Cycle 4
Connection 1-2.

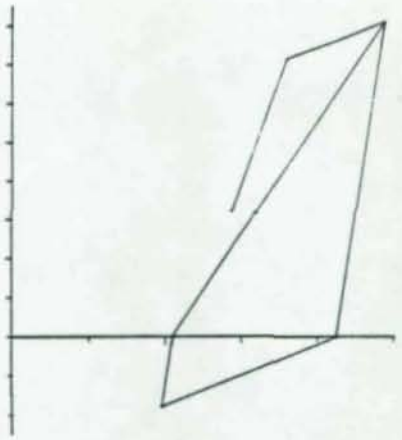


h) Test 3 : Load Cycle 4
Connection 3-4.

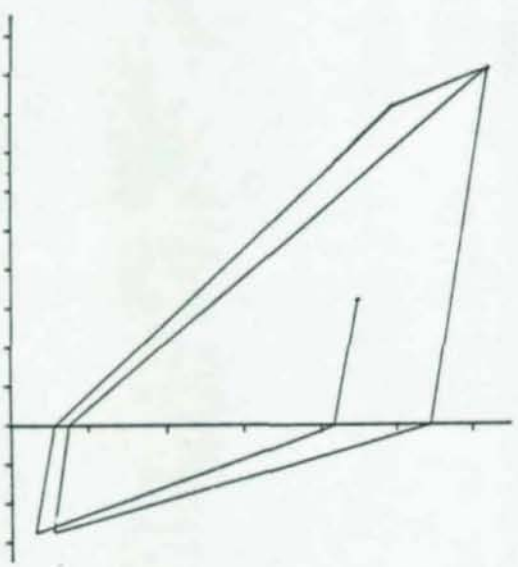
Figure 49 continued. Model Generation of Test 3 Connection Curves using Test 4 Monotonic Curve.



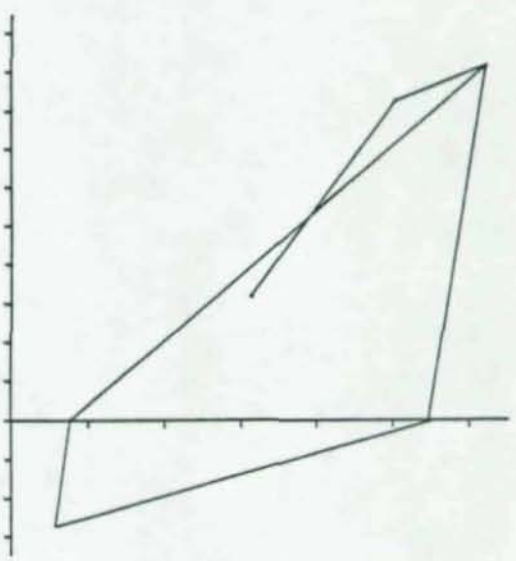
i) Test 3 : Load Cycle 5
Connection 1-2.



j) Test 3 : Load Cycle 5
Connection 3-4.

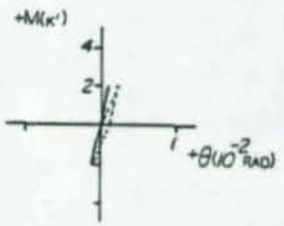


k) Test 3 : Load Cycle 6
Connection 1-2.

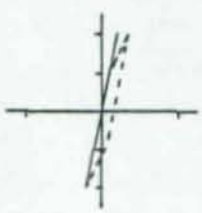


l) Test 3 : Load Cycle 6
Connection 3-4.

Figure 49 continued. Model Generation of Test 3 Connection Curves using Test 4 Monotonic Curve.

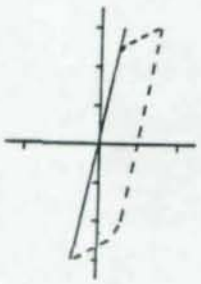


a) Test 3 : Load Cycle 1
Joint Moment-Rotation.

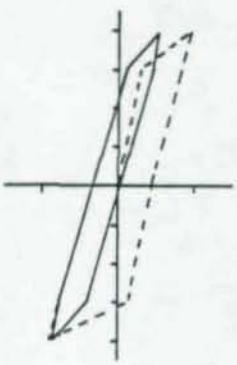


b) Test 3 : Load Cycle 2
Joint Moment-Rotation.

Figure 50. Model Generation of Test 3 Joint Moment-Rotation Hystereses using Test 4 Monotonic Curve.

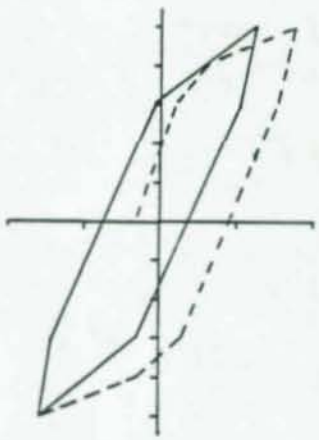


c) Test 3 : Load Cycle 3
Joint Moment-Rotation.

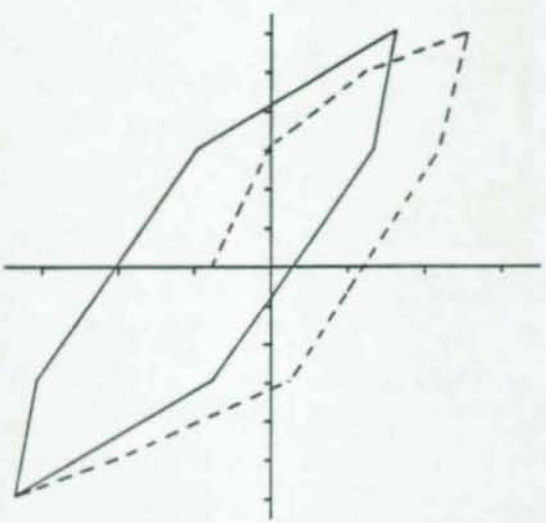


d) Test 3 : Load Cycle 4
Joint Moment-Rotation.

Figure 50 continued. Model Generation of Test 3 Joint Moment-Rotation Hystereses using Test 4 Monotonic Curve.

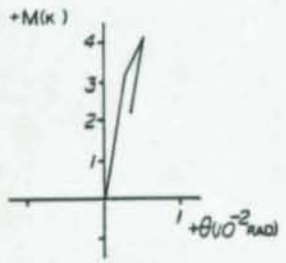


e) Test 3 : Load Cycle 5
Joint Moment-Rotation.

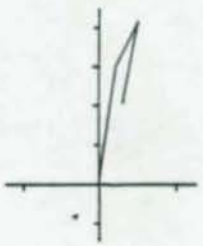


f) Test 3 : Load Cycle 6
Joint Moment-Rotation.

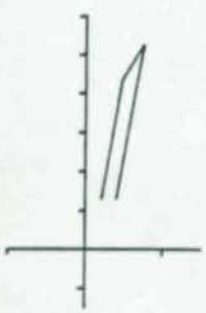
Figure 50 continued. Model Generation of Test 3 Joint Moment-Rotation Hystereses using Test 4 Monotonic Curve.



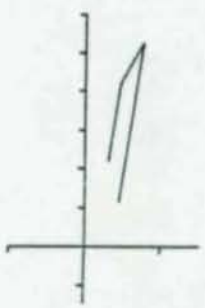
a) Test 3 : Load Cycle 1
Connection 1-2.



b) Test 3 : Load Cycle 1
Connection 3-4.

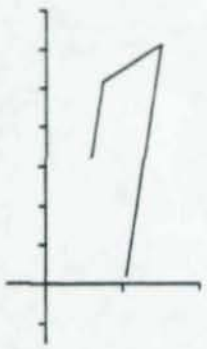


c) Test 3 : Load Cycle 2
Connection 1-2.

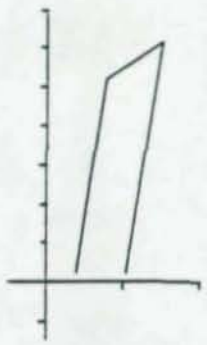


d) Test 3 : Load Cycle 2
Connection 3-4.

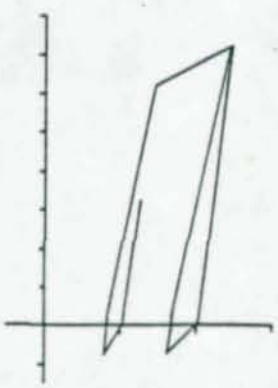
Figure 51. Model Generation of Test 3 Connection Curves using Test 3 Initial Connection Loading Peaks Curve.



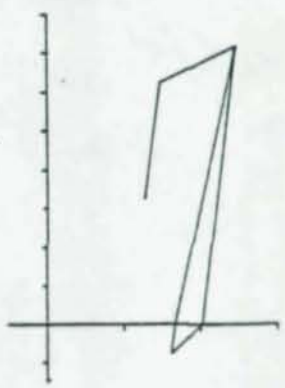
e) Test 3 : Load Cycle 3
Connection 1-2.



f) Test 3 : Load Cycle 3
Connection 3-4.

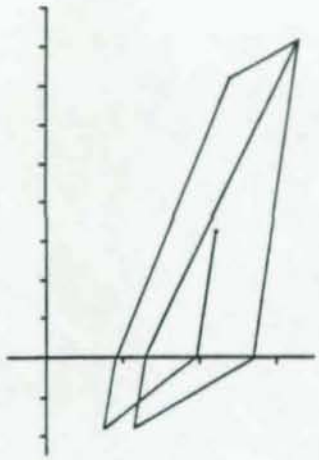


g) Test 3 : Load Cycle 4
Connection 1-2.

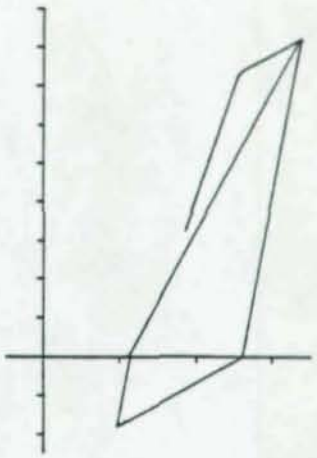


h) Test 3 : Load Cycle 4
Connection 3-4.

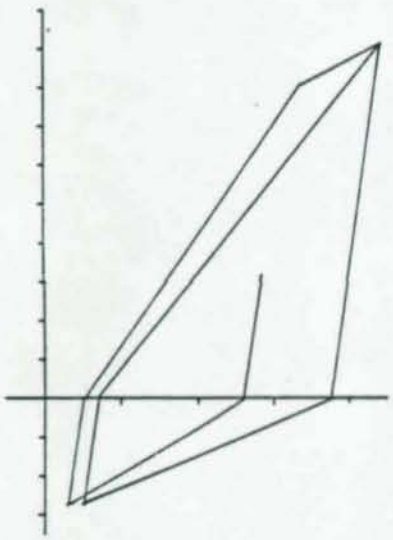
Figure 51 continued. Model Generation of Test 3 Connection Curves using Test 3 Initial Connection Loading Peaks Curve.



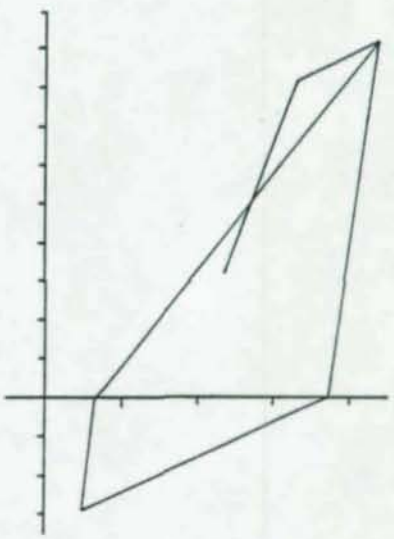
i) Test 3 : Load Cycle 5
Connection 1-2.



j) Test 3 : Load Cycle 5
Connection 3-4.

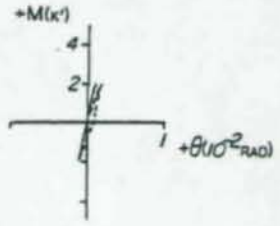


k) Test 3 : Load Cycle 6
Connection 1-2.



l) Test 3 : Load Cycle 6
Connection 3-4.

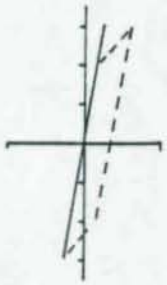
Figure 51 continued. Model Generation of Test 3 Connection Curves using Test 3 Initial Connection Loading Peaks Curve.



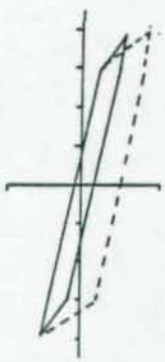
a) Test 3 : Load Cycle 1
Joint Moment-Rotation.

b) Test 3 : Load Cycle 2
Joint Moment-Rotation.

Figure 52. Model Generation of Test 3 Joint Moment-Rotation Hystereses using Test 3 Initial Peaks Curve.

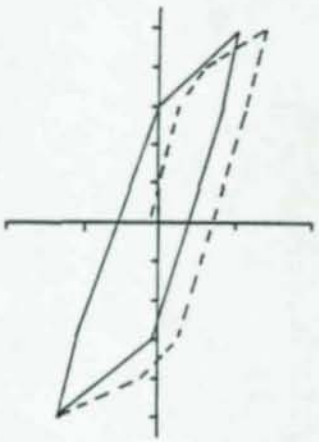


c) Test 3 : Load Cycle 3
Joint Moment-Rotation.

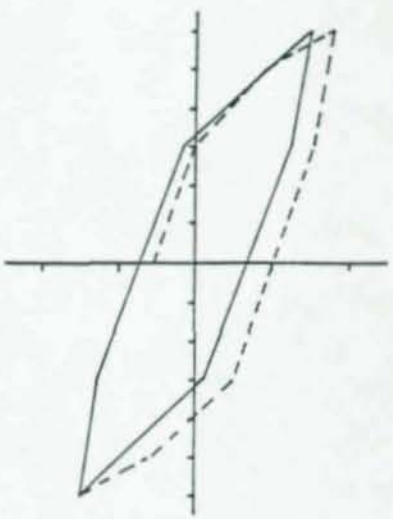


d) Test 3 : Load Cycle 4
Joint Moment-Rotation.

Figure 52 continued. Model Generation of Test 3 Joint Moment-Rotation Hystereses using Test 3 Initial Peaks Curve.



e) Test 3 : Load Cycle 5
Joint Moment-Rotation.



f) Test 3 : Load Cycle 6
Joint Moment-Rotation.

Figure 52 continued. Model Generation of Test 3 Joint Moment-Rotation Hystereses using Test 3 Initial Peaks Curve.

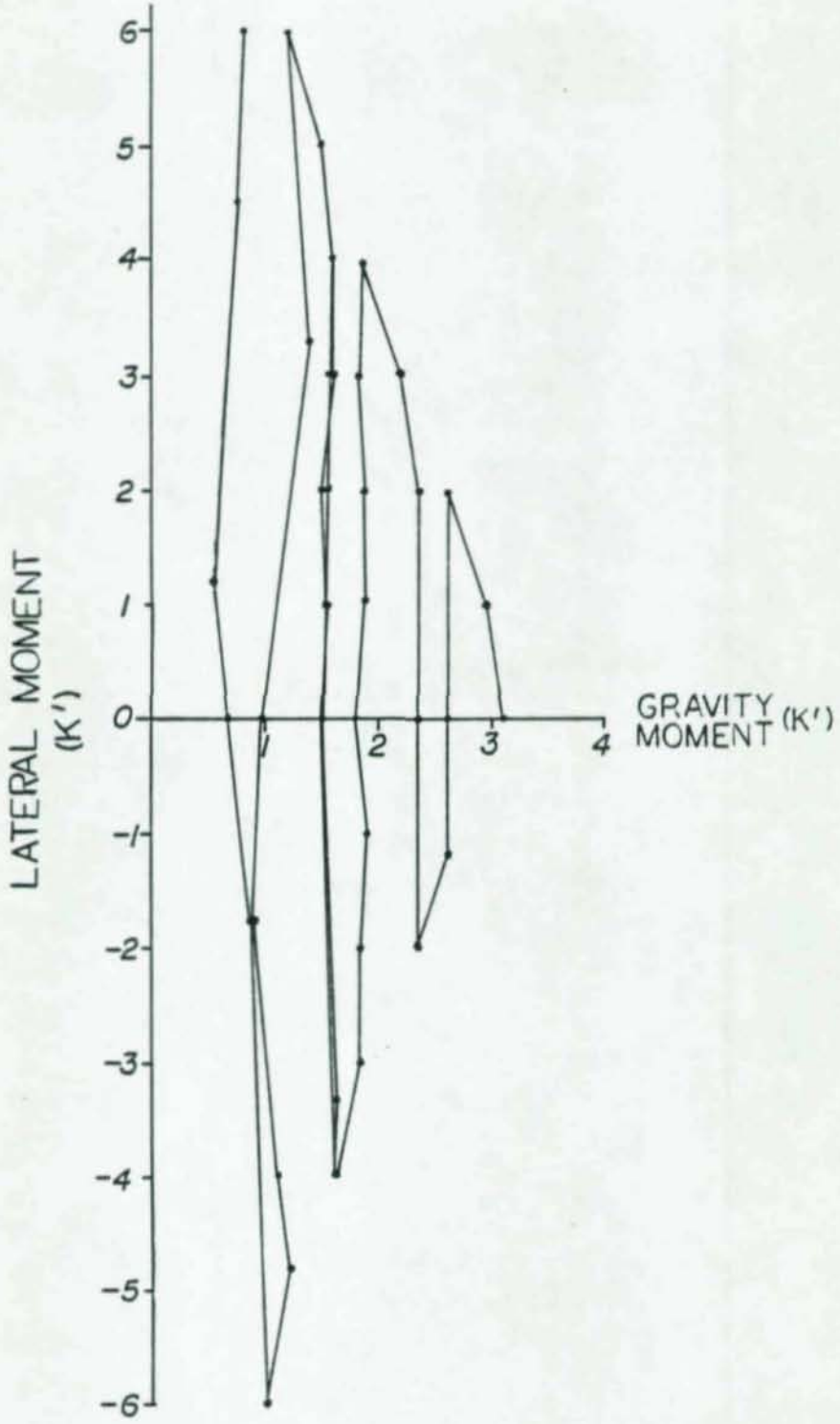


Figure 53. Model Generation of Test 5 Gravity Moment Shakedown using Test 4 Monotonic Curve.

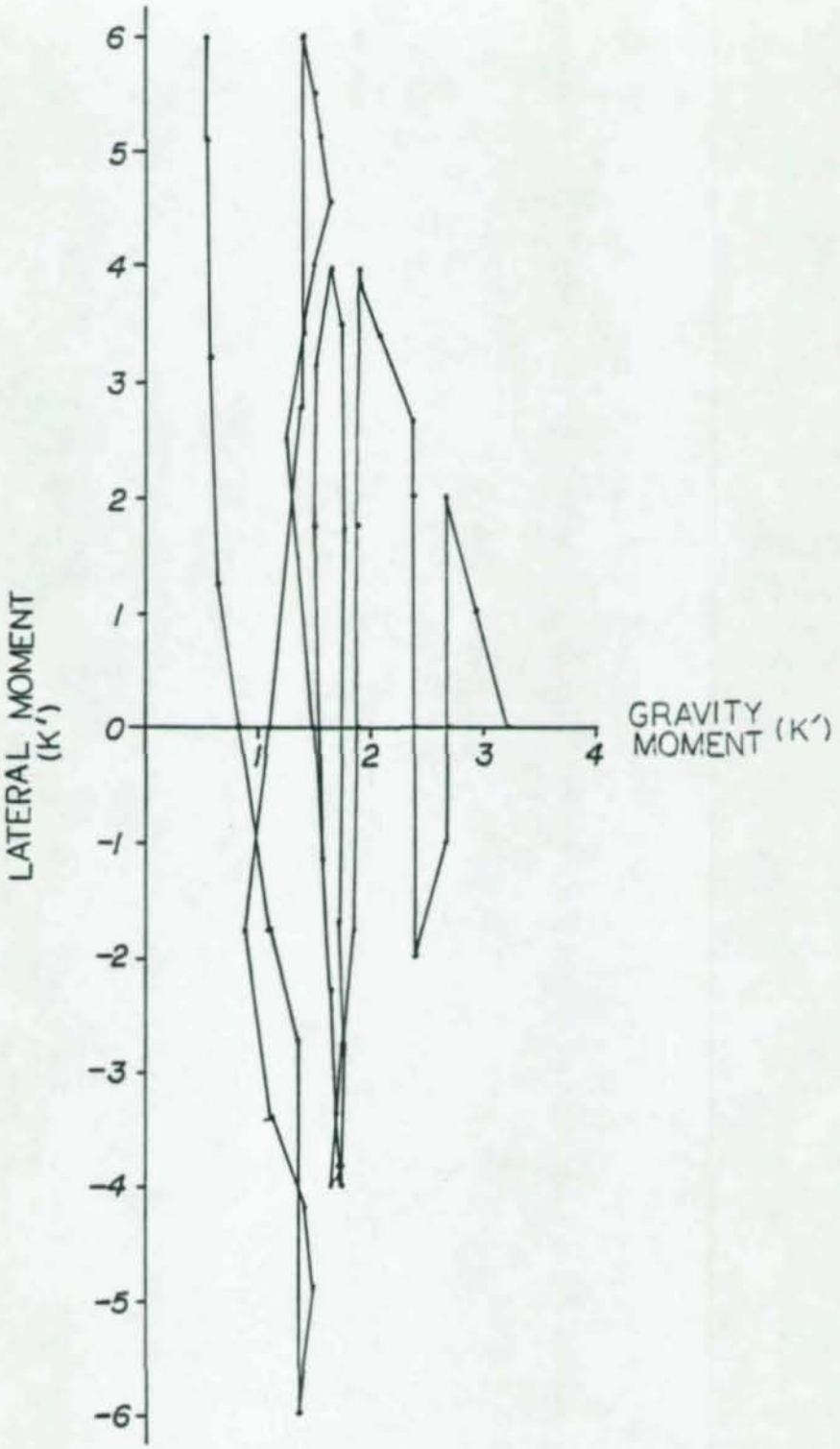
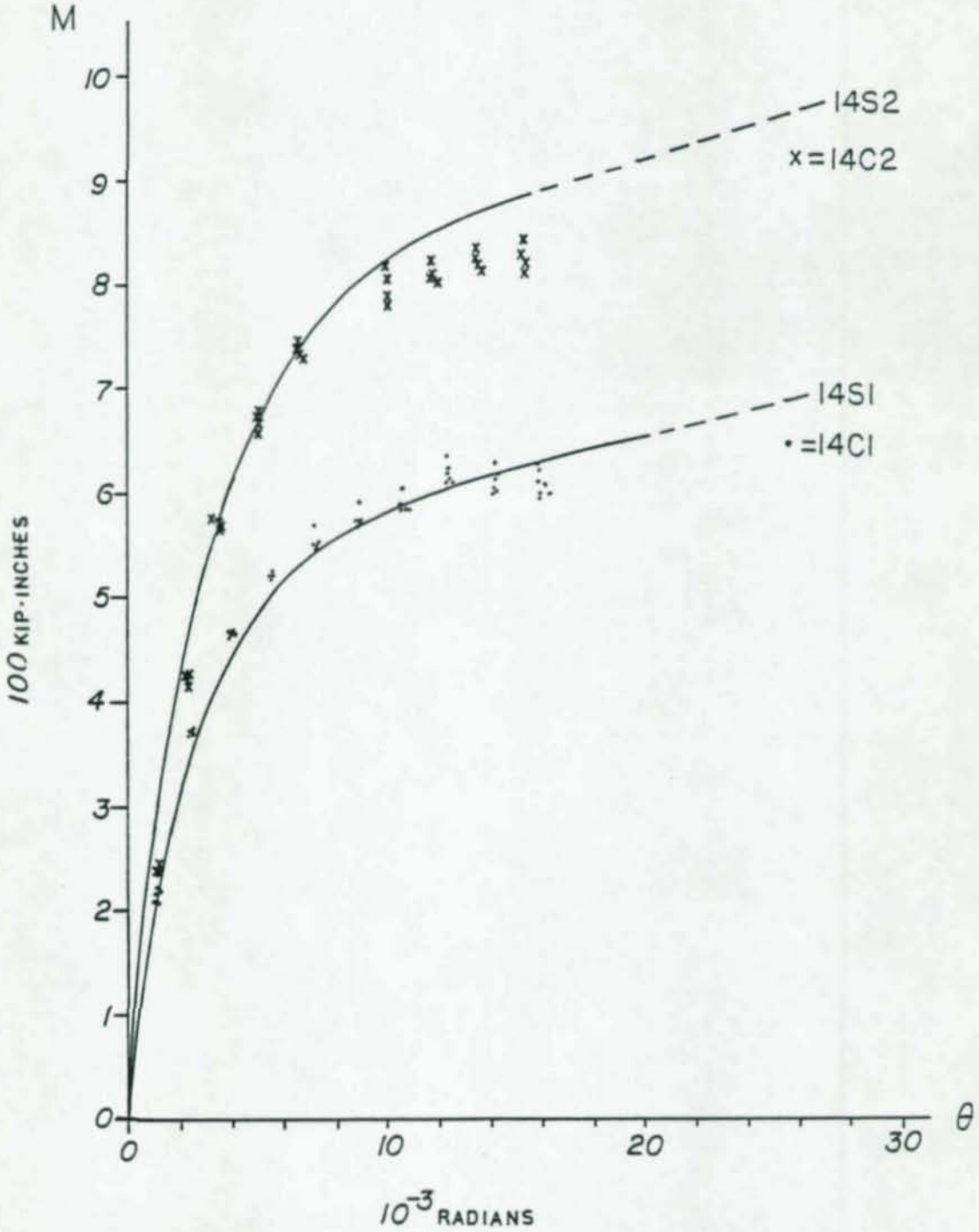
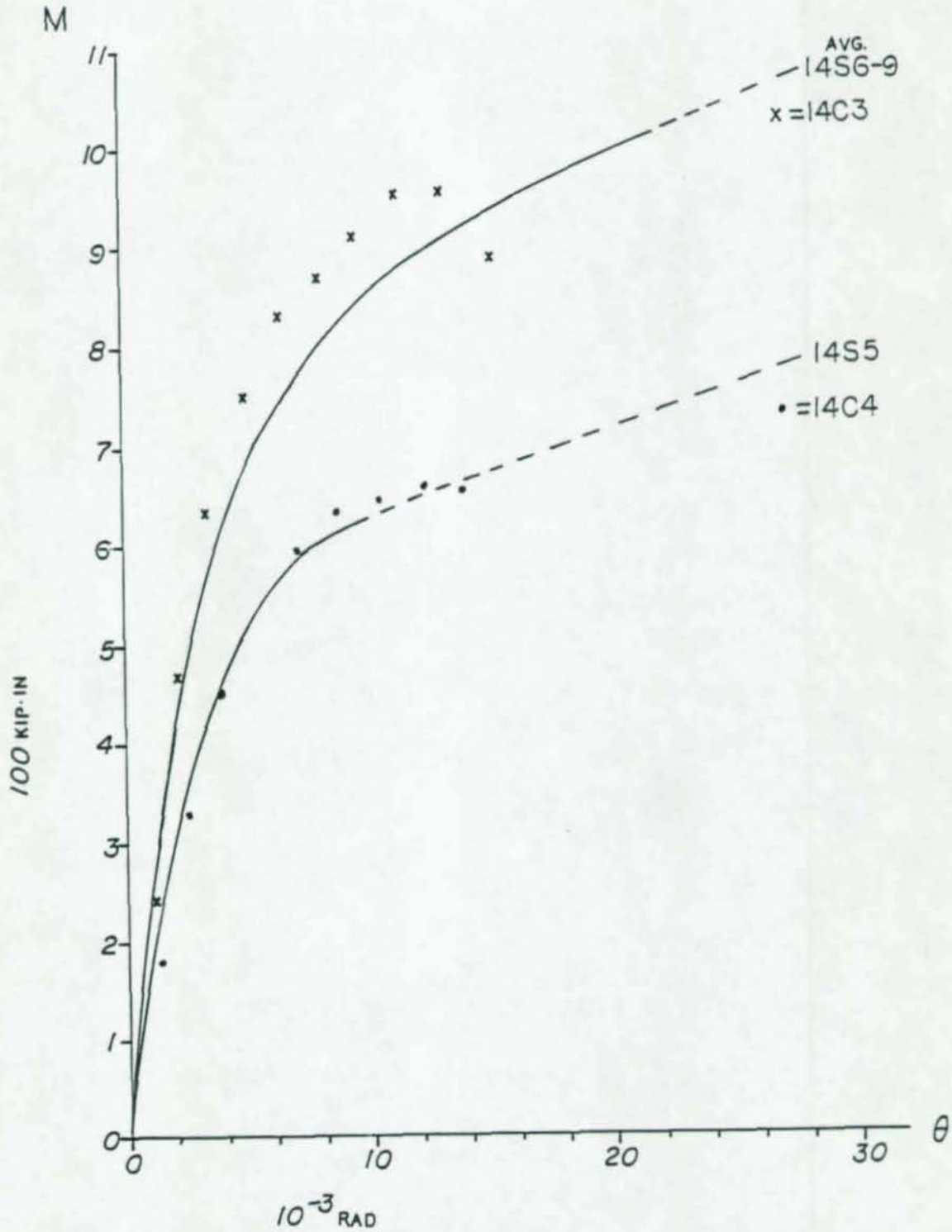


Figure 54. Model Generation of Test 5 Gravity Moment Shakedown using Test 5 Initial Connection Loading Peaks Curve.



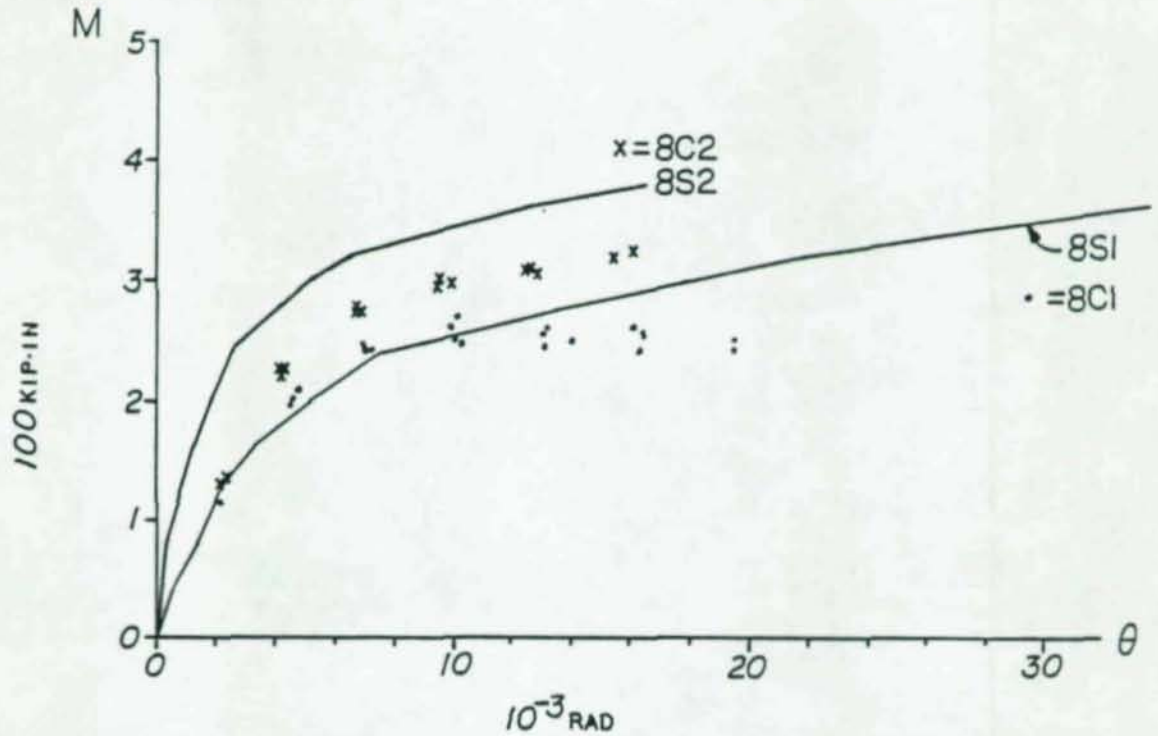
a) Static: 14S1&2 Cyclic: 14C1&2

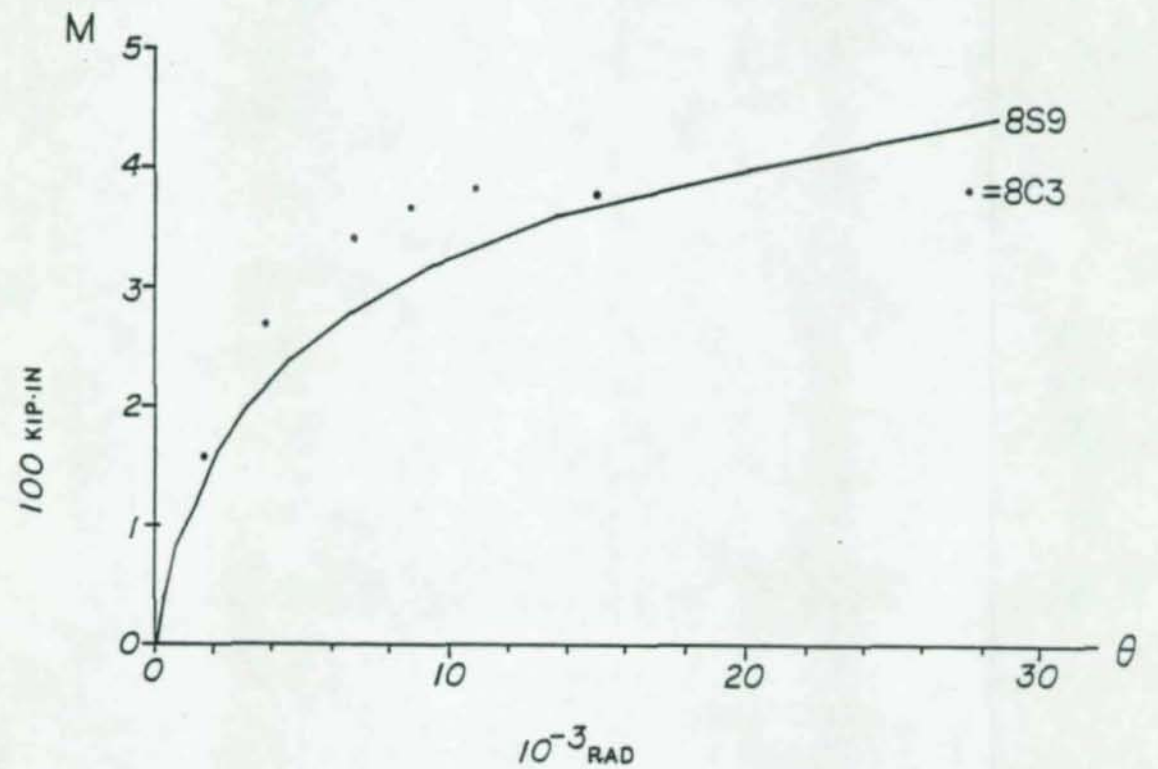
Figure 55. South Carolina Test Data (Ref. 14): Monotonic Tests (S) and Corresponding Cyclic Test Peaks (C).



b) Static: 14S5 & Avg.6&9 Cyclic: 14C3&4

Figure 55 continued. South Carolina Test Data (Ref. 14): Monotonic Tests (S) and Corresponding Cyclic Test Peaks (C).





c) Static: 8S1,2,&9 Cylclic:8C1,2,&3

Figure 55 continued. South Carolina Test Data (Ref. 14): Monotonic Tests (S) and Corresponding Cyclic Test Peaks (C).

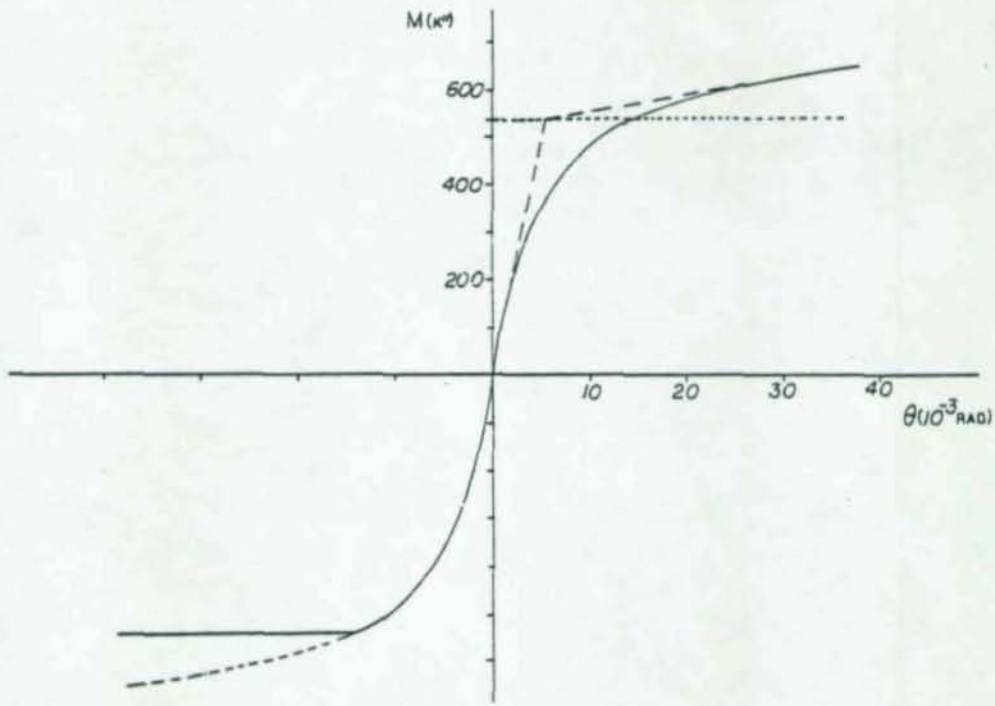


Figure 56. Modified 14S1 Loading Curve for Model Generation of Test 14C1.

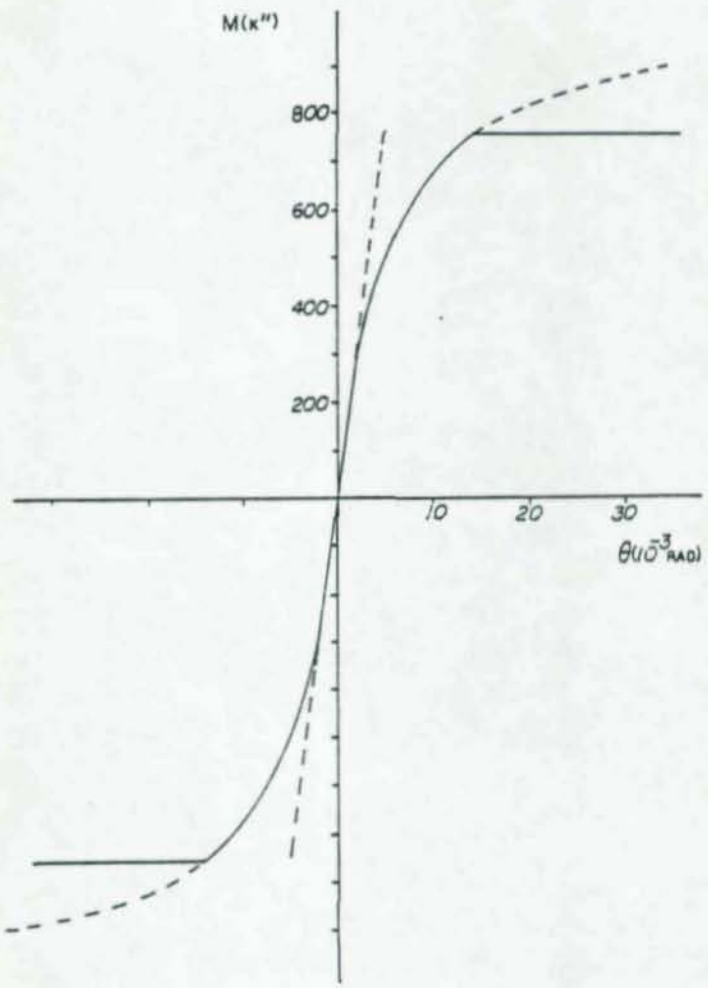


Figure 57. Modified 14S2 Loading Curve for Model Generation of Test 14C2.

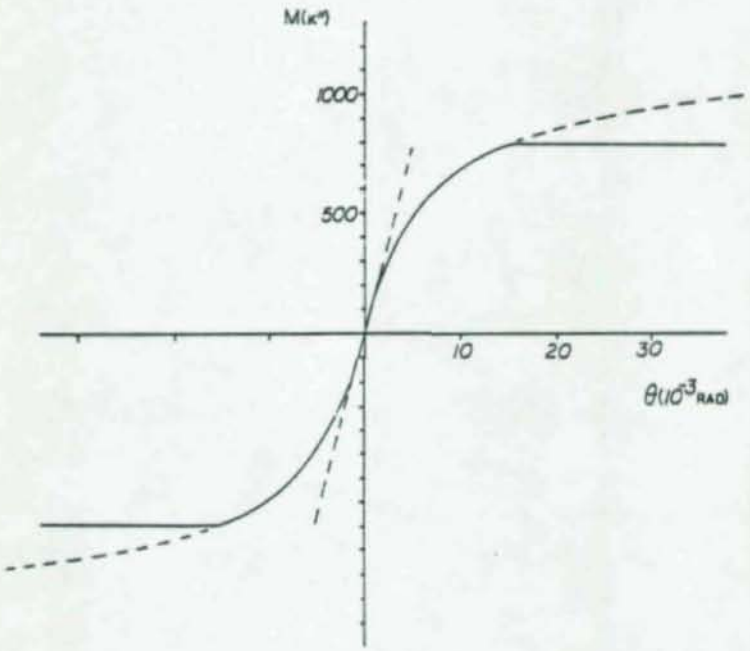


Figure 58. Modified 14S69 avg. Loading Curve for Model Generation of Test 14C3.

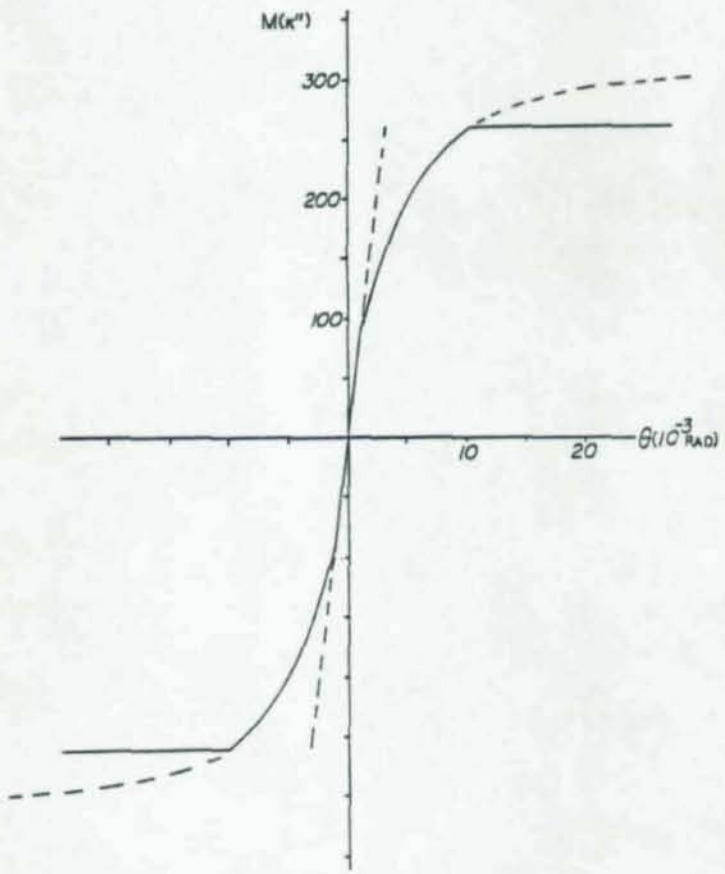


Figure 59. Modified 8S1 Loading Curve for Model Generation of Test 8C1.

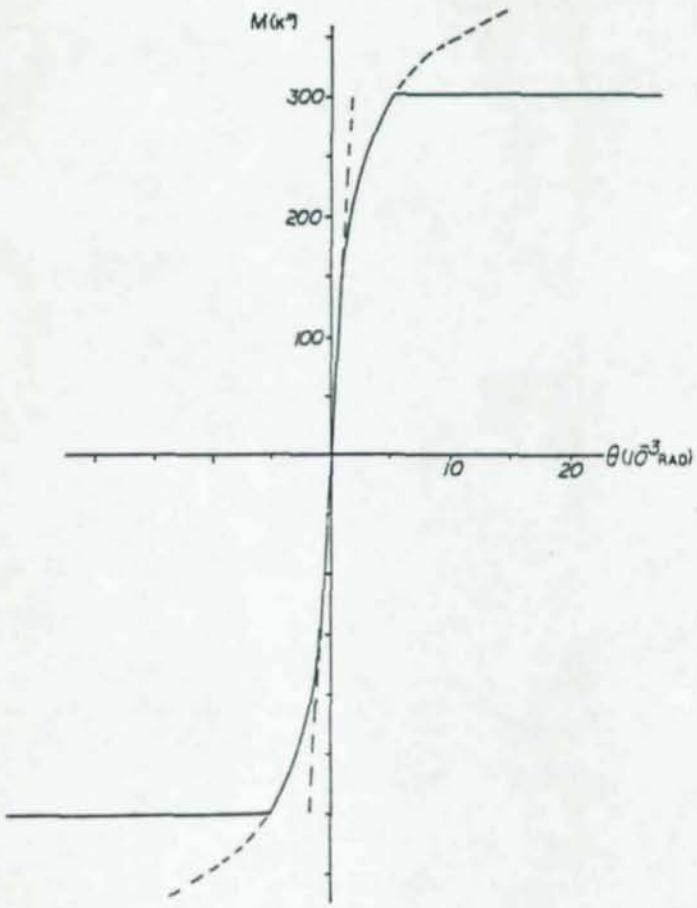
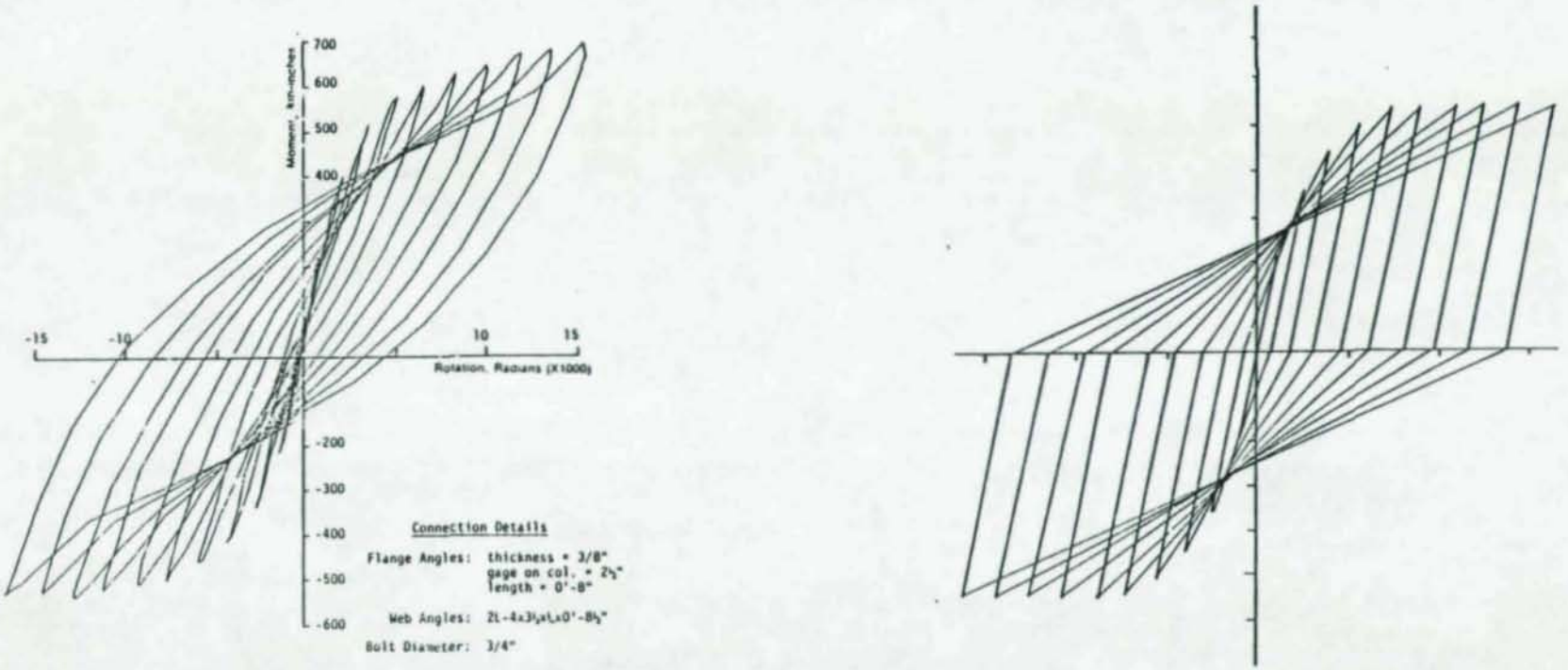
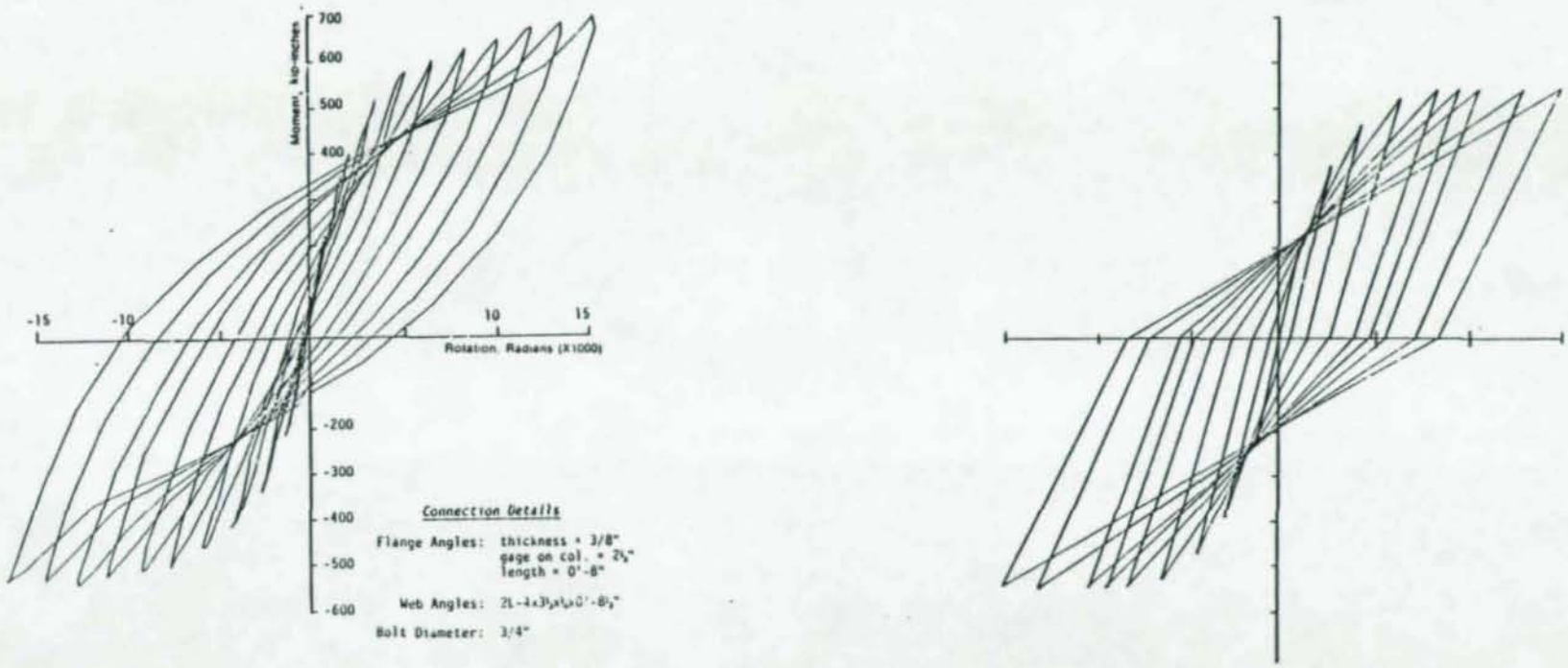


Figure 60. Modified 8S2 Loading Curve for Model Generation of Test 8C2.



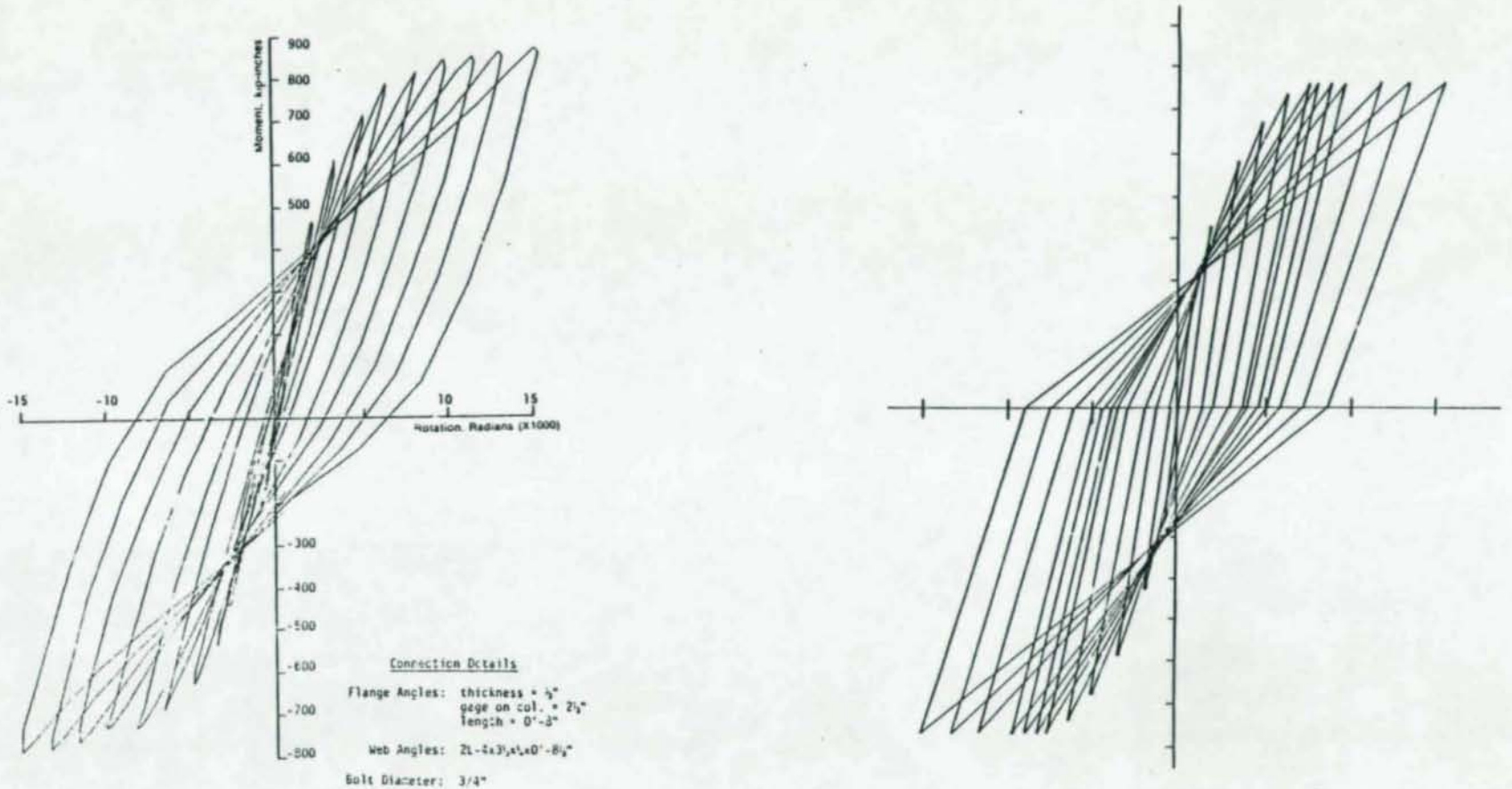
STABLE HYSTERESIS LOOPS FOR SPECIMEN 14C1

Figure 61. South Carolina Test 14C1 (Ref. 14) & Model Generated Results Using Initial Elastic Unloading Stiffness.



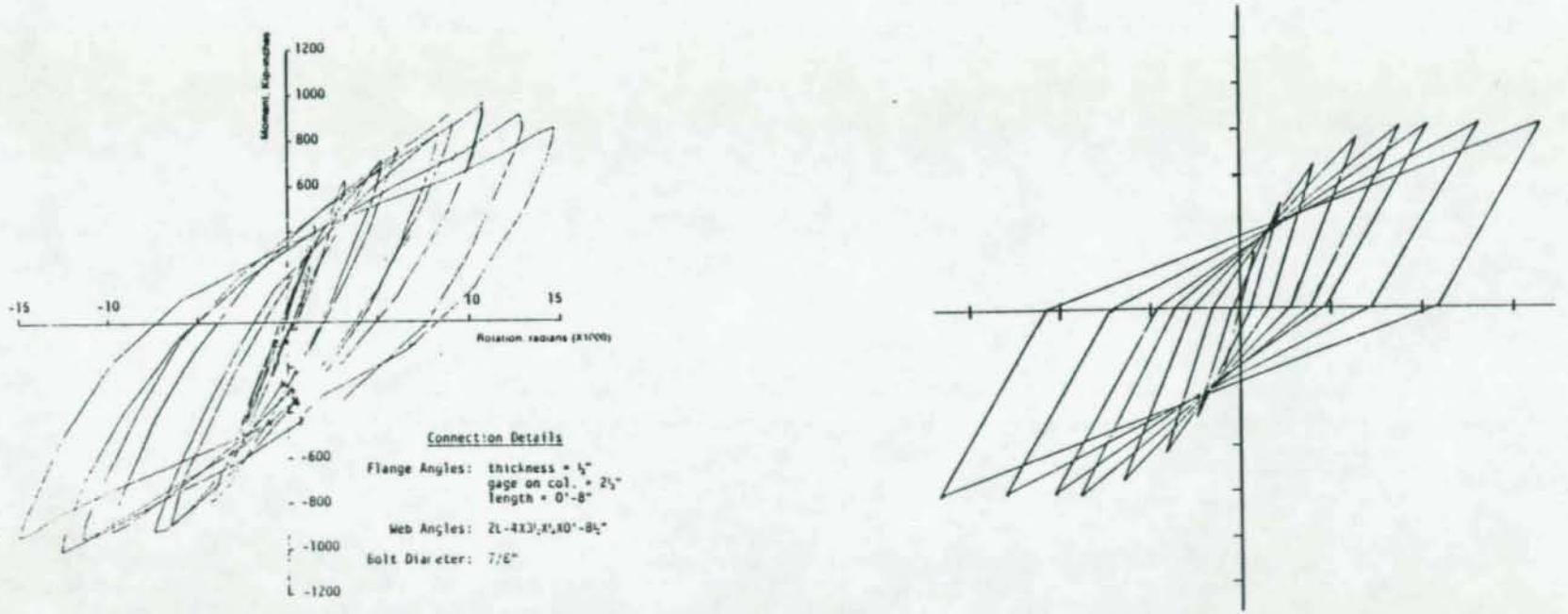
STABLE HYSTERESIS LOOPS FOR SPECIMEN 14C1

Figure 62. South Carolina Test 14C1 (Ref. 14) & Model Generated Results Using Unloading Stiffness Reduction (Degenerative Model).



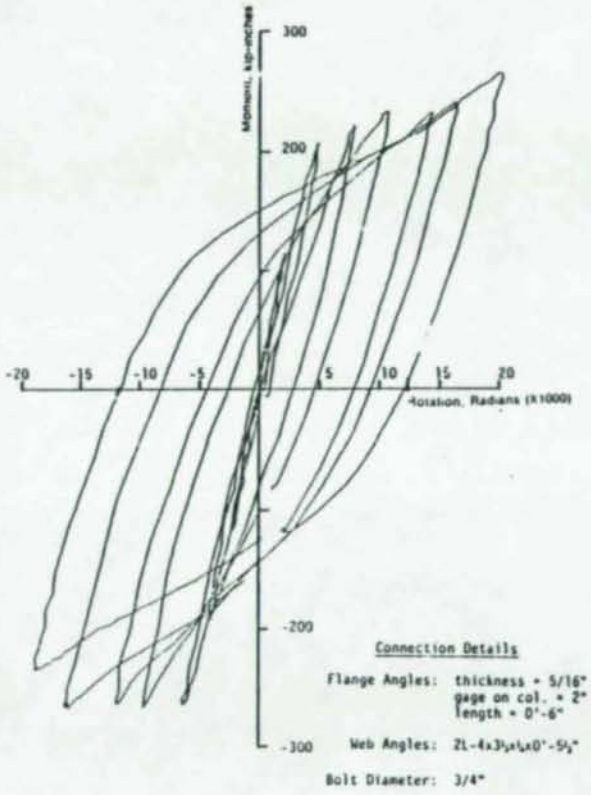
STABLE HYSTERESIS LOOPS FOR SPECIMEN 14C2

Figure 63. South Carolina Test 14C2 (Ref. 14) & Model Generated Results Using Unloading Stiffness Reduction (Degenerative Model).



STABLE HYSTERESIS LOOPS FOR SPECIMEN 14C3

Figure 64. South Carolina Test 14C3 (Ref. 14) & Model Generated Results Using Unloading Stiffness Reduction (Degenerative Model).



STABLE HYSTERESIS LOOPS FOR SPECIMEN 8C1

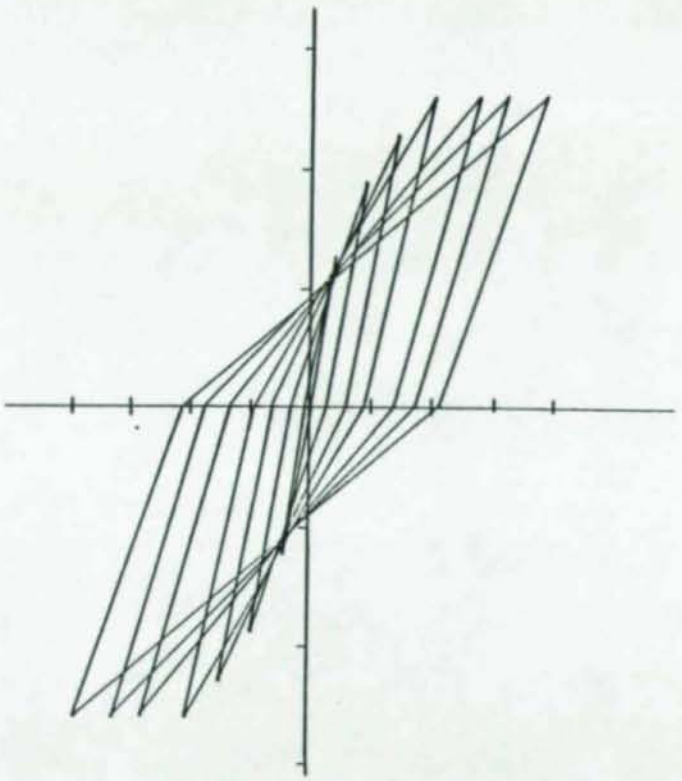
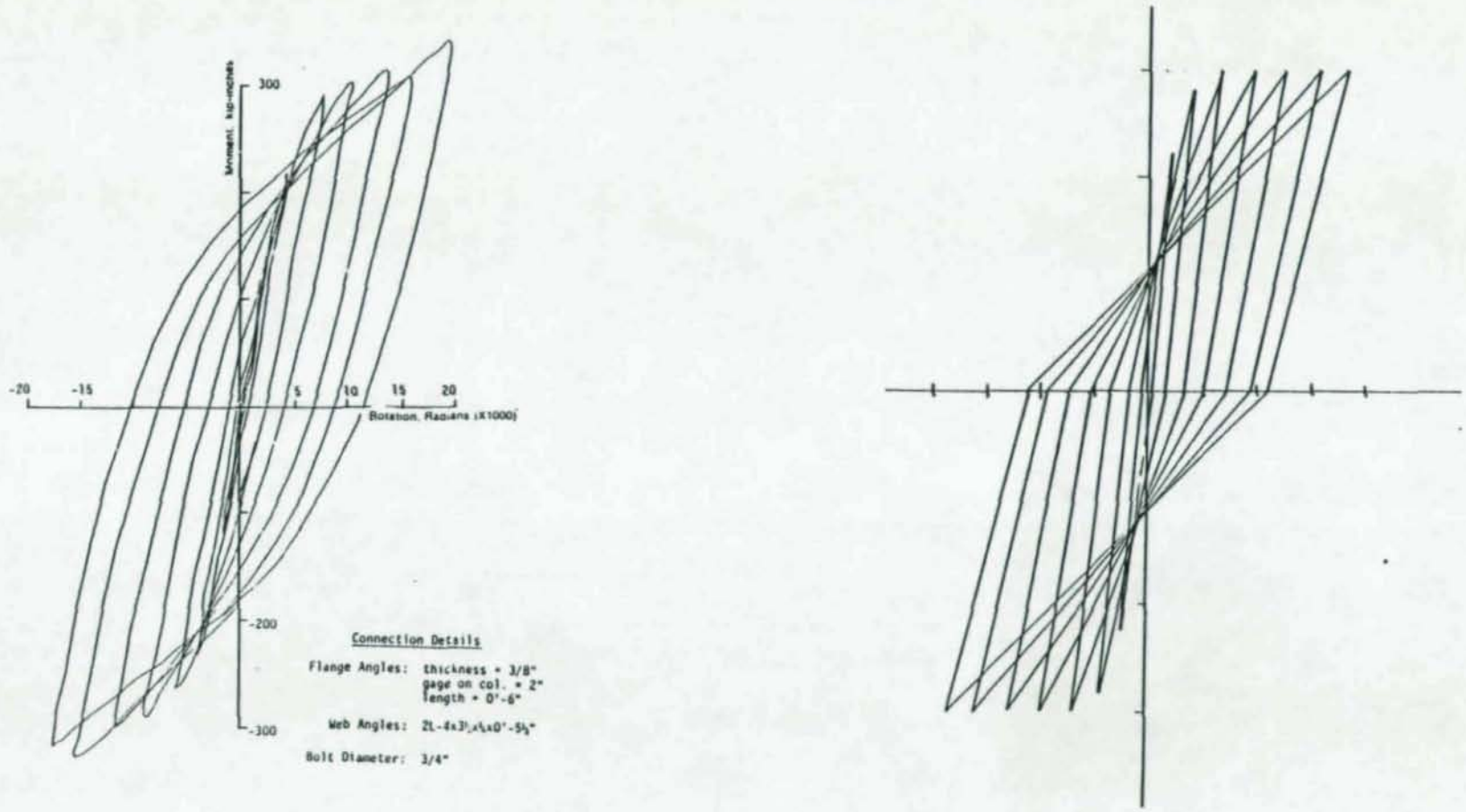


Figure 65. South Carolina Test 8C1 (Ref. 14) & Model Generated Results Using Unloading Stiffness Reduction (Degenerative Model).



STABLE HYSTERESIS LOOPS FOR SPECIMEN 8C2

Figure 66. South Carolina Test 8C2 (Ref. 14) & Model Generated Results Using Unloading Stiffness Reduction (Degenerative Modle).

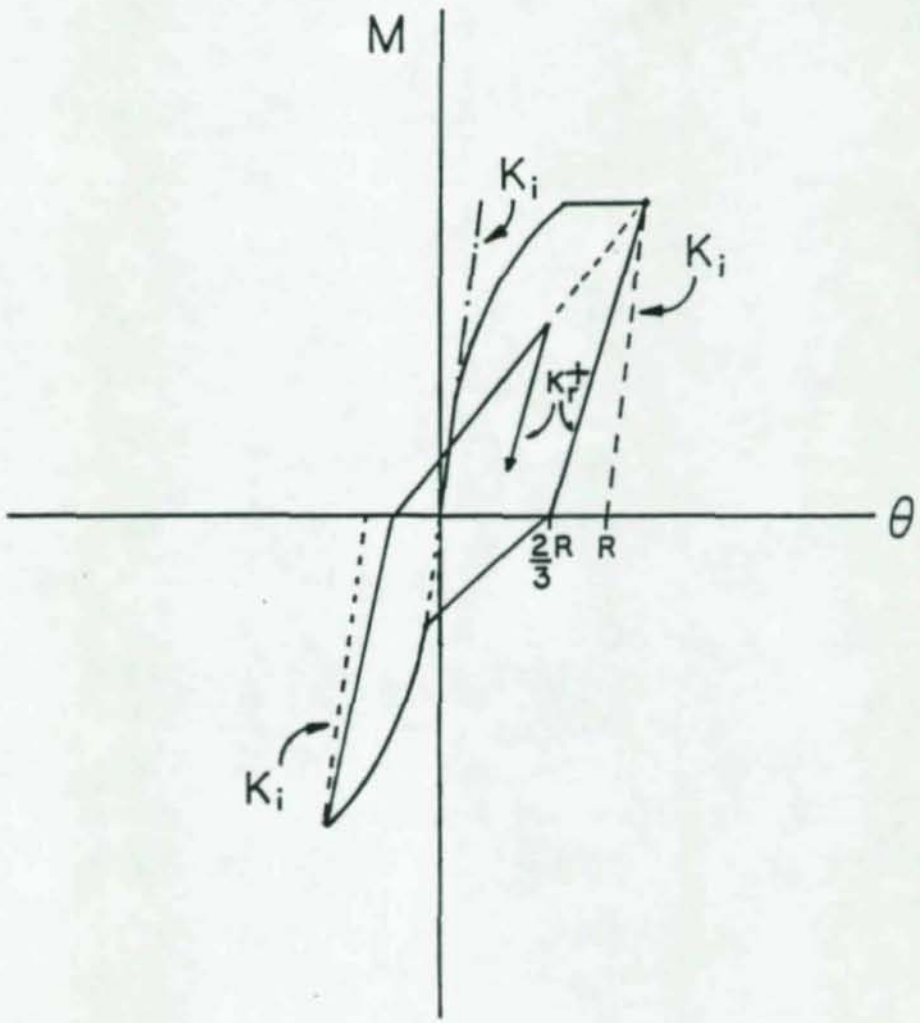
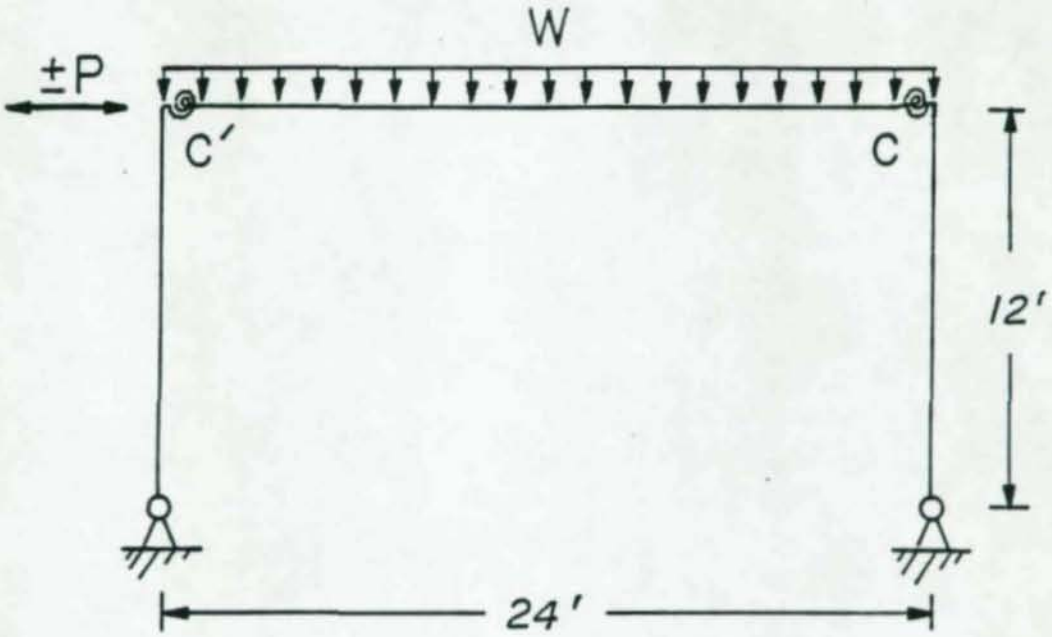


Figure 67. Degenerative Model Example. Ki=Initial Elastic Stiffness. Kr=Reduced Stiffness.



CONNECTIONS – 14S-C2 A36

WEB		2L - 4"×3-1/2"×1/4"×8-1/2"
FLANGE		2L - 6"×4"×1/2"×8"
BOLTS		A325 FXS 3/4" DIA.
$K_{\varepsilon_i} = Z^{-1}$	≈	$(2.265 \times 10^{-6}$ RAD/KIP-IN$)^{-1}$

COLUMNS – W10 x 26 A36

I = 144 IN^4

BEAM – W14 x 30 A36

I = 291 IN^4

Figure 68. Example Design Frame.

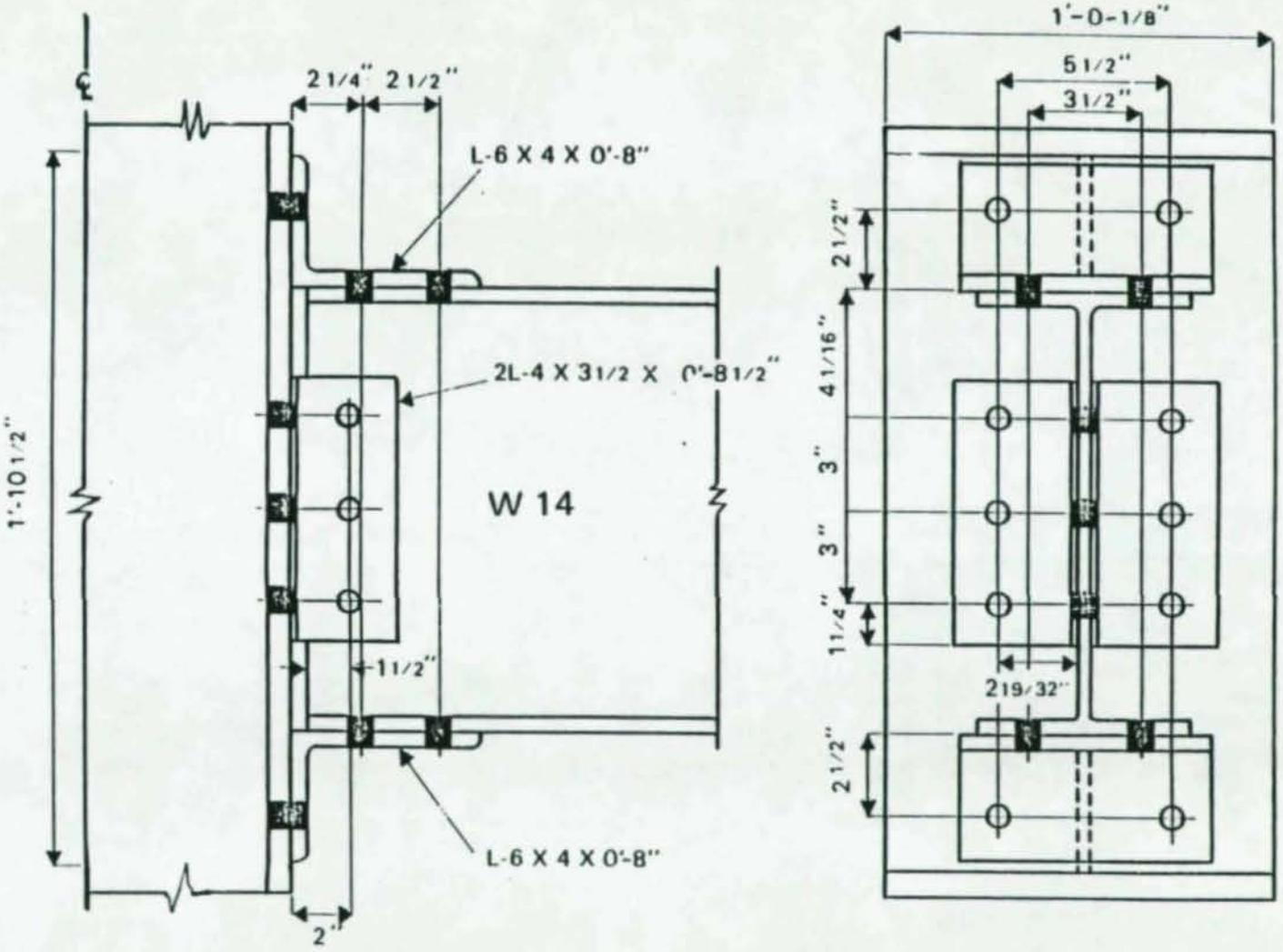


Figure 69. "Flexible Wind" Connection (14S&C2, Ref. 14).

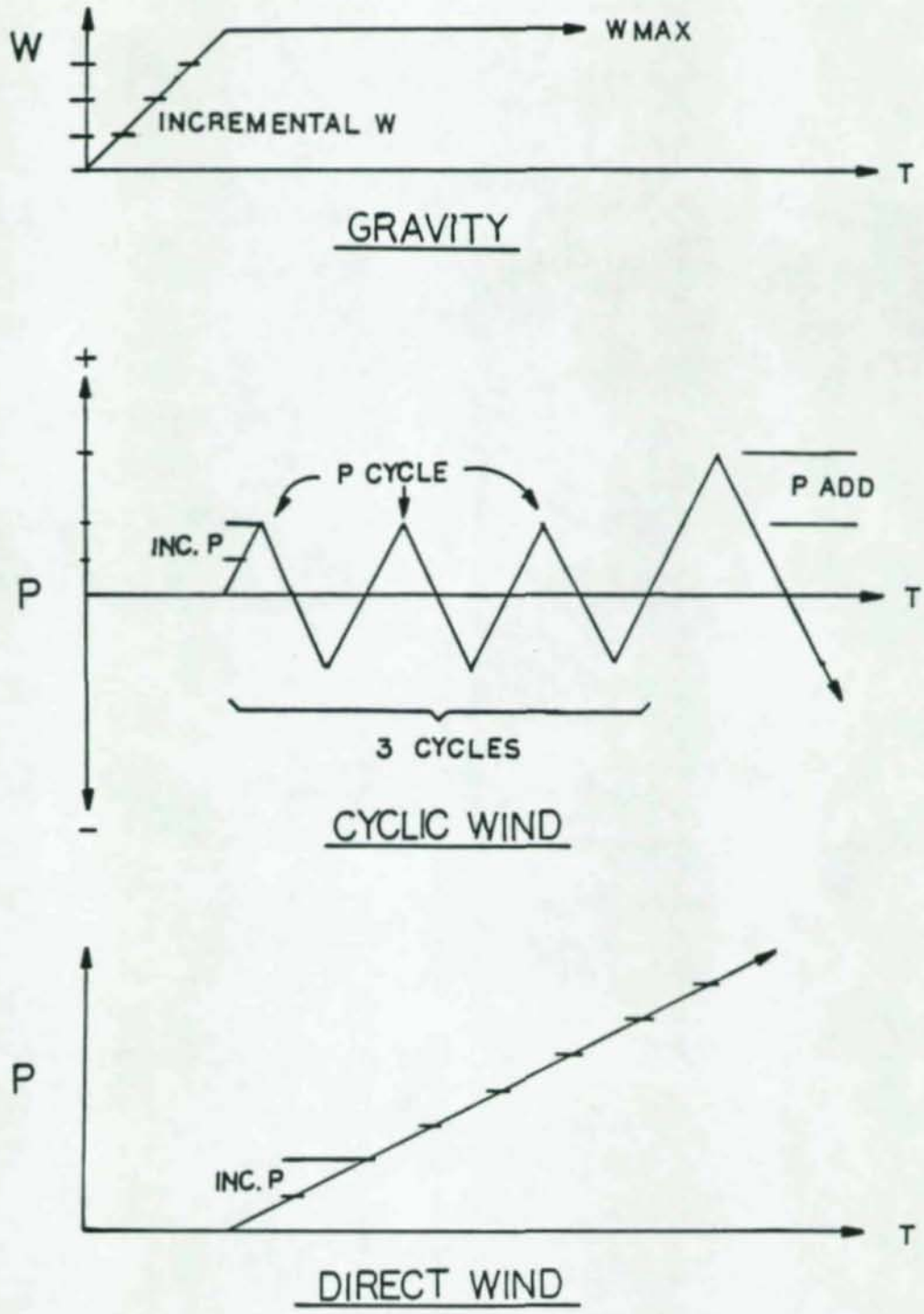


Figure 70. Analysis Loading Schemes.

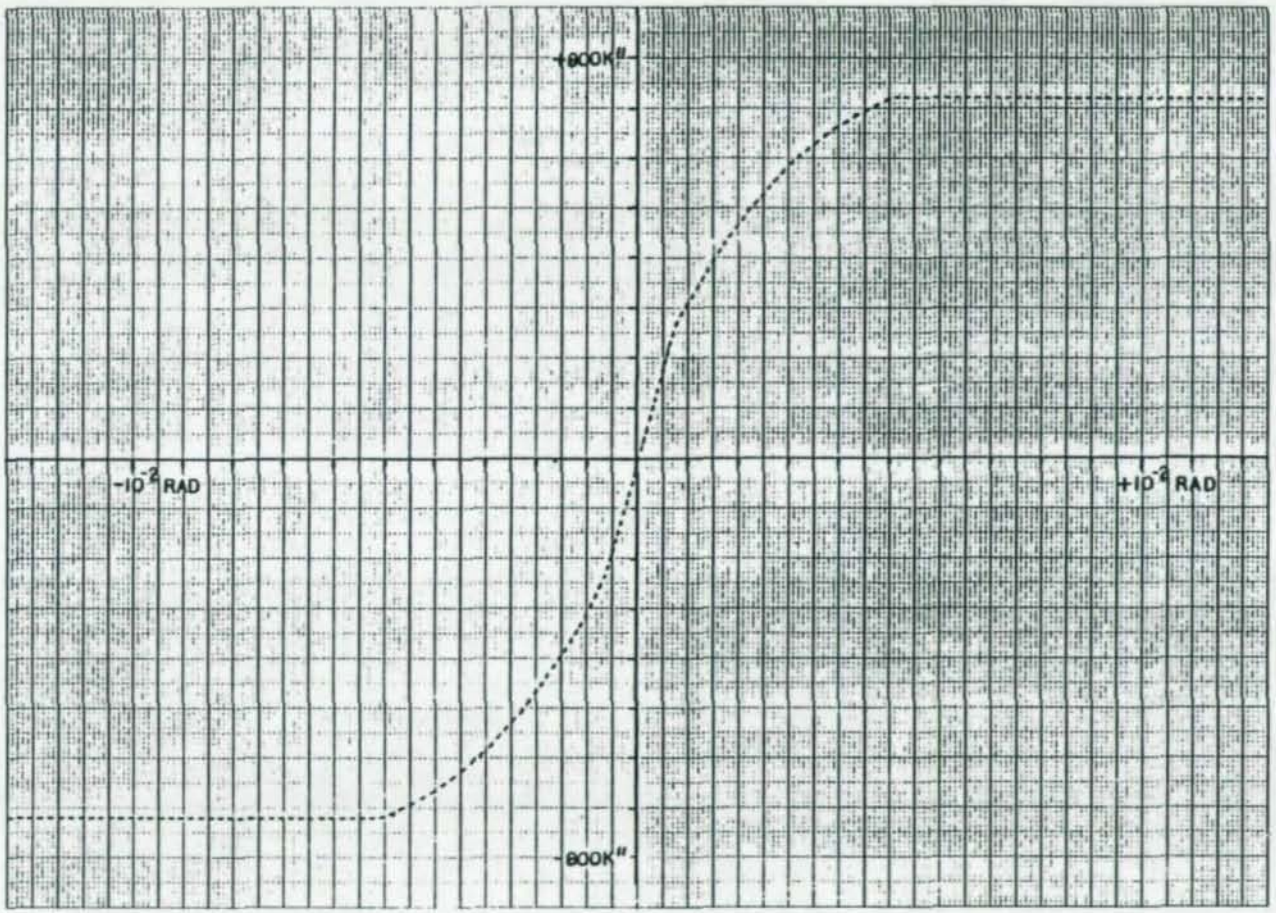


Figure 71. Connection Model Loading Curve (from Ref. 14, 14S&C2).

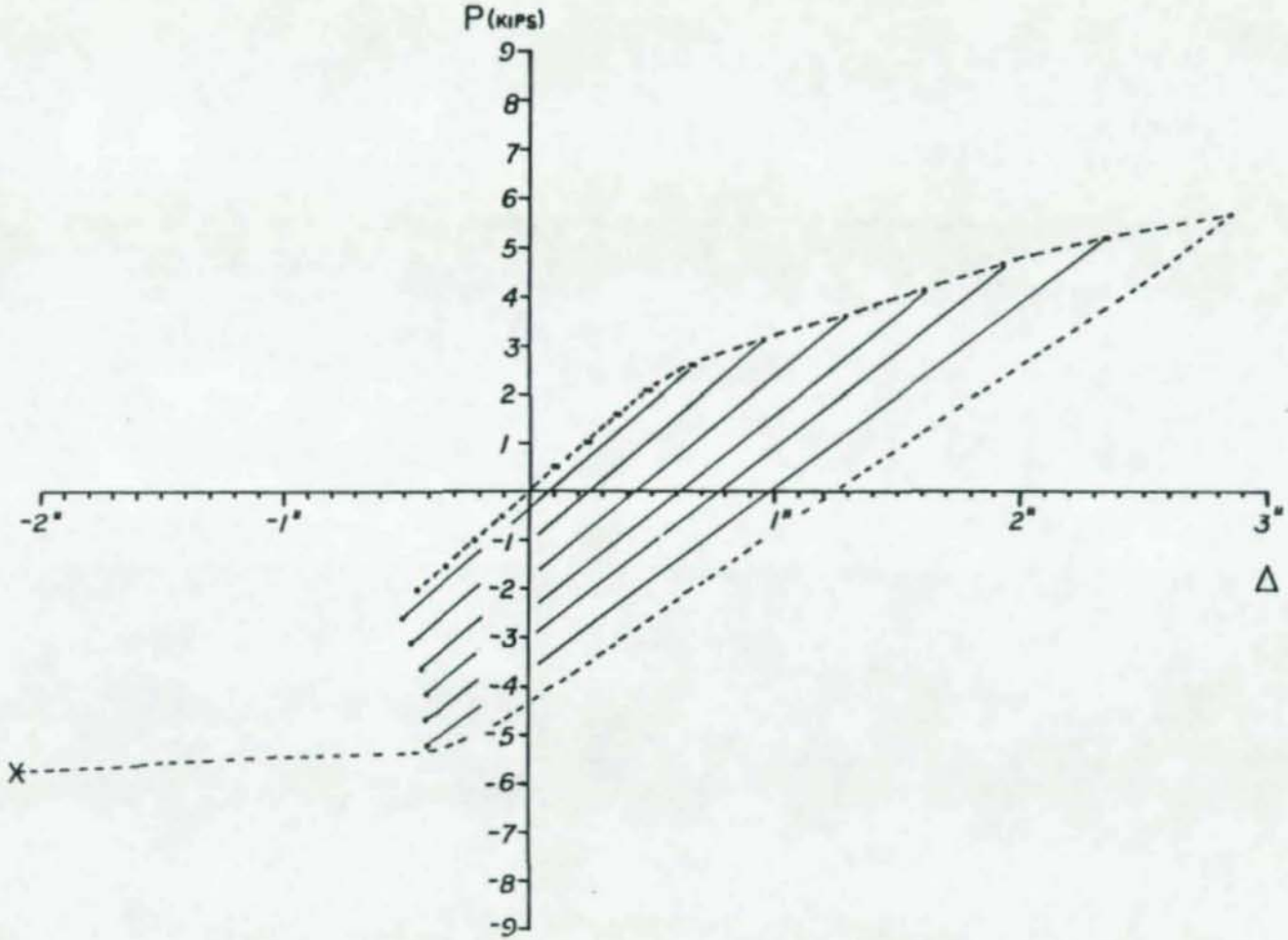
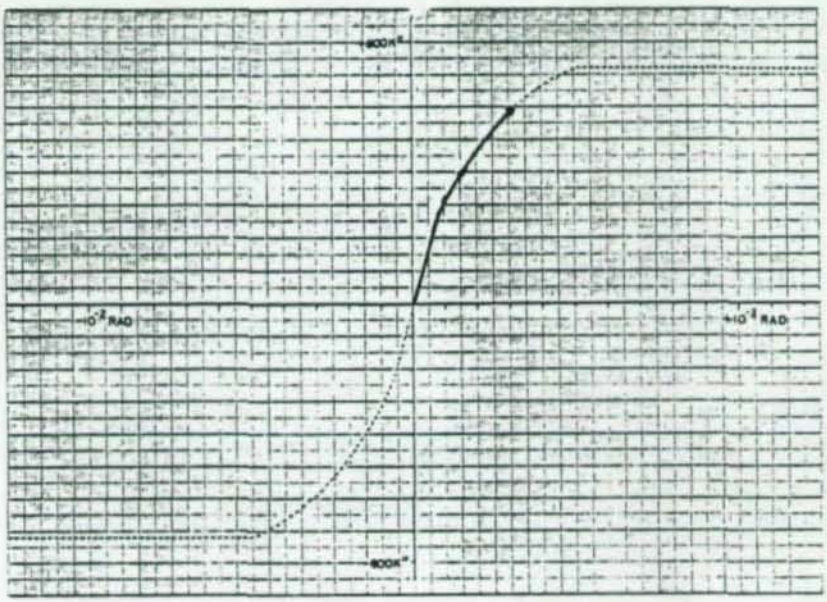
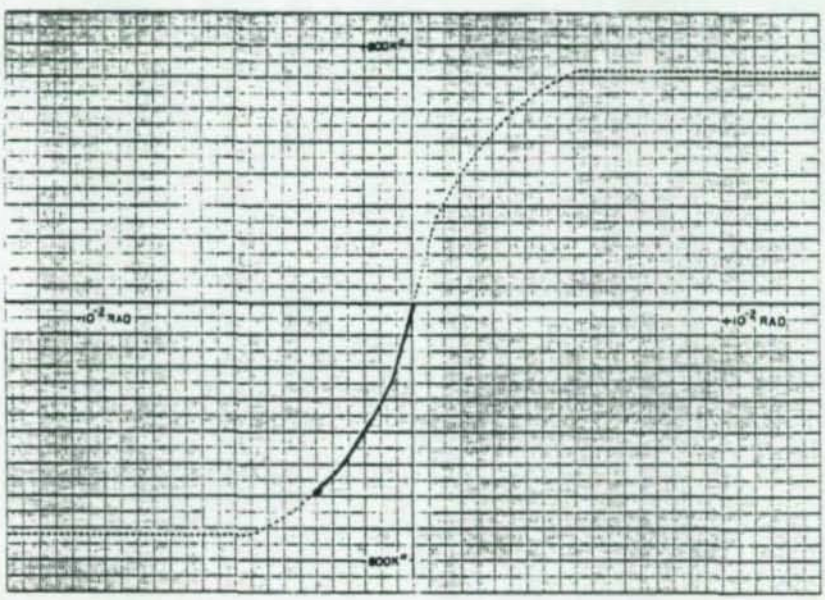


Figure 72. Lateral Loads vs. Lateral Deflection for Design Frame Cyclic Peaks.

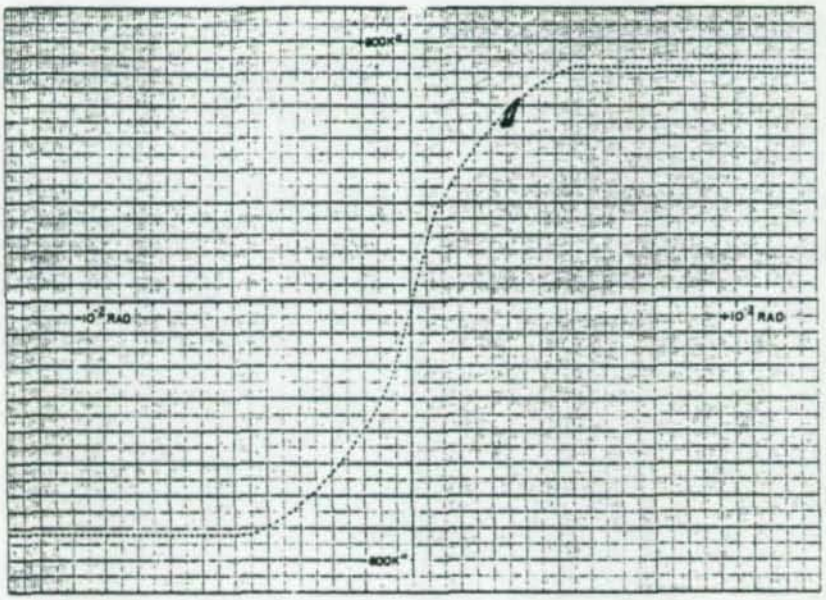


a) Left Connection; Initial Gravity Conditions.

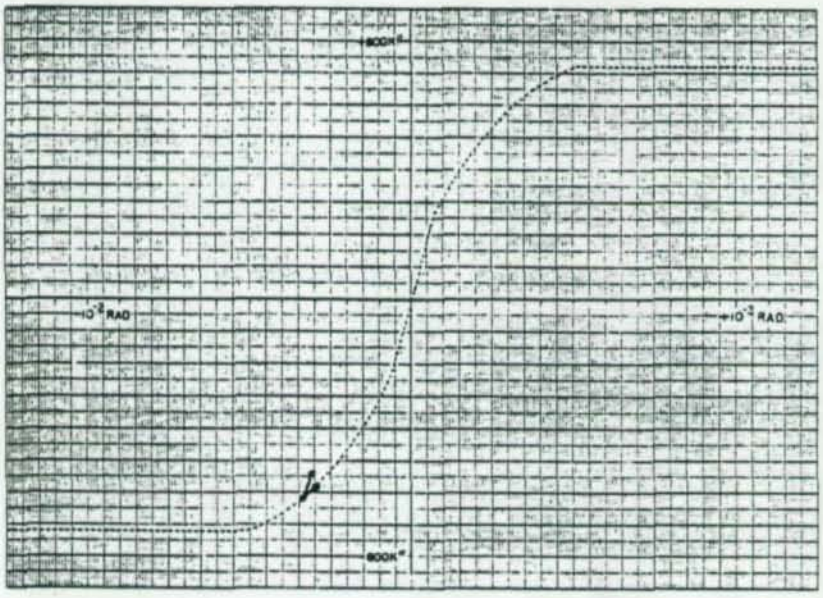


b) Right Connection; Initial Gravity Conditions.

Figure 73. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.

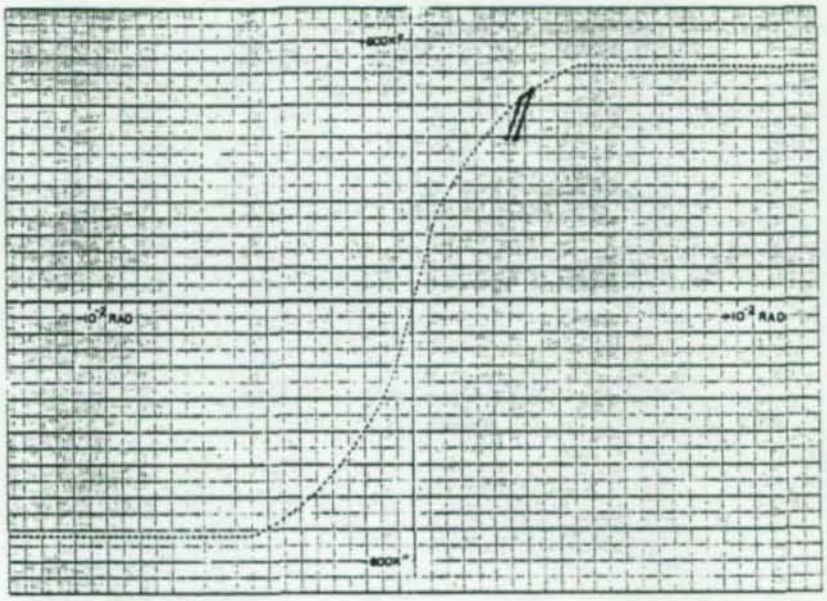


c) Left Connection; Cyclic Peak P=+/-0.52 kips.

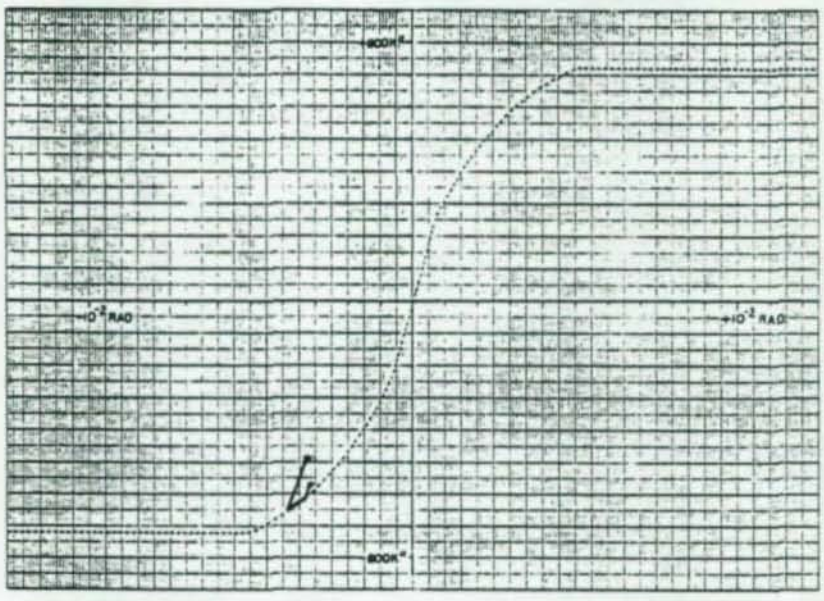


d) Right Connection; Cyclic Peak P=+/-0.52 kips.

Figure 73 continued. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.

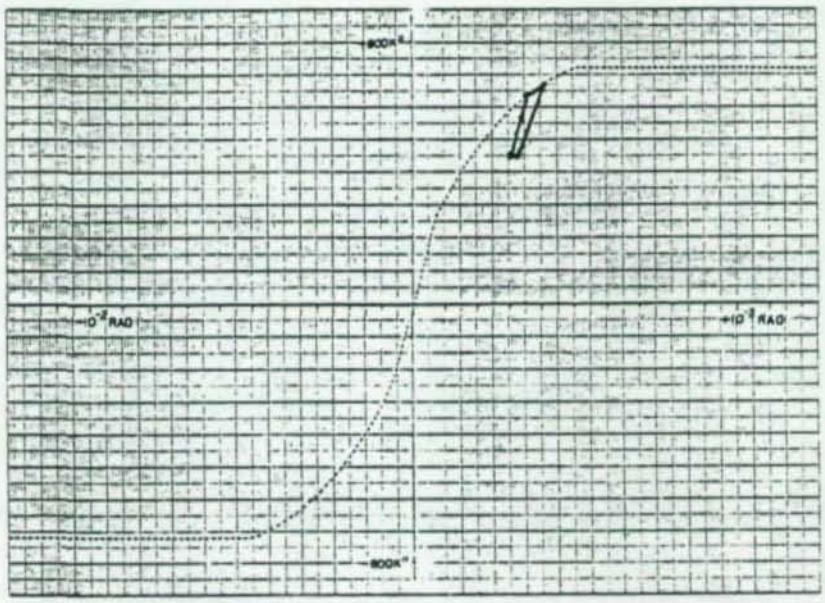


e) Left Connection; Cyclic Peak P=+/-1.04 kips.

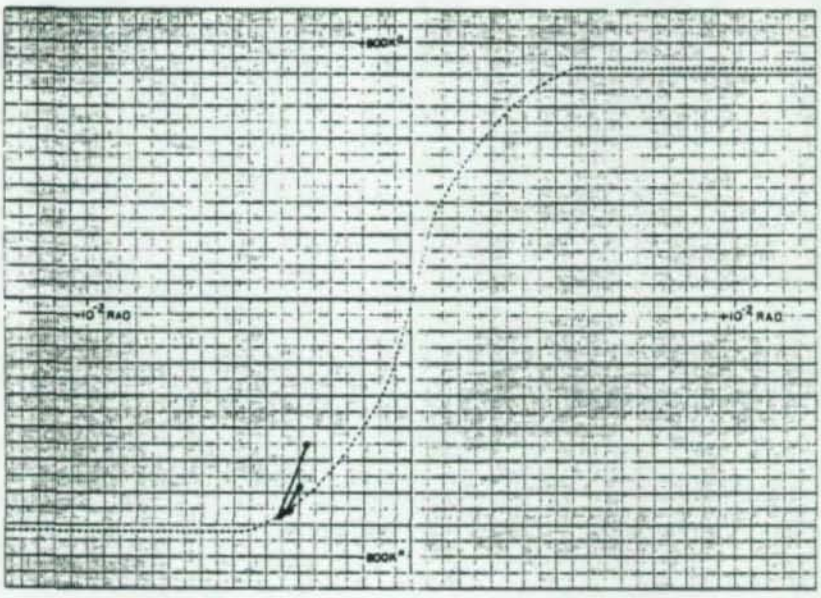


f) Right Connection; Cyclic Peak P=+/-1.04 kips.

Figure 73 continued. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.

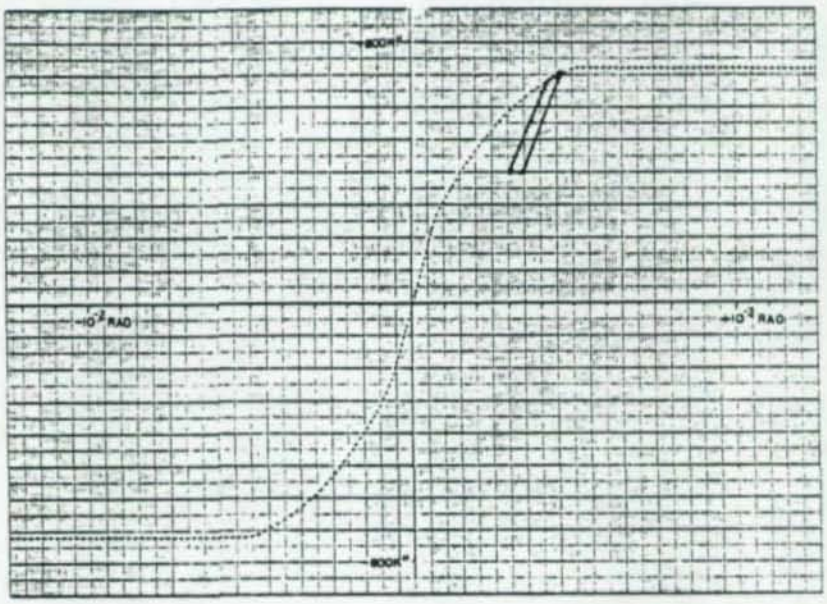


g) Left Connection; Cyclic Peak P=+/-1.56 kips.

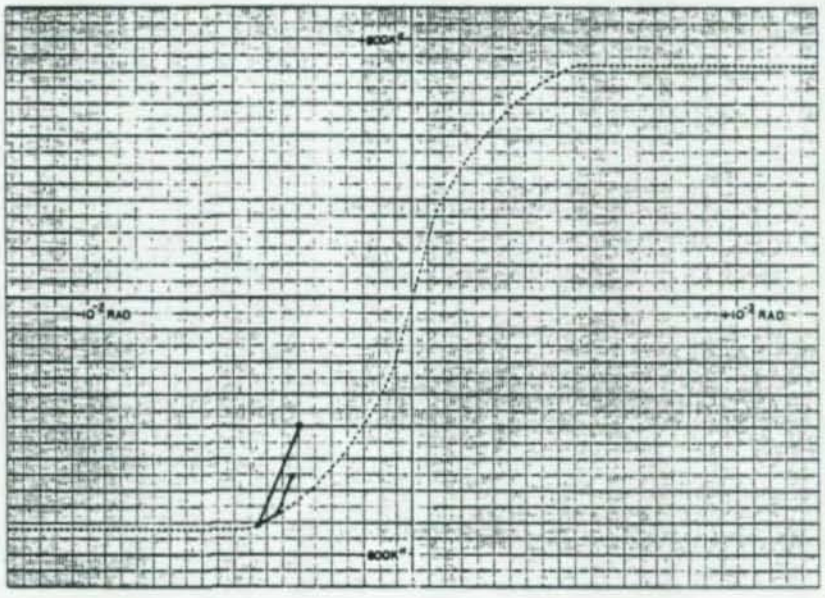


h) Right Connection; Cyclic Peak P==/-1.56 kips.

Figure 73 continued. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.

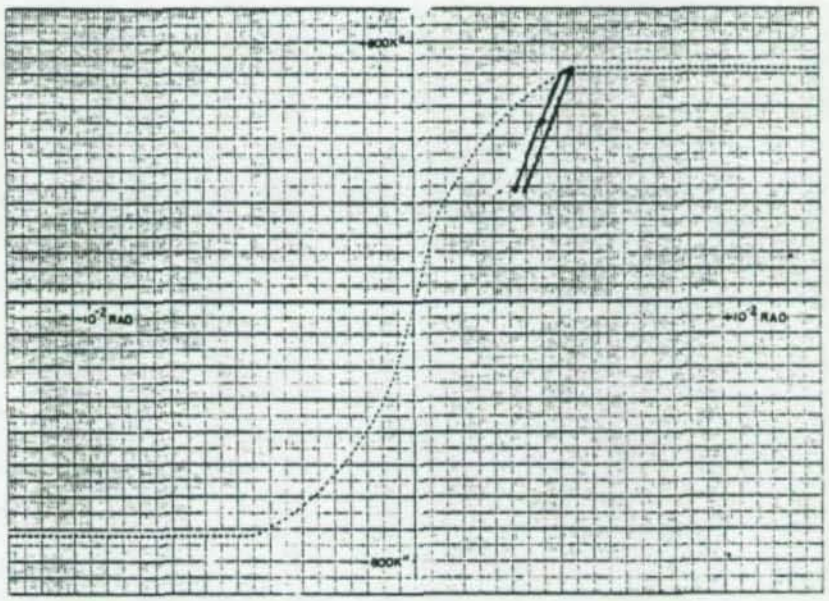


i) Left Connection; Cyclic Peak P=+/-2.08 kips.

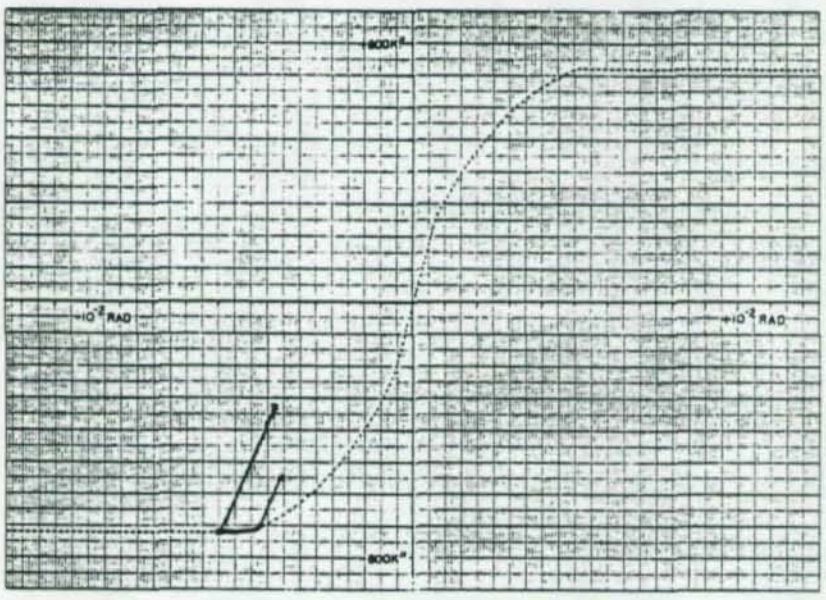


j) Right Connection; Cyclic Peak P=+/-2.08 kips.

Figure 73 continued. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.

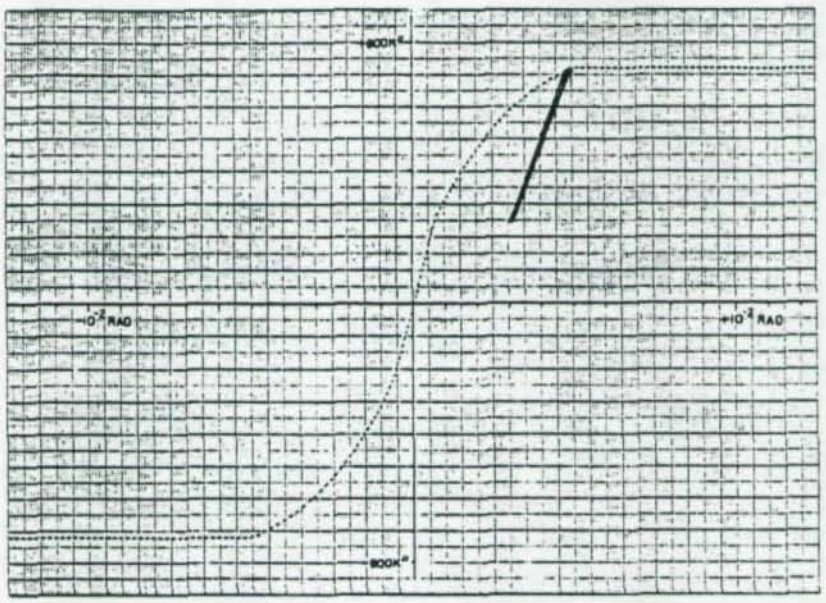


k) Left Connection; Cyclic Peak P=+/-2.60 kips.

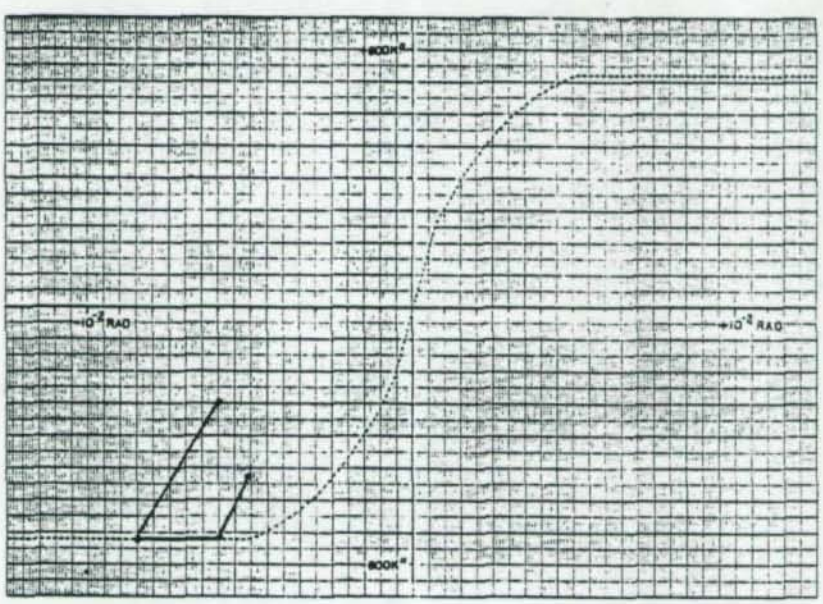


l) Right Connection; Cyclic Peak P=+/-2.60 kips.

Figure 73 continued. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.

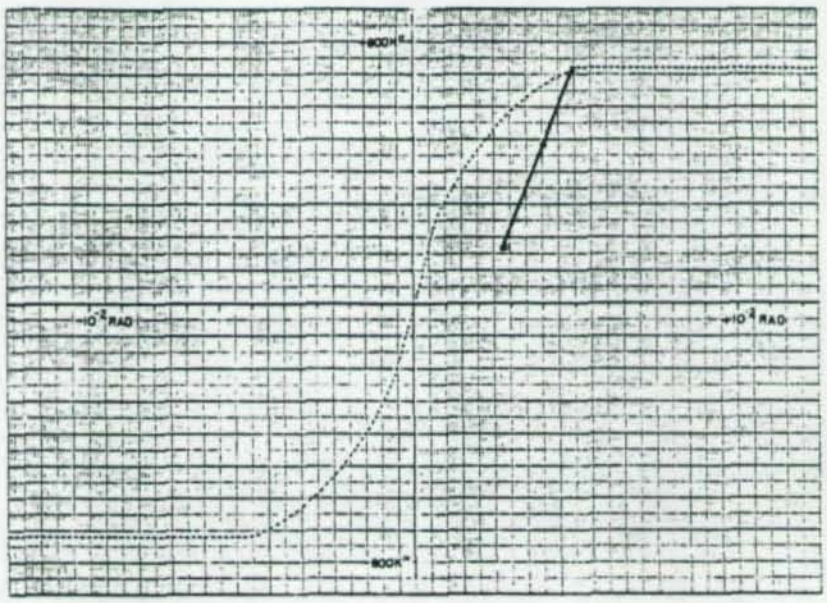


m) Left Connection; Cyclic Peak P=+/-3.12 kips.

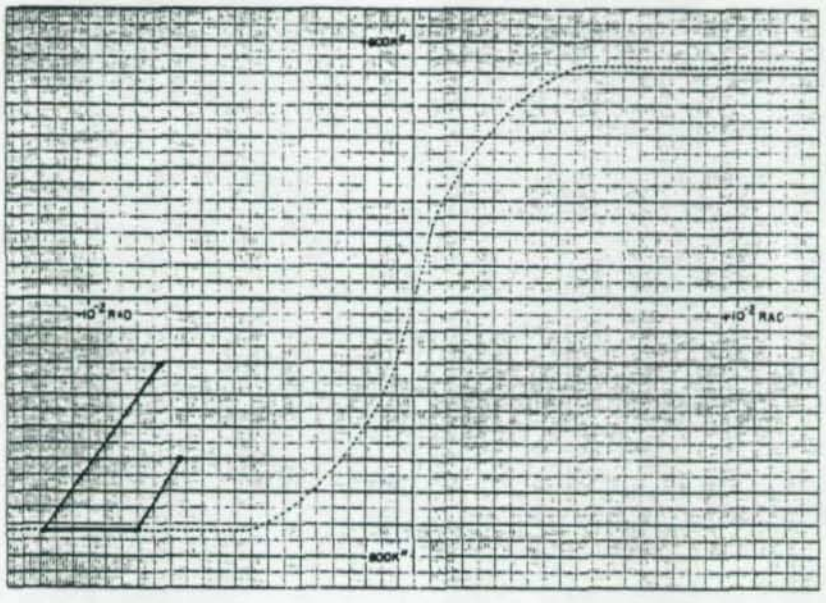


n) Right Connection; Cyclic Peak P=+/- 3.12 kips.

Figure 73 continued. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.

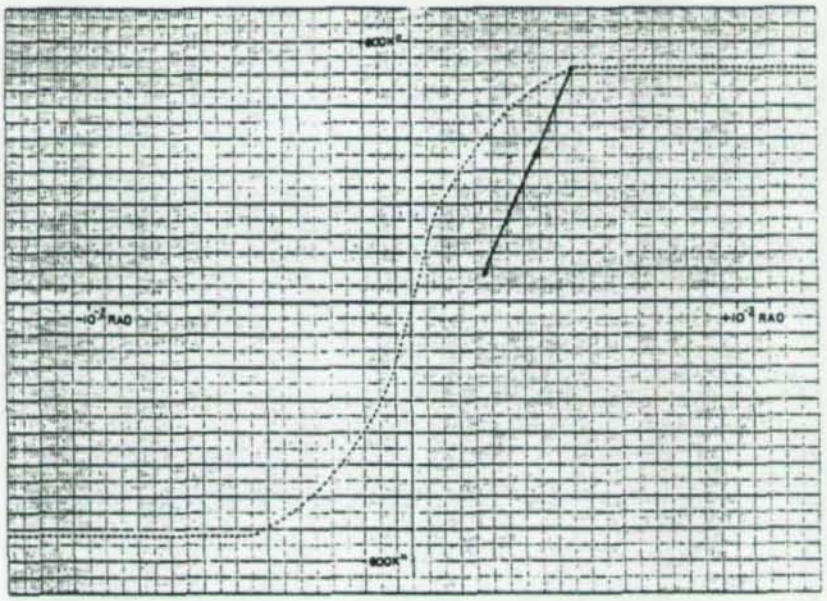


o) Left Connection; Cyclic Peak P=+/-3.64 kips.

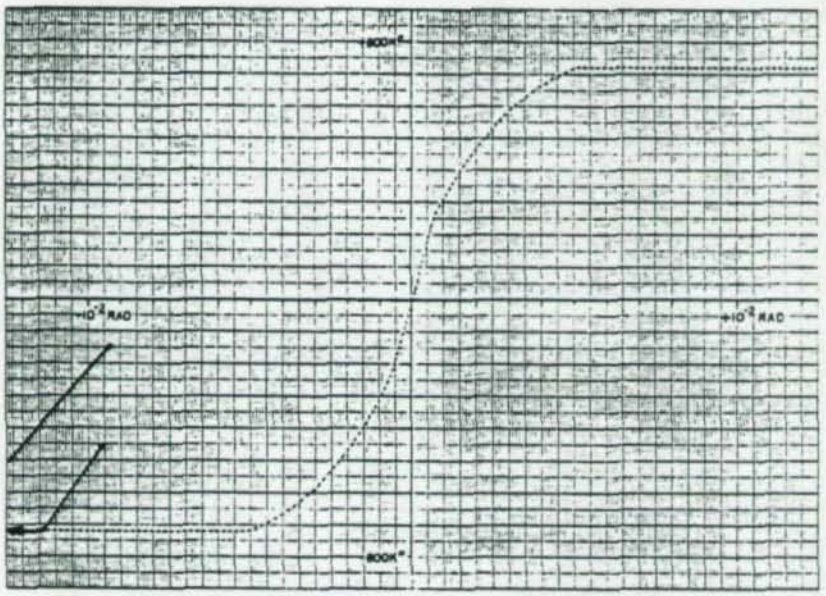


p) Right Connection; Cyclic Peak P=+/-3.64 kips.

Figure 73 continued. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.

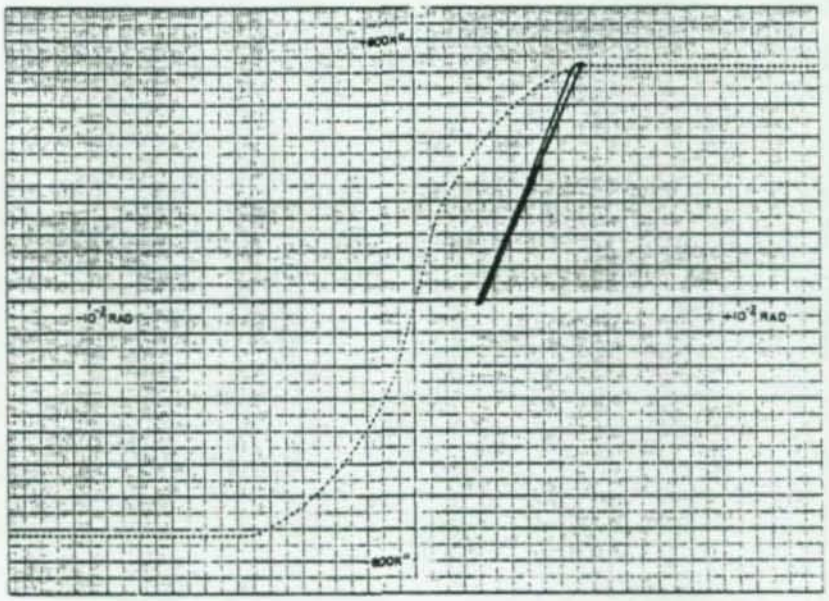


q) Left Connection; Cyclic Peak P=+/-4.16 kips.

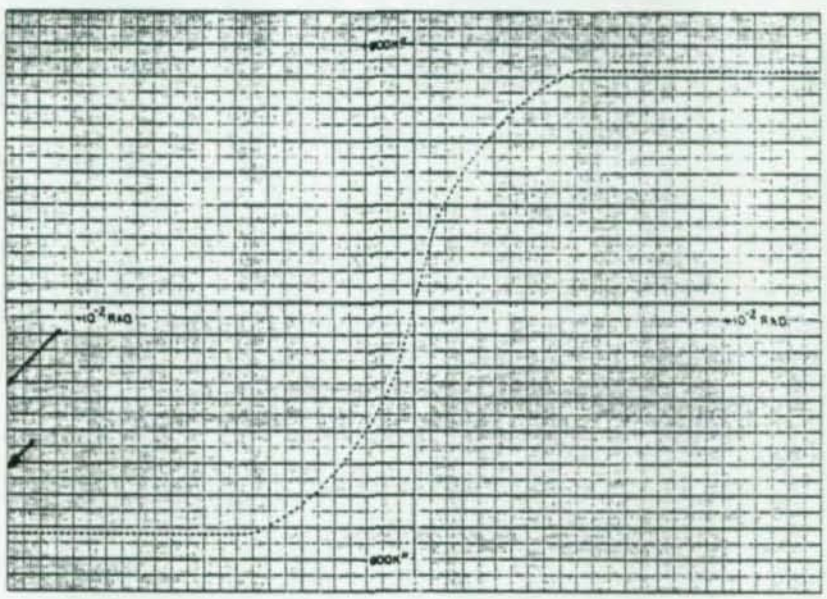


r) Right Connection; Cyclic Peak P=+/-4.16 kips.

Figure 73 continued. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.

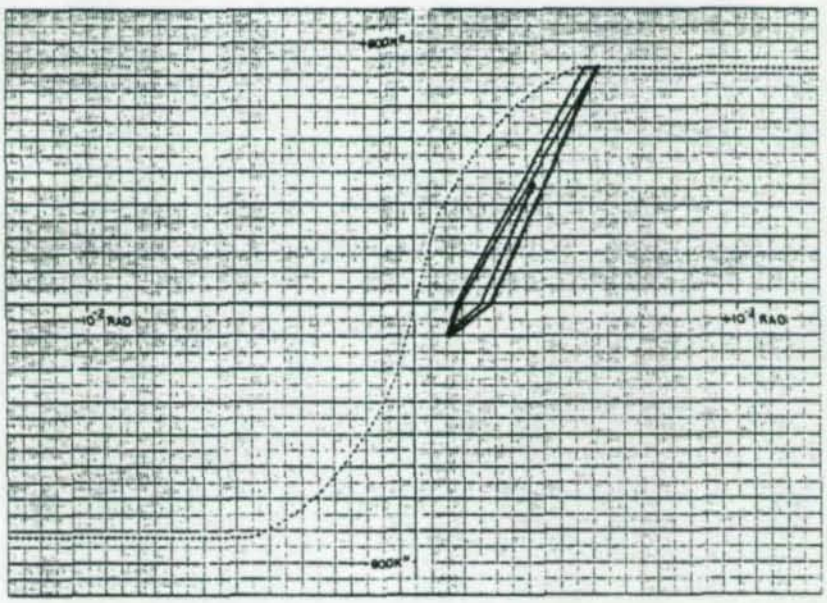


s) Left Connection; Cyclic Peak P=+/-4.68 kips.

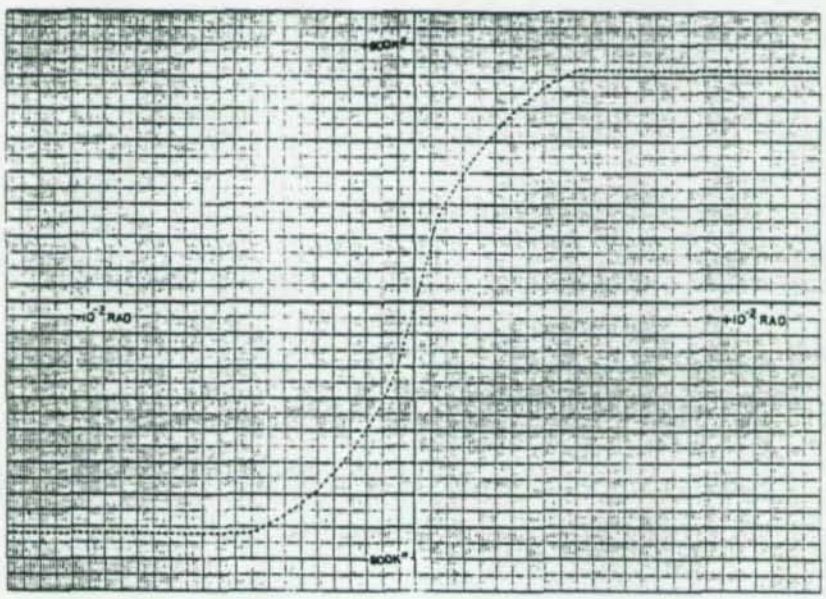


t) Right Connection; Cyclic Peak P=+/-4.68 kips.

Figure 73 continued. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.

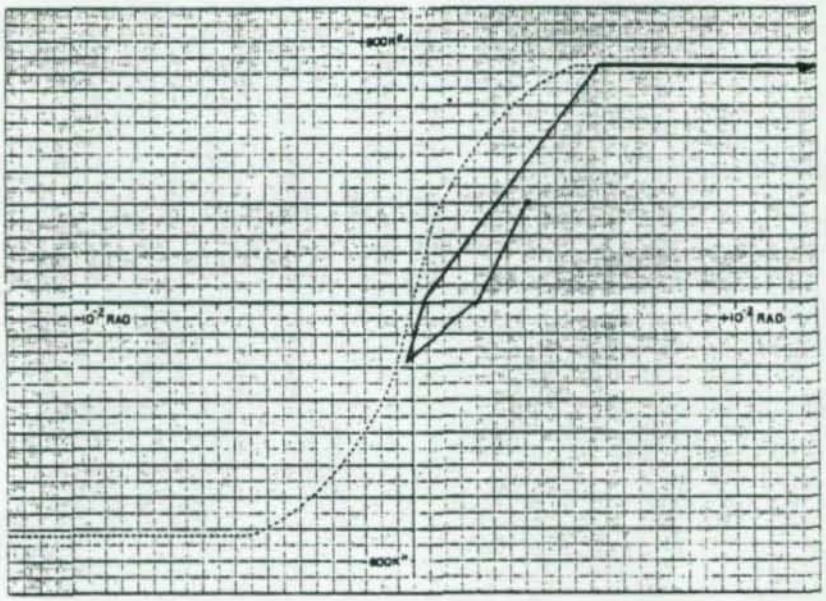


u) Left Connection; Cyclic Peak P=+/-5.2 kips.

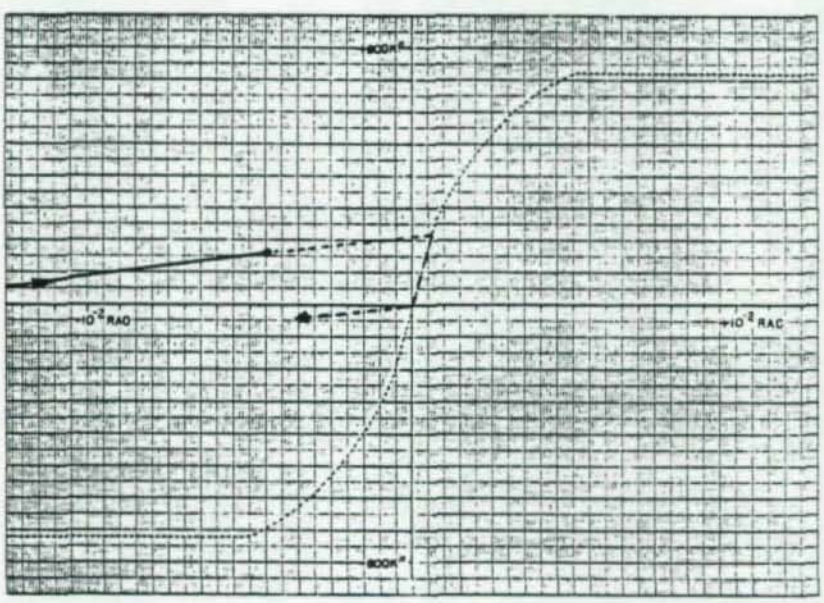


v) Right Connection; Cyclic Peak P=+/-5.2 kips.

Figure 73 continued. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.



w) Left Connection; Cyclic Peak P=+/-5.72 kips.



x) Right Connection; Cyclic Peak P=+/-5.72 kips.

Figure 73 continued. Connection Moment vs. Rotation Response for Design Frame under Cylic Loading.

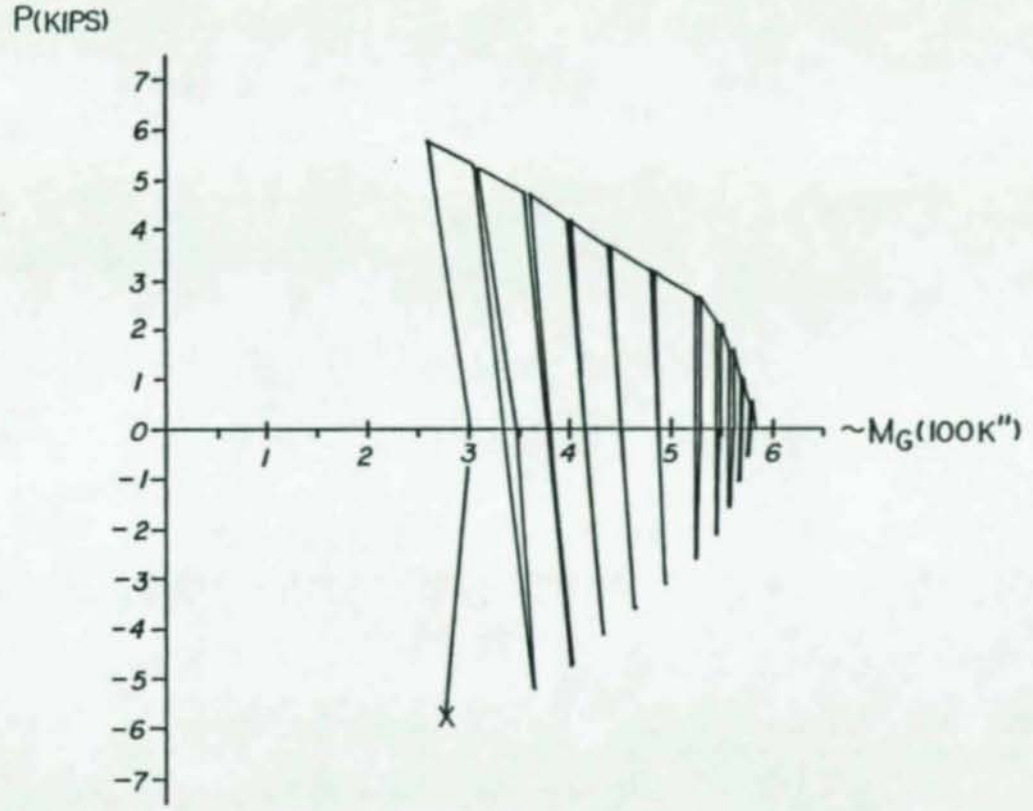


Figure 74. Approximate Gravity Moment Shakedown for Design Frame Under Cyclic Lateral Loading. Average Connection Moment vs. Lateral Load.

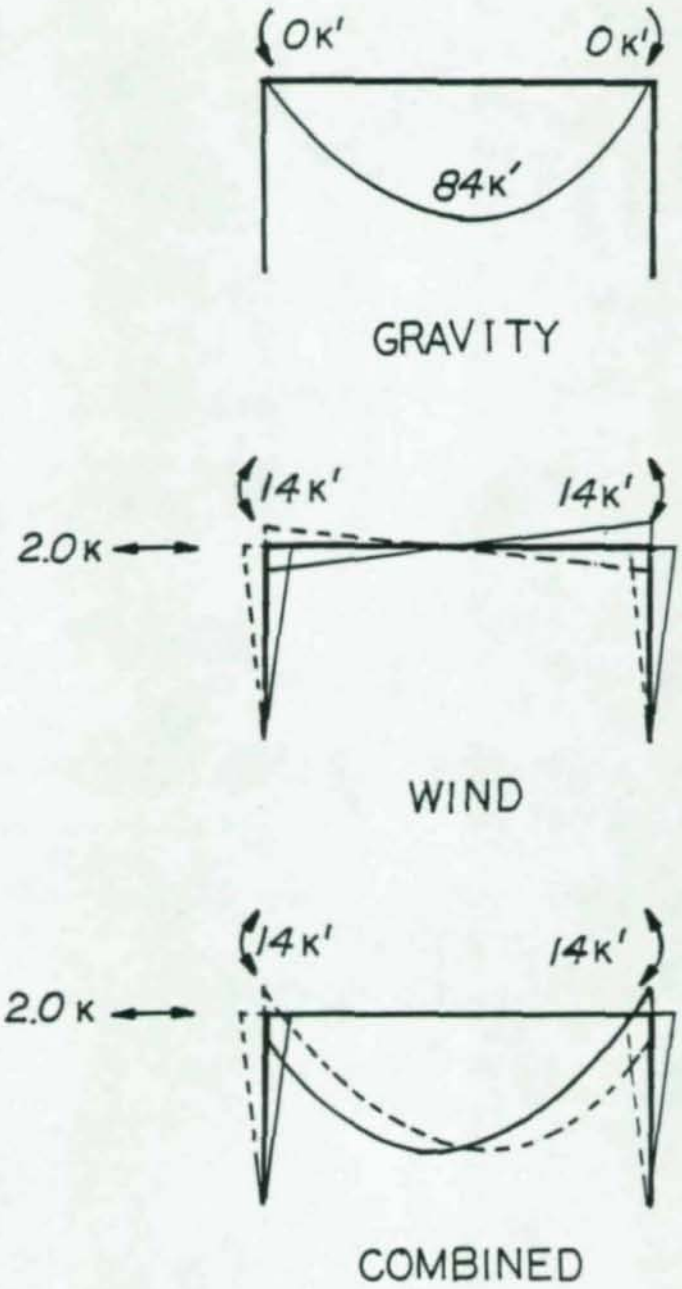
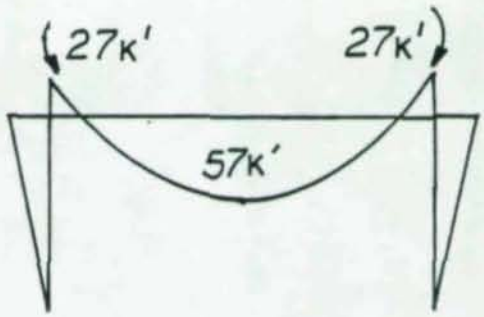
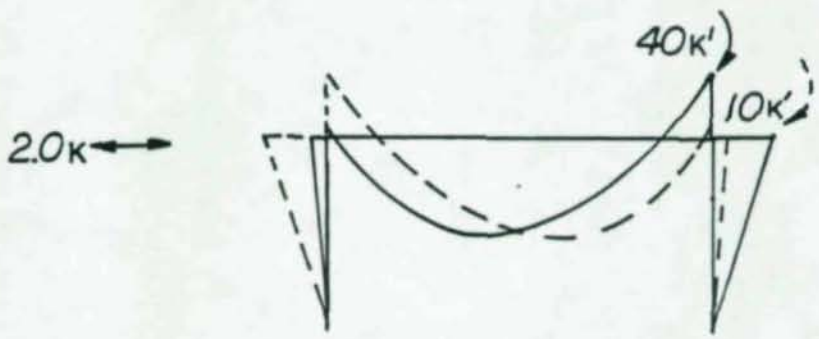


Figure 75. "Flexible Wind" Design Moment Distributions at Design Loads after "Shakedown".



GRAVITY



COMBINED

Figure 76. Analysis Moment Distribution at Design Loads After Cycling.

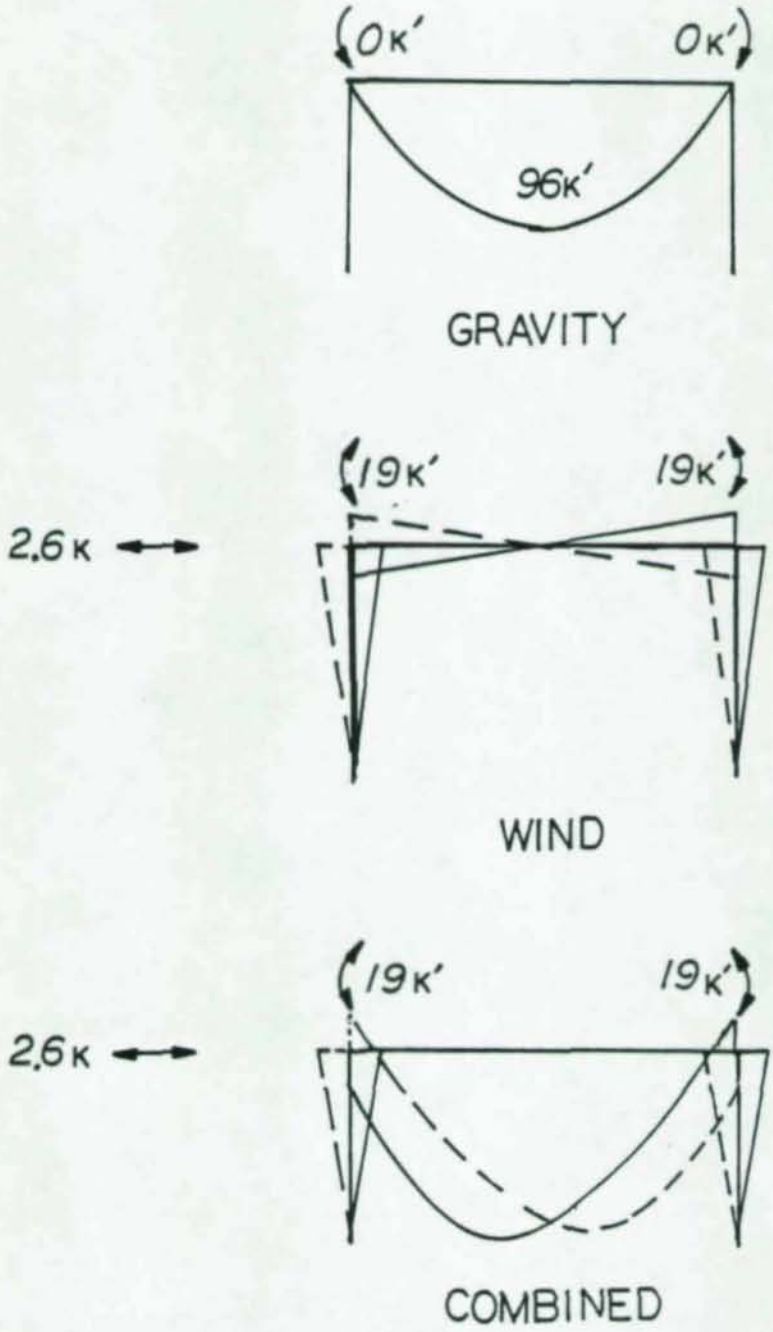
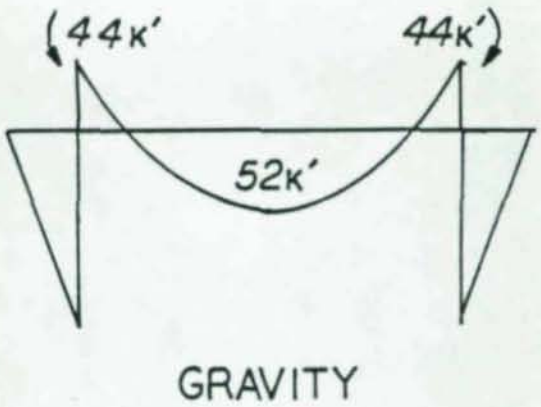


Figure 77. "Flexible Wind" Design Moment Distributions at Factored Loads after "Shakedown".



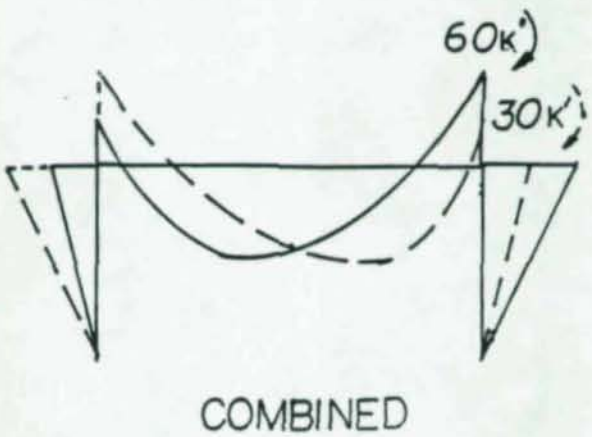


Figure 78. Analysis Moment Distribution at Factored Loads after Cycling.

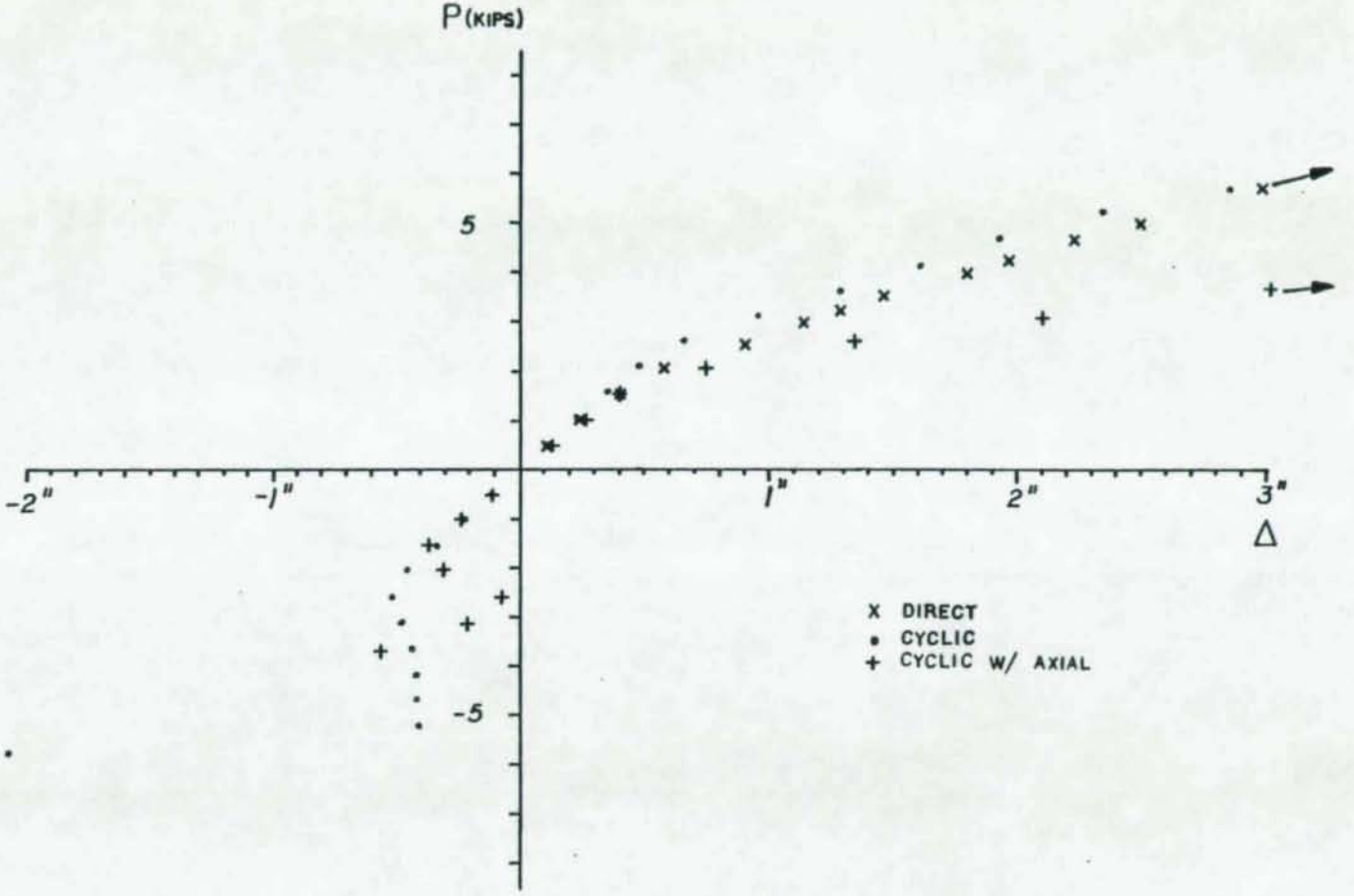


Figure 79. Lateral Load vs. Deflection for Design Frame under Lateral Loadings; Direct Wind, Cyclic Wind, Cyclic Wind with Additional Axial Column Loads.

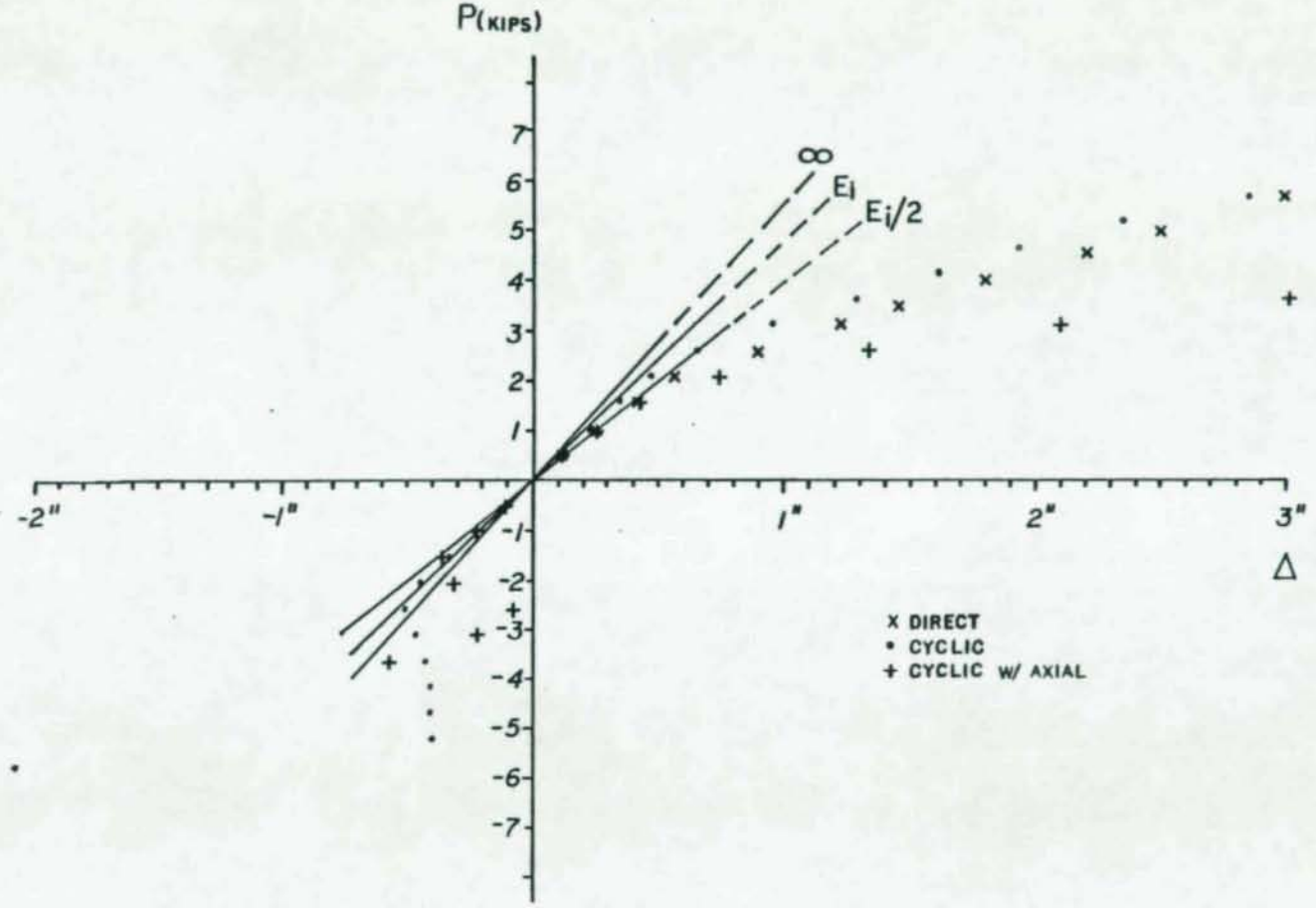


Figure 80. Elastic Stiffness Approximations.

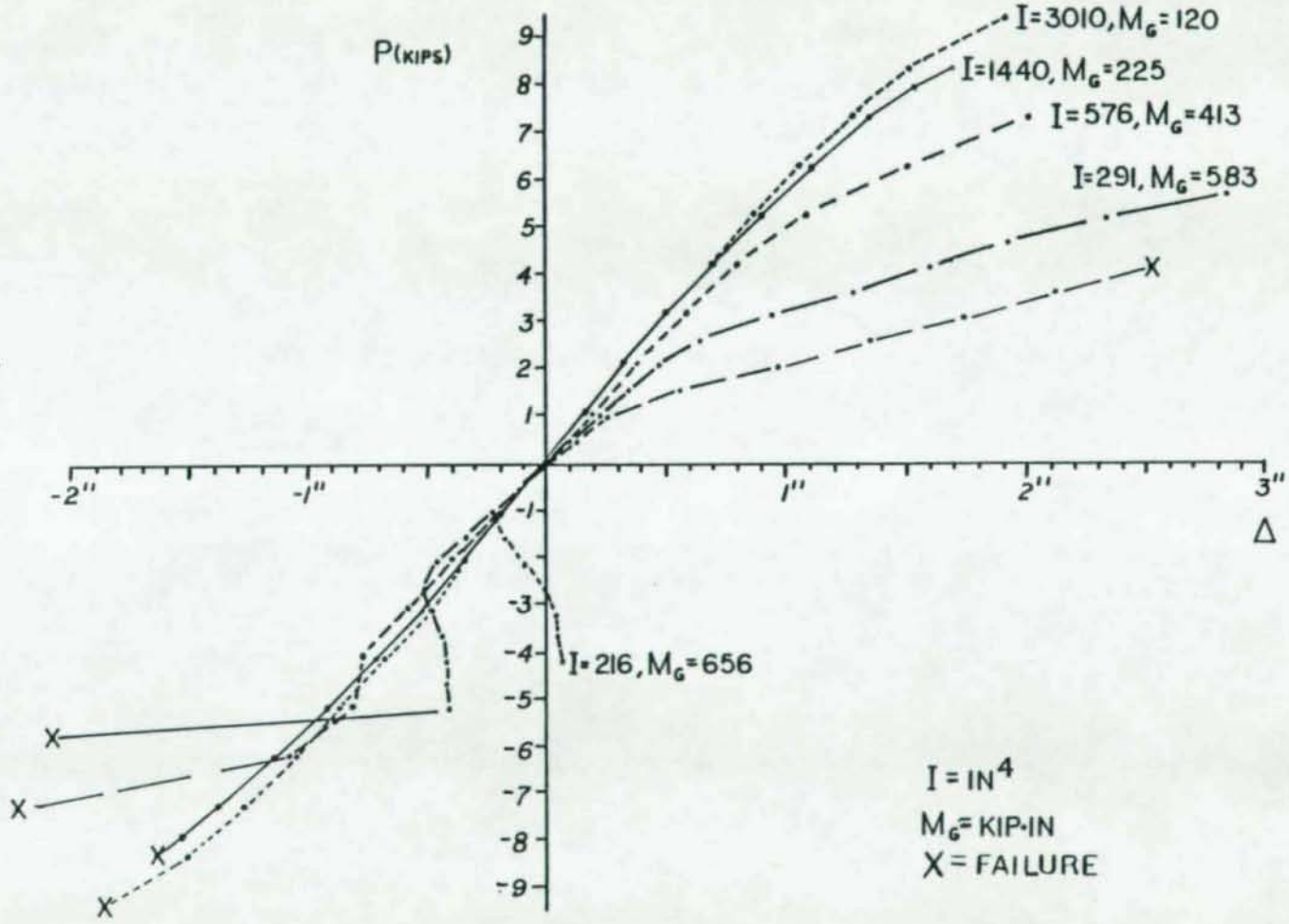


Figure 81. Cyclic Lateral Peak Loads vs. Lateral Deflection and Initial Gravity Moments, Mg, for Various Beam Moments of Inertia, I.

6. ACKNOWLEDGEMENTS

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The manuscript was prepared in part and under the guidance of Ms. Kathryn Schallert, Secretary, Department of Civil Engineering, Washington University.

7. APPENDICES

APPENDIX 7.1

Connection Test Data

Table 7.1.1. Tests Performed

Test/Type	Initial Pg (kips)	Maximum Pg (kips)	Maximum Pw (kips)	Comments
1/Cyclic	0.00	1.25	3.00	Trial Test
2/Cyclic	0.67	0.67	3.50	Constant Gravity Moment
3/Cyclic	2.15	2.15	3.00	Constant Gravity Moment
4/Static	0.00	0.00	3.00	Monotonic
5/Cyclic	2.14	0.00	1.50	Constant Gravity Angle
6/Cyclic	0.00	0.00	3.00	Pure Cyclic Bolt Slip
7/Cyclic	0.00	0.00	3.50	Pure Cyclic Increased Bolt Tightening

TEST 1: CYCLE 1. POSITIVE JOINT MOMENT A= 43" a= 9" D_{12}= 4.375" D_{34}= 4.75"

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{12}$ ($\times 10^{-2}$ rads)	theta $_{34}$ ($\times 10^{-2}$ rads)	theta avg. ($\times 10^{-2}$ rads)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.5	.008	.003	-.002	-.007	.114	.105	.110
0.7	.015	.006	-.003	-.013	.206	.211	.208
0.95	.025	.010	-.006	-.023	.343	.358	.350
1.05	.030	.012	-.008	-.027	.411	.411	.411
1.15	.035	.014	-.009	-.032	.480	.484	.482
1.25	.040	.016	-.011	-.038	.549	.568	.559
1.39	.050	.020	-.015	-.047	.686	.674	.680
1.48	.060	.025	-.019	-.057	.800	.800	.800
1.53	.070	.029	-.023	-.065	.949	.884	.916
1.61	.080	.033	-.027	-.076	1.074	1.032	1.053
1.725	.103	.043	-.037	-.093	1.371	1.179	1.275
1.885	.125	.054	-.046	-.120	1.634	1.558	1.596
2.03	.150	.065	-.056	-.146	1.943	1.895	1.919
2.15	.174	.075	-.065	-.170	2.263	2.210	2.236
2.3	.200	.087	-.075	-.195	2.583	2.526	2.555
1.35	.184	.079	-.066	-.174	2.400	2.273	2.337
0.85	.173	.075	-.062	-.163	2.240	2.126	2.183
0.50	.164	.072	-.059	-.153	2.103	1.979	2.041
0.25	.154	.069	-.057	-.144	1.943	1.832	1.887
0.00	.135	.062	-.054	-.133	1.669	1.663	1.666

TEST 1: CYCLE 2. NEGATIVE JOINT MOMENT

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{12}$ ($\times 10^{-2}$ rads)	theta $_{34}$ ($\times 10^{-2}$ rads)	theta $_{avg.}$ ($\times 10^{-2}$ rads)
-.125	.121	.057	-.048	-.115	1.451	1.400	1.426
-.25	.108	.052	-.043	-.099	1.291	1.179	1.235
-.375	.092	.046	-.035	-.082	1.051	.989	1.020
-.5	.078	.040	-.027	-.065	.869	.800	.834
-.75	.048	.028	-.013	-.028	.457	.316	.387
-1.0	.010	.012	.005	.034	-.457	-.611	-.534
-1.25	-.038	-.008	.026	.092	-.686	-1.389	-1.036
-1.375	-.061	-.017	.034	.115	-1.306	-1.705	-1.355
-1.5	-.093	-.031	.049	.152	-1.417	-2.168	-1.793
-1.625	-.128	-.046	.070	.203	-1.874	-2.800	-2.337
-1.75	-.160	-.060	.092	.259	-2.286	-3.516	-2.901
-1.875	-.196	-.077	.105	.293	-2.720	-3.958	-3.339
-2.00	-.218	-.087	.112	.312	-2.994	-4.211	-3.602
-2.125	-.246	-.097	.121	.340	-3.406	-4.611	-4.008
-2.30	-.277	-.111	.130	.367	-3.794	-4.989	-4.392
-2.00	-.273	-.109	.129	.363	-3.749	-4.926	-4.337
-1.45	-.261	-.104	.128	.352	-3.589	-4.716	-4.152
-1.00	-.252	-.100	.121	.342	-3.474	-4.653	-4.063
-.50	-.233	-.092	.115	.323	-3.223	-4.379	-3.801
-.25	-.220	-.086	.111	.310	-3.063	-4.189	-3.626
0.00	-.189	-.075	.101	.280	-2.606	-3.768	-3.187

TEST 1: CYCLE 3. POSITIVE JOINT MOMENT

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{12}$ ($\times 10^{-2}$ rads)	theta $_{34}$ ($\times 10^{-2}$ rads)	theta $_{avg.}$ ($\times 10^{-2}$ rads)
.25	-.150	-.060	.088	.251	-2.060	-3.432	-2.721
.50	-.105	-.043	.075	.197	-1.417	-2.526	-1.972
.75	-.060	-.025	.055	.131	-.800	-1.600	-1.200
1.00	.085	.038	.005	.002	1.074	.069	.572
1.25	.139	.061	-.016	-.061	1.783	.947	1.365
1.50	.173	.073	-.038	-.106	2.286	1.430	1.858
1.75	.196	.082	-.050	-.135	2.400	1.789	2.095
2.00	.220	.092	-.060	-.162	2.926	2.147	2.537
2.30	.253	.105	-.073	-.194	3.383	2.547	2.965
1.75	.242	.100	-.065	-.179	3.246	2.400	2.823
1.25	.230	.095	-.061	-.165	3.086	2.189	2.638
.75	.216	.090	-.056	-.149	2.880	1.958	2.419
0.0	.168	.075	-.048	-.110	2.126	1.305	1.716
TEST 1: CYCLE 4. NEGATIVE JOINT MOMENT							
-0.25	.112	.054	-.018	-.042	1.326	.505	.916
-.050	.050	.030	.020	.067	.457	-.989	-.126
-0.75	-.036	-.001	.046	.146	-.663	-2.105	-1.384
-1.00	-.132	-.051	.103	.285	-1.951	-3.830	-2.842
-1.25	-.183	-.073	.113	.313	-2.514	-4.211	-3.362
-1.50	-.215	-.086	.121	.339	-2.949	-4.589	-3.769
-1.75	-.240	-.096	.128	.351	-3.291	-4.695	-3.993
-2.00	-.262	-.105	.134	.373	-3.589	-5.032	-4.311
-2.30	-.286	-.115	.143	.399	-3.910	-5.389	-4.650
-1.75	-.272	-.110	.138	.386	-3.703	-5.221	-4.462
-1.25	-.260	-.104	.134	.374	-3.566	-5.053	-4.310
-0.75	-.243	-.097	.129	.360	-3.337	-4.863	-4.100
-0.25	-.219	-.088	.123	.340	-2.994	-4.568	-3.781
0.00	-.189	-.078	.113	.312	-2.537	-4.189	-3.363

TEST 1: CYCLE 5, POSITIVE JOINT MOMENT

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta$_{-2}$ 12 (x 10^{-2} rads)	theta$_{-2}$ 34 (x 10^{-2} rads)	theta$_{-2}$ avg. (x 10^{-2} rads)
0.50	-0.089	-.039	0.076	0.191	-1.143	-2.421	-1.782
slip							
0.50	0.086	.040	.024	.047	1.050	-0.484	0.283
1.00	.156	.066	-.014	-.048	2.057	0.716	1.387
1.50	.210	.086	-.045	-.123	2.834	1.642	2.238
2.00	.242	.099	-.061	-.167	3.270	2.232	2.751
2.30	.253	.106	-.068	-.186	3.497	2.484	2.991
1.75	.250	.102	-.063	-.173	3.383	2.316	2.850
1.25	.239	.098	-.058	-.160	3.223	2.147	2.685
0.75	.225	.093	-.053	-.143	3.017	1.895	2.456
0.25	.201	.085	-.044	-.118	2.651	1.559	2.105
0.00	.172	.076	-.037	-.101	2.194	1.347	1.771

TEST 1: CYCLE 6. NEGATIVE JOINT MOMENT

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	θ_{12} ($\times 10^{-2}$ rads)	θ_{34} ($\times 10^{-2}$ rads)	$\theta_{avg.}$ ($\times 10^{-2}$ rads)
-0.25	0.118	0.055	-.010	-.024	1.440	0.295	0.868
-0.50	.001	.007	.040	.122	-0.137	-1.726	-.932
-1.00	-.170	-.068	.118	.320	-2.331	-4.253	-3.292
-1.50	-.239	-.097	.132	.378	-3.246	-5.179	-4.213
-2.15	-.287	-.115	.136	.396	-3.931	-5.474	-4.703
-2.30	-.295	-.120	.146	.403	-4.000	-5.411	-4.705
-2.375	-.305	-.124	.153	.413	-5.166	-5.537	-5.352
-1.50	-.285	-.115	.150	.390	-3.886	-5.137	-4.512
-0.50	-.250	-.100	.131	.360	-3.429	-4.821	-4.125
0.00	-.205	-.084	.118	.321	-2.766	-4.274	-3.520
-0.50	-.233	-.093	.127	.348	-3.200	-4.653	-3.927
-1.00	-.256	-.102	.135	.370	-3.520	-4.947	-4.234
-2.00	-.293	-.118	.146	.402	-4.000	-5.390	-4.695
-2.30	-.304	-.123	.149	.412	-4.137	-5.537	-4.837
-2.50	-.317	-.129	.154	.424	-4.297	-5.684	-4.991
-2.75	-.352	-.144	.167	.461	-4.750	-6.189	-5.470
-3.00	-.390	-.157	.181	.496	-5.326	-6.695	-6.011
-2.50	-.378	-.152	.177	.487	-5.074	-6.526	-5.800
-1.75	-.362	-.147	.170	.470	-4.914	-6.316	-5.615
-1.00	-.342	-.140	.163	.449	-4.617	-6.021	-5.319
-0.50	-.324	-.133	.156	.431	-4.361	-5.789	-5.075
0.00	-.284	-.115	.143	.396	-3.863	-5.326	-4.595

TEST 1: CYCLE 7. POSITIVE JOINT MOMENT

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	θ_{12} ($\times 10^{-2}$ rads)	θ_{34} ($\times 10^{-2}$ rads)	$\theta_{avg.}$ ($\times 10^{-2}$ rads)
0.05	-.155	-.068	.088	.267	-1.089	-3.768	-2.879
shift							
0.05	.053	.030	.015	.071	.526	-1.179	-0.327
1.00	.128	.055	-.018	-.026	1.669	.169	0.919
1.50	.202	.083	-.054	-.116	2.720	1.305	2.013
2.00	.242	.099	-.072	-.163	3.269	1.916	2.593
2.50	.277	.113	-.087	-.200	3.749	2.379	3.064
2.75	.301	.122	-.095	-.226	4.090	2.758	3.424
3.00	.335	.137	-.105	-.260	4.523	3.260	3.892
2.50	.329	.135	-.101	-.250	4.434	3.137	3.786
1.75	.315	.129	-.095	-.234	4.251	2.926	3.589
shift							
1.00	.298	.122	-.088	-.211	4.022	2.589	3.306
0.50	.280	.116	-.082	-.189	3.749	2.253	3.001
0.00	.230	.099	-.069	-.148	2.994	1.663	2.329

TEST 1: CYCLE 8. NEGATIVE JOINT MOMENT

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta4 (inches)	theta_{2} 12 ($\times 10^{-2}$ rads)	theta_{2} 34 ($\times 10^{-2}$ rads)	theta_{2} avg. ($\times 10^{-2}$ rads)
-0.25	0.171	0.076	-0.037	-0.073	2.171	0.758	1.465
-0.50	.051	.025	.021	.194	0.594	-3.642	-1.524
0.00	-.032	-.012	.072	.218	-0.457	-3.074	-1.766
-0.25	-.062	-.025	.082	.254	-0.846	-3.621	-2.235
-0.50	-.096	-.038	.093	.276	-1.326	-3.853	-2.590
-1.00	-.181	-.072	.120	.358	-2.491	-5.011	-3.751
-1.50	-.287	-.116	.141	.415	-3.909	-5.768	-4.839
-2.00	-.337	-.136	.152	.448	-4.594	-6.232	-5.413
shift							
-2.13	-.348	-.141	.156	.457	-4.731	-6.337	-5.534
-2.50	-.365	-.148	.163	.474	-4.960	-6.547	-5.750
-2.75	-.380	-.154	.169	.490	-5.166	-6.758	-5.962
-3.00	-.398	-.160	.177	.509	-5.440	-6.989	-6.215
-2.50	-.386	-.155	.172	.497	-5.280	-6.842	-6.061
-1.75	-.369	-.149	.165	.480	-5.029	-6.632	-5.831
-1.00	-.347	-.140	.157	.459	-4.731	-6.358	-5.545
-0.50	-.327	-.133	.150	.440	-4.434	-6.105	-5.270
0.00	-.285	-.115	.135	.399	-3.886	-5.558	-4.722
TEST 1: CYCLE 9.		POSITIVE JOINT MOMENT					
0.25	-.229	-.094	.116	.322	-3.086	-4.337	-3.172
0.50	-.142	-.062	.077	.208	-1.829	-2.758	-2.294
shift							
0.50	.093	.045	.005	.024	1.097	-0.400	0.349
1.00	.177	.075	-.037	-.090	2.331	1.116	1.723
1.50	.249	.100	-.070	-.180	3.406	2.316	2.861
2.00	.288	.115	-.087	-.225	3.954	2.905	3.430
2.50	.315	.128	-.096	-.255	4.274	3.347	3.811
2.75	.330	.134	-.101	-.270	4.480	3.558	4.019
3.00	.350	.142	-.108	-.291	4.754	3.853	4.304
2.50	.343	.139	-.104	-.281	4.663	3.726	4.195
1.75	.328	.133	-.098	-.262	4.457	3.453	3.955
1.00	.309	.126	-.090	-.240	4.183	3.160	3.672
0.50	.291	.119	-.084	-.218	3.931	2.821	3.376
0.00	.237	.101	-.071	-.178	3.109	2.254	2.681

TEST 1: CYCLE 10. GRAVITY MOMENT & POSISTIVE WIND MOMENT

A= 42.25" a= 9.75"
D_{12}= 4.375" D_{34}= 4.75"

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta$_2$ 12 (x 10^{-2} rads)	theta$_2$ 34 (x 10^{-2} rads)	theta joint (x 10^{-2} rads)
.313	0.00	.187	.081	-.068	-.130	2.423	2.484	2.454
.625	0.00	.053	.021	-.078	-.204	0.731	2.653	1.692
.938	0.00	-.065	-.022	-.082	-.220	-0.983	2.905	0.961
1.25	0.00	-.177	-.070	-.085	-.231	-2.446	3.074	0.314
"	0.25	-.166	-.068	-.088	-.241	-2.240	3.221	0.491
"	0.50	-.164	-.065	-.092	-.244	-2.263	3.200	0.469
"	0.75	-.148	-.059	-.095	-.260	-2.034	3.474	0.720
"	1.00	-.120	-.049	-.098	-.270	-1.623	3.621	0.999
"	1.25	-.082	-.033	-.102	-.280	-1.120	3.747	1.314
"	1.50	-.030	-.012	-.109	-.302	-0.411	4.063	1.826
"	1.25	-.035	-.014	-.107	-.295	-0.480	3.958	1.739
"	1.00	-.041	-.018	-.105	-.288	-0.526	3.874	1.674
"	0.75	-.066	-.026	-.101	-.279	-0.914	3.747	1.417
"	0.50	-.110	-.044	-.098	-.270	-1.509	3.621	1.056
"	0.25	-.155	-.061	-.095	-.268	-2.149	3.642	0.747
1.25	0.00	-.179	-.069	-.094	-.254	-2.514	3.368	0.427

TEST 1: CYCLE 11. GRAVITY MOMENT & NEGATIVE WIND MOMENT

1.25	-0.25	-.262	-.104	-.088	-.240	-3.611	3.200	-0.206
"	-0.50	-.299	-.118	-.088	-.225	-4.137	2.884	-0.627
"	-0.75	-.328	-.122	-.075	-.200	-4.640	2.632	-1.004
"	-1.00	-.358	-.142	-.070	-.189	-4.937	2.505	-1.216
"	-1.25	-.377	-.150	-.059	-.150	-5.189	1.916	-1.637
"	-1.50	-.399	-.158	.000	.003	-5.509	-.063	-2.723
"	-1.25	-.392	-.155	-.006	-.013	-5.417	.147	-2.635
"	-1.00	-.386	-.153	-.013	-.034	-5.326	.442	-2.442
"	-0.75	-.380	-.151	-.022	-.060	-5.234	.989	-2.123
"	-0.50	-.374	-.149	-.036	-.102	-5.143	1.389	-1.877
"	-0.25	-.365	-.145	-.067	-.173	-5.029	2.232	-1.388
1.25	0.00	-.359	-.143	-.089	-.213	-4.937	2.611	-1.163
.938	"	-.355	-.142	-.088	-.221	-4.868	2.800	-1.034
.625	"	-.345	-.138	-.080	-.212	-4.731	2.779	-0.976
.313	"	-.327	-.130	-.075	-.196	-4.503	2.547	-0.978
.156	"	-.313	-.125	-.071	-.184	-4.300	2.379	-0.961
0.00	0.00	-.296	-.119	-.067	-.170	-4.046	2.168	-0.939

TEST 2: CYCLIC SERIES 1. A= 42.5" a= 10" $D_{12} = D_{34} = 4$"

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta 12 (x 10^{-2} rads)	theta 34 (x 10^{-2} rads)	theta joint (x 10^{-2} rads)
0.125	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.25	"	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.375	"	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.50	"	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.625	"	-.002	-.001	0.000	-.002	-.025	.050	.0125
0.667	"	-.002	-.001	0.000	-.002	-.025	.050	.0125
0.667	0.125	-.002	-.001	0.000	-.004	-.025	.100	.0375
"	0.25	-.001	-.001	-.002	-.005	.000	.075	.0375
"	0.125	-.001	-.001	-.001	-.005	.000	.100	.050
"	0.00	-.002	-.001	-.001	-.003	.000	.050	.025
"	-0.125	-.007	-.004	-.002	-.006	-.075	.100	.0125
"	-0.25	-.010	-.005	-.002	-.005	-.125	.075	-.025
"	-0.125	-.008	-.004	-.002	-.006	-.100	.100	.000
"	0.00	-.006	-.003	-.004	-.008	-.075	.100	.0125
"	0.125	-.004	-.002	-.005	-.011	-.050	.150	.050
"	0.25	-.002	-.001	-.005	-.012	-.025	.175	.075
"	0.125	-.002	-.001	-.005	-.011	-.025	.150	.0625
"	0.00	-.004	-.002	-.004	-.010	-.050	.150	.050
"	-0.125	-.007	-.004	-.003	-.008	-.075	.125	.025
"	-0.25	-.010	-.005	-.002	-.006	-.125	.100	-.0125
"	-0.125	-.009	-.004	-.003	-.007	-.125	.100	-.0125
0.667	0.00	-.006	-.003	-.004	-.009	-.075	.100	.0125

TEST 2: CYCLIC SERIES 2.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{12}$ ($\times 10^{-2}$ rads)	theta $_{34}$ ($\times 10^{-2}$ rads)	theta $_{joint}$ ($\times 10^{-2}$ rads)
0.667	0.125	-.004	-.002	-.005	-.012	-0.050	0.175	0.063
"	.25	-.002	-.001	-.006	-.014	-0.025	0.200	.088
"	.375	0.000	0.000	-.007	-.017	0.000	0.250	.125
"	.50	.002	.001	-.008	-.019	0.025	0.275	.150
"	.375	.001	.001	-.007	-.017	0.000	0.250	.125
"	.25	0.000	.006	-.006	-.015	-0.150	0.225	.038
"	0.000	-.003	-.002	-.004	-.011	-0.025	0.175	.075
"	-.25	-.009	-.005	-.003	-.008	-0.100	0.125	.063
"	-.50	-.016	-.007	-.001	-.005	-0.225	0.100	-.063
"	-.25	-.012	-.005	-.003	-.008	-0.175	0.100	-.038
"	0.00	-.007	-.003	-.004	-.011	-0.100	0.175	.038
"	.25	-.004	-.002	-.005	-.014	-0.050	0.225	.038
"	.50	0.000	0.000	-.007	-.018	0.000	0.275	.138
"	.25	-.001	0.000	-.005	-.015	-0.025	0.250	.113
"	0.00	-.005	-.002	-.004	-.012	-0.075	0.200	.063
"	-.25	-.011	-.005	-.003	-.011	-0.150	0.200	.025
"	-.50	-.016	-.007	-.001	-.008	-0.225	0.175	-.025
"	-.25	-.012	-.005	-.003	-.011	-0.175	0.200	.013
0.667	0.00	-.018	-.004	-.005	-.013	-0.350	0.200	-.075

TEST 2: CYCLIC SERIES 3.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta$_{-2}$ 12 (x 10^{-2} rads)	theta$_{-2}$ 34 (x 10^{-2} rads)	theta$_{-2}$ joint (x 10^{-2} rads)
0.667	0.50	0.000	0.000	-.007	-.021	0.000	.350	.175
"	.75	.003	.002	-.009	-.024	.025	.375	.200
"	1.00	.007	.003	-.010	-.029	.100	.475	.288
"	.75	.005	.002	-.009	-.027	.075	.450	.263
"	.50	.002	.001	-.008	-.024	.025	.400	.213
"	0.00	-.004	-.002	-.005	-.016	-.025	.275	.125
"	-.50	-.014	-.006	-.002	-.011	-.200	.225	.013
"	-.75	-.019	-.008	.000	-.006	-.275	.150	-.063
"	-1.00	-.025	-.010	.002	-.001	-.375	.075	-.150
"	-.75	-.022	-.009	.000	-.004	-.325	.100	-.113
"	-.50	-.019	-.008	-.001	-.008	-.275	.175	-.050
"	0.00	-.010	-.004	-.005	-.015	-.150	.250	.050
"	.50	-.002	.000	-.008	-.019	-.050	.275	.113
"	.75	.002	.001	-.009	-.027	.025	.450	.238
"	1.00	.006	.003	-.011	-.031	.075	.500	.288
"	.75	.004	.002	-.010	-.029	.050	.475	.263
"	.50	.001	.001	-.008	-.025	.000	.425	.213
"	0.00	-.006	-.003	-.005	-.017	-.075	.300	.113
"	-.50	-.016	-.007	-.002	-.010	-.225	.200	-.013
"	-.75	-.020	-.009	.000	-.006	-.275	.150	-.063
"	-1.00	-.025	-.011	.002	-.002	-.350	.100	-.125
"	-.75	-.022	-.010	.000	-.004	-.300	.100	-.100
"	-.50	-.018	-.008	-.001	-.008	-.250	.175	-.038
0.667	0.00	-.010	-.004	-.005	-.015	-.150	.250	.050

TEST 2: CYCLIC SERIES 4.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{12}$ ($\times 10^{-2}$ rads)	theta $_{34}$ ($\times 10^{-2}$ rads)	theta $_{joint}$ ($\times 10^{-2}$ rads)
0.667	0.50	-.002	0.000	-.008	-.024	-.050	.400	.175
"	1.00	.005	.003	-.011	-.031	.000	.500	.275
"	1.25	.009	.005	-.013	-.037	.100	.600	.350
"	1.50	.015	.008	-.016	-.046	.175	.750	.463
"	1.00	.010	.005	-.013	-.040	.125	.675	.400
"	0.50	.004	.002	-.010	-.033	.050	.575	.313
"	0.00	-.004	-.002	-.006	-.025	-.050	.475	.213
"	-0.50	-.017	-.007	-.003	-.016	-.250	.325	.038
"	-1.00	-.026	-.011	.001	-.006	-.375	.175	-.100
"	-1.25	-.032	-.013	.003	.008	-.475	-.125	-.300
"	-1.50	-.040	-.016	.005	.007	-.600	-.050	-.325
"	-1.00	-.033	-.012	.002	-.001	-.525	.075	-.225
"	-0.50	-.025	-.008	-.002	-.010	-.425	.200	-.113
"	0.00	-.015	-.005	-.005	-.018	-.250	.325	.038
"	0.50	-.007	-.001	-.010	-.031	-.150	.525	.188
"	1.00	.002	.003	-.013	-.040	0.000	.675	.338
"	1.25	.007	.005	-.015	-.045	.050	.750	.400
"	1.50	.012	.007	-.017	-.051	.125	.850	.488
"	1.00	.006	.004	-.014	-.044	.050	.750	.400
"	0.50	-.001	0.000	-.010	-.036	-.025	.650	.313
"	0.00	-.009	-.004	-.007	-.027	-.125	.500	.188
"	-0.50	-.022	-.009	-.003	-.018	-.325	.375	.025
"	-1.00	-.031	-.012	.001	-.006	-.475	.175	-.150
"	-1.25	-.035	-.014	.003	-.001	-.525	.100	-.213
"	-1.50	-.041	-.016	.005	.004	-.625	.025	-.300
"	-1.00	-.034	-.013	.002	-.003	-.525	.125	-.200
"	-0.50	-.025	-.008	-.002	-.011	-.425	.225	-.100
0.667	0.00	-.016	-.004	-.006	-.020	-.300	.350	.025

TEST 2: CYCLIC SERIES 5.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{2}^{12}$ ($\times 10^{-2}$ rads)	theta $_{2}^{34}$ ($\times 10^{-2}$ rads)	theta $_{2}^{joint}$ ($\times 10^{-2}$ rads)
0.667	0.750	-.003	.001	-.012	-.038	-0.100	0.650	0.275
"	1.500	.011	.007	-.018	-.053	0.100	0.875	0.488
"	1.750	.019	.010	-.022	-.063	0.225	1.025	0.625
"	2.000	.028	.013	-.026	-.074	0.375	1.200	0.788
"	1.500	.022	.010	-.023	-.068	0.300	1.125	0.713
"	0.750	.010	.004	-.017	-.055	0.150	0.950	0.550
"	0.000	-.006	-.003	-.010	-.039	-0.075	0.725	0.325
"	-0.750	-.028	-.010	-.002	-.022	-0.450	0.500	0.003
"	-1.500	-.043	-.016	.003	-.003	-0.675	0.150	-0.263
"	-1.750	-.052	-.020	.006	.004	-0.800	0.050	-0.375
"	-2.000	-.066	-.025	.010	.015	-1.025	-0.125	-0.575
"	-1.500	-.060	-.022	.006	.006	-0.950	0.000	-0.475
"	-0.750	-.047	-.015	.000	-.007	-0.800	0.175	-0.313
"	0.000	-.030	-.008	-.007	-.023	-0.550	0.400	-0.008
"	0.750	-.006	.002	-.014	-.044	-0.200	0.750	0.275
"	1.500	.011	.009	-.021	-.064	0.050	1.075	0.563
"	1.750	.018	.011	-.024	-.072	0.175	1.200	0.688
"	2.000	.026	.015	-.028	-.083	0.275	1.375	0.825
"	1.500	.020	.011	-.025	-.077	0.225	1.300	0.763
"	0.750	.009	.005	-.019	-.062	0.100	1.075	0.588
"	0.000	-.009	-.002	-.011	-.044	-0.175	0.825	0.325
"	-0.750	-.032	-.010	-.004	-.026	-0.550	0.550	0.000
"	-1.500	-.051	-.019	.003	-.005	-0.800	0.200	-0.300
"	-1.750	-.059	-.021	.006	.002	-0.950	-0.100	-0.525
"	-2.000	-.068	-.024	.009	.011	-1.100	-0.050	-0.525
"	-1.500	-.062	-.021	.005	.003	-1.025	0.050	-0.488
"	-0.750	-.048	-.015	-.001	-.011	-0.825	0.250	-0.288
0.667	0.000	-.029	-.007	-.008	-.028	-0.550	0.500	-0.025

TEST 2: CYCLIC SERIES 6.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{-2}$ 12 (x 10^{-2} rads)	theta $_{-2}$ 34 (x 10^{-2} rads)	theta $_{-2}$ joint (x 10^{-2} rads)
0.667	1.00	-.004	.003	-.017	-.054	-0.175	0.925	0.375
"	2.00	.023	.013	-.028	-.083	0.250	1.375	0.813
"	2.25	.035	.017	-.034	-.098	0.450	1.600	1.025
"	2.50	.060	.025	-.048	-.133	0.875	2.125	1.505
"	1.75	.052	.020	-.043	-.123	0.800	2.000	1.400
"	0.50	.028	.009	-.033	-.100	0.475	1.675	1.075
"	0.00	.009	.003	-.028	-.088	0.150	1.500	0.825
"	-1.60	-.054	-.020	-.007	-.031	-0.850	0.600	-0.125
"	-2.00	-.069	-.025	0.000	-.010	-1.100	0.250	-0.425
"	-2.25	-.084	-.030	.007	.016	-1.350	-0.225	-0.785
"	-2.50	-.102	-.038	.015	.036	-1.600	-0.525	-1.063
"	-1.75	-.096	-.033	.010	.026	-1.575	-0.400	-0.988
"	-0.50	-.071	-.022	-.001	-.003	-1.225	0.050	-0.588
"	0.00	-.053	-.015	-.008	-.021	-0.950	0.325	-0.313
"	1.00	-.023	-.005	-.017	-.048	-0.450	0.775	0.163
"	2.00	.025	.013	-.037	-.104	0.030	1.675	0.988
"	2.25	.038	.018	-.044	-.121	0.050	1.925	1.213
"	2.50	.054	.023	-.052	-.142	0.775	2.250	1.513
"	1.75	.045	.018	-.047	-.131	0.675	2.100	1.388
"	0.50	.019	.006	-.035	-.105	0.325	1.750	1.038
"	0.00	-.001	-.001	-.029	-.093	0.000	1.600	0.800
"	-1.00	-.049	-.018	-.015	-.054	-0.775	0.975	0.100
"	-2.00	-.094	-.034	.005	.006	-1.500	-0.025	-0.763
"	-2.25	-.105	-.039	.011	.022	-1.650	-0.275	-0.963
"	-2.50	-.128	-.048	.020	.047	-2.000	-0.675	-1.338
"	-1.75	-.118	-.043	.013	.033	-1.875	-0.500	-1.188
"	-0.50	-.095	-.032	.001	.004	-1.575	-0.075	-0.825
"	0.00	-.080	-.025	-.004	-.012	-1.375	0.200	-0.588
"	1.00	-.042	-.011	-.020	-.054	-0.775	0.850	0.038
"	2.00	.020	.010	-.043	-.129	0.250	2.150	1.200
"	2.25	.036	.016	-.050	-.135	0.500	2.125	1.313
"	2.50	.054	.023	-.056	-.152	0.775	2.400	1.588
"	1.75	.045	.017	-.051	-.142	0.700	2.275	1.488
"	0.50	.017	.005	-.039	-.116	0.300	1.925	1.113
0.667	0.00	-.008	-.005	-.033	-.102	0.075	1.725	0.900
0.500	"	-.009	-.005	-.032	-.100	0.100	1.700	0.900
0.000	0.00	-.006	-.004	-.029	-.096	0.050	1.675	0.863

TEST 2: CYCLIC SERIES 7.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{-2}$ 12 (x 10^{-2} rads)	theta $_{-2}$ 34 (x 10^{-2} rads)	theta $_{-2}$ joint (x 10^{-2} rads)
0.00	0.00	-.006	-.004	-.025	-.093	-0.050	1.700	0.825
0.67	0.00	-.010	-.005	-.027	-.097	-0.125	1.750	0.813
"	-1.00	-.059	-.022	-.015	-.066	-0.925	1.275	0.175
"	-2.00	-.110	-.041	.009	-.000	-1.725	0.225	-0.750
"	-2.50	-.135	-.049	.022	.038	-2.150	-0.400	-1.275
"	-2.75	-.158	-.060	.031	.063	-2.450	-0.800	-1.625
"	-3.00	-.190	-.073	.045	.098	-2.925	-1.375	-2.125
"	-2.50	-.183	-.070	.041	.092	-2.825	-1.275	-2.050
"	-1.25	-.163	-.061	.030	.067	-2.550	-0.925	-1.738
"	-0.50	-.148	-.055	.022	.048	-2.323	-0.650	-1.488
"	0.00	-.131	-.048	.015	.029	-2.075	-0.350	-1.213
"	1.00	-.092	-.034	-.006	-.033	-1.450	0.675	-0.388
"	2.00	-.005	-.002	-.038	-.116	-0.075	1.950	0.713
"	2.50	.055	.020	-.058	-.165	0.875	2.675	1.775
"	2.75	.085	.029	-.068	-.193	1.400	3.125	2.263
"	3.00	.115	.040	-.082	-.227	1.875	3.625	2.750
"	2.50	.110	.036	-.079	-.221	1.850	3.550	2.700
"	1.10	.086	.025	-.067	-.194	1.525	3.175	2.350
"	0.50	.067	.019	-.060	-.178	1.200	2.950	2.075
"	0.00	.034	.007	-.054	-.160	0.675	2.650	1.663
"	-1.00	-.042	-.020	-.036	-.114	-0.550	1.950	0.700
"	-2.00	-.146	-.057	.005	-.004	-2.225	0.225	-1.000
"	-2.50	-.172	-.066	.028	.061	-2.650	-0.825	-1.738
"	-2.75	-.182	-.070	.037	.084	-2.800	-1.175	-1.988
"	-3.00	-.196	-.075	.047	.110	-3.025	-1.575	-2.300
"	-2.50	-.190	-.072	.042	.090	-2.950	-1.200	-2.075
"	-1.25	-.169	-.063	.030	.072	-2.650	-1.050	-1.850
"	-0.50	-.147	-.053	.022	.051	-2.350	-0.725	-1.538
"	0.00	-.136	-.050	.013	.026	-2.150	-0.325	-1.238
"	1.00	-.086	-.031	-.010	-.037	-1.375	0.675	-0.350
"	2.00	.032	.011	-.060	-.172	0.525	2.800	1.663
"	2.50	.082	.028	-.077	-.216	1.350	3.475	2.413
"	2.75	.102	.035	-.085	-.232	1.675	3.675	2.675
"	3.00	.122	.042	-.094	-.256	2.000	4.050	3.025
"	2.38	.114	.037	-.088	-.246	1.925	3.950	2.938
"	1.18	.092	.028	-.077	-.223	1.600	3.650	2.625
"	0.50	.070	.020	-.070	-.203	1.250	3.325	2.288
"	0.00	.038	.007	-.063	-.188	0.775	3.125	1.950

TEST 2: CYCLIC SERIES 7 cont.

"	-1.00	-.045	-.020	-.044	-.133	-0.625	2.225	0.800
"	-2.00	-.156	-.060	.000	-.017	-2.400	0.425	-0.988
"	-2.50	-.182	-.070	.025	.052	-2.800	-0.675	-1.738
"	-2.75	-.193	-.073	.034	.079	-3.000	-1.125	-2.063
"	-3.00	-.210	-.080	.046	.111	-3.250	-1.625	-2.438
"	-2.50	-.203	-.076	.041	.098	-3.175	-1.425	-2.300
"	-1.25	-.182	-.067	.029	.072	-2.875	-1.075	-1.975
"	-0.50	-.165	-.060	.020	.047	-2.625	-0.675	-1.650
"	0.00	-.148	-.054	.012	.027	-2.350	-0.375	-1.363
"	1.00	-.094	-.035	-.014	-.045	-1.475	0.775	-0.350
"	2.00	.035	.012	-.066	-.186	0.575	3.000	1.788
"	3.00	.123	.042	-.094	-.255	2.025	4.025	3.025
"	3.25	.139	.048	-.101	-.274	2.275	4.325	3.300
"	3.50	.173	.060	-.116	-.314	2.825	4.950	3.888
"	2.50	.159	.053	-.109	-.300	2.650	4.775	3.713
0.67	1.25	.138	.045	-.099	-.277	2.325	4.450	3.388
0.67	0.00	.080	.024	-.085	-.236	1.400	3.775	2.588
0.00	0.00	.085	.025	-.082	-.231	1.500	3.725	2.613

TEST 2: CYCLIC SERIES 8.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{-2}$ 12 (x 10^{-2} rads)	theta $_{-2}$ 34 (x 10^{-2} rads)	theta $_{-2}$ joint (x 10^{-2} rads)
0.667	0.00	.079	.024	-.084	-.234	1.375	3.750	2.563
"	-1.00	.009	.002	-.068	-.193	0.175	3.125	1.650
"	-2.00	-.132	-.051	-.016	-.048	-2.025	0.800	-.613
"	-3.00	-.208	-.079	.043	.109	-3.225	-1.650	-2.438
"	-3.25	-.225	-.085	.053	.135	-3.500	-2.100	-2.775
"	-3.50	-.256	-.099	.079	.196	-3.950	-2.950	-3.435
"	-2.50	-.244	-.093	.073	.182	-3.775	-2.725	-3.250
"	-1.25	-.221	-.082	.060	.154	-3.475	-2.350	-2.913
"	-0.50	-.205	-.075	.050	.129	-3.250	-1.975	-2.613
"	0.00	-.182	-.067	.042	.106	-2.875	-1.600	-2.238
"	1.00	-.120	-.047	.010	.017	-1.825	-0.175	-1.00
"	2.00	.057	.012	-.060	-.177	1.150	2.925	2.038
"	3.00	.157	.052	-.106	-.290	2.625	4.600	3.613
"	3.25	.171	.058	-.110	-.301	2.825	4.775	3.800
"	3.50	.204	.072	-.118	-.320	3.300	5.050	4.175
"	2.50	.186	.063	-.109	-.301	3.075	4.800	3.938
"	1.25	.164	.053	-.099	-.278	2.775	4.475	3.625
"	0.50	.141	.045	-.091	-.256	2.400	4.125	3.263
"	0.00	.108	.034	-.086	-.242	1.850	3.900	2.875
"	-1.00	.015	.002	-.061	-.173	0.325	2.800	1.563
"	-2.00	-.115	-.036	-.010	-.031	-1.975	-0.525	-0.725
"	-3.00	-.242	-.092	.061	.147	-3.750	-2.200	-2.975
"	-3.25	-.255	-.098	.072	.176	-3.925	-2.600	-3.263
"	-3.50	-.278	-.105	.086	.209	-4.325	-3.075	-3.700
"	-2.50	-.263	-.098	.078	.196	-4.125	-2.950	-3.538
"	-1.25	-.247	-.093	.066	.167	-3.850	-2.525	-3.188
"	-0.50	-.227	-.085	.055	.141	-3.475	-2.150	-2.813
"	0.00	-.205	-.076	.047	.116	-3.225	-1.725	-2.475
"	1.00	-.141	-.055	.012	.022	-2.150	-0.250	-1.200
"	2.00	.061	.018	-.074	-.216	1.075	3.550	2.313
"	3.00	.175	.060	-.112	-.304	2.875	4.800	3.838
"	3.25	.194	.068	-.119	-.319	3.150	5.000	4.075
0.67	3.50	.217	.078	-.126	-.336	3.475	5.250	4.363

TEST 3: CYCLIC SERIES 1. A= 43" a= 9" D_{12}= D_{34}= 4.0" D_{20}= D_{30}= 4.0"

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{12}$ ($\times 10^{-2}$ rads)	theta $_{34}$ ($\times 10^{-2}$ rads)	theta $_{joint}$ ($\times 10^{-2}$ rads)
0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.41	"	-.005	-.002	-.001	-.003	-0.075	0.050	-0.013
0.81	"	-.006	-.002	-.002	-.004	-0.100	0.050	-0.025
1.21	"	-.008	-.003	-.003	-.007	-0.125	0.100	-0.013
1.61	"	-.013	-.005	-.004	-.010	-0.200	0.150	-0.025
2.15	0.00	-.022	-.009	-.005	-.014	-0.325	0.225	-0.050
"	0.25	-.020	-.008	-.007	-.018	-0.300	0.275	-0.013
"	0.50	-.016	-.006	-.008	-.022	-0.250	0.350	0.050
"	0.25	-.019	-.008	-.007	-.020	-0.275	0.325	0.025
"	0.00	-.023	-.010	-.006	-.018	-0.325	0.300	-0.013
"	-0.25	-.030	-.012	-.004	-.013	-0.450	0.225	-0.113
"	-0.50	-.040	-.016	-.002	-.009	-0.600	0.175	-0.213
"	-0.25	-.037	-.015	-.005	-.013	-0.550	0.200	-0.175
"	0.00	-.034	-.014	-.006	-.016	-0.500	0.250	-0.125
"	0.25	-.030	-.012	-.007	-.020	-0.450	0.325	-0.063
"	0.50	-.026	-.010	-.008	-.022	-0.400	0.350	-0.025
"	0.25	-.028	-.012	-.007	-.020	-0.400	0.325	-0.038
"	0.00	-.032	-.013	-.006	-.018	-0.475	0.300	-0.088
"	-0.25	-.037	-.015	-.005	-.013	-0.550	0.200	-0.175
"	-0.50	-.041	-.017	-.003	-.010	-0.600	0.175	-0.213
"	-0.25	-.038	-.016	-.005	-.014	-0.550	0.225	-0.163
2.15	0.00	-.034	-.014	-.006	-.017	-0.500	0.275	-0.113

TEST 3: CYCLIC SERIES 2.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{-2}$ 12 (x 10^{-2} rads)	theta $_{-2}$ 34 (x 10^{-2} rads)	theta $_{-2}$ joint (x 10^{-2} rads)
2.15	0.50	-.026	-.010	-.009	-.023	-0.400	0.350	-0.250
"	0.75	-.021	-.007	-.010	-.028	-0.350	0.450	0.050
"	1.00	-.016	-.005	-.013	-.034	-0.275	0.525	0.125
"	0.50	-.022	-.009	-.010	-.029	-0.325	0.475	0.075
"	0.00	-.030	-.012	-.008	-.025	-0.450	0.425	-0.013
"	-0.50	-.040	-.017	-.005	-.019	-0.575	0.350	-0.113
"	-0.75	-.049	-.021	-.003	-.014	-0.700	0.275	-0.213
"	-1.00	-.065	-.026	.000	-.008	-0.975	0.200	-0.388
"	-0.50	-.059	-.022	-.004	-.016	-0.925	0.300	-0.313
"	0.00	-.052	-.020	-.007	-.023	-0.800	0.400	-0.200
"	0.50	-.044	-.016	-.004	-.029	-0.700	0.625	-0.038
"	0.75	-.040	-.014	-.012	-.033	-0.650	0.525	-0.063
"	1.00	-.034	-.012	-.014	-.038	-0.550	0.600	0.025
"	0.50	-	-	-	-	-	-	-
"	0.00	-.046	-.012	-.009	-.028	-0.850	0.475	-0.188
"	-0.50	-.058	-.022	-.005	-.020	-0.900	0.375	-0.264
"	-0.75	-.063	-.025	-.003	-.015	-0.950	0.300	-0.325
"	-1.00	-.071	-.029	-.001	-.010	-1.050	0.225	-0.413
"	-0.50	-.063	-.025	-.005	-.020	-0.950	0.375	-0.288
2.15	0.00	-.055	-.021	-.008	-.025	-0.850	0.425	-0.213

TEST 3: CYCLIC SERIES 3.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{12}$ ($\times 10^{-2}$ rads)	theta $_{34}$ ($\times 10^{-2}$ rads)	theta $_{joint}$ ($\times 10^{-2}$ rads)
2.15	0.50	-.048	-.018	-.010	-.031	-0.750	0.525	-0.113
"	1.00	-.040	-.014	-.013	-.036	-0.650	0.575	-0.038
"	1.25	-.032	-.010	-.017	-.050	-0.550	0.825	0.138
"	1.50	-.025	-.007	-.025	-.077	-0.450	1.300	0.425
"	1.00	-.029	-.009	-.022	-.072	-0.500	1.250	0.375
"	0.50	-.036	-.012	-.019	-.065	-0.600	1.150	0.275
"	0.00	-.045	-.016	-.018	-.061	-0.725	1.075	0.125
"	-0.50	-.059	-.022	-.014	-.053	-0.925	0.975	0.025
"	-1.00	-.069	-.028	-.010	-.043	-1.025	0.825	-0.100
"	-1.25	-.096	-.035	-.008	-.037	-1.525	0.730	-0.400
"	-1.50	-.109	-.040	-.004	-.024	-1.725	0.500	-0.613
"	-1.00	-.102	-.038	-.008	-.032	-1.600	0.600	-0.500
"	-0.50	-.095	-.033	-.011	-.041	-1.550	0.750	-0.400
"	0.00	-.088	-.030	-.016	-.050	-1.450	0.850	-0.300
"	0.50	-.082	-.027	-.019	-.059	-1.375	1.000	-0.188
"	1.00	-.072	-.022	-.022	-.070	-1.250	1.200	-0.025
"	1.25	-.066	-.020	-.024	-.074	-1.150	1.250	0.050
"	1.50	-.061	-.016	-.035	-.101	-1.125	1.650	0.263
"	1.00	-.064	-.019	-.031	-.094	-1.175	1.575	0.225
"	0.50	-.070	-.023	-.028	-.088	-1.175	1.500	0.163
"	0.00	-.079	-.027	-.025	-.083	-1.300	1.450	0.075
"	-0.50	-.089	-.032	-.021	-.075	-1.425	1.350	-0.038
"	-1.00	-.100	-.037	-.017	-.066	-1.575	1.225	-0.175
"	-1.25	-.105	-.040	-.016	-.063	-1.625	1.175	-0.225
"	-1.50	-.110	-.041	-.012	-.054	-1.725	1.050	-0.338
"	-1.00	-.103	-.038	-.015	-.059	-1.625	1.100	-0.265
"	-0.50	-.096	-.034	-.019	-.067	-1.550	1.200	-0.175
2.15	0.00	-.089	-.030	-.023	-.075	-1.475	1.300	-0.088

TEST 3: CYCLIC SERIES 4.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{-2}$ 12 (x 10^{-2} rads)	theta $_{-2}$ 34 (x 10^{-2} rads)	theta $_{-2}$ joint (x 10^{-2} rads)
2.15.	0.50	-.082	-.027	-.025	-.082	-1.375	1.425	0.025
"	1.00	-.072	-.022	-.030	-.091	-1.250	1.525	0.138
"	1.50	-.063	-.019	-.039	-.112	-1.100	1.825	0.363
"	1.75	-.054	-.014	-.045	-.126	-1.000	2.025	0.513
"	2.00	-.040	-.010	-.053	-.146	-0.750	2.325	0.788
"	1.50	-.045	-.012	-.050	-.141	-0.825	2.275	0.725
"	0.50	-.059	-.019	-.044	-.126	-1.000	2.050	0.525
"	0.00	-.064	-.024	-.041	-.122	-1.000	2.025	0.513
"	-0.50	-.083	-.030	-.036	-.113	-1.325	1.925	0.300
"	-1.00	-.096	-.036	-.032	-.102	-1.500	1.750	0.125
"	-1.50	-.113	-.042	-.028	-.090	-1.775	1.550	-0.113
"	-1.75	-.125	-.047	-.023	-.080	-1.950	1.425	-0.263
"	-2.00	-.140	-.052	-.018	-.066	-2.200	1.200	-0.500
"	-1.50	-.134	-.050	-.021	-.073	-2.100	1.300	-0.400
"	-0.50	-.116	-.041	-.030	-.091	-1.875	1.525	-0.175
"	0.00	-.110	-.038	-.036	-.104	-1.800	1.700	-0.050
"	0.60	-.100	-.034	-.042	-.117	-1.650	1.878	0.113
"	1.00	-.091	-.030	-.046	-.127	-1.525	2.025	0.250
"	1.50	-.084	-.025	-.051	-.144	-1.475	2.325	0.425
"	1.75	-.075	-.022	-.055	-.148	-1.325	2.325	0.500
"	2.00	-.064	-.017	-.064	-.171	-1.175	2.675	0.750
"	1.50	-.069	-.020	-.061	-.166	-1.225	2.630	0.700
"	0.50	-.083	-.027	-.054	-.151	-1.400	2.425	0.513
"	0.00	-.093	-.032	-.051	-.145	-1.525	2.350	0.413
"	-0.50	-.107	-.039	-.047	-.136	-1.700	2.225	0.263
"	-1.00	-.118	-.043	-.042	-.126	-1.875	2.100	0.113
"	-1.50	-.131	-.049	-.038	-.114	-2.050	1.900	-0.075
"	-1.75	-.138	-.052	-.034	-.104	-2.150	1.750	-0.200
"	-2.00	-.145	-.054	-.030	-.093	-2.225	1.575	-0.350
"	-1.50	-.130	-.052	-.033	-.100	-2.200	1.675	-0.263
"	-0.50	-.131	-.043	-.042	-.118	-1.950	1.900	-0.025
2.15	0.00	-.113	-.040	-.046	-.129	-1.825	2.075	0.125

TEST 3: CYCLIC SERIES 5.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta 12 ($\times 10^{-2}$ rads)	theta 34 ($\times 10^{-2}$ rads)	theta joint ($\times 10^{-2}$ rads)
2.15	1.00	-.094	-.031	-.054	-.150	-1.575	2.400	0.413
"	2.00	-.074	-.021	-.068	-.176	-1.325	2.700	0.688
"	2.25	-.060	-.015	-.074	-.197	-1.250	3.075	0.913
"	2.50	-.045	-.010	-.084	-.220	-0.875	3.400	1.263
"	1.75	-.053	-.014	-.080	-.212	-0.975	3.300	1.163
"	0.50	-.075	-.025	-.064	-.192	-1.250	3.200	0.975
"	0.00	-.087	-.030	-.068	-.188	-1.425	3.000	0.788
"	-1.00	-.122	-.045	-.058	-.164	-1.925	2.650	0.363
"	-2.00	-.154	-.058	-.046	-.132	-2.400	2.150	-0.125
"	-2.25	-.170	-.064	-.040	-.116	-2.650	1.900	-0.375
"	-2.50	-.196	-.075	-.031	-.093	-3.025	1.550	-.738
"	-1.75	-.187	-.071	-.038	-.107	-2.900	1.725	-.588
"	-0.50	-.164	-.060	-.050	-.133	-2.600	2.075	-.263
"	0.00	-.156	-.055	-.057	-.148	-2.525	2.275	-.125
"	1.00	-.132	-.044	-.069	-.180	-2.200	2.775	.288
"	2.00	-.110	-.035	-.083	-.215	-1.875	3.300	.713
"	2.25	-.099	-.031	-.087	-.225	-1.700	3.450	.875
"	2.50	-.080	-.024	-.095	-.244	-1.400	3.725	1.163
"	1.75	-.092	-.030	-.090	-.233	-1.550	3.575	1.013
"	0.50	-.108	-.040	-.080	-.215	-1.700	3.375	.838
"	0.00	-.128	-.046	-.079	-.211	-2.050	3.300	.625
"	-1.00	-.160	-.060	-.068	-.187	-2.500	2.975	.238
"	-2.00	-.185	-.070	-.055	-.155	-2.875	2.500	-.188
"	-2.25	-.192	-.074	-.052	-.143	-2.950	2.275	-.338
"	-2.50	-.199	-.073	-.047	-.129	-3.150	2.050	-.550
"	-1.75	-.189	-.072	-.052	-.139	-2.925	2.175	-.375
"	-0.50	-.168	-.062	-.064	-.165	-2.650	2.525	-.063
2.15	0.00	-.160	-.058	-.070	-.181	-2.550	2.775	.113

TEST 3: CYCLIC SERIES 6.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{12}$ ($\times 10^{-2}$ rads)	theta $_{34}$ ($\times 10^{-2}$ rads)	theta $_{joint}$ ($\times 10^{-2}$ rads)
2.15	1.25	-.124	-.045	-.082	-.212	-1.975	3.250	0.638
"	2.50	-.098	-.030	-.097	-.249	-1.700	3.800	1.050
"	2.75	-.082	-.025	-.105	-.267	-1.425	4.050	1.313
"	3.00	-.059	-.016	-.120	-.298	-1.075	4.450	1.688
"	1.75	-.077	-.025	-.112	-.282	-1.300	4.250	1.475
"	0.50	-.102	-.037	-.102	-.263	-1.750	4.025	1.138
"	0.00	-.121	-.045	-.099	-.258	-1.900	3.975	1.038
"	-1.25	-.170	-.064	-.086	-.229	-2.650	3.575	0.463
"	-2.50	-.205	-.079	-.062	-.170	-3.150	2.700	-0.225
"	-2.75	-.230	-.090	-.054	-.145	-3.500	2.275	-0.613
"	-3.00	-.259	-.103	-.045	-.120	-3.900	1.875	-1.013
"	-1.75	-.244	-.095	-.057	-.143	-3.725	2.150	-0.788
"	-0.50	-.221	-.089	-.070	-.174	-3.300	2.600	-0.350
"	0.00	-.214	-.081	-.069	-.198	-3.323	3.225	-0.050
0.00	0.00	-	-	-	-	-	-	-
2.15	1.25	-.186	-.067	-.103	-.246	-2.975	3.575	0.300
"	2.50	-.147	-.051	-.115	-.287	-2.400	4.300	0.950
"	2.75	-.133	-.046	-.119	-.296	-2.125	4.425	1.150
"	3.00	-.115	-.039	-.124	-.309	-1.900	4.625	1.363
"	1.75	-.135	-.049	-.116	-.292	-2.150	4.400	1.125
"	0.50	-.159	-.060	-.102	-.270	-2.475	4.100	0.813
"	0.00	-.176	-.067	-.105	-.267	-2.725	4.050	0.663
"	-1.25	-.221	-.086	-.092	-.240	-3.375	3.700	0.163
"	-2.50	-.252	-.100	-.066	-.177	-3.825	2.775	-0.525
"	-2.75	-.262	-.104	-.060	-.160	-3.950	2.500	-0.725
"	-3.00	-.280	-.112	-.052	-.140	-4.200	2.200	-1.000
"	-1.75	-.262	-.104	-.065	-.164	-3.950	2.475	-0.738
"	-0.50	-.242	-.096	-.078	-.194	-3.650	2.900	-0.375
"	0.00	-.231	-.089	-.086	-.215	-3.550	3.225	-0.163
"	1.25	-.201	-.074	-.105	-.262	-3.175	3.925	0.375
"	2.50	-.160	-.056	-.121	-.300	-2.600	4.475	0.938
"	2.75	-.147	-.051	-.124	-.308	-2.400	4.600	1.100
"	3.00	-.131	-.045	-.128	-.319	-2.150	4.775	1.313
"	1.75	-.150	-.045	-.120	-.304	-2.400	4.600	1.100
"	0.50	-.176	-.067	-.111	-.281	-2.725	4.250	0.763
2.15	0.00	-.191	-.074	-.109	-.276	-2.925	4.150	0.600
0.81	0.00	-.182	-.070	-.103	-.262	-2.800	3.975	0.588
0.00	0.00	-.179	-.069	-.101	-.257	-2.750	3.900	0.575

TEST 4: MONOTOMIC LOADING CURVE $D_{12} = D_{34} = 4.0"$ $D_{20} = D_{30} = 4.0"$ $A = 43.5"$ $a = 9.5"$

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta $_{-2}^{12}$ (x 10^{-2} rads)	theta $_{-2}^{34}$ (x 10^{-2} rads)	theta $_{-2}^{joint}$ (x 10^{-2} rads)
0.00	0.00	.000	.000	.000	.000	0.000	0.000	0.000
	0.25	.003	.001	-.001	-.003	0.050	0.050	0.050
	0.50	.006	.003	-.003	-.007	0.075	0.100	0.088
	0.75	.009	.004	-.005	-.010	0.125	0.125	0.125
	1.00	.012	.006	-.006	-.014	0.150	0.200	0.175
	1.25	.017	.008	-.009	-.020	0.225	0.275	0.250
	1.50	.022	.011	-.011	-.025	0.275	0.350	0.313
	1.75	.030	.014	-.014	-.033	0.400	0.475	0.438
	2.00	.042	.019	-.018	-.043	0.600	0.625	0.613
	2.25	.056	.024	-.024	-.061	0.800	0.925	0.863
	2.50	.070	.029	-.029	-.074	1.025	1.125	1.075
	2.75	.088	.036	-.036	-.087	1.300	1.275	1.288
	3.00	.140	.059	-.063	-.150	2.025	2.200	2.113
	3.25	.179	.077	-.082	-.198	2.550	2.900	2.725
	3.50	.219	.095	-.104	-.244	3.100	3.500	3.300
	3.00	.213	.092	-.099	-.235	3.025	3.425	3.225
	2.50	.206	.088	-.095	-.229	2.950	3.350	3.150
	2.00	.199	.085	-.091	-.222	2.850	3.275	3.063
	1.50	.191	.081	-.087	-.214	2.750	3.175	2.963
	1.00	.182	.077	-.083	-.205	2.625	3.050	2.838
	0.50	.169	.070	-.078	-.196	2.475	2.950	2.713
	0.25	.162	.067	-.076	-.193	2.375	2.925	2.650
	0.00	.152	.063	-.075	-.190	2.225	2.875	2.550

TEST 5: CYCLIC SERIES 1. A=43" a=8.5" $D_{12}=D_{34}=4.0"$ $D_{20}=D_{30}=4.0"$

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$theta_2$ 12 ($x10^{-2}$ rads)	$theta_2$ 34 ($x10^{-2}$ rads)	$theta_2$ joint ($x10^{-2}$ rads)
0.00	0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2.14	0.00	-0.017	-0.007	-0.008	-0.018	-0.250	0.250	0.000
2.00	0.25	-0.015	-0.007	-0.010	-0.021	-0.200	0.275	0.038
1.98	0.50	-0.011	-0.005	-0.011	-0.025	-0.150	0.350	0.100
1.96	0.25	-0.012	-0.005	-0.010	-0.023	-0.175	0.325	0.075
1.94	0.00	-0.015	-0.007	-0.009	-0.020	-0.200	0.275	0.038
1.94	-0.25	-0.020	-0.009	-0.007	-0.016	-0.275	0.225	-0.025
1.93	-0.50	-0.024	-0.011	-0.005	-0.011	-0.325	0.150	-0.088
1.82	0.00	-0.018	-0.008	-0.008	-0.018	-0.250	0.250	0.000
1.84	0.25	-0.015	-0.007	-0.010	-0.021	-0.200	0.275	0.038
1.89	0.50	-0.012	-0.005	-0.012	-0.024	-0.175	0.300	0.063
1.84	0.00	-0.016	-0.008	-0.010	-0.019	-0.200	0.225	0.013
1.82	-0.25	-0.021	-0.010	-0.008	-0.015	-0.275	0.175	-0.050
1.86	-0.50	-0.025	-0.011	-0.006	-0.010	-0.350	0.100	-0.075
1.82	0.00	-0.018	-0.008	-0.009	-0.018	-0.250	0.225	-0.013

TEST 5: CYCLIC SERIES 2.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$theta_2$ 12 ($x10^{-2}$ rads)	$theta_2$ 34 ($x10^{-2}$ rads)	$theta_2$ joint ($x10^{-2}$ rads)
1.88	0.50	-0.012	-0.005	-0.013	-0.025	-0.175	0.300	0.063
1.76	0.75	-0.008	-0.003	-0.015	-0.031	-0.125	0.400	0.138
1.71	1.00	-0.002	0.000	-0.019	-0.045	-0.050	0.650	0.300
1.26	0.50	-0.003	-0.001	-0.016	-0.038	-0.050	0.550	0.250
1.03	0.00	-0.008	-0.004	-0.013	-0.031	-0.100	0.450	0.175
1.10	-0.50	-0.018	-0.008	-0.008	-0.020	-0.250	0.300	0.025
1.28	-0.75	-0.024	-0.011	-0.006	-0.017	-0.325	0.275	-0.025
1.61	-1.00	-0.035	-0.015	-0.005	-0.015	-0.500	0.250	-0.130
0.91	-0.50	-0.024	-0.011	-0.007	-0.017	-0.325	0.250	-0.038
0.78	0.00	-0.013	-0.005	-0.010	-0.025	-0.200	0.375	0.088
1.04	0.50	-0.009	-0.004	-0.015	-0.033	-0.125	0.450	0.163
1.36	0.75	-0.007	-0.003	-0.018	-0.038	-0.100	0.500	0.200
1.62	1.00	-0.005	-0.002	-0.020	-0.044	-0.125	0.600	0.238
1.22	0.50	-0.007	-0.003	-0.017	-0.037	-0.100	0.500	0.200
0.86	0.00	-0.010	-0.005	-0.013	-0.028	-0.125	0.375	0.125
0.96	-0.50	-0.022	-0.010	-0.009	-0.018	-0.300	0.225	-0.038
1.22	-0.75	-0.029	-0.014	-0.008	-0.016	-0.375	0.200	-0.088
1.60	-1.00	-0.039	-0.017	-0.007	-0.015	-0.550	0.200	-0.175
0.78	-0.50	-0.024	-0.012	-0.008	-0.017	-0.300	0.225	-0.038
0.55	0.00	-0.016	-0.007	-0.011	-0.023	-0.225	0.300	0.038

TEST 5: CYCLIC SERIES 3.

Pg (kips)	Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$theta_{2}$ 12 ($x10^{-2}$ rads)	$theta_{2}$ 34 ($x10^{-2}$ rads)	$theta_{2}$ joint ($x10^{-2}$ rads)
0.99	0.50	-0.010	-0.004	-0.016	-0.033	-0.150	0.425	0.138
1.72	1.00	-0.008	-0.002	-0.022	-0.046	-0.150	0.600	0.225
1.76	1.25	-0.004	-0.001	-0.029	-0.067	-0.075	0.950	0.438
1.50	1.50	0.006	0.003	-0.035	-0.083	0.075	1.200	0.638
0.92	0.50	-0.003	-0.002	-0.028	-0.066	-0.025	0.950	0.463
0.02	0.00	-0.006	-0.003	-0.021	-0.052	-0.075	0.775	0.350
0.85	-0.50	-0.022	-0.010	-0.019	-0.048	-0.300	0.725	0.213
1.42	-1.00	-0.037	-0.017	-0.017	-0.042	-0.500	0.675	0.063
1.42	-1.25	-0.044	-0.019	-0.014	-0.034	-0.625	0.500	-0.063
0.90	-1.50	-0.055	-0.025	-0.010	-0.025	-0.750	0.375	-0.188
0.83	-0.50	-0.040	-0.017	-0.017	-0.038	-0.575	0.525	-0.025
0.00	0.00	-0.022	-0.009	-0.018	-0.040	-0.375	0.550	0.113
1.04	1.00	-0.009	-0.003	-0.030	-0.068	-0.150	0.950	0.400
1.14	1.25	-0.004	-0.001	-0.032	-0.075	-0.075	1.075	0.500
1.09	1.50	0.001	0.001	-0.035	-0.080	0.000	1.125	0.563
0.93	0.50	-0.009	-0.004	-0.028	-0.066	-0.125	0.950	0.413
0.00	0.00	-0.012	-0.006	-0.021	-0.050	-0.150	0.725	0.288
1.00	-1.00	-0.045	-0.020	-0.015	-0.037	-0.625	0.550	-0.038
1.04	-1.25	-0.051	-0.023	-0.013	-0.030	-0.700	0.425	-0.138
1.06	-1.50	-0.057	-0.025	-0.010	-0.024	-0.800	0.350	-0.225
0.76	-0.50	-0.041	-0.018	-0.017	-0.038	-0.575	0.525	-0.025
0.00	0.00	-0.022	-0.009	-0.018	-0.040	-0.325	0.550	0.113

TEST 6: CYCLIC SERIES 1. A=43.5" a=9" $D_{12}=D_{34}=4"$ $D_{20}=D_{30}=4"$

P_w (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$theta_2$ 12 ($x10^{-2}$ rads)	$theta_2$ 34 ($x10^{-2}$ rads)	$theta_2$ joint ($x10^{-2}$ rads)
0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.25	0.005	0.002	-0.002	-0.003	0.075	0.025	0.050
0.50	0.010	0.005	-0.004	-0.009	0.125	0.125	0.125
0.25	0.009	0.004	-0.004	-0.007	0.125	0.075	0.100
0.00	0.005	0.002	-0.002	-0.002	0.075	0.000	0.038
-0.25	-0.003	-0.002	0.002	0.006	-0.025	-0.100	-0.063
-0.50	-0.009	-0.004	0.004	0.013	-0.125	-0.225	-0.175
-0.25	-0.006	-0.002	0.003	0.009	-0.088	-0.163	-0.125
0.00	-0.001	0.000	0.000	0.004	-0.025	-0.100	-0.063
0.25	0.005	0.002	-0.002	-0.001	0.063	0.025	0.044
0.50	0.010	0.005	-0.005	-0.008	0.138	0.075	0.106
0.25	0.008	0.003	-0.003	-0.004	0.125	0.025	0.075
0.00	0.003	0.001	-0.001	0.000	0.050	0.025	0.038
-0.50	-0.008	-0.004	0.004	0.012	-0.100	-0.200	-0.150
-0.25	-0.005	-0.002	0.003	0.009	-0.075	-0.163	-0.119
0.00	-0.001	0.000	0.000	0.004	-0.025	-0.088	-0.056

TEST 6: CYCLIC SERIES 2.

Rw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$theta_{2\ 12}$ ($x10^{-2}$ rads)	$theta_{2\ 34}$ ($x10^{-2}$ rads)	$theta_{2\ joint}$ ($x10^{-2}$ rads)
0.50	0.010	0.004	-0.004	-0.005	0.150	0.025	0.088
0.75	0.016	0.007	-0.006	-0.011	0.225	0.125	0.175
1.00	0.026	0.010	-0.009	-0.019	0.400	0.250	0.325
0.50	0.019	0.007	-0.007	-0.014	0.300	0.175	0.238
0.00	0.008	0.002	-0.003	-0.005	0.150	0.050	0.100
-0.50	-0.006	-0.002	0.003	0.010	-0.100	-0.175	-0.138
-0.75	-0.012	-0.005	0.006	0.017	-0.175	-0.275	-0.225
-1.00	-0.021	-0.009	0.009	0.026	-0.300	-0.413	-0.356
-0.50	-0.014	-0.005	0.005	0.018	-0.225	-0.325	-0.275
0.00	-0.001	0.000	0.001	0.007	-0.025	-0.150	-0.088
0.50	0.010	0.004	-0.003	-0.004	0.150	0.025	0.088
0.75	0.016	0.006	-0.006	-0.010	0.250	0.100	0.175
1.00	0.022	0.009	-0.008	-0.017	0.338	0.225	0.281
0.50	0.017	0.006	-0.006	-0.011	0.275	0.175	0.200
0.00	0.002	0.000	-0.002	-0.002	0.050	0.000	0.025
-0.50	-0.011	-0.005	0.004	0.011	-0.150	-0.188	-0.169
-0.75	-0.018	-0.008	0.006	0.018	-0.250	-0.300	-0.275
-1.00	-0.026	-0.011	0.009	0.025	-0.375	-0.400	-0.388
-0.50	-0.018	-0.008	0.005	0.018	-0.250	-0.325	-0.288
0.00	-0.005	-0.002	0.001	0.006	-0.075	-0.125	-0.100

TEST 6: CYCLIC SERIES 3.

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	θ_{2} 12 ($\times 10^{-2}$ rads)	θ_{2} 34 ($\times 10^{-2}$ rads)	θ_{2} joint ($\times 10^{-2}$ rads)
0.50	0.006	0.002	-0.004	-0.004	0.100	0.000	0.050
1.00	0.018	0.007	-0.008	-0.017	0.288	0.225	0.256
1.25	0.026	0.009	-0.012	-0.026	0.425	0.350	0.388
1.50	0.040	0.014	-0.019	-0.042	0.650	0.588	0.619
1.00	0.034	0.011	-0.016	-0.037	0.575	0.513	0.544
0.50	0.024	0.007	-0.012	-0.027	0.425	0.375	0.400
0.00	0.009	0.002	-0.006	-0.015	0.175	0.225	0.200
-0.50	-0.007	-0.004	0.001	0.003	-0.075	-0.050	-0.063
-1.00	-0.022	-0.010	0.007	0.020	-0.300	-0.313	-0.306
-1.25	-0.035	-0.014	0.011	0.031	-0.525	-0.500	-0.513
-1.50	-0.055	-0.020	0.018	0.051	-0.875	-0.825	-0.850
-1.00	-0.044	-0.016	0.014	0.042	-0.700	-0.713	-0.706
-0.50	-0.030	-0.011	0.009	0.029	-0.488	-0.500	-0.494
0.00	-0.014	-0.005	0.002	0.010	-0.225	-0.200	-0.213
0.50	0.000	0.000	-0.003	-0.005	0.000	0.050	0.025
1.00	0.015	0.005	-0.009	-0.019	0.250	0.250	0.250
1.25	0.022	0.008	-0.012	-0.028	0.350	0.400	0.375
1.50	0.032	0.011	-0.017	-0.038	0.525	0.525	0.525
1.00	0.026	0.008	-0.014	-0.032	0.450	0.450	0.450
0.50	0.015	0.004	-0.010	-0.022	0.275	0.300	0.288
0.00	-0.001	-0.002	-0.003	-0.007	-0.025	0.100	0.038
-0.50	-0.016	-0.007	0.004	0.013	-0.225	-0.225	-0.225
-1.00	-0.034	-0.013	0.010	0.030	-0.525	-0.500	-0.513
-1.25	-0.044	-0.016	0.014	0.040	-0.688	-0.663	-0.675
-1.50	-0.054	-0.020	0.017	0.050	-0.850	-0.825	-0.838
-1.00	-0.044	-0.016	0.014	0.041	-0.700	-0.688	-0.694
-0.50	-0.030	-0.010	0.008	0.028	-0.500	-0.500	-0.500
0.00	-0.013	-0.005	0.002	0.009	-0.200	-0.175	-0.188

TEST 6: CYCLIC SERIES 4.

P_w (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$\theta_{2\ 12}$ ($\times 10^{-2}$ rads)	$\theta_{2\ 34}$ ($\times 10^{-2}$ rads)	$\theta_{2\ joint}$ ($\times 10^{-2}$ rads)
1.00	0.015	0.005	-0.010	-0.021	0.250	0.275	0.263
1.50	0.033	0.011	-0.017	-0.038	0.550	0.525	0.538
1.75	0.051	0.017	-0.024	-0.059	0.850	0.875	0.863
2.00	0.073	0.025	-0.034	-0.084	1.188	1.250	1.219
1.50	0.067	0.022	-0.031	-0.076	1.125	1.125	1.125
0.50	0.041	0.012	-0.021	-0.051	0.725	0.750	0.738
0.00	0.021	0.006	-0.013	-0.032	0.375	0.475	0.425
-1.00	-0.020	-0.009	0.003	0.012	-0.275	-0.225	-0.250
-1.50	-0.045	-0.017	0.013	0.040	-0.700	-0.675	-0.688
-1.75	-0.073	-0.025	0.023	0.070	-1.200	-1.175	-1.188
-2.00	-0.103	-0.035	0.038	0.111	-1.700	-1.825	-1.763
-1.50	-0.094	-0.031	0.035	0.104	-1.575	-1.725	-1.650
-0.50	-0.064	-0.021	0.024	0.077	-1.075	-1.325	-1.200
0.00	-0.041	-0.014	0.016	0.052	-0.675	-0.900	-0.788
1.00	0.005	0.001	-0.001	-0.002	0.100	0.025	0.063
1.50	0.035	0.011	-0.015	-0.037	0.600	0.550	0.575
1.75	0.050	0.016	-0.024	-0.060	0.850	0.900	0.875
2.00	0.063	0.021	-0.031	-0.077	1.050	1.150	1.100
1.50	0.058	0.018	-0.029	-0.071	1.000	1.050	1.025
0.50	0.031	0.008	-0.017	-0.042	0.575	0.625	0.600
0.00	0.010	0.001	-0.010	-0.032	0.225	0.550	0.388
-1.00	-0.040	-0.015	0.010	0.032	-0.625	-0.550	-0.588
-1.50	-0.074	-0.026	0.025	0.075	-1.200	-1.250	-1.225
-1.75	-0.092	-0.031	0.032	0.097	-1.525	-1.625	-1.575
-2.00	-0.106	-0.036	0.040	0.117	-1.750	-1.925	-1.838
-1.50	-0.097	-0.032	0.037	0.109	-1.625	-1.800	-1.713
-0.50	-0.066	-0.022	0.026	0.080	-1.100	-1.350	-1.225
0.00	-0.042	-0.015	0.015	0.063	-0.675	-1.200	-0.938

TEST 6: CYCLIC SERIES 5.

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$\theta_{2\ 12}$ ($\times 10^{-2}$ rads)	$\theta_{2\ 34}$ ($\times 10^{-2}$ rads)	$\theta_{2\ joint}$ ($\times 10^{-2}$ rads)
1.00	0.006	0.001	0.001	0.002	0.125	-0.025	0.050
2.00	0.072	0.024	-0.031	-0.077	1.200	1.115	1.175
2.25	0.091	0.030	-0.038	-0.094	1.525	1.400	1.463
2.50	0.118	0.042	-0.048	-0.120	1.900	1.800	1.850
1.50	0.104	0.035	-0.040	-0.103	1.725	1.575	1.650
0.50	0.076	0.025	-0.029	-0.076	1.275	1.175	1.225
0.00	0.054	0.017	-0.022	-0.055	0.925	0.825	0.875
-1.00	-0.011	-0.004	0.002	0.012	-0.175	-0.250	-0.213
-2.00	-0.102	-0.034	0.047	0.134	-1.700	-2.175	-1.938
-2.25	-0.132	-0.045	0.075	0.196	-2.175	-3.025	-2.600
-2.50	-0.175	-0.064	0.095	0.249	-2.775	-3.850	-3.313
-1.50	-0.159	-0.056	0.088	0.231	-2.575	-3.575	-3.075
-0.50	-0.131	-0.050	0.077	0.204	-2.025	-3.175	-2.600
0.00	-0.104	-0.035	0.067	0.177	-1.725	-2.750	-2.238
1.00	-0.029	-0.013	0.039	0.094	-0.400	-1.375	-0.888
2.00	0.117	0.041	-0.032	-0.085	1.900	1.325	1.613
2.25	0.138	0.050	-0.039	-0.102	2.200	1.575	1.888
2.50	0.164	0.061	-0.047	-0.121	2.575	1.850	2.213
1.50	0.147	0.053	-0.037	-0.101	2.350	1.600	1.975
0.50	0.116	0.040	-0.025	-0.068	1.900	1.075	1.488
0.00	0.090	0.032	-0.018	-0.048	1.450	0.750	1.100
-1.00	0.011	0.006	0.011	0.033	0.125	-0.055	-0.213
-2.00	-0.134	-0.044	0.086	0.218	-2.250	-3.300	-2.775
-2.25	-0.158	-0.055	0.094	0.242	-2.575	-3.700	-3.138
-2.50	-0.190	-0.070	0.117	0.279	-3.000	-4.050	-3.525
-1.50	-0.171	-0.060	0.099	0.260	-2.775	-4.025	-3.400
-0.50	-0.143	-0.049	0.088	0.232	-2.350	-3.600	-2.975
0.00	-0.117	-0.040	0.079	0.204	-1.925	-3.125	-2.525

TEST 6: CYCLIC SERIES 6.

P w (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta$_2$ 12 (x10^{-2}rads)	theta$_2$ 34 (x10^{-2}rads)	theta$_2$ joint (x10^{-2}rads)
1.00	-0.034	-0.015	0.049	0.111	-0.475	-1.550	-1.125
2.00	0.137	0.049	-0.029	-0.078	2.200	1.225	1.713
2.50	0.174	0.065	-0.044	-0.114	2.725	1.750	2.113
2.75	0.196	0.075	-0.052	-0.135	3.025	2.025	2.550
3.00	0.288	0.090	-0.066	-0.170	3.450	2.600	3.025
2.00	0.210	0.080	-0.057	-0.151	3.250	2.350	2.800
0.50	0.174	0.065	-0.039	-0.109	2.725	1.750	2.238
0.00	0.146	0.055	-0.032	-0.090	2.275	1.450	1.863
-1.00	0.060	0.026	-0.001	0.003	0.850	-0.100	0.375
-2.00	-0.145	-0.052	0.092	0.239	-2.325	-3.675	-3.000
-2.50	-0.199	-0.073	0.112	0.290	-3.150	-4.450	-3.800
-2.75	-0.241	-0.091	0.127	0.323	-3.750	-5.000	-4.375
-3.00	-0.287	-0.108	0.143	0.365	-4.475	-5.550	-5.013
-2.00	-0.265	-0.099	0.134	0.347	-4.150	-5.325	-4.738
-0.50	-0.237	-0.084	0.119	0.311	-3.825	-4.800	-4.313
0.00	-0.205	-0.076	0.110	0.286	-3.225	-4.400	-3.813
1.00	-0.118	-0.046	0.077	0.179	-1.800	-2.550	-2.175
2.00	0.131	0.047	-0.017	-0.053	2.100	0.900	1.500
2.50	0.205	0.078	-0.049	-0.134	3.175	2.125	2.650
2.75	0.225	0.088	-0.059	-0.156	3.425	2.425	2.925
3.00	0.249	0.098	-0.068	-0.181	3.775	2.825	3.300
2.00	0.230	0.089	-0.058	-0.160	3.525	2.550	3.038
0.50	0.191	0.072	-0.040	-0.118	2.975	1.950	2.463
0.00	0.161	0.062	-0.032	-0.097	2.475	1.625	2.050
-1.00	0.060	0.028	0.012	0.028	0.800	-0.400	0.200
-2.00	-0.182	-0.065	0.112	0.284	-2.925	-4.300	-3.613
-2.50	-0.253	-0.096	0.134	0.343	-3.925	-5.225	-4.575
-2.75	-0.278	-0.105	0.143	0.364	-4.325	-5.525	-4.925
-3.00	-0.309	-0.116	0.155	0.390	-4.825	-5.900	-5.363
-2.00	-0.286	-0.107	0.145	0.370	-4.475	-5.625	-5.050
-0.50	-0.243	-0.091	0.128	0.331	-3.800	-5.025	-4.413
0.00	-0.219	-0.083	0.119	0.305	-3.400	-4.650	-4.025
1.00	-0.103	-0.042	0.076	0.175	-1.525	-2.475	-2.000
2.00	0.162	0.060	-0.019	-0.063	2.550	1.100	1.825
2.50	0.212	0.082	-0.045	-0.129	3.250	2.100	2.675
2.75	0.226	0.088	-0.053	-0.147	3.450	2.350	2.900
3.00	0.240	0.095	-0.059	-0.163	3.625	2.600	3.113
2.00	0.223	0.086	-0.050	-0.146	3.425	2.400	2.913
0.50	0.185	0.070	-0.032	-0.102	2.875	1.750	2.313
0.00	0.155	0.059	-0.025	-0.080	2.400	1.375	1.888

TEST 7: CYCLIC SERIES 1. A=43.25" a=8" $D_{12}=D_{34}=D_{20}=D_{30}=4"$

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$\theta_{2\ 12}$ ($\times 10^{-2}$ rads)	$\theta_{2\ 34}$ ($\times 10^{-2}$ rads)	$\theta_{2\ joint}$ ($\times 10^{-2}$ rads)
0.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.25	0.004	0.001	-0.002	-0.003	0.075	0.025	0.050
0.50	0.007	0.003	-0.003	-0.006	0.100	0.075	0.088
0.00	0.001	0.000	0.000	-0.001	0.025	0.025	0.025
-0.25	-0.004	-0.002	0.002	0.003	-0.050	-0.025	-0.038
-0.50	-0.008	-0.004	0.003	0.007	-0.100	-0.100	-0.100
0.00	-0.001	-0.001	0.000	0.000	0.000	0.000	0.000
0.25	0.004	0.001	-0.002	-0.003	0.075	0.025	0.050
0.50	0.007	0.003	-0.003	-0.006	0.100	0.075	0.088
0.00	0.001	0.000	0.000	0.000	0.025	0.000	0.013
-0.25	-0.003	-0.002	0.002	0.003	-0.025	-0.025	-0.025
-0.50	-0.007	-0.003	0.003	0.007	-0.100	-0.100	-0.100
0.00	-0.001	-0.001	0.000	0.000	0.000	0.000	0.000

TEST 7: CYCLIC SERIES 2.

P w (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta$_{12}$ ($\times 10^{-2}$ rads)	theta$_{34}$ ($\times 10^{-2}$ rads)	theta$_{joint}$ ($\times 10^{-2}$ rads)
0.50	0.007	0.002	-0.003	-0.006	0.125	0.075	0.100
0.75	0.010	0.004	-0.005	-0.009	0.150	0.100	0.125
1.00	0.014	0.006	-0.006	-0.012	0.200	0.150	0.175
0.50	0.009	0.003	-0.004	-0.007	0.150	0.075	0.113
0.00	0.002	0.000	-0.001	-0.001	0.050	0.000	0.025
-0.50	-0.006	-0.004	0.003	0.006	-0.050	-0.075	-0.063
-0.75	-0.010	-0.005	0.005	0.010	-0.125	-0.125	-0.125
-1.00	-0.014	-0.007	0.007	0.013	-0.175	-0.150	-0.163
-0.50	-0.007	-0.004	0.004	0.008	-0.075	-0.100	-0.088
0.00	-0.001	-0.001	0.000	0.000	0.000	0.000	0.000
0.50	0.008	0.003	-0.003	-0.006	0.125	0.075	0.100
0.75	0.011	0.004	-0.005	-0.010	0.175	0.125	0.150
1.00	0.015	0.006	-0.007	-0.013	0.225	0.150	0.188
0.50	0.009	0.004	-0.004	-0.008	0.125	0.100	0.113
0.00	0.001	0.000	0.000	-0.001	0.025	0.025	0.025
-0.50	-0.007	-0.004	0.003	0.007	-0.075	-0.100	-0.088
-0.75	-0.010	-0.005	0.005	0.010	-0.125	-0.125	-0.125
-1.00	-0.014	-0.007	0.007	0.014	-0.175	-0.175	-0.175
-0.50	-0.007	-0.004	0.003	0.007	-0.075	-0.100	-0.088
0.00	-0.001	-0.001	0.000	0.000	0.000	0.000	0.000

TEST 7: CYCLIC SERIES 3.

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$theta_{2}$ 12 ($x10^{-2}$ rads)	$theta_{2}$ 34 ($x10^{-2}$ rads)	$theta_{2}$ joint ($x10^{-2}$ rads)
0.50	0.007	0.002	-0.003	-0.005	0.125	0.050	0.875
1.00	0.014	0.006	-0.006	-0.012	0.200	0.150	0.175
1.25	0.018	0.008	-0.008	-0.016	0.250	0.200	0.225
1.50	0.023	0.010	-0.010	-0.020	0.325	0.250	0.288
0.50	0.011	0.004	-0.004	-0.008	0.175	0.100	0.138
0.00	0.002	0.000	-0.001	-0.002	0.050	0.025	0.038
-0.50	-0.006	-0.004	0.003	0.006	-0.050	-0.075	-0.063
-1.00	-0.014	-0.007	0.007	0.013	-0.175	-0.150	-0.163
-1.25	-0.018	-0.009	0.008	0.017	-0.225	-0.225	-0.225
-1.50	-0.023	-0.011	0.010	0.021	-0.300	-0.275	-0.288
-0.50	-0.008	-0.005	0.004	0.008	-0.075	-0.100	-0.088
0.00	-0.001	-0.001	0.000	0.000	0.000	0.000	0.000
0.50	0.008	0.003	-0.003	-0.007	0.125	0.100	0.113
1.00	0.016	0.007	-0.007	-0.014	0.225	0.175	0.200
1.25	0.020	0.008	-0.009	-0.018	0.300	0.225	0.263
1.50	0.023	0.010	-0.011	-0.021	0.325	0.250	0.288
0.50	0.011	0.004	-0.004	-0.008	0.175	0.100	0.113
0.00	0.002	0.000	-0.001	-0.001	0.050	0.000	0.025
-0.50	-0.007	-0.004	0.003	0.007	-0.075	-0.100	-0.088
-1.00	-0.015	-0.008	0.007	0.014	-0.175	-0.175	-0.175
-1.25	-0.019	-0.010	0.009	0.018	-0.225	-0.225	-0.225
-1.50	-0.023	-0.012	0.010	0.022	-0.275	-0.300	-0.288
-0.50	-0.008	-0.005	0.004	0.008	-0.075	-0.100	-0.088
0.00	-0.001	-0.001	0.000	0.000	0.000	0.000	0.000

TEST 7: CYCLIC SERIES 4.

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$\theta_{2\ 12}$ ($\times 10^{-2}$ rads)	$\theta_{2\ 34}$ ($\times 10^{-2}$ rads)	$\theta_{2\ joint}$ ($\times 10^{-2}$ rads)
0.50	0.008	0.003	-0.003	-0.007	0.125	0.100	0.113
1.00	0.015	0.007	-0.007	-0.014	0.200	0.175	0.188
1.50	0.023	0.010	-0.011	-0.021	0.325	0.250	0.288
1.75	0.027	0.012	-0.013	-0.026	0.375	0.325	0.350
2.00	0.032	0.014	-0.015	-0.031	0.450	0.400	0.425
1.50	0.027	0.012	-0.012	-0.025	0.395	0.325	0.355
0.50	0.013	0.005	-0.005	-0.011	0.200	0.150	0.175
0.00	0.004	0.001	-0.002	-0.004	0.075	0.050	0.063
-0.50	-0.005	-0.003	0.002	0.004	-0.100	-0.050	-0.075
-1.00	-0.013	-0.007	0.006	0.012	-0.125	-0.150	-0.138
-1.50	-0.022	-0.011	0.010	0.020	-0.275	-0.250	-0.238
-1.75	-0.027	-0.013	0.012	0.024	-0.350	-0.300	-0.325
-2.00	-0.032	-0.016	0.014	0.030	-0.400	-0.400	-0.400
-1.50	-0.024	-0.012	0.011	0.022	-0.300	-0.275	-0.288
-0.50	-0.009	-0.005	0.004	0.009	-0.100	-0.125	-0.113
0.00	-0.002	-0.002	0.000	0.000	0.000	0.000	0.000
0.50	0.007	0.003	-0.003	-0.007	0.100	0.100	0.100
1.00	0.015	0.006	-0.007	-0.015	0.225	0.200	0.213
1.50	0.023	0.010	-0.011	-0.022	0.325	0.275	0.300
1.75	0.027	0.012	-0.013	-0.026	0.375	0.325	0.350
2.00	0.031	0.014	-0.015	-0.031	0.425	0.400	0.413
1.50	0.025	0.011	-0.012	-0.024	0.350	0.300	0.325
0.50	0.012	0.004	-0.005	-0.010	0.200	0.125	0.163
0.00	0.002	0.000	-0.001	-0.003	0.050	0.050	0.050
-0.50	-0.007	-0.005	0.003	0.006	-0.050	-0.075	-0.063
-1.00	-0.015	-0.008	0.007	0.013	-0.175	-0.150	-0.163
-1.50	-0.023	-0.012	0.010	0.021	-0.275	-0.275	-0.275
-1.75	-0.028	-0.014	0.012	0.025	-0.350	-0.325	-0.338
-2.00	-0.032	-0.016	0.014	0.029	-0.400	-0.375	-0.388
-1.50	-0.025	-0.012	0.011	0.022	-0.325	-0.275	-0.300
-0.50	-0.010	-0.005	0.004	0.008	-0.125	-0.100	-0.113
0.00	-0.002	-0.002	0.000	0.000	0.000	0.000	0.000

TEST 7: CYCLIC SERIES 5.

P_w (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$theta_{2}$ 12 ($x10^{-2}$ rads)	$theta_{2}$ 34 ($x10^{-2}$ rads)	$theta_{2}$ joint ($x10^{-2}$ rads)
0.50	0.006	0.002	-0.003	-0.006	0.100	0.075	0.088
1.00	0.014	0.006	-0.007	-0.012	0.200	0.125	0.163
1.50	0.022	0.009	-0.011	-0.021	0.325	0.250	0.288
2.00	0.029	0.013	-0.015	-0.029	0.400	0.350	0.375
2.25	0.034	0.015	-0.017	-0.034	0.475	0.425	0.450
2.50	0.040	0.018	-0.021	-0.040	0.550	0.475	0.573
1.50	0.028	0.013	-0.014	-0.028	0.375	0.350	0.363
0.50	0.013	0.005	-0.007	-0.013	0.200	0.150	0.175
0.00	0.005	0.001	-0.003	-0.007	0.100	0.100	0.100
-0.50	-0.004	-0.003	0.001	0.002	-0.025	-0.025	-0.025
-1.00	-0.012	-0.007	0.005	0.010	-0.125	-0.125	-0.125
-1.50	-0.021	-0.010	0.009	0.017	-0.275	-0.200	-0.238
-2.00	-0.031	-0.015	0.013	0.027	-0.400	-0.350	-0.375
-2.25	-0.037	-0.018	0.016	0.032	-0.475	-0.400	-0.438
-2.50	-0.046	-0.022	0.019	0.039	-0.600	-0.500	-0.550
-1.50	-0.029	-0.014	0.012	0.024	-0.375	-0.300	-0.338
-0.50	-0.014	-0.007	0.005	0.010	-0.175	-0.125	-0.150
0.00	-0.005	-0.003	0.001	0.001	-0.050	0.000	-0.025
0.50	0.003	0.001	-0.003	-0.006	0.050	0.075	0.063
1.00	0.012	0.005	-0.007	-0.014	0.175	0.175	0.175
1.50	0.021	0.009	-0.011	-0.022	0.300	0.275	0.288
2.00	0.031	0.014	-0.016	-0.031	0.425	0.375	0.400
2.25	0.036	0.016	-0.018	-0.035	0.500	0.425	0.463
2.50	0.041	0.019	-0.021	-0.040	0.550	0.475	0.513
1.50	0.027	0.012	-0.014	-0.026	0.375	0.300	0.338
0.50	0.012	0.005	-0.006	-0.012	0.175	0.150	0.163
0.00	0.003	0.001	-0.002	-0.005	0.050	0.075	0.064
-0.50	-0.007	-0.004	0.003	0.004	-0.075	-0.025	-0.050
-1.00	-0.015	-0.008	0.007	0.012	-0.175	-0.125	-0.150
-1.50	-0.025	-0.012	0.011	0.021	-0.325	-0.250	-0.288
-2.00	-0.035	-0.017	0.015	0.030	-0.450	-0.375	-0.413
-2.25	-0.040	-0.019	0.017	0.034	-0.525	-0.425	-0.475
-2.50	-0.046	-0.022	0.019	0.040	-0.600	-0.550	-0.575
-1.50	-0.029	-0.014	0.012	0.025	-0.375	-0.325	-0.350
-0.50	-0.014	-0.007	0.005	0.010	-0.175	-0.125	-0.150
0.00	-0.005	-0.003	0.001	0.001	-0.050	0.000	-0.025

TEST 7: CYCLIC SERIES 6.

P_w (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$theta_{2\ 12}$ ($x10^{-2}$ rads)	$theta_{2\ 34}$ ($x10^{-2}$ rads)	$theta_{2\ joint}$ ($x10^{-2}$ rads)
0.50	0.004	0.001	-0.003	-0.006	0.075	0.075	0.075
1.00	0.013	0.006	-0.007	-0.014	0.175	0.175	0.175
1.50	0.022	0.010	-0.011	-0.022	0.300	0.275	0.288
2.00	0.032	0.014	-0.016	-0.031	0.450	0.375	0.388
2.50	0.041	0.019	-0.021	-0.040	0.550	0.525	0.538
2.75	0.048	0.022	-0.024	-0.047	0.650	0.575	0.613
3.00	0.066	0.029	-0.031	-0.068	0.925	0.925	0.925
1.50	0.045	0.018	-0.020	-0.047	0.675	0.675	0.675
0.50	0.028	0.010	-0.013	-0.032	0.450	0.475	0.463
0.00	0.017	0.005	-0.009	-0.025	0.300	0.400	0.350
-0.50	0.007	0.000	-0.004	-0.015	0.175	0.225	0.200
-1.00	-0.003	-0.004	0.001	-0.005	0.025	0.100	0.063
-1.50	-0.014	-0.009	0.005	0.004	-0.125	0.025	-0.050
-2.00	-0.028	-0.015	0.011	0.016	-0.325	-0.125	-0.225
-2.50	-0.045	-0.021	0.017	0.031	-0.600	-0.350	-0.475
-2.75	-0.060	-0.027	0.022	0.043	-0.825	-0.525	-0.675
-3.00	-0.075	-0.032	0.028	0.058	-1.075	-0.750	-0.913
-1.50	-0.050	-0.020	0.017	0.035	-0.750	-0.450	-0.600
-0.50	-0.032	-0.012	0.009	0.018	-0.500	-0.225	-0.363
0.00	-0.023	-0.007	0.003	0.007	-0.400	-0.100	-0.250
0.50	-0.013	-0.003	-0.001	-0.001	-0.250	0.000	-0.125
1.00	-0.002	0.002	-0.006	-0.011	-0.100	0.125	0.013
1.50	0.011	0.007	-0.011	-0.021	0.100	0.250	0.175
2.00	0.027	0.014	-0.018	-0.035	0.325	0.425	0.375
2.50	0.047	0.021	-0.025	-0.052	0.650	0.675	0.663
2.75	0.057	0.025	-0.028	-0.062	0.800	0.850	0.825
3.00	0.068	0.030	-0.033	-0.074	0.950	1.025	0.988
1.50	0.046	0.019	-0.022	-0.051	0.675	0.725	0.700
0.50	0.027	0.010	-0.014	-0.036	0.425	0.550	0.488
0.00	0.016	0.004	-0.010	-0.023	0.300	0.325	0.313

Pg (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$\theta_{2\ 12}$ ($\times 10^{-2}$ rads)	$\theta_{2\ 34}$ ($\times 10^{-2}$ rads)	$\theta_{2\ joint}$ ($\times 10^{-2}$ rads)
-0.50	0.005	0.000	-0.004	-0.016	0.125	0.300	0.213
-1.00	-0.006	-0.005	0.001	-0.005	-0.025	0.150	0.063
-1.50	-0.021	-0.012	0.007	0.007	-0.225	0.000	-0.113
-2.00	-0.039	-0.019	0.014	0.022	-0.500	-0.200	-0.350
-2.50	-0.060	-0.026	0.021	0.042	-0.850	-0.525	-0.688
-2.75	-0.069	-0.030	0.025	0.050	-0.975	-0.625	-0.800
-3.00	-0.080	-0.034	0.029	0.061	-1.150	-0.800	-0.975
-1.50	-0.053	-0.021	0.018	0.037	-0.800	-0.475	-0.638
-0.50	-0.035	-0.013	0.010	0.020	-0.550	-0.250	-0.400
0.00	-0.025	-0.008	0.004	0.008	-0.425	-0.100	-0.263
0.50	-0.014	-0.003	-0.001	-0.001	-0.275	0.000	-0.138
1.00	-0.002	0.002	-0.006	-0.011	-0.100	0.125	0.013
1.50	0.012	0.008	-0.012	-0.024	0.100	0.300	0.200
2.00	0.032	0.015	-0.020	-0.039	0.425	0.475	0.450
2.50	0.053	0.023	-0.027	-0.058	0.750	0.775	0.763
2.75	0.064	0.028	-0.030	-0.069	0.900	0.975	0.938
3.00	0.074	0.032	-0.034	-0.078	1.050	1.100	1.075
1.50	0.052	0.021	-0.023	-0.055	0.775	0.800	0.788
0.50	0.032	0.012	-0.015	-0.038	0.500	0.575	0.538
0.00	0.021	0.007	-0.010	-0.029	0.350	0.475	0.413

TEST 7: CYCLIC SERIES 7.

Pw (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	$theta_{12}$ ($x10^{-2}$ rads)	$theta_{34}$ ($x10^{-2}$ rads)	$theta_{joint}$ ($x10^{-2}$ rads)
0.50	0.028	0.010	-0.014	-0.035	0.450	0.525	0.488
1.00	0.037	0.014	-0.017	-0.043	0.575	0.650	0.613
1.50	0.046	0.018	-0.021	-0.051	0.700	0.750	0.725
2.00	0.055	0.023	-0.025	-0.060	0.800	0.875	0.838
2.50	0.064	0.027	-0.030	-0.069	0.925	0.975	0.950
3.00	0.073	0.031	-0.034	-0.078	1.050	1.100	1.075
3.25	0.078	0.034	-0.037	-0.084	1.100	1.175	1.138
3.50	0.092	0.040	-0.043	-0.098	1.300	1.375	1.338
2.50	0.078	0.033	-0.036	-0.084	1.125	1.200	1.163
1.50	0.062	0.025	-0.028	-0.068	0.925	1.000	0.963
0.50	0.043	0.016	-0.020	-0.050	0.675	0.700	0.688
0.00	0.032	0.011	-0.015	-0.041	0.525	0.650	0.588
-0.50	0.020	0.005	-0.009	-0.029	0.375	0.500	0.438
-1.00	0.009	0.000	-0.004	-0.017	0.225	0.325	0.275
-1.50	-0.010	-0.007	0.003	-0.003	-0.075	0.150	0.038
-2.00	-0.032	-0.016	0.011	0.025	-0.400	-0.350	-0.375
-2.50	-0.057	-0.025	0.021	0.046	-0.800	-0.625	-0.713
-3.00	-0.081	-0.034	0.030	0.063	-1.125	-0.825	-0.975
-3.25	-0.093	-0.040	0.036	0.077	-1.325	-1.025	-1.175
-3.50	-0.105	-0.045	0.042	0.092	-1.500	-1.250	-1.375
-2.50	-0.089	-0.037	0.034	0.076	-1.300	-1.050	-1.175
-1.50	-0.072	-0.028	0.027	0.061	-1.100	-0.850	-0.975
-0.50	-0.053	-0.019	0.018	0.043	-0.850	-0.625	-0.738
0.00	-0.042	-0.014	0.012	0.030	-0.700	-0.450	-0.575

Pg (kips)	delta 1 (inches)	delta 2 (inches)	delta 3 (inches)	delta 4 (inches)	theta$_2$ 12 ($\times 10^{-2}$ rads)	theta$_2$ 34 ($\times 10^{-2}$ rads)	theta$_2$ joint ($\times 10^{-2}$ rads)
0.50	-0.031	-0.008	0.007	0.020	-0.575	-0.325	-0.450
1.00	-0.017	-0.002	0.001	0.008	-0.375	-0.175	-0.275
1.50	0.003	0.005	-0.006	-0.007	-0.050	0.025	-0.038
2.00	0.028	0.014	-0.016	-0.029	0.350	0.325	0.375
2.50	0.056	0.028	-0.027	-0.059	0.700	0.800	0.750
3.00	0.081	0.036	-0.037	-0.084	1.125	1.175	1.150
3.25	0.092	0.040	-0.042	-0.095	1.300	1.325	1.313
3.50	0.103	0.045	-0.046	-0.105	1.450	1.475	1.463
2.50	0.089	0.038	-0.039	-0.091	1.275	1.300	1.288
1.50	0.072	0.030	-0.031	-0.075	1.050	1.100	1.075
0.50	0.051	0.020	-0.022	-0.056	0.775	0.850	0.813
0.00	0.039	0.014	-0.017	-0.046	0.625	0.725	0.675
-0.50	0.020	0.008	-0.010	-0.033	0.300	0.575	0.438
-1.00	0.012	0.002	-0.004	-0.019	0.250	0.375	0.313
-1.50	-0.011	-0.007	0.003	-0.002	-0.100	0.125	0.013
-2.00	-0.041	-0.018	0.014	0.023	-0.575	-0.225	-0.400
-2.50	-0.070	-0.030	0.026	0.054	-1.000	-0.700	-0.850
-3.00	-0.095	-0.040	0.037	0.079	-1.375	-1.050	-1.213
-3.25	-0.104	-0.045	0.041	0.090	-1.475	-1.225	-1.350
-3.50	-0.115	-0.050	0.045	0.101	-1.625	-1.400	-1.513
-2.50	-0.097	-0.040	0.037	0.084	-1.425	-1.175	-1.300
-1.50	-0.078	-0.031	0.029	0.063	-1.175	-0.850	-1.013
-0.50	-0.059	-0.021	0.020	0.049	-0.950	-0.725	-0.838
0.00	-0.048	-0.016	0.014	0.036	-0.800	-0.550	-0.675

APPENDIX 7.2

Semi-Rigid Analysis Program Listing

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c . . . . . .
c .                                                                .
c .   Program Name   -      P R _ P O R T A L                     .
c .                                                                .
c .   Program language-    VAX FORTRAN V4.5-219                   .
c .                                                                .
c .   Programmers     -                                           .
c .                         Deryl L. Earsom & C. Thomas Jan        .
c .                         Department of Civil Engineering        .
c .                         Washington University, St. Louis.      .
c .   Director       -                                           .
c .                         Professor Thomas G. Harmon             .
c .                         Department of Civil Engineering        .
c .                         Washington University, St. Louis.      .
c .   Purposes       -                                           .
c .                         Analysis of a simply pinned            .
c .                         column portal frame with               .
c .                         partially rigid beam-column            .
c .                         connections under the effect of        .
c .                         varying gravity and cyclically         .
c .                         varying lateral loads.                 .
c .                         Incorporates a connection              .
c .                         moment-rotation hysteresis model       .
c .                         developed in                           .
c .                         SEMI-RIGID CONNECTIONS UNDER           .
c .                         VARYING GRAVITY AND CYCLICALLY         .
c .                         VARYING LATERAL LOADS,                 .
c .                         D.L. Earsom,                           .
c .                         Master of Science Thesis, 1986         .
c .                         Department of Civil Engineering,       .
c .                         Washington University, St. Louis.      .
c .                         Missouri 63130.                        .
c .   Restrictions   -                                           .
c .                         The moment rotation curve of the       .
c .                         connection must be given as a          .
c .                         function of rotation. The              .
c .                         gravity load must be applied           .
c .                         first and then held constant           .
c .                         with the cyclic lateral load           .
c .                         application.                           .
c .   Date           -                                           .
c .                          9/21/86        Setup                 .
c .                         10/13/86        Compiled              .
c .   Subroutines    -                                           .
c .                         INITIAL_VALUES                         .
c .                         W_LOADER                               .
c .                         P_LOADER                               .
c .                         ASSEMBLE_INC_LOADS                     .
c .                         ASSEMBLE_STIFFNESS                     .
c .                         CALC_INC_DISPLACEMENTS                 .
c .                         FULL_SOLVER                            .
c .                         CALC_INC_MEMBER_END_ACTIONS            .
c .                         TOTAL_MEMBER_END_ACTIONS               .
c .                         TOTAL_DISPLACEMENTS                    .
c .                         CONNECTION                             .
c .                         MOMENT_MAGNIFICATION                   .
c .                         UNBALANCED_MOMENTS                     .
c .                         CHECK_UNBALANCED_MOMENTS               .
c .                         REITERATE                              .
c .                         NONCONVERGENCE                         .
```

```
c .                                  OUTPUT                              .
c .     Functions       -                                                .
c .                                  FUNCTION_M                          .
c .                                  FUNCTION_K                          .
c .     I/O Devices     -                                                .
c .                                                                      .
c . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . . .
      Program PR_PORTAL
c .
c .     declaration of all variables used
c .
c .     WDONE=0,1: Flag for completion of distributed load application.
c .     PDONE=0,1: Flag for completion of cyclic lateral load applicat.
c .     I=0,11: Iteration index for each load increment.
c .     C=3,4: Connection index for K23 & K45 respectively.
c .     BAL=0,1: Flag denoting if unbalanced forces are within acceptable
c .              limits.
c .     REDO=0,1: Flag denoting whether reiteration is allowed or whether
c .               iteration limit has been exceeded.
c .     M=0,1: Flag denoting a load increment cycle.
c .     NEWLOAD=STATEMENT NO. 100: Beginning of an incremental load appl.
c .     LOOP=STATEMENT NO. 200: Beginning of the iteration loop.
c .     EXIT=STATEMENT NO. 300
c .     PRINT=0,1: Flag denoting whether or not output is to be printed.
c .
      INTEGER WDONE, PDONE, I, C, BAL, REDO, M,
     #  NEWLOAD, LOOP, EXIT, PRINT
      ASSIGN 100 TO NEWLOAD
      ASSIGN 200 TO LOOP
      ASSIGN 300 TO EXIT
c .
      TYPE *, ' [PR_PORTAL]'
      Call INITIAL_VALUES (WDONE,PDONE,C,BAL,REDO)
      TYPE *, '*****************  OUTPUT FORMAT ***************** '
      TYPE *, ' LOAD CYCLE, ', ' DISTRIBUTED LOAD, ', ' LATERAL LOAD'
      TYPE *, ' MEMBER END ACTIONS AM(1-5)'
      TYPE *, ' RELATIVE DISPLACEMENT DC(3), DC(4)'
      TYPE *, ' DISPLACEMENTS D(1-5)'
      TYPE *, ' AXIAL LOADS V1, ', ' V2, ', ' CONN. STIFFNESSES K23, K45'
100   IF (WDONE .EQ. 0) THEN
            CALL W_LOADER (WDONE,PRINT)
      END IF
      IF ((WDONE .EQ. 1) .AND. (PDONE .EQ. 0)) THEN
            CALL P_LOADER (PDONE,PRINT)
      END IF
      IF ((WDONE .EQ. 1) .AND. (PDONE .EQ. 1)) THEN
            GOTO EXIT
      END IF
c .   Set iteration and load increment indicies to 0.
      I=0
      M=0
c .   Change external incremental loads to Combined Joint Load Vector,
c .   including unbalanced forces if any.
      CALL ASSEMBLE_INC_LOADS
200   CONTINUE
c .   Revise direct stiffness matrix for calculated change in connection
c .   stiffness.
      CALL ASSEMBLE_STIFFNESS
c .   Do direct stiffness analysis for incremental displacement.
      CALL CALC_INC_DISPLACEMENTS (WDONE,PRINT)
```

```
c .     Calculate incremental member end actions from member stiffnesses
c .     and incremental displacements.
        CALL CALC_INC_MEMBER_END_ACTIONS (WDONE)
c .     Sum total displacements.
        CALL TOTAL_DISPLACEMENTS
c .     Sum total member end actions.
        CALL TOTAL_MEMBER_END_ACTIONS (WDONE)
c .     If a new load increment is being applied, then find the moment
c .     magnification due to lateral dispacement and column axial loads
c .     to determine direction of forces and to obtain an adequate mag-
c .     nitude before assessing connection unbalanced forces and new
c .     stiffnesses.
C.      IF (M .EQ. 0) THEN
C.              CALL MOMENT_MAGNIFICATION (M)
C.              M=1
C.              GOTO LOOP
C.      END IF
C.      IF (PRINT .EQ. 1) THEN
C.      TYPE *, I
C.      END IF
c .     Set connection index for connection K23.
        C=3
C.      IF (PRINT .EQ. 1) THEN
C.      TYPE *, C
C.      END IF
c .     Find unbalanced forces in connection due to previously used
c .     tangential stiffness and revise tangential stiffness.
        CALL CONNECTION (C,PRINT,I)
c .     Set connection index for connection K43.
        C=4
C.      IF (PRINT .EQ. 1) THEN
C.      TYPE *, C
C.      END IF
c .     Repeat for connection K43.
        CALL CONNECTION (C,PRINT,I)
c .     Calculate and update joint 2 & 3 moment magnification resulting from
c .     column axial load and new displacement as an approximation to the
c .     instability effects.
        CALL MOMENT_MAGNIFICATION (M)
c .     Sum unbalanced forces from connections and moment magnification.
        CALL UNBALANCED_MOMENTS (PRINT)
c .     Check to see if unbalanced forces are within acceptable limits.
        CALL CHECK_UNBALANCED_MOMENTS (BAL)
c .     If unbalanced forces are acceptably small, then output results
c .     and call for a new load increment to be applied. If not, change
c .     unbalanced forces to external joint loads and reiterate unless
c .     have exceeded the number of iterations allowed.
        IF (BAL .EQ. 1) THEN
                CALL OUTPUT (PRINT)
                GOTO NEWLOAD
        ELSE
                CALL REITERATE (I,REDO)
                IF (REDO .EQ. 1) THEN
                        GOTO LOOP
                ELSE
                        CALL NONCONVERGENCE
                END IF
        END IF
c .
300     STOP  ' [GOOD BYE!]'
```

```
      END
c .
c ..........................................................................
c .
      Subroutine INITIAL_VALUES (WDONE,PDONE,C,BAL,REDO)
      Real*4 L, H, E, IB, IC,  S11, S12, S13, S14, S15, S21, S22,
     # S23, S24, S25, S31, S32, S33, S34, S35, S41, S42, S43, S44,
     # S45, S51, S52, S53, S54, S55, WSTEP, PSTEP, W, P, DRP, DP,
     # NEWV3, NEWV4, V3, V4, WMAX, PMAX, INCW, INCP, P_ADD, P_CYCLE,
     # INC_AJ(5), INC_AML(5), INC_AC(5), AM(5), INC_D(5), D(5),
     # AJ(5), UNBL_AJ(5), INC_AMA(2), INC_AMB(2), INC_AMC(2),
     # KE, ME, DE, MY, K23, K45, LASTCM(3:4), LASTDC(3:4),
     # POSMAXCM(3:4), POSMAXDC(3:4), NEGMAXCM(3:4), NEGMAXDC(3:4),
     # TPOSMAXCM(3:4), TPOSMAXDC(3:4), TNEGMAXCM(3:4), TNEGMAXDC(3:4),
     # DC(3:4), DR(3:4), POSDR(3:4), NEGDR(3:4), CM(3:4), K(3:4),
     # UNBLCM(3:4), UNBLV3D1, UNBLV4D1
      Integer PDONE, WDONE, N, C, Z, REDO, BAL, FR(3:4)
      Common /GEOMETRY/ L, H, E, IB, IC
      Common /W_LOAD/ W, WMAX, WSTEP, INCW
      Common /P_LOAD/ P, P_CYCLE, PSTEP, INCP, P_ADD, PMAX, N
      Common /RIGID/ S11, S12, S13, S14, S15, S21, S22, S23, S24,
     # S25, S31, S32, S33, S34, S35, S41, S42, S43, S44, S45, S51,
     # S52, S53, S54, S55
      Common /SPRING/ K23, K45
      Common /NEW_CONN_STIFF/ K
      Common /CONNECT/ DRP, DP, KE, ME, DE, MY,
     # DC, CM, LASTDC, LASTCM, DR, POSDR, NEGDR,
     # POSMAXCM, POSMAXDC, NEGMAXCM, NEGMAXDC,
     # TPOSMAXCM, TPOSMAXDC, TNEGMAXCM, TNEGMAXDC, FR
      Common /AXIAL/ V3, V4, NEWV3, NEWV4, UNBLV3D1, UNBLV4D1
      Common /LOADS/ INC_AJ, INC_AML, INC_AC, INC_AMA,
     # INC_AMB, INC_AMC, AJ, AM, UNBL_AJ, UNBLCM
      Common /DISPLACE/ D, INC_D
      L=24.0E00*12.0E00          ! FRAME DESCRIPTION:     L= BEAM SPAN.
      H=12.0E00*12.0E00          !                        H= HEIGHT.
      E=29000.0E00               !                        E= YOUNG'S MOD.
c .   moment of inertia of beam IB and columns IC
      IB=216.0E00                ! WEAK BEAM W14x30, I=291in**4 Mp=170Kip*in
      IC=144.0E00                ! HVY BEAM W14x233, I=3010in**4 Mp=15,720kip*in
c .   Accept *, IB, IC           ! COLUMN W10x26, I=144in**4 Mp=1397kip*in
      S11=3.0E00*E*IC/H/H/H            ! MEMBER STIFFNESS
      S12=3.0E00*E*IC/H/H              ! COEFFICIENTS
      S13=0.0E00
      S14=0.0E00
      S15=S12
      S21=S12
      S22=3.0E00*E*IC/H
      S23=0.0E00
      S24=0.0E00
      S25=0.0E00
      S31=0.0E00
      S32=0.0E00
      S33=4.0E00*E*IB/L
      S34=2.0E00*E*IB/L
      S35=0.0E00
      S41=0.0E00
      S42=0.0E00
      S43=S34
      S44=S33
      S45=0.0E00
```

```
      S51=S12
      S52=0.0E00
      S53=0.0E00
      S54=0.0E00
      S55=S22
      WSTEP=0.0E00     ! LOADING PARAMETERS, SEE SUBROUTINES W_LOADER
      PSTEP=0.0E00     ! & P_LOADER FOR VARIABLE DEFINITIONS.
      W=0.0E00
      P=0.0E00
      PDONE=0
      WDONE=0
      WMAX=1.0E00/6.0E00
      PMAX=5.2E00
C .   Accept *, WMAX, PMAX
      INCW=WMAX/10.00
      INCP=PMAX/40.0E00
      N=1
      P_ADD=PMAX/5.0E00
C .   Accept *, P_ADD
      P_CYCLE=P_ADD
      C=0
      DO Z=1,5
            INC_AJ(Z)=0.0E00
            INC_AML(Z)=0.0E00
            INC_AC(Z)=0.0E00
            AM(Z)=0.0E00
            INC_D(Z)=0.0E00
            D(Z)=0.0E00
            AJ(Z)=0.0E00
            UNBL_AJ(Z)=0.0E00
      END DO
      DO Z=1,2
            INC_AMA(Z)=0.0E00
            INC_AMB(Z)=0.0E00
            INC_AMC(Z)=0.0E00
      END DO
      KE=3.67164E05
      ME=242.328E00
      DE=0.0007E00
      MY=720.0E00
C .   Accept *, KE, ME, DE, MY
      K23=KE
      K45=KE
      DO Z=3,4                          ! DECLARED ARRAYS
            LASTCM(Z)=0.0E00            ! WITH INDICES 3 AND 4
            LASTDC(Z)=0.0E00
            POSMAXCM(Z)=0.0E00
            POSMAXDC(Z)=0.0E00
            NEGMAXCM(Z)=0.0E00
            NEGMAXDC(Z)=0.0E00
            TPOSMAXCM(Z)=0.0E00
            TPOSMAXDC(Z)=0.0E00
            TNEGMAXCM(Z)=0.0E00
            TNEGMAXDC(Z)=0.0E00
            DC(Z)=0.0E00
            FR(Z)=0                     ! IT'S AN INTEGER
            DR(Z)=0.0E00
            POSDR(Z)=0.0E00
            NEGDR(Z)=0.0E00
            CM(Z)=0.0E00
```

```
                K(Z)=0.0E00
                UNBLCM(Z)=0.0E00
        END DO
        DRP=0.0E00
        DP=0.0E00
        NEWV3=0.0E00
        NEWV4=0.0E00
        V3=0.0E00
        V4=0.0E00
        UNBLV3D1=0.0E00
        UNBLV4D1=0.0E00
        REDO=0
        BAL=0
        Return
        End
C .
C ..........................................................................
C .
        Subroutine W_LOADER (WDONE,PRINT)
C .
C .     VARIALBLES:
C .     WDONE=0.1       : A flag for completion of distributed load.
C .     WMAX=given      : The maximum distributed load to be applied
C .                       to the beam.
C .     INCW=given      : The incremental distributed load to be applied
C .                       at each load step.
C .     WSTEP=INCW or 0 : The incremental distributed load to be applied
C .                       and balanced at this particular load step.
C .     W .le. WMAX     : The current value of the distributed load, to
C .                       be increased by WSTEP.
C .
C .                     !!!!!!!!!!!!!!!!!!!!!!!!!!!!
C .
        Real*4 W, WMAX, WSTEP, INCW
        Integer WDONE, PRINT
        Common /W_LOAD/ W, WMAX, WSTEP, INCW
C .
        If (W .LT. WMAX) then
                WSTEP=INCW
                PRINT=1
        else
                WSTEP=0
                WDONE=1
C .             PRINT=1
        End if
        W=W+WSTEP
        Return
        End
C .
C ..........................................................................
C .
        Subroutine P_LOADER (PDONE,PRINT)
C .
C .     VARIABLES:
C .     PDONE=0.1       : A flag for the completion of the series of
C .                       cyclical lateral load application.
C .     PMAX=given      : The maximum lateral load to be cycled.
C .     INCP=given      : The incremental change in P, the lateral load,
C .                       at each load step.
C .     PSTEP=+-INCP,0  : The load step to be applied and balanced.
```

```
c .     N=1-7           : An index which counts the number of lateral
c .                       load cycles.
c .     P_CYCLE .LE. PMAX       : The current magnitude of each cycle.
c .     P_ADD=given     : The difference between succesive load cycles.
c .     P .le. PMAX     : The current value of the lateral load.
c .     PRINT           : Output flag.
c .
c .                     !!!!!!!!!!!!!!!!!!!
c .
        Real*4 P, P_CYCLE, PSTEP, INCP, P_ADD, PMAX
        Integer N, PDONE, PRINT
        Common /P_LOAD/ P, P_CYCLE, PSTEP, INCP, P_ADD, PMAX, N
        If ((MOD(N,2) .NE. 0) .AND. (N .LT. 7)) then
                PRINT=0
                If (ABS(P_CYCLE-P) .GT. 0.01E00) then
C .                     If (P .LT. P_CYCLE) then
                                PSTEP=INCP
                                P=P+PSTEP
                        else
                                N=N+1
                        End if
                        If ((1.00E00-ABS(P)/P_CYCLE) .LT. 0.01E00) then
                                PRINT=1
                        End if
        End if
        If ((MOD(N,2) .EQ. 0) .AND. (N .LE. 6)) then
                PRINT=0
                If (ABS(P+P_CYCLE) .GT. 0.01E00) then
c .                     If (P .GT. -P_CYCLE) then
                                PSTEP=-INCP
                                P=P+PSTEP
                        else
                                PSTEP=INCP
                                P=P+PSTEP
                                IF (N .EQ. 6) THEN
                                        P=-P_CYCLE
                                END IF
                                N=N+1
                        End if
                        If ((1.0E00-ABS(P)/P_CYCLE) .LT. 0.01E00) then
                                PRINT=1
                        End if
        End if
        If (N .EQ. 7) then
                PRINT=0
                If (ABS(P+PINC) .LT. 0.01E00) then
                        PRINT=1
                        PSTEP=PINC
                        P=P+PSTEP
                        If ((1.0E00-ABS(P_CYCLE)/PMAX) .LE. 0.01E00) then
                                PDONE=1
                        Else
                                P_CYCLE=P_CYCLE+P_ADD
                                N=1
                        End if
                Else
                        PRINT=0
                        PSTEP=INCP
                        P=P+PSTEP
                End if
```

```
        End if
        IF ((N .EQ. 1) .OR. (N .EQ. 2)) THEN
        PRINT=1
        END IF
        IF (ABS(P) .LT. 0.1E00) THEN
        PRINT=1
        END IF
        IF ((1.0E00-(P_CYCLE/PMAX)) .LT. 0.01E00) THEN
                PRINT=1
        END IF
        Return
        End
C .
C ......................................................................
C .
        Subroutine ASSEMBLE_INC_LOADS
        Real*4 INC_AJ(5), PSTEP, INC_AML(5), WSTEP, L, INC_AC(5),
C .     unused common reals
     #  INC_AMA(2), INC_AMB(2), INC_AMC(2), AJ(5), AM(5), UNBL_AJ(5),
     #  UNBLCM(3:4), W, WMAX, INCW, P, P_CYCLE, INCP, P_ADD, PMAX,
     #  H, E, IB, IC
        Integer Z,
C .     unused common integers
     #  N
        Common /LOADS/ INC_AJ, INC_AML, INC_AC, INC_AMA,
     #  INC_AMB, INC_AMC, AJ, AM, UNBL_AJ, UNBLCM
        Common /W_LOAD/ W, WMAX, WSTEP, INCW
        Common /P_LOAD/ P, P_CYCLE, PSTEP, INCP, P_ADD, PMAX, N
        Common /GEOMETRY/ L, H, E, IB, IC
        INC_AJ(1)=PSTEP
        DO Z=2,5
                INC_AJ(Z)=0.0E00
        END DO
        DO Z=1,2
                INC_AML(Z)=0.0E00
        END DO
        INC_AML(3)=WSTEP*L*L/12.0E00
        INC_AML(4)=-WSTEP*L*L/12.0E00
        INC_AML(5)=0.0E00
        DO Z=1,5
                INC_AC(Z)=INC_AJ(Z)-INC_AML(Z)
        END DO
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' INC_AC='
C .     TYPE *, INC_AC
C .     END IF
        RETURN
        END
C .
C ......................................................................
C .
        Subroutine ASSEMBLE_STIFFNESS
        Real*4 S(5,5), S11, S12, S13, S14, S15, S21, S22, S23, S24,
     #  S25, S31, S32, S33, S34, S35, S41, S42, S43, S44, S45, S51,
     #  S52, S53, S54, S55, K23, K45
        Common /RIGID/ S11, S12, S13, S14, S15, S21, S22, S23, S24,
     #  S25, S31, S32, S33, S34, S35, S41, S42, S43, S44, S45, S51,
     #  S52, S53, S54, S55
        Common /SPRING/ K23, K45
        Common /STIFF/ S
```

```
      S(1,1)=2.0E00*S11
      S(1,2)=S12
      S(1,3)=S13
      S(1,4)=S14
      S(1,5)=S15
      S(2,1)=S21
      S(2,2)=S22+K23
      S(2,3)=S23-K23
      S(2,4)=S24
      S(2,5)=S25
      S(3,1)=S31
      S(3,2)=S32-K23
      S(3,3)=S33-K23
      S(3,4)=S34
      S(3,5)=S35
      S(4,1)=S41
      S(4,2)=S42
      S(4,3)=S43
      S(4,4)=S44+K45
      S(4,5)=S45-K45
      S(5,1)=S51
      S(5,2)=S52
      S(5,3)=S53
      S(5,4)=S54-K45
      S(5,5)=S55+K45
      RETURN
      END
C .
C ..................................................................
C .
      Subroutine CALC_INC_DISPLACEMENTS (WDONE,PRINT)
      Real*4 S(5,5), INC_AC(5), INC_D(5), A(25), DET, B(5,1), AC(5),
C .   unused common reals
     # D(5), INC_AJ(5), INC_AML(5), INC_AMA(2), INC_AMB(2),
     # INC_AMC(2), AJ(5), AM(5), UNBL_AJ(5), UNBLCM(3:4)
      Integer WDONE, Z, Y, X, L(5), M(5), PRINT
      Common /DISPLACE/ D, INC_D
      Common /STIFF/ S
      Common /LOADS/ INC_AJ, INC_AML, INC_AC, INC_AMA,
     # INC_AMB, INC_AMC, AJ, AM, UNBL_AJ, UNBLCM
      DO Z=1,25
            A(Z)=0.0E00
      END DO
      DET=0.0E00
      DO Z=1,5
            L(Z)=0
            M(Z)=0
            B(Z,1)=0.0E00
            INC_D(Z)=0.0E00
      END DO
C .   Type *, ' INCREMENTAL NODAL FORCES INC_AC(5x1) :'
      DO Z=1,5
            DO Y=1,5
                  X=5*(Y-1)+Z
                  A(X)=S(Z,Y)
            END DO
C .         Type *, INC_AC(Z)
            B(Z,1)=INC_AC(Z)
            AC(Z)=AC(Z)+INC_AC(Z)
      END DO
```

```
C .      IF (PRINT .EQ. 1) THEN
C .      TYPE *, 'NODAL FORCES AC(5x1) :'
C .      TYPE *, AC
C .      END IF
C .      Type *, ' STIFFNESS S(5x5) :'
C .      Type *, S
C .      Type *, ' FULL STIFFNESS A(25) :'
C .      Type *, A
         Call FULL_SOLVER(A,5,DET,L,M,25)
C .      Type *, ' INCREMENTAL NODAL DISPLACEMENTS INC_D(5x1) :'
         DO Z=1,5
               DO Y=1,5
                     X=5*(Y-1)+Z
                     INC_D(Z)=INC_D(Z)+A(X)*B(Y,1)
               END DO
         END DO
         IF (WDONE .EQ. 0) THEN
         INC_D(1)=0.0E00
         END IF
         Return
         End
C .
C ..........................................................................
C .
      SUBROUTINE FULL_SOLVER(A,N,D,L,M,NN)
      DIMENSION A(NN),L(N),M(N)
      D=1.0
      NK=-N
      DO 80 K=1,N
      NK=NK+N
      L(K)=K
      M(K)=K
      KK=NK+K
      BIGA=A(KK)
      DO 20 J=K,N
      IZ=N*(J-1)
      DO 20 I=K,N
      IJ=IZ+I
   10 IF( ABS(BIGA)- ABS(A(IJ))) 15,20,20
   15 BIGA=A(IJ)
      L(K)=I
      M(K)=J
   20 CONTINUE
      J=L(K)
      IF(J-K) 35,35,25
   25 KI=K-N
      DO 30 I=1,N
      KI=KI+N
      HOLD=-A(KI)
      JI=KI-K+J
      A(KI)=A(JI)
   30 A(JI) =HOLD
   35 I=M(K)
      IF(I-K) 45,45,38
   38 JP=N*(I-1)
      DO 40 J=1,N
      JK=NK+J
      JI=JP+J
      HOLD=-A(JK)
      A(JK)=A(JI)
```

```
     40 A(JI) =HOLD
     45 IF(BIGA) 48,46,48
     46 D=0.0
        RETURN
     48 DO 55 I=1,N
        IF(I-K) 50,55,50
     50 IK=NK+I
        A(IK)=A(IK)/(-BIGA)
     55 CONTINUE
        DO 65 I=1,N
        IK=NK+I
        HOLD=A(IK)
        IJ=I-N
        DO 65 J=1,N
        IJ=IJ+N
        IF(I-K) 60,65,60
     60 IF(J-K) 62,65,62
     62 KJ=IJ-I+K
        A(IJ)=HOLD*A(KJ)+A(IJ)
     65 CONTINUE
        KJ=K-N
        DO 75 J=1,N
        KJ=KJ+N
        IF(J-K) 70,75,70
     70 A(KJ)=A(KJ)/BIGA
     75 CONTINUE
        D=D*BIGA
        A(KK)=1.0/BIGA
     80 CONTINUE
        K=N
    100 K=(K-1)
        IF(K) 150,150,105
    105 I=L(K)
        IF(I-K) 120,120,108
    108 JQ=N*(K-1)
        JR=N*(I-1)
        DO 110 J=1,N
        JK=JQ+J
        HOLD=A(JK)
        JI=JR+J
        A(JK)=-A(JI)
    110 A(JI) =HOLD
    120 J=M(K)
        IF(J-K) 100,100,125
    125 KI=K-N
        DO 130 I=1,N
        KI=KI+N
        HOLD=A(KI)
        JI=KI-K+J
        A(KI)=-A(JI)
    130 A(JI) =HOLD
        GO TO 100
    150 RETURN
        END
C .
C ..............................................................................
C .
        Subroutine CALC_INC_MEMBER_END_ACTIONS (WDONE)
        Real*4 INC_AMA(2), INC_AMB(2), INC_AMC(2), INC_D(5), INC_AM(5),
     #  INC_AML(5), S11, S12, S13, S14, S15, S21, S22, S23, S24, S25,
```

```
     #  S31, S32, S33, S34, S35, S41, S42, S43, S44, S45, S51, S52,
     #  S53, S54, S55,
c .       unused common reals
     #  INC_AJ(5), INC_AC(5), AJ(5), AM(5), UNBL_AJ(5), UNBLCM(3:4), D(5)
          Common /LOADS/ INC_AJ, INC_AML, INC_AC, INC_AMA,
     #  INC_AMB, INC_AMC, AJ, AM, UNBL_AJ, UNBLCM
          INTEGER WDONE
          Common /DISPLACE/ D, INC_D
          Common /RIGID/ S11, S12, S13, S14, S15, S21, S22, S23, S24,
     #  S25, S31, S32, S33, S34, S35, S41, S42, S43, S44, S45, S51,
     #  S52, S53, S54, S55
          Common /INC_MEMBER/ INC_AM
          INC_AMA(1)=S11*INC_D(1)+S12*INC_D(2)
          INC_AMA(2)=S21*INC_D(1)+(S22)*INC_D(2)
          INC_AMB(1)=INC_AML(3)+(S33)*INC_D(3)+S34*INC_D(4)
          INC_AMB(2)=INC_AML(4)+S43*INC_D(3)+(S44)*INC_D(4)
          INC_AMC(1)=S11*INC_D(1)+S15*INC_D(5)
          INC_AMC(2)=S51*INC_D(1)+(S55)*INC_D(5)
          INC_AM(1)=INC_AMA(1)+INC_AMC(1)
          INC_AM(2)=INC_AMA(2)
          INC_AM(3)=INC_AMB(1)
          INC_AM(4)=INC_AMB(2)
          INC_AM(5)=INC_AMC(2)
          IF (WDONE .EQ. 0) THEN
          INC_AM(1)=0.0E00
          END IF
          RETURN
          END
c .
c ..........................................................................
c .
          Subroutine TOTAL_MEMBER_END_ACTIONS (WDONE)
          Real*4 AM(5), INC_AM(5),
c .       unused common reals
     #  INC_AJ(5), INC_AML(5), INC_AC(5), INC_AMA(2), INC_AMB(2),
     #  INC_AMC(2), AJ(5), UNBL_AJ(5), UNBLCM(3:4)
          Integer Z, WDONE
          Common /LOADS/ INC_AJ, INC_AML, INC_AC, INC_AMA,
     #  INC_AMB, INC_AMC, AJ, AM, UNBL_AJ, UNBLCM
          Common /INC_MEMBER/ INC_AM
          DO Z=1,5
                  AM(Z)=AM(Z)+INC_AM(Z)
          END DO
          IF (WDONE .EQ. 0) THEN
          AM(1)=0.0E00
          END IF
          RETURN
          END
c .
c ..........................................................................
c .
          Subroutine TOTAL_DISPLACEMENTS
          Real*4 D(5), INC_D(5)
          Integer Z
          Common /DISPLACE/ D, INC_D
          DO Z=1,5
                  D(Z)=D(Z)+INC_D(Z)
          END DO
          RETURN
          END
```

```
C .
C ..........................................................................
C .
      Subroutine CONNECTION (C,PRINT,I)
C .
C .       This subroutine uses the total displacements, member end
C .     actions and load history (updated internally) to determine the
C .     revised connection stiffness and unbalanced connection moments.
C .       The first part of the actual subroutine determines the
C .     direction of loading and displacements.
C .       Next, it determines the sign and then the specific loading
C .     or unloading path.
C .       Knowing the specific path, the load history, and direction,
C .     the unbalanced moments and stiffness can be determined.
C .       The subroutine is to be used seperately for each connection.
C .     Thus, the parameters are two member arrays indexed for each
C .     particular connection.
C .
C .     Variables:
C .
C .     I=0-10: Iteration index,counts unbalanced moment iterations.
C .             I=0 for incremental load application.
C .     C=3 or 4: Index for connection 2-3 or 4-5.
C .     Wstep=+-incw,0: Load increment from subroutine W_LOADER.
C .     Pstep=+-incp,0:  "      "       "      "       P_LOADER.
C .     LASTCM(C)=(real): A two-member array. The previously calculated
C .                       connection moment for LASTDC(C).
C .     LASTDC(C)=(real): A two-member array. The previously calculated
C .                       total displacement at connections.
C .     AM(C)=(real): A five member array. Current calculated member end
C .                   actions from main program (also connection moments)
C .     DC(C)=(real): Current calculated connection end displacements.
C .                   DC(3)=D(3)-D(2) and DC(4)=D(4)-D(5).
C .     D(C)=(real):  A five member array. Current calculated member end
C .                   displacements from the main program.
C .     POSMAXCM(C)=(real): A two member array. The maximum positve
C .                         connection moment ( CM(C) )  calculated
C .                         internally from POSMAXDC(C). A load history
C .                         parameter.
C .     POSMAXDC(C)=(real): A two member array. The maximum positive
C .                         relative displacement ( DC(C) ) recorded at
C .                         a connection. A load history parameter.
C .     NEGMAXCM(C)=(real): A two member array. See POSMAXCM(C).
C .     NEGMAXDC(C)=(real): A two member array. See POSMAXDC(C).
C .     TPOSMAXCM(C)=(real): A two member array. The moment on the positive
C .                          extrema envelope from which unloading occured.
C .     TPOSMAXDC(C)=(real): A two member array. The deflection correspond-
C .                          ing to TPOSMAXCM(C).
C .     TNEGMAXCM(C)=(real): A two member array. The moment on the negative
C .                          extrema envelope from which unloading occured.
C .     TNEGMAXDC(C)=(real): A two member array. The deflection correspond-
C .                          ing to TNEGMAXCM(C).
C .     Ke=(real given): A real constant. The initial or elastic stiff-
C .                      ness of the connections. Given positive.
C .     De=(real given): A real constant. The relative displacement at
C .                      the elastic limit of the connection. Given
C .                      positive.
C .     Me=(real given): A real constant. The connection moment at the
C .                      elastic limit of the connection. Given positive.
C .     My=(real given): A real constant. The cyclic yield moment of the
```

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c .                    connection. Given positive.
c .     DR(C)=(real): A two member array. Internally calculated residual
c .                   relative displacement during unloading. A load
c .                   history parameter.
c .     POSDR(C)=(real): A two member array. Internally calculated residual
c .                      relative displacement of the extrema envelope.
c .     NEGDR(C)=(real): A two member array. Internally calculated residual
c .                      relative displacement of the extrema envelope.
c .     DP =(real): A temporary value used in calculating DR(C).
c .     DRP=(real): "     "       "     "    "     "       "   .
c .     FR(C)=(0,1): A two member array. A flag denoting that unloading
c .                  has occured. A load history parameter.
c .     CM(C)=(real): A two member array. The actual connection moment
c .                   corresponding to relative connection displacement
c .                   and load history.
c .     K(C)=(real): A two member array. The updated stiffness of the
c .                  connection coresponding to DC(C) and CM(C).
c .     POSK(C)=(real): A two member array. Constant positive moment
c .                     unloading stiffness.
c .     NEGK(C)=(real): A two member array. Constant negative moment
c .                     unloading stiffness.
c .     UNBLCM(C)=(real): A two member array. The unbalanced connection
c .                       moment resulting from the linear approximation
c .                       of the connection stiffness. Defined in negative
c .                       sense. UNBLCM(C)=CM(C)-AM(C).
c .     FUNCTIONS:
c .     FUNC_M[!DC(C)!]=(real): A given function which describes the pos.
c .                             monotonic moment-rotation curve of the
c .                             connection. A function of the relative
c .                             connection
c .                             displcement only. All values given positive
c .     FUNC_K[!DC(C)!]=(real): A given function for the monotonic stiffness
c .                             of the connection (see above). A function of
c .                             the connection relative displacement only.
c .                             All values given positive.
c .
c .           !!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!!
c .
      Real*4 LASTAM(3:4), LASTD(3:4), AM(5), WSTEP, PSTEP, DRP, DP, D(5),
     # KE, ME, DE, MY, LASTCM(3:4), LASTDC(3:4), POSMAXCM(3:4),
     # POSMAXDC(3:4), NEGMAXCM(3:4), NEGMAXDC(3:4), TPOSMAXCM(3:4),
     # TPOSMAXDC(3:4), TNEGMAXCM(3:4), TNEGMAXDC(3:4), DC(3:4),
     # DR(3:4), POSDR(3:4), NEGDR(3:4), CM(3:4), K(3:4), POSK(3:4),
     # NEGK(3:4), UNBLCM(3:4),
c .   unused common reals
     # W, WMAX, INCW, P, P_CYCLE, INCP, P_ADD, PMAX, INC_AJ(5),
     # INC_AML(5),INC_AC(5), INC_AMA(2), INC_AMB(2), INC_AMC(2),
     # AJ(5),UNBL_AJ(5), INC_D(5)
      Integer I, C, FR(3:4), ITERAT, MINC, MDEC, MSAME, PMSAME,
     # NMSAME, NGMINC, PSMINC, PMLINC, PLYINC, PMCINC, PSMDEC,
     # NGMDEC, NMLDEC, NLYDEC, NMCDEC, PRINT,
c .   unused common integers
     # N
      Common /LAST/ LASTAM, LASTD
      Common /DISPLACE/ D, INC_D
      Common /LOADS/ INC_AJ, INC_AML, INC_AC, INC_AMA,
     # INC_AMB, INC_AMC, AJ, AM, UNBL_AJ, UNBLCM
      Common /NEW_CONN_STIFF/ K
      Common /W_LOAD/ W, WMAX, WSTEP, INCW
      Common /P_LOAD/ P, P_CYCLE, PSTEP, INCP, P_ADD, PMAX, N
```

```
      Common /CONNECT/ DRP, DP, KE, ME, DE, MY,
     #  DC, CM, LASTDC, LASTCM, DR, POSDR, NEGDR,
     #  POSMAXCM, POSMAXDC, NEGMAXCM, NEGMAXDC,
     #  TPOSMAXCM, TPOSMAXDC, TNEGMAXCM, TNEGMAXDC, FR
      Common /EXTREMA/ POSK, NEGK
C .
      ASSIGN 100 TO ITERAT
      ASSIGN 150 TO MINC
      ASSIGN 200 TO MDEC
      ASSIGN 250 TO MSAME
      ASSIGN 300 TO PMSAME
      ASSIGN 350 TO NMSAME
      ASSIGN 400 TO NGMINC
      ASSIGN 450 TO PSMINC
      ASSIGN 500 TO PMLINC
      ASSIGN 550 TO PLYINC
      ASSIGN 600 TO PMCINC
      ASSIGN 650 TO PSMDEC
      ASSIGN 700 TO NGMDEC
      ASSIGN 750 TO NMLDEC
      ASSIGN 800 TO NLYDEC
      ASSIGN 850 TO NMCDEC
C .
C .   IF (PRINT .EQ. 1) THEN
C .   TYPE *, ' C=', C
C .   TYPE *, ' AM=', AM(C)
C .   END IF
      If (C .EQ. 3) then                    ! Calculating the relative
            DC(C)=D(2)-D(C)                 ! displacement of the con-
      else                                  ! nection.
            DC(C)=D(5)-D(C)
      End if
C .
      If (I .NE. 0) then                    ! If I=0, this is an incre-
            goto ITERAT                     ! mental load step loop and
      End if                                ! the direction of the mo-
      If (Wstep .GT. 0.0001) then           ! ments can be readily de-
            If (C .EQ. 4) then              ! termined.
                  goto MDEC
            else
                  goto MINC                 ! MDEC=Moment is decreasing
            End if
      End if
C .   If (PRINT .EQ. 1) THEN                ! MINC=Moment is increasing
C .   TYPE *, ' LASTCM(C)=', LASTCM(C)
C .   END IF
      If (AM(C) .LT. LASTCM(C)) then
            goto MDEC
      else
            goto MINC
      End if
C .
100   continue                              ! ITERAT
C .
C .   IF (PRINT .EQ. 1) THEN
C .   TYPE *, ' ITERATE'
C .   END IF
      If ((ABS(AM(C)-LASTCM(C))) .LE. 1.00E00) THEN
            IF ((ABS(DC(C)-LASTDC(C))) .LT. 1.0E-05) THEN
            GOTO MSAME
```

```
                END IF
        END IF
        If (LASTCM(C) .GT. AM(C)) then
                goto MDEC
        ELSE
                goto MINC                               ! MSAME=Moment is unchanged
        End if
C .
150     continue                                        ! MINC
C .
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' MINC'
C .     END IF
        If (AM(C) .LT. 0) then
                goto NGMINC                             ! NGMINC=Negative moment
        else                                            ! and increasing.
                goto PSMINC                             ! PSMINC=Positive moment
        End if                                          ! and increasing.
C .
200     continue                                        ! MDEC
C .
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' MDEC'
C .     END IF
        If (AM(C) .GT. 0) then                          ! PSMDEC=Positive moment
                goto PSMDEC                             ! and decreasing
        else                                            ! NGMDEC=Negative moment
                goto NGMDEC                             ! and decreasing
        End if
C .
250     continue                                        ! MSAME
C .
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' MSAME'
C .     END IF
        If (AM(C) .GE. 0) then
                goto PMSAME                             ! PMSAME=Positive moment
        else                                            ! and unchanged.
                goto NMSAME                             ! NMSAME=Negative moment
        End if                                          ! and unchanged.
C .
300     continue                                        ! PMSAME
C .                                                     ! Positive moment unchanged.
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' PMSAME'
C .     END IF
        If (DC(C) .GT. POSMAXDC(C)) then
                POSMAXDC(C)=DC(C)                       ! Check to see if current
        End if                                          ! displacement is maximum.
        LASTDC(C)=DC(C)
        LASTCM(C)=AM(C)
        UNBLCM(C)=0
        Return
C .
350     continue                                        ! NMSAME
C .                                                     ! Negative moment unchanged.
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' NMSAME'
C .     END IF
        If (DC(C) .LT. NEGMAXDC(C)) then                ! Check to see if current
```

```
                NEGMAXDC(C)=DC(C)                    ! displacement is maximum.
        End if
        LASTDC(C)=DC(C)
        LASTCM(C)=AM(C)
        UNBLCM(C)=0
        Return
C .
400     continue                                     ! NGMINC
C .                                                  ! Negative moment and inc-
C .                                                  ! reasing.
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' NGMINC'
C .     END IF
        IF (NEGMAXDC(C) .EQ. (-De)) then
                NEGMAXDC(C)=LASTDC(C)
        END IF
410     If (LASTDC(C) .EQ. NEGMAXDC(C)) then         ! Extrema unloading.
                DP=LASTCM(C)/Ke                      ! Elastic displacement mag.
                DRP=LASTDC(C)-DP                     ! Elastic res. displacement.
                IF (DRP .GT. 0.00E00) THEN
                DR(C)=DRP
                ELSE
                DR(C)=DRP+ABS(DRP)/3.0E00            ! Elastic residual displace-
                END IF
                FR(C)=1          !DR(C) .NE. 0       ! ment reduced by 1/3.
                K(C)=-LASTCM(C)/(DR(C)-LASTDC(C))      ! Linear unloading stiff
                NEGK(C)=K(C)                           ! for negative moment.
                NEGDR(C)=DR(C)                         ! Extrema res. displ.
        End if
        K(C)=NEGK(C)
        IF ((ABS(K(C))) .LT. 10.00) THEN             ! Constant neg. unloading
                NEGMAXDC(C)=LASTDC(C)
                GOTO 410
        END IF
        IF (NEGMAXDC(C) .GE. -DE) THEN
                K(C)=KE
        END IF
        DP=LASTCM(C)/K(C)                            ! stiffness.
        DR(C)=LASTDC(C)-DP                           ! Unloading M=0 intersect.
        If (DC(C) .GT. DR(C)) then                   ! Check if relative displ-
                LASTDC(C)=DR(C)                      ! acement corresponds to
                LASTCM(C)=0                          ! positive moment.
                GOTO PMLINC
        End if
        CM(C)=LASTCM(C)+K(C)*(DC(C)-LASTDC(C))       ! DC(C) .GT. LASTDC(C).
        UNBLCM(C)=CM(C)-AM(C)                        ! Unbalanced conn. moment.
        LASTCM(C)=CM(C)                              ! Update load history.
        LASTDC(C)=DC(C)
        Return
C .
450     continue                                     ! PSMINC
C .                                                  ! Positive moment and inc-
C .                                                  ! reasing.
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' PSMINC'
C .     END IF
        If (FR(C) .EQ. 0) then                       ! No residual displacement
                goto PMCINC                          ! therefore on monotonic
        else                                         ! curve.
                goto PMLINC                          ! Or on a linear path.
```

```
        End if
c .
500     continue                                        ! PMLINC
c .                                                     ! Positive moment linearly
c .                                                     ! increasing.
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' PMLINC'
C .     END IF
        IF ((LASTCM(C) .LE. 0.0E00) .AND. (LASTAM(C) .LE. 0.00E00)) THEN
        If (POSMAXCM(C) .LE. Me) then                   ! The moment always increas-
                POSMAXCM(C)=Me                          ! es linearly to at least
                POSMAXDC(C)=De                          ! the elastic limit: Me,De.
                TPOSMAXCM(C)=Me
                TPOSMAXDC(C)=Me
        End if
        END IF
        If (POSMAXCM(C) .GE. My) then                   ! PLYINC=Positive moment
                goto PLYINC                             ! linearly increasing to
        End if                                          ! yield moment,My.
        If (LASTCM(C) .LE. 0.00) then                   ! For first crossing into
                If (TPOSMAXCM(C) .EQ. POSMAXCM(C)) then ! positive moment.
                        NEGDR(C)=DR(C)
                End if
        End if
        IF (LASTCM(C) .GT. POSMAXCM(C)) THEN
                DR(C)=NEGDR(C)
        END IF
        If (DR(C) .NE. NEGDR(C)) then                           ! On a load path
c .                                                             ! interior
                K(C)=TPOSMAXCM(C)/(TPOSMAXDC(C)-DR(C))          ! of the extrema hy-
                CM(C)=K(C)*(DC(C)-DR(C))                        ! steresis.
                If (CM(C) .GT. TPOSMAXCM(C)) then               ! Back on extrema
                        DR(C)=NEGDR(C)                          ! hysteresis.
                End if
        End if
        If (DR(C) .EQ. NEGDR(C)) then                   ! On extrema hysteresis.
                K(C)=POSMAXCM(C)/(POSMAXDC(C)-DR(C))    ! Updated conn. stiff.
                CM(C)=K(C)*(DC(C)-DR(C))                ! Actual conn. moment.
                If (CM(C) .GE. POSMAXCM(C)) then        ! Check to see if back
                        CM(C)=FUNCTION_M(DC,C,PRINT)    ! on the monotonic
c .                                                     ! curve.
                        K(C)=FUNCTION_K(DC,C,PRINT)     ! If so, recalculate CM
                        FR(C)=0                         ! and K,make FR=0.
                        If (CM(C) .GE. My) then         ! Check to see if CM on
                                CM(C)=My                ! on curve is .GT. My.
                                K(C)=100.0E00           ! If so, recalculate CM
                        End if                          ! and K,and update the
                        POSMAXCM(C)=CM(C)               ! load history.
                        POSMAXDC(C)=DC(C)
                End if
                TPOSMAXCM(C)=CM(C)
                TPOSMAXDC(C)=DC(C)
        END IF
        IF ((POSMAXDC(C) .LE. De) .AND. (NEGMAXDC(C) .GE. (-De))) THEN
                DR(C)=0.00
                K(C)=KE
                CM(C)=KE*DC(C)
                NEGDR(C)=0.00
        End if
        UNBLCM(C)=CM(C)-AM(C)
```

```
      LASTCM(C)=CM(C)
      LASTDC(C)=DC(C)
      Return
C .
550   continue                                        ! PLYINC
C .                                                   ! Positive moment linearly
C .                                                   ! increasing to My.
C .   IF (PRINT .EQ. 1) THEN
C .   TYPE *, ' PLYINC'
C .   END IF
      If (LASTCM(C) .LE. 0) then                      ! First crossing into
            If (TPOSMAXCM(C) .EQ. POSMAXCM(C)) then   ! positive moment
                  NEGDR(C)=DR(C)                      ! region.
            End if
      End if
      If (DR(C) .NE. NEGDR(C)) then
            K(C)=TPOSMAXCM(C)/(TPOSMAXDC(C)-DR(C))
            CM(C)=K(C)*(DC(C)-DR(C))
            IF (CM(C) .GE. TPOSMAXDC(C)) then
                  DR(C)=NEGDR(C)
            End if
      End if
      If (DR(C) .EQ. NEGDR(C)) then
            K(C)=POSMAXCM(C)/(POSMAXDC(C)-DR(C))
            CM(C)=K(C)*(DC(C)-DR(C))
            If (CM(C) .GE. My) then
                  CM(C)=My
                  K(C)=100.0E00
                  POSMAXCM(C)=CM(C)
                  POSMAXDC(C)=DC(C)
            End if
            TPOSMAXCM(C)=CM(C)
            TPOSMAXDC(C)=DC(C)
      End if
      UNBLCM(C)=CM(C)-AM(C)
      LASTCM(C)=CM(C)
      LASTDC(C)=DC(C)
      Return
C .
600   continue                                        ! PMCINC
C .                                                   ! Positive moment on mono-
C .                                                   ! tonic curve and increas-
C .   IF (PRINT .EQ. 1) THEN
C .   TYPE *, ' PMCINC'
C .   END IF
      If (LASTCM(C) .LE. My) then                     ! ing.
            CM(C)=FUNCTION_M(DC,C,PRINT)
            K(C)=FUNCTION_K(DC,C,PRINT)
            If (CM(C) .GE. My) then                   ! Check to see if new con-
                  CM(C)=My                            ! nection moment is .gt.
                  K(C)=100.0E00                       !       ! yield moment,My.
            End if
      else
            CM(C)=My
            K(C)=100.0E00
      End if
      UNBLCM(C)=CM(C)-AM(C)                           ! Update load history.
      POSMAXCM(C)=CM(C)
      POSMAXDC(C)=DC(C)
      TPOSMAXCM(C)=CM(C)
```

```
      TPOSMAXDC(C)=DC(C)
      LASTCM(C)=CM(C)
      LASTDC(C)=DC(C)
      FR(C)=0
      Return
C .
650   continue                                        ! PSMDEC
C .                                                   ! Positive moment decreas-
C .                                                   ! ing.
C .   IF (PRINT .EQ. 1) THEN
C .   TYPE *, ' PSMDEC'
C .   END IF
      IF (POSMAXDC(C) .EQ. (De)) THEN
            POSMAXDC(C)=LASTDC(C)
      END IF
660   If (LASTDC(C) .EQ. POSMAXDC(C)) then            ! Unloading from extrema
            DP=LASTCM(C)/Ke                           ! Elastic unloading displ.
            DRP=LASTDC(C)-DP                          ! Elastic res. displ.
            IF (DRP .LT. 0.00E00) THEN
                  DR(C)=DRP
            ELSE
                  DR(C)=DRP-ABS(DRP)/3.0E00! Residual displacement.
            END IF
            FR(C)=1
            K(C)=LASTCM(C)/(LASTDC(C)-DR(C))! New unloading stiffness.
            POSDR(C)=DR(C)                            ! Pos. extrema res. displ.
            POSK(C)=K(C)                              ! Pos. unloading stiffness.
      End if
      K(C)=POSK(C)
      IF ((ABS(K(C))) .LT. 10.00) THEN                ! Constant positive un-
            POSMAXDC(C)=LASTDC(C)
            GOTO 660
      END IF
      IF (POSMAXDC(C) .LE. DE) THEN
            K(C)=Ke
      END IF
      DP=LASTCM(C)/K(C)
      DR(C)=LASTDC(C)-DP
      If (DC(C) .LT. DR(C)) then                      ! Check to see if DC(C)
            LASTDC(C)=DR(C)                           ! corresponds to a neg-
            LASTCM(C)=0.00E00                         ! ative moment.
            GOTO NMLDEC
      End if
      CM(C)=LASTCM(C)-K(C)*(LASTDC(C)-DC(C))
      UNBLCM(C)=CM(C)-AM(C)                           ! Unbalanced moment.
      LASTCM(C)=CM(C)                                 ! Update load history.
      LASTDC(C)=DC(C)
      Return
C .
700   continue                                        ! NGMDEC
C .                                                   ! Negative moment and
C .                                                   ! decreasing.
C .   IF (PRINT .EQ. 1) THEN
C .   TYPE *, ' NGMDEC'
C .   END IF
      If (FR(C) .EQ. 0) then                          ! If DR=0,on monotonic
            goto NMCDEC                               ! curve,else on linear
      else                                            ! load path,NMLDEC.
            goto NMLDEC                               ! NMCDEC=On negative mo-
      End if                                          ! ment curve & decreasing.
```

```
C .
750     continue                                        ! NMLDEC
C .                                                     ! Negative moment linearly
C .                                                     ! decreasing,NMLDEC.
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' MNLDEC'
C .     END IF
        IF ((LASTCM(C) .GE. 0.0E00) .AND. (LASTAM(C) .GE. 0.00E00)) THEN
        If (NEGMAXCM(C) .GE. -Me) then                  ! Linear to elastic limit
                NEGMAXCM(C)=-Me                         ! at least.
                NEGMAXDC(C)=-De
                TNEGMAXCM(C)=-Me
                TNEGMAXDC(C)=-De
        End if
        END IF
        If (NEGMAXCM(C) .LE. -My) then                  ! Check to see if loading
                goto NLYDEC                             ! to yield line.
        End if
        If (LASTCM(C) .GE. 0.00) then
                If (TNEGMAXCM(C) .EQ. NEGMAXCM(C)) then
                        POSDR(C)=DR(C)
                End if
        End if
        If (DR(C) .NE. POSDR(C)) then
                K(C)=-TNEGMAXCM(C)/(DR(C)-TNEGMAXDC(C))
                CM(C)=K(C)*(DC(C)-DR(C))
                If (CM(C) .LT. TNEGMAXCM(C)) then
                        DR(C)=POSDR(C)
                End if
        End if
        If (DR(C) .EQ. POSDR(C)) then
                K(C)=-NEGMAXCM(C)/(DR(C)-NEGMAXDC(C))
                CM(C)=K(C)*(DC(C)-DR(C))
                If (CM(C) .LE. NEGMAXCM(C)) then        ! Check to see if have
                        CM(C)=-FUNCTION_M(DC,C,PRINT)   ! intersected monotonic
                        K(C)=-FUNCTION_K(DC,C,PRINT)    ! loading curve.
                        FR(C)=0
                        If (CM(C) .LE. -My) then        ! Check to see if have
                                CM(C)=-My               ! exceeded yield moment.
                                K(C)=100.0E00
                        End if
                        NEGMAXCM(C)=CM(C)               ! Update load history.
                        NEGMAXDC(C)=DC(C)
                End if
                TNEGMAXCM(C)=CM(C)
                TNEGMAXDC(C)=DC(C)
        End if
        IF ((NEGMAXDC(C) .GE. -DE) .AND. (POSMAXDC(C) .LE. DE)) THEN
                DR(C)=0.00
                K(C)=KE
                CM(C)=KE*DC(C)
        END IF
        UNBLCM(C)=CM(C)-AM(C)
        LASTCM(C)=CM(C)
        LASTDC(C)=DC(C)
        Return
C .                                                     .
800     continue                                        ! NLYDEC
C .                                                     ! Negative moment linearly
C .                                                     ! decreasing to -My.
```

```
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' NLYDEC'
C .     END IF
        If (LASTCM(C) .GE. 0) then                          ! First crossing into
                If (TNEGMAXCM(C) .EQ. NEGMAXCM(C)) then     ! negative moment
                        POSDR(C)=DR(C)                      ! region.
                End if
        End if
        If (DR(C) .NE. POSDR(C)) then
                K(C)=-TNEGMAXCM(C)/(DR(C)-TNEGMAXDC(C))
                CM(C)=K(C)*(DC(C)-DR(C))
                If (CM(C) .LE. TNEGMAXCM(C)) then
                        DR(C)=POSDR(C)
                End if
        End if
        If (DR(C) .EQ. POSDR(C)) then
                K(C)=-NEGMAXCM(C)/(DR(C)-NEGMAXDC(C))
                CM(C)=K(C)*(DC(C)-DR(C))
                If (CM(C) .LE. -My) then
                        CM(C)=-My                           ! Check that moments do not
                        K(C)=100.0E00                       ! exceed yield, My.
                        NEGMAXCM(C)=CM(C)
                        NEGMAXDC(C)=DC(C)
                End if
                TNEGMAXCM(C)=CM(C)
                TNEGMAXDC(C)=DC(C)
        End if
        UNBLCM(C)=CM(C)-AM(C)
        LASTCM(C)=CM(C)
        LASTDC(C)=DC(C)
        Return
C .
850     continue                                            ! NMCDEC
C .                                                         ! Negative moment decreasing
C .                                                         ! along monotonic curve.
C .     IF (PRINT .EQ. 1) THEN
C .     TYPE *, ' NMCDEC'
C .     END IF
        If (LASTCM(C) .GE. -My) then
                CM(C)=-FUNCTION_M(DC,C,PRINT)               ! Get conn. moment.
                K(C)=FUNCTION_K(DC,C,PRINT)                 ! Get conn. stiffness.
                If (CM(C) .LE. -My) then                    ! Check that conn. moment
                        CM(C)=-My                           ! .lt. yield, My.
                        K(C)=100.0E00
                End if
        else
                CM(C)=-My
                K(C)=100.0E00
        End if
        UNBLCM(C)=CM(C)-AM(C)                               ! Unbalanced conn. moment.
        NEGMAXCM(C)=CM(C)                                   ! Update loading history.
        NEGMAXDC(C)=DC(C)
        TNEGMAXCM(C)=CM(C)
        TNEGMAXDC(C)=DC(C)
        LASTCM(C)=CM(C)
        LASTDC(C)=DC(C)
        FR(C)=0
        Return
        END
C .
```

```
c ..........................................................................
c .
      Subroutine MOMENT_MAGNIFICATION (M)
      Real*4 NEWV3, NEWV4, W, L, AM(5), D(5), INC_D(5), V3, V4,
     # UNBLV3D1, UNBLV4D1, INC_AC(5), INC_AML(5),
c .   unused common reals
     # WMAX, WSTEP, INCW, H, E, IB, IC, INC_AJ(5),
     # INC_AMA(2), INC_AMB(2), INC_AMC(2), AJ(5), UNBL_AJ(5),
     # UNBLCM(3:4)
      Common /AXIAL/ V3, V4, NEWV3, NEWV4, UNBLV3D1, UNBLV4D1
      Common /W_LOAD/ W, WMAX, WSTEP, INCW
      Common /GEOMETRY/ L, H, E, IB, IC
      Common /LOADS/ INC_AJ, INC_AML, INC_AC, INC_AMA,
     # INC_AMB, INC_AMC, AJ, AM, UNBL_AJ, UNBLCM
      Common /DISPLACE/ D, INC_D
      INTEGER M, Z
      NEWV3=W*L/2+(AM(3)+AM(4))/L
      NEWV4=W*L/2-(AM(3)+AM(4))/L
      UNBLV3D1=NEWV3*D(1)-V3*(D(1)-INC_D(1))
      UNBLV4D1=NEWV4*D(1)-V4*(D(1)-INC_D(1))
      V3=NEWV3
      V4=NEWV4
C.    IF ( M .EQ. 0) THEN
C.            INC_AC(1)=0.0E00
C.            INC_AC(2)=-UNBLV3D1
C.            INC_AC(3)=0.0E00
C.            INC_AC(4)=0.0E00
C.            INC_AC(5)=-UNBLV4D1
C.            DO Z=1,5
C.             INC_AML(Z)=0.0E00
C.            END DO
C.    END IF
      RETURN
      END
c .
c ..........................................................................
c .
      Subroutine UNBALANCED_MOMENTS (PRINT)
      Real*4 UNBL_AJ(5), UNBLCM(3:4), UNBLV3D1, UNBLV4D1,
c .   unused common reals
     # INC_AJ(5), INC_AML(5), INC_AC(5), INC_AMA(2), INC_AMB(2),
     # INC_AMC(2), AJ(5), AM(5), V3, V4, NEWV3, NEWV4
      Common /LOADS/ INC_AJ, INC_AML, INC_AC, INC_AMA,
     # INC_AMB, INC_AMC, AJ, AM, UNBL_AJ, UNBLCM
      Common /AXIAL/ V3, V4, NEWV3, NEWV4, UNBLV3D1, UNBLV4D1
      Integer Z, PRINT
      UNBL_AJ(1)=0.0E00
      UNBL_AJ(2)=-UNBLCM(3)-UNBLV3D1
      UNBL_AJ(3)=UNBLCM(3)
      UNBL_AJ(4)=UNBLCM(4)
      UNBL_AJ(5)=-UNBLCM(4)-UNBLV4D1
C .   IF (PRINT .EQ. 1) THEN
C .   TYPE *, ' UNBALANCED NODAL FORCES :'
C .   TYPE *, UNBL_AJ
C .   END IF
      RETURN
      END
c .
c ..........................................................................
c .
```

```
      Subroutine CHECK_UNBALANCED_MOMENTS (BAL)
      Real*4 UNBL_AJ(5),
c .   unused common reals
     # INC_AJ(5), INC_AML(5), INC_AC(5), INC_AMA(2), INC_AMB(2),
     # INC_AMC(2), AJ(5), AM(5), UNBLCM(3:4)
      Common /LOADS/ INC_AJ, INC_AML, INC_AC, INC_AMA,
     # INC_AMB, INC_AMC, AJ, AM, UNBL_AJ, UNBLCM
      Integer BAL, Z
      BAL=1
      DO Z=2,5
            IF(ABS(UNBL_AJ(Z)) .GE. 1.00E00) THEN
                  BAL=0
            END IF
      END DO
      RETURN
      END
c .
c ..................................................................
c .
      Subroutine REITERATE (I,REDO)
      Real*4 INC_AC(5), UNBL_AJ(5), INC_AML(5), K(3:4), K23, K45,
c .   unused common reals
     # INC_AJ(5), INC_AMA(2), INC_AMB(2), INC_AMC(2), AJ(5), AM(5),
     # UNBLCM(3:4), DRP, DP, KE, ME, DE, MY, DC(3:4),
     # CM(3:4), LASTDC(3:4), LASTCM(3:4), DR(3:4), POSDR(3:4),
     # NEGDR(3:4), POSMAXCM(3:4), POSMAXDC(3:4),
     # NEGMAXCM(3:4), NEGMAXDC(3:4), TPOSMAXCM(3:4), TPOSMAXDC(3:4),
     # TNEGMAXCM(3:4), TNEGMAXDC(3:4)
      Integer I, REDO, Z,
c .   unused common integers
     # FR(3:4)
      Common /LOADS/ INC_AJ, INC_AML, INC_AC, INC_AMA,
     # INC_AMB, INC_AMC, AJ, AM, UNBL_AJ, UNBLCM
      Common /NEW_CONN_STIFF/ K
      Common /CONNECT/ DRP, DP, KE, ME, DE, MY,
     # DC, CM, LASTDC, LASTCM, DR, POSDR, NEGDR,
     # POSMAXCM, POSMAXDC, NEGMAXCM, NEGMAXDC,
     # TPOSMAXCM, TPOSMAXDC, TNEGMAXCM, TNEGMAXDC, FR
      Common /SPRING/ K23, K45
      I=I+1
      REDO=1
      K23=K(3)
      K45=K(4)
      DO Z=1,5
            INC_AC(Z)=UNBL_AJ(Z)
            INC_AML(Z)=0.0E00
      END DO
      IF (I .GT. 10) THEN
            REDO=0
      END IF
      RETURN
      END
c .
c ..................................................................
c .
      Subroutine NONCONVERGENCE
      Real*4 W, P,
c .   unused common reals
     # WMAX, WSTEP, INCW, P_CYCLE, PSTEP, INCP, P_ADD, PMAX
c .   unused common integers
```

```
      Integer N
      Common /W_LOAD/ W, WMAX, WSTEP, INCW
      Common /P_LOAD/ P, P_CYCLE, PSTEP, INCP, P_ADD, PMAX, N
c .   WRITE W,P, "NONCONVERGENCE AFTER 10 ITERATIONS"
      Type *, N, W, P, ' NONCONVERGENCE AFTER 10 ITERATIONS'
      RETURN
      END
c .
c ..........................................................................
c .
      Subroutine OUTPUT (PRINT)
      Real*4 W, P, AM(5), D(5), K23, K45, V3, V4, K(3:4), R(3:4),
     # LASTAM(3:4), LASTD(3:4),
c .   unused common reals
     # WMAX, WSTEP, INCW, P_CYCLE, PSTEP, INCP, P_ADD, PMAX,
     # INC_AJ(5), INC_AML(5), INC_AC(5), INC_AMA(2), INC_AMB(2),
     # INC_AMC(2), AJ(5), UNBL_AJ(5), UNBLCM(3:4), INC_D(5),
     # NEWV3, NEWV4, UNBLV3D1, UNBLV4D1
c .   unused common integers
      Integer N, PRINT
      Common /LAST/ LASTAM, LASTD
      Common /NEW_CONN_STIFF/ K
      Common /W_LOAD/ W, WMAX, WSTEP, INCW
      Common /P_LOAD/ P, P_CYCLE, PSTEP, INCP, P_ADD, PMAX, N
      Common /LOADS/ INC_AJ, INC_AML, INC_AC, INC_AMA,
     # INC_AMB, INC_AMC, AJ, AM, UNBL_AJ, UNBLCM
      Common /DISPLACE/ D, INC_D
      Common /SPRING/ K23, K45
      Common /AXIAL/ V3, V4, NEWV3, NEWV4, UNBLV3D1, UNBLV4D1
      K23=K(3)
      K45=K(4)
      R(3)=D(2)-D(3)
      LASTD(3)=R(3)
      R(4)=D(5)-D(4)
      LASTD(4)=R(4)
      LASTAM(3)=AM(3)
      LASTAM(4)=AM(4)
c .   WRITE,W,P,AM,D,K23,K45,V3,V4
      IF (PRINT .EQ. 1) THEN
      Type *, ' '
      Type *, N, W, P
      Type *, AM
      TYPE *, R(3), R(4)
      Type *, D
      Type *, V3, V4, K23, K45
      TYPE *, ' '
      END IF
      RETURN
      END
c .
c ..........................................................................
c .
      Function FUNCTION_M(DC,J,PRINT)
      Real*4 X, A, B, C, D, DC(3:4)
      INTEGER PRINT, J
      A=1.34856666E09
      B=-29703882.4E00
      C=238347.279E00
      D=99.871082
      X=ABS(DC(J))
```

```
      FUNCTION_M=A*X*X*X+B*X*X+C*X+D
      IF (X .LT. 0.0007) THEN
            FUNCTION_M=3.67164E05*X
      END IF
      IF (X .GT. 0.0053) THEN
            FUNCTION_M=720.00E00+(X-0.0053)*100.0E00
      END IF
C .   IF (PRINT .EQ. 1) THEN
C .   TYPE *, ' FUNCTION_M: '
C .   TYPE *, ' C=', J
C .   TYPE *, ' ABS(DC)=', X
C .   TYPE *, ' M=', FUNCTION_M
C .   END IF
      Return
      End
c .
c ------------------------------------------------------------------------
c .
      Function FUNCTION_K(DC,J,PRINT)
      Real*4 X, A, B, C, DC(3:4)
      INTEGER PRINT, J
      A=1.34856666E09
      B=-29703882.4E00
      C=238347.279E00
      X=ABS(DC(J))
      FUNCTION_K=3.0E00*A*X*X+2.0E00*B*X+C
      IF (X .LT. 0.0007) THEN
            FUNCTION_K=3.67164E05
      END IF
      IF (X .GT. 0.0053) THEN
            FUNCTION_K=100.00E00
      END IF
C .   IF (PRINT .EQ. 1) THEN
C .   TYPE *, ' FUNCTION_K:'
C .   TYPE *, ' C=', J
C .   TYPE *, ' ABS(DC)=', X
C .   TYPE *, ' K=', FUNCTION_K
C .   END IF
      Return
      End
c .
c ------------------------------------------------------------------------
```

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