Computer-Aided Design of Steel Structures with Flexible Connections

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Summary

The influence of connection flexibility on the behavior of steel framed structures has long been recognized, however, due to the difficulty of accurately modeling connection effects in analysis, these effects are usually not considered explicitly in design. This paper describes the development and application of a computer-aided design system for including semi-rigid connection behavior in the analysis and design of two and three dimensional buildings. The system utilizes interactive computer-graphics to provide a convenient means of defining and characterizing joint behavior for design.

Inelastic connection behavior is modeled using nonlinear moment rotation curves that are implemented in an analysis and design program which can account for both geometric and material nonlinear behavior in framed structures. For design, connection response is characterized using a library of standarized moment-rotation curves which are calibrated to experimental test data for various connection configurations. Two case studies are presented which

demonstrate the influence of connection flexibility in evaluating strength and serviceability limit states. Also considered is the effect of semi-rigid connections on the ultimate limit load of the structure considered. The computer-aided analysis and design methodology which is presented provides an approach for taking reasonable account of connection effects during the design phase, prior to final detailing of the connections.

COMPUTER-AIDED DESIGN OF STEEL STRUCTURES WITH FLEXIBLE CONNECTIONS

The influence of connection flexibility on the behavior of steel framed structures has long been recognized by engineers. However, because of uncertainties in predicting joint response and difficulties associated with incorporating it in analysis, inelastic joint flexibility is usually not considered explicitly in design. Consequently, in spite of much research there is still incomplete understanding of joint effects and their significance, and need for convenient methods for including these in analysis and design.

Several trends in building design and construction are increasing the importance of incorporating joint behavior in design. These include: 1) the development of inelastic limit state design procedures which require more realistic analysis of actual response, 2) growing emphasis for evaluating inelastic structural response to earthquakes and other extreme loadings, and 3) structural challenges posed by innovations in architecture and construction. Advances in computer technology, particularly the availability of low cost engineering workstations, are providing the means for performing more realistic analyses of structures including joint behavior.

This paper describes the development and application of a computer-aided system for including semi-rigid connection behavior in the analysis and design of three dimensional building frames. A key aspect of the proposed method is the introduction of a standardized connection model which facilitates the incorporation of semi-rigid connection behavior during the preliminary and final stages of design. The analytic formulation used for modelling connection response is based on a discrete nonlinear rotational spring which is implemented in a program for the analysis and design of three dimensional steel structures. The analysis is based on a finite element approach where the structure is discretized into 3-D inelastic beam-column line elements connected by either rigid or semi-rigid connections. The computer-aided analysis and design system utilizes interactive menu-driven graphics for definition of the structural geometry and properties, characterization of connection behavior, control of the analysis and design process, and display of structural response.

The paper is organized as follows: 1) a description of the moment-rotation behavior model used for the connection, 2) a brief description of the beam-column element formulation and the computer-aided analysis and design system, 3) a presentation of two case studies which demonstrate use of the system in the investigation of the influence of partially restrained connections on frame behavior, and 4) a summary and conclusions.

STANDARDIZED MOMENT-ROTATION MODEL FOR CONNECTIONS

In the past, many techniques have been proposed for representing the moment-rotation behavior of semi-rigid connections, some based on simple linear approximations and others on more sophisticated nonlinear functions. The model used in this work is based on a nonlinear equation first presented by Richard and Abbott (1975), and later by Kishi et.al. (1988). Using this model, the moment-rotation relationship of the connection is given by the following equation:

$$M = \frac{(K_e - K_p)\theta}{\left[1 + \left[\frac{(K_e - K_p)\,\theta}{M_o}\right]^n\right]^{1/n}} + K_p\theta \tag{1a}$$

In Eq. 1a, M is the moment corresponding to the connection rotation, θ. The parameters, K_e, K_p, and M_o, are independent variables which are related to the moment-rotation behavior as shown in Fig. 1, and n controls the shape of the curve. This model was chosen because it represents observed experimental data well, it is convenient to implement in the computer program described below, and the four parameters are derived from a rational interpretation of response. One advantage of this model is that it encompasses more simple models. For example, Eq. 1a becomes a simple linear model if $K_e = K_p$, an elastic-plastic model if $K_p = 0$, and a bilinear model if n is large.

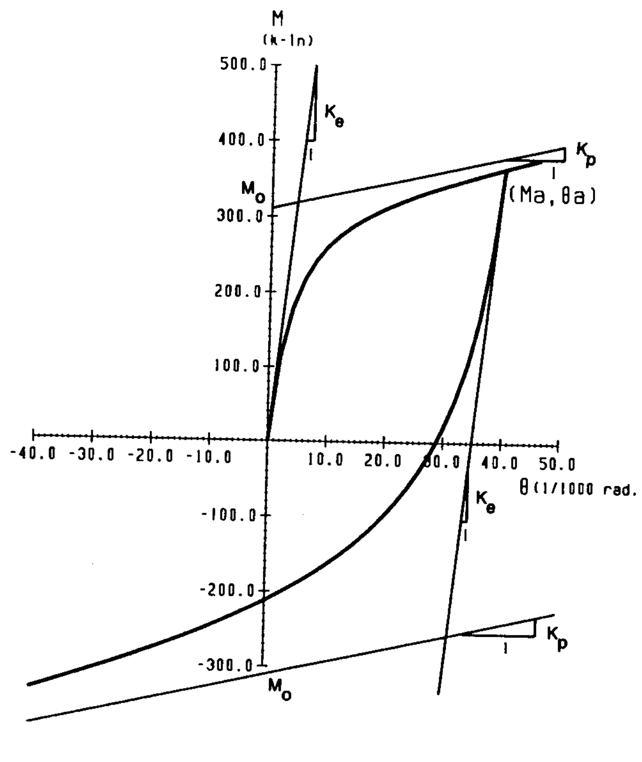


Figure 1. Moment-Rotation Model for Inelastic Connection Response.

To allow for unloading of the connections associated with nonproportional loading and inelastic force redistribution, the unloading curve shown in Fig. 1 was developed (Hsieh 1990). This portion of the moment-rotation curve is given by the following equation, where the peak moments and rotations reached during the initial loading are M_a and θ_a:

$$M = M_a - \frac{(K_e - K_p)(\theta_a - \theta)}{\left[1 + \left[\frac{(K_e - K_p)(\theta_a - \theta)}{2\,M_o}\right]^n\right]^{1/n}} - K_p(\theta_a - \theta) \qquad (1b)$$

In practice, a major obstacle to including semi-rigid connection behavior in the overall analysis and design is the difficulty of defining the parameters of the moment-rotation curve. Connection behavior is an integration of many effects including the connection type, geometry, materials, detailing, workmanship, etc. In particular, during design of the overall structural system it is difficult (if not impossible) to precisely establish the parameters which define the moment-rotation behavior since usually the exact connection is not completely detailed until late in the design process. One solution to this is the development of standardized connection reference curves which are based on experimental test data and normalized to be amenable to design.

To generalize Eqs. 1a and 1b for use in design, the moment-rotation expressions are normalized with respect to a reference value of moment which is defined herein as the *nominal connection capacity*, M_{cn}. The normalized expressions are identical to Eqs. 1a & b except that M, K_e, K_p, and M_o are replaced by $M'=M/M_{cn}$, $K_e'=K_e/M_{cn}$, $K_p'=K_p/M_{cn}$, and $M_o'= M_o/M_{cn}$. An example is presented below to show how the normalized curves are developed for top- and seat-angle connections with double web angles.

Using a standard curve fitting technique, Eq. 1a was calibrated to experimental data for top- and seat-angle connections with double web angles (TSAW) as shown in Fig. 2. The data in this case are based on tests conducted by Azizinamini which are included in the Kishi and Chen data base (Kishi 1986). The curves shown in Fig. 2 were normalized by a value of M_{cn} equal to the moment resisted at an applied rotation of 0.02 radians. This value was chosen after considering several alternate normalization schemes, further details of which are reported by Hsieh (1990). The normalization results in the set of curves shown in Fig. 3. For a given type of connection, this procedure provides a convenient means of condensing the data from a large number of tests by eliminating variations due to scale effects.

From the normalized curves shown in Fig. 3, the three standard reference curves shown in Fig. 4 were developed. The center (TSAW-Ave) curve in Fig. 4 was obtained by fitting a curve through the average of the set of curves in Fig. 3. The upper and lower curves in Fig. 4 reflect a variation from the average curve of plus or minus two standard deviations. Assuming the

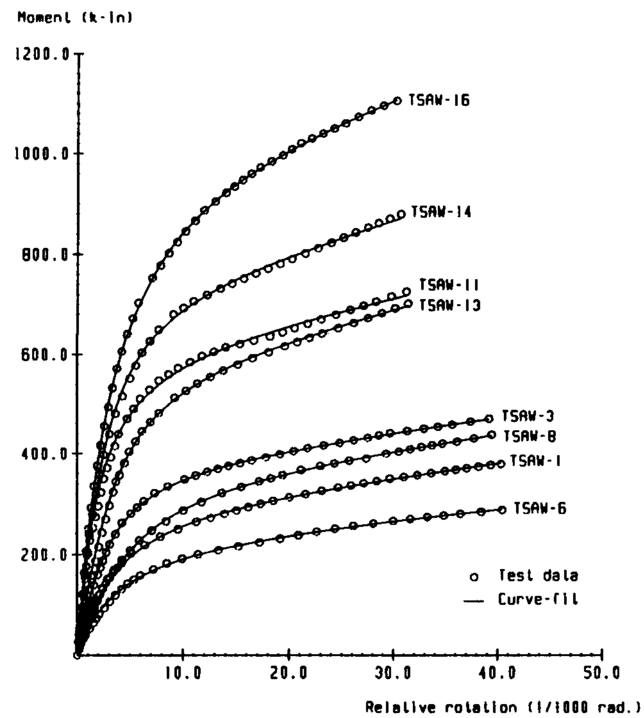


Figure 2. Moment-Rotation Behavior for TSAW Connections.

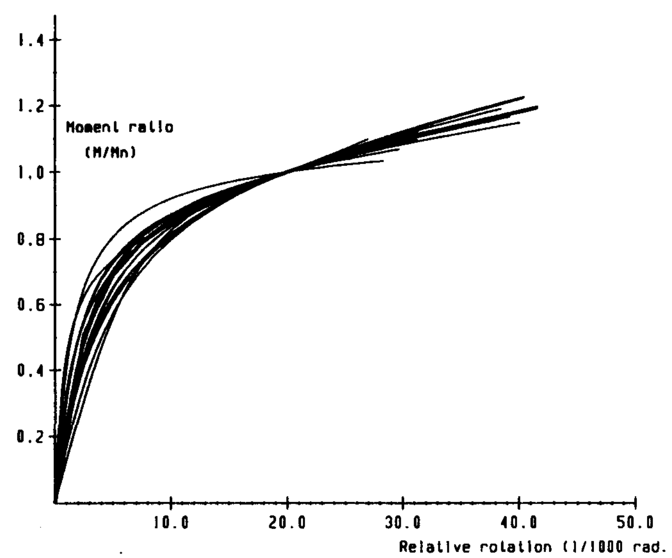


Figure 3. Normalized Moment-Rotation Behavior for TSAW Connections.

variation in connection response is random and normally distributed, the region between the upper and lower curves in Fig. 4 encompasses roughly 95% of the sampled data. Currently, similar curves are being developed by the authors for additional connection types. Parameters for the three curves shown in Fig. 4 are presented in Table 1.

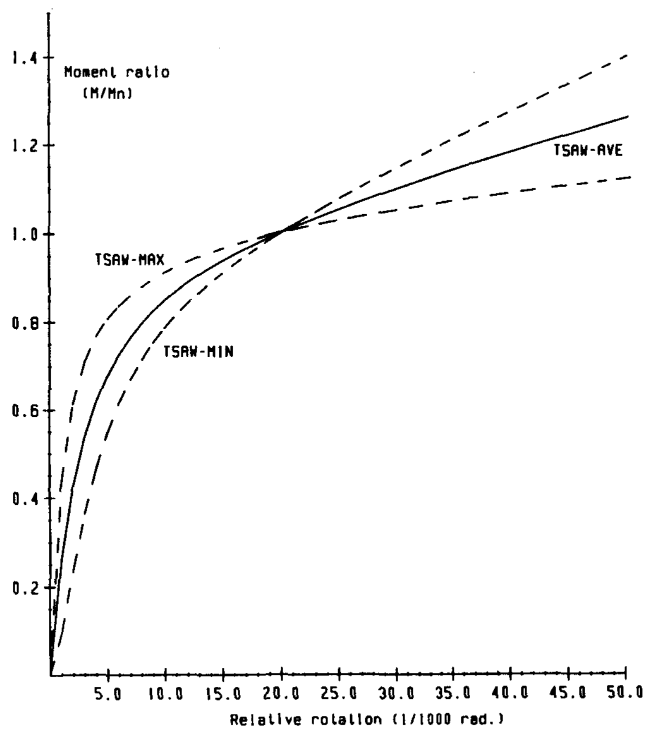


Figure 4. Standardized Moment-Rotation Curves, for TSAV Connections.

Table 1 Standard Reference Curve Parameters for TSAW Connections

Curve	M_o'	K_e'	K_p'	n
TSAW-Max	1.0	430	2.6	1.2
TSAW-Ave	0.9	270	6.9	1.3
TSAW-Min	0.8	100	12.4	3.3

The aim of this approach is to establish a library of standard reference curves for common connection configurations. Then, for analysis of the overall structure, only the connection type and nominal capacity would need to be defined. As shown in the first example below, standard curves such as those shown in Fig. 4 can be used to investigate the range of expected structural behavior. By establishing realistic upper and lower bounds of response based on the type and strength of the connection, the structure can be reliably designed without unnecessary concern over the precise behavior of the final connection detail.

A remaining question in the proposed method is how to calculate the value of M_{cn} for design of the final connection detail. M_{cn} defined by the moment sustained at a rotation of approximately 0.02 radians, is M_{cn} representative of a nominal capacity which could be calculated based on plastic mechanism design procedures such as those in the AISC Engineering Detailing Manual. A preliminary investigation of the calculation for shows that the AISC procedures provide a low value for this moment compared to measured test data. Alternative procedures, such as those developed by Wu (1988) and others, are being reviewed and improved models for calculating M_{cn} are currently being studied.

COMPUTER-AIDED ANALYSIS AND DESIGN SYSTEM

For this research, the semi-rigid connection model was implemented in an analysis and design program for steel structures called CU-STAND. CU-STAND is an interactive-graphics program which is capable of both geometric and material nonlinear analysis of 3 dimensional structures (Ziemian et.al. 1990, Hsieh et.al. 1989, Deierlein et.al. 1989). Geometric nonlinear behavior is modelled through a second order analysis using an updated Lagrangian formulation with geometric element stiffness matrices. Material nonlinear (inelastic) response is included through a concentrated plasticity model which is based on a three parameter yield surface. The yield surface provides an elastic-plastic model which includes the influence of major- and minor-axis bending and axial loads on member yielding.

In the analysis, the zero-length connection springs defined by the model described in the previous section are attached to 3-D beam-column elements. The beam-column elements have 6 degrees of freedom at each end, and the connection implementation allows for definition of two rotational springs at each end, corresponding to the major- and minor-bending axes. In CU-STAND, the connection properties are defined interactively using the menu shown in Fig. 5 and then attached to specified members. For design purposes, the nominal connection strength, M_{cn} may be defined either as a fixed value or as a fraction of the nominal moment capacity of the connected member. The latter option is useful when the semi-automated redesign features of CU-STAND are used in an iterative analysis/design process to determine the required steel section sizes.

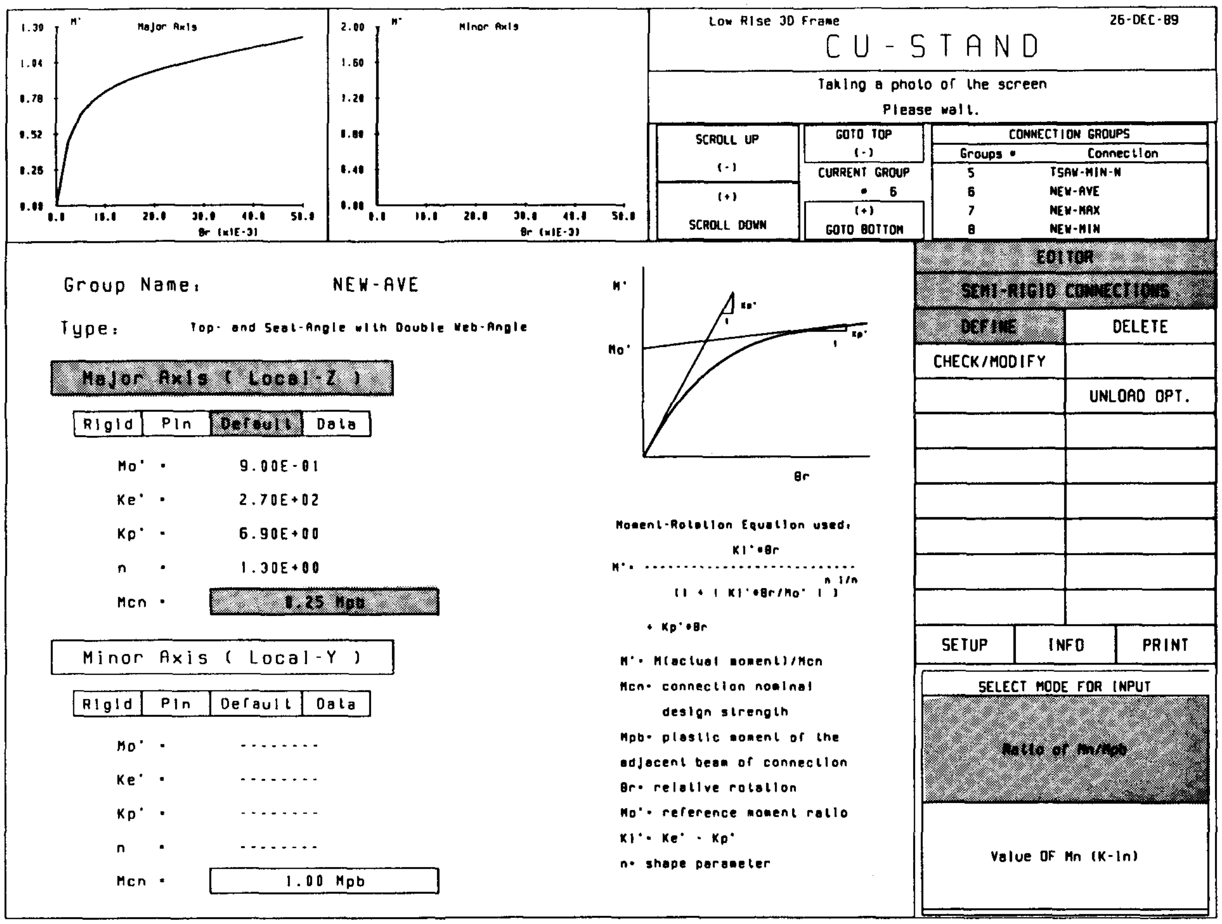


Figure 5. Connection Definition Menu of CU-STAND.

CASE STUDIES

Two case studies are presented to demonstrate application of the system for the analysis and design of frames with partially restrained connections. Investigation of the sensitivity of the overall structural behavior to variations in the assumed connection properties is also included.

Two Dimensional Frame

The two story partially restrained (PR) frame shown in Fig. 6 was designed based on the AISC-LRFD Specification (1986) for the loads shown. The beam-column connections are assumed to be TSAW connections whose behavior is defined the average standard curve (TSAW-Ave) presented previously in Fig. 4 and Table 1. The design was based on a second-order analysis where the dead, live, and wind loads were applied proportionally up to the full factored loads per the load combinations given by the Specification (Eqs. A4-1 to A4-6, AISC-

LRFD 1986). The final member sizes (shown in Fig. 6) were controlled by the gravity load combinations without lateral load. In the analysis and design procedure, the nominal connection strength, M_{cn} was set equal to 25% of the plastic moment, M_{pb} of the adjacent beam. In this way, during reanalysis of trial designs, the connection strength was automatically updated to correspond to the current member sizes.

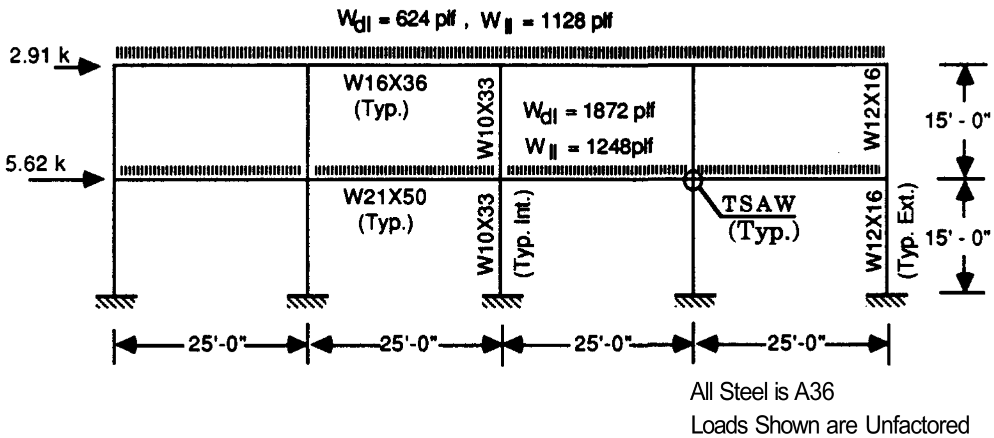


Figure 6. Elevation of Two Dimensional Building Frame.

After the members were designed using the average connection curve (TSAW-Ave), the behavior was evaluated for a range of connection parameters. Both the service and strength limit states were investigated for four cases: three PR frames with connections defined by the average, upper, and lower bound curves shown in Fig. 4 (TSAW-Ave, TSAW-Max, TSAW-Min), and one fully restrained (FR) frame with rigid connections. In the three PR frames, the connection strength was kept constant with $M_{cn} = 0.25M_{pb}$.

Second-order analyses were performed to evaluate the lateral drift under service conditions using the load combination given in Table 2a. In these analyses, proportional loading was used where loads were applied incrementally. The analysis included the geometric nonlinear behavior of the overall system and the nonlinear connection response (which is largely the result of local inelasticity). Gravity loads were included because of their contribution to the total connection deformation and hence to the change in connection stiffness during loading. As shown by the data presented in Table 2a, the frames with semi-rigid connections had drifts on the order of 1.83 to 1.93 times that for the frame with rigid connections. Also, there was relatively little difference in drift due to the variation in the moment-rotation model used for the three semi-rigid cases. Finally, for all four cases, the calculated service load wind drift was less than H/500 = 0.72 inches.

Table 2a - Roof Drift Under Service Load

Loading	Roof Drift (inches)			
	Rigid (FR)	Semi-Rigid (PR)		
		TSAW-Max	TSAW-Ave	TSAW-Min
1.ODL + 0.2LL + 1.OWL	0.29	0.53	0.56	0.56

Second-order analyses were performed to evaluate the strength of the frames at the full factored load and at the limit point. In addition to nonlinear connection and geometric response, these analyses included member plastification through the elastic-plastic yield surface model described previously. The load-deformation response under gravity plus wind loads (1.2DL + 0.5LL + 1.3 WL) is shown in Fig. 7 where the roof drift is plotted versus the applied load ratio. In this case, the applied load ratio of 1.0 corresponds to the full factored load, and as shown, the maximum limit point for all the frames exceeded the full factored load.

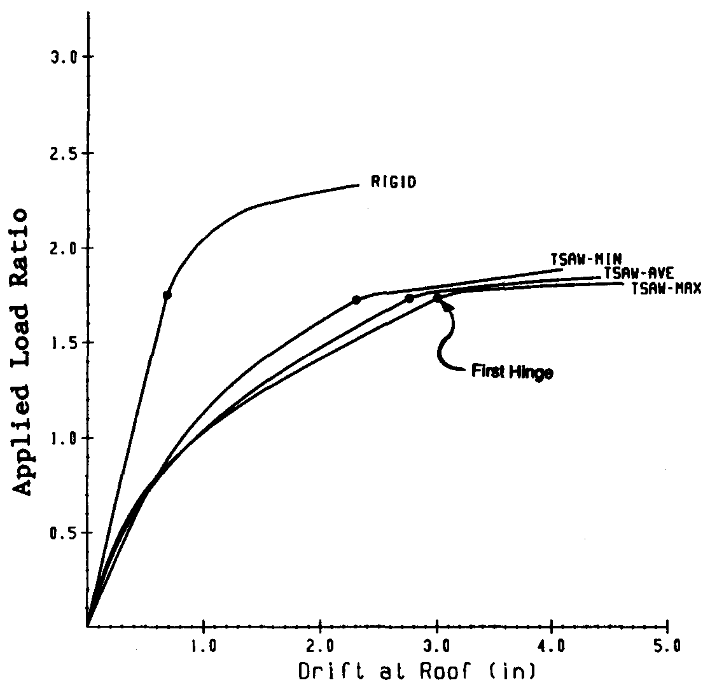


Figure 7. Inelastic Load-Deflection Response of 2-D Frame.

As shown in Fig. 7, there was considerable variation in response between the frames with semi-rigid versus rigid connections. As in the service load analyses, the variation between the three PR frames was rather small, particularly at low loads. The FR frame was stiffer throughout the entire range of loading, and for the load combination presented in Fig. 7, the maximum limit strength for the FR frame was approximately 25% greater than that for the PR frames. Note that at large deformations the PR frame with TSAW-Min connections carried a slightly higher load than that with TSAW-Max connections. This is due to the fact that at large connection rotations (greater than 0.02 radians) the TSAW-Min connection resists a larger moment (see Fig. 4). Finally, as indicated in Fig. 7, for the lateral loading combination in all four frames, the first plastic hinge formed at roughly the same applied load ratios (1.72 to 1.75) which were well above the full factored load.

A summary of the results from second-order analyses that included both member and connection inelasticity is presented in Table 2b. The applied load ratios are listed for the load at which the first hinge occurred and at the limit point. As in Fig. 7, an applied load ratio of 1.0 corresponds to the full factored load. Several observations can be drawn from the data in this table. First, as noted previously, the difference in the three curves used to model the semi-rigid connections did not have a significant influence on the overall structural response. Also, for all three load combinations, the first plastic hinge occurred at approximately the same load ratio for the rigid and semi-rigid frames, although these hinges did not necessarily form at the same locations in the frames. In general, the first hinges occurred near the midspan of the beams in the PR frames and at the beam ends in the FR frames.

Table 2b - Applied Load Ratios for Factored Load Combinations

Criteria & Loading		Applied Load Ratio			
		Rigid (FR)	Semi-Rigid (PR)		
			TSAW-Max	TSAW-Ave	TSAW-Min
1st Hinge	$1.2DL + 1.6L_f + 0.5L_r$	1.28	1.23	1.21	1.20
	$1.2DL + 0.5L_f + 1.6L_r$	1.24	1.19	1.18	1.18
	$1.2DL + 0.5LL + 1.3WL$	1.75	1.74	1.73	1.72
Limit Point	$1.2DL + 1.6L_f + 0.5L_r$	1.87	1.42[1]	1.54[1,2]	1.63[1]
	$1.2DL + 0.5L_f + 1.6L_r$	1.70	1.63	1.65[1]	1.62[1]
	$1.2DL + 0.5LL + 1.3WL$	2.33	1.81	1.84	1.89

Notes: 1 For the load ratios noted, the associated connection rotations are in excess of 50×10^{-3} radians.
2 For this case, the maximum limit load ratio is equal to 1.31 at a maximum connection rotation of 50×10^{-3} radians.

For all load combinations, the limit point of the FR frame was greater than that of the PR frames, where the load ratio's for the PR frames ranged from 0.76 to 0.97 of those for the FR frame. As indicated by Note 1 in Table 2b, in some cases, the joint rotations corresponding to the limit point in the PR frames exceeded 0.05 radians which is beyond the limits of most experimental data. Also, calculation of the exact limit point in these cases was sometimes influenced by the numerical convergence of the solution algorithm. Hence, for analysis and design of PR frames, it is advisable to place a restriction on the maximum limit point based on a realistic upper limit of the connection rotation. As an example, consider the case indicated by Note 2 in Table 2b where the limit point load ratio of 1.54 corresponded to a maximum connection rotation of 0.177 radians. If the maximum rotation were limited to 0.05 radians, the corresponding load ratio would have been 1.31 which is 15% less than the peak of 1.54, but still 30% greater than the full factored load.

In routine design, the maximum limit strength of the frame is usually not calculated, but rather member design checks at the full factored load are used to ensure that the structure can safely resist the applied loads. For the frames considered in this example, the maximum values of the AISC-LRFD member interaction design checks (LRFD Eqs. Hl-la&b) were calculated as the following:

Rigid:	0.97	Semi-Rigid:	TSAW-Max	0.89
			TSAW-Ave	0.92
			TSAW-Min	0.94

These are the maximum values for each of the load combinations listed in Table 2b calculated using a second-order analyses at an applied load ratio of 1.0. The value of 0.92 ($\leq$ 1.0) for the TSAW-Ave case controlled the original design of the frame. It is interesting to note that these checks seem to infer that the PR frames are stronger than the FR frame (ie. the AISC-LRFD force interaction equations are satisfied by a larger margin in the PR frames). Clearly, as seen from the data in Table 2b this is not the case since the FR frame consistently reached larger applied load ratio's at the limit point. This discrepancy demonstrates the type of inconsistency which can arise when the strength assessment is based on member by member design checks which do not fully account for overall system behavior and the inelastic force redistribution which will occur in the structure.

In Table 2c, the maximum connection rotations are summarized for the analyses presented above. The connection deformations at service loads were all less than 0.008 radians, and under the full factored loads were less than 0.013 radians. As noted previously, at the limit point some of the calculated rotations were considerably beyond the limits of reported experimental data. Hence, when interpreting the results of such analyses, a practical limit should be set on the realistic rotation capacity of the connection. Based on this example, large connection deformations seem to be of greater concern for the gravity load only combinations. The issue of limiting connection deformations is analogous to situations in plastic (inelastic) design, where the inelastic rotation demand should be checked against the rotation capacity mplied by member compactness requirements.

Table 2c - Maximum Connection Deformation

Criteria & Loading		θ (X 10^{-3} radians)		
		TSAW-Max	TSAW-Ave	TSAW-Min
Service	1.0DL + 1.0LL	7	8	8
	1.0DL + 0.2LL + 1.0WL	5	5	6
Full Factored	$1.2DL + 1.6L_f + 0.5L_r$	9	10	10
	$1.2DL + 0.5L_f + 1.6L_r$	12	13	13
	1.2DL + 0.5LL + 1.3WL	12	13	13
Limit Point	$1.2DL + 1.6L_f + 0.5L_r$	168[1]	177[1]	357[1]
	$1.2DL + 0.5L_f + 1.6L_r$	37	80[1]	191[1]
	1.2DL + 0.5LL + 1.3WL	39	42	47

Notes: 1 Calculated joint rotations in excess of 50 X 10^{-3} radians exceed the range of most of the experimental data.

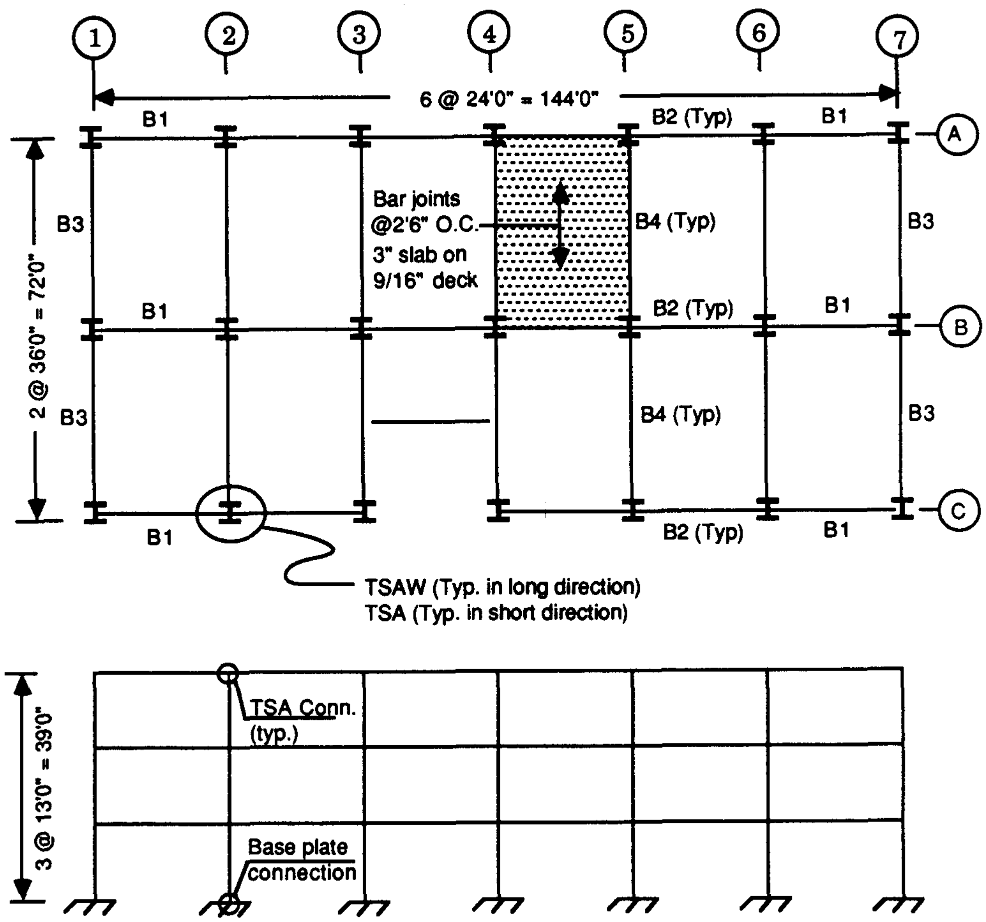
Three Dimensional Frame

The three story frame shown in Fig. 8 was designed assuming rigid connections based on the AISC-LRFD Specification provisions using a second-order analysis. Semi-rigid connections were then introduced into the structure and a comparison of the behavior is reported below. In this design, the floors were modelled as rigid diaphragms and the service loads were equal to the following:

DL: Roof 25 psf, Floor 75 psf
LL: Roof 27 psf, Floor 42 psf (these are reduced per UBC 88)
WL: 20 psf on projected area in each direction.

Note that with the framing system shown in Fig. 8, the bents in the short direction carry almost all of the gravity load. The sizes of beams B3 and B4 and the columns were governed by the factored gravity load combination. In the long direction, beams Bl and B2 were selected to limit the service load wind drift to approximately H/500.

Given the member sizes obtained from the FR frame analysis and design, the structure was then reanalyzed in a similar manner as the previous example. In addition to the FR frame, two PR frames with the same member sizes were investigated. In both PR frames, top- and seat-angle with web angle (TSAW) details were used to connect beams B3 and B4 to the strong axes of the columns and top and seat angles (TSA) were used to connect beams Bl and B2 to the weak axes of the columns (see Fig. 8). For bending about the strong axis of the beams, all semi-rigid connections were modeled as described above and for bending about the weak axis of the beams, (in the plane of the floor) the connections were assumed to be rigid. In addition, in the PR frames, flexibility at the foundation was included by modelling the fixed base plate connections as semi-rigid extended end plate connections. The nominal capacity of the beam-column connections in the two PR frames was: $M_{cn} = 0.25\ M_{pb}$ and $M_{cn} = 0.40\ M_{pb}$, respectively. In both PR frames, the base plate connection strength was kept constant with $M_{cn} = 0.75\ M_{pb}$.



MEMBER SIZES (All steel Is A-36)

Column	Story	Section	Beam	Story	Section
A.1.A.7, C.1.C.7	1,2 3	W14X34 x26	B1	1,2 3	W16X26
A.2-6.C.2-6	1,2 3	x61 X43	B2	1,2 3	" "
B.1.B.7	1,2 3	x38 x22	B3	1,2 3	W21x44 W16X26
B.2-6	1,2 3	X61 x30	B4	1,2 3	W24X68 W21x44

Figure 8 Three Dimensional Building Frame

The lateral drift under service loads was calculated for the FR and PR frames using a second-order analysis for wind parallel to the long and short directions of the building. The calculated drifts from these analyses are summarized in Table 3a. As in the previous example, the drift was considerably larger for the semi-rigid frames. Also, the large difference in drift between the two semi-rigid frames indicates how the behavior is strongly influenced by the nominal moment capacity of the connection (relative to the adjacent members). This is a different situation from the previous example where the connection *strength* was held constant while the *shape* of the moment-rotation curve was varied. For wind in the long direction, drifts for the PR frames with $M_{cn} = 0.40\ M_{pb}$ and $M_{cn} = 0.25\ M_{pb}$ were 1.26 and 1.50 times that of the FR frame, respectively. In the short direction, the respective ratios of the drift were 2.02 and 3.05. As noted above, in the original design for rigid connections, the beams in the long direction were sized to limit the wind drift to approximately H/500 = 0.94 inches. Therefore, it is not surprising that for the PR frames, drifts in the long direction were considerably in excess of H/500. On the other hand, in the short direction where gravity loads governed the original design, drifts for the PR frames were closer to the H/500 limit.

Table 3a - Roof Drift Under Service Loads

Loading	Roof Drift (inches)		
	Rigid (FR)	Semi-Rigid (PR)	
		$M_{cn} = 0.40M_{pb}$	$M_{cn} = 0.25M_{pb}$
1.0DL + 0.2LL + LOW-LONG	0.97	1.22	1.46
1.0DL + 0.2LL + LOW-SHORT	0.39	0.79	1.19
" (nonproportional)	0.39	--	0.92
1.0W -SHORT	0.35	--	0.64

The service load drift in the short direction was also calculated assuming non-proportional gravity and wind loading and for wind only loading. For non-proportional loading, the gravity loads were applied incrementally up to the full service load (1.0DL + 0.2LL), and then the wind load was applied incrementally. As indicated in Table 3a, the variation in loadings had a significant influence on the results for the PR frame with $M_{cn} = 0.25\ M_{pb}$ but not for the FR frame. For the non-proportional and wind only loadings, the drift in the PR frame was 0.77 and 0.54 times that of the proportional loading case, respectively. A comparison between the load-deformation response for the proportional and non-proportional loading is shown in Fig. 9 where the applied load ratio of 1.0 corresponds to the full load combination, 1.0DL + 0.2LL +1.0 WL. The response under non-proportional loading was more linear since once the gravity load was applied, the connection stiffness did not change much during subsequent wind loading. Intuitively, the non-proportional

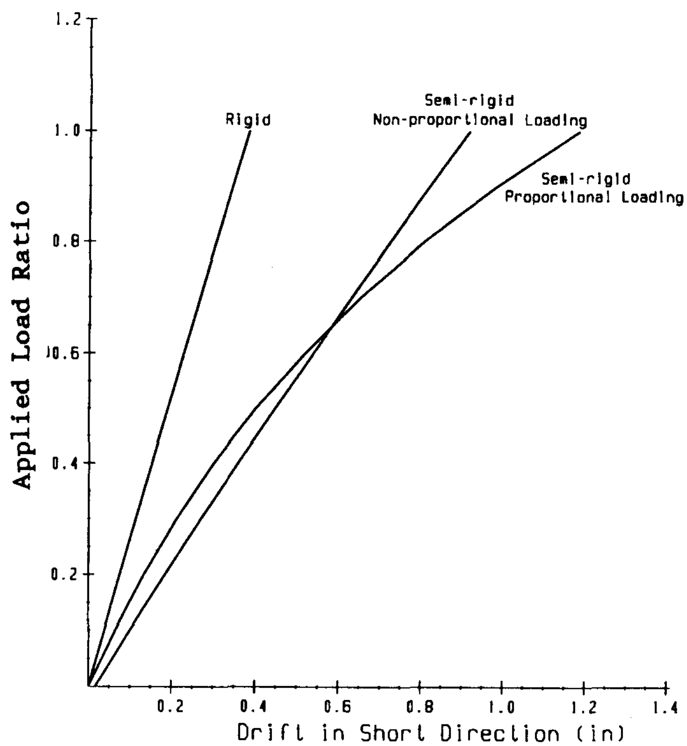


Figure 9. Service Load Drift of 3-D Frame.

loading seems more realistic. However, there are many issues related to construction sequence and cyclic load effects which are not considered in this analysis. Hence, at this stage it is premature to advocate non-proportional over proportional loading. In any case, the variation in response does suggest the need for further study regarding appropriate load combinations for serviceability checks and means for handling load sequence effects.

The strength limit state was investigated for the FR frame and the PR frame with $M_{cn} = 0.25\ M_{pb}$. The load-deformation response for wind in the long direction is shown in Fig. 10 where the applied load ratio of 1.0 corresponds to the full factored load, 1.2DL + 0.5LL + 1.3W-LONG. As in the previous example, the FR frame was both stiffer and reached a higher limit point than the PR frame. A summary of the analyses for additional load combinations is given in Table 3b. For the gravity load only combination and the combination with wind in the short direction (where gravity loads dominated) the first hinge occurred at roughly a 10% lower load in the FR versus the PR frame.

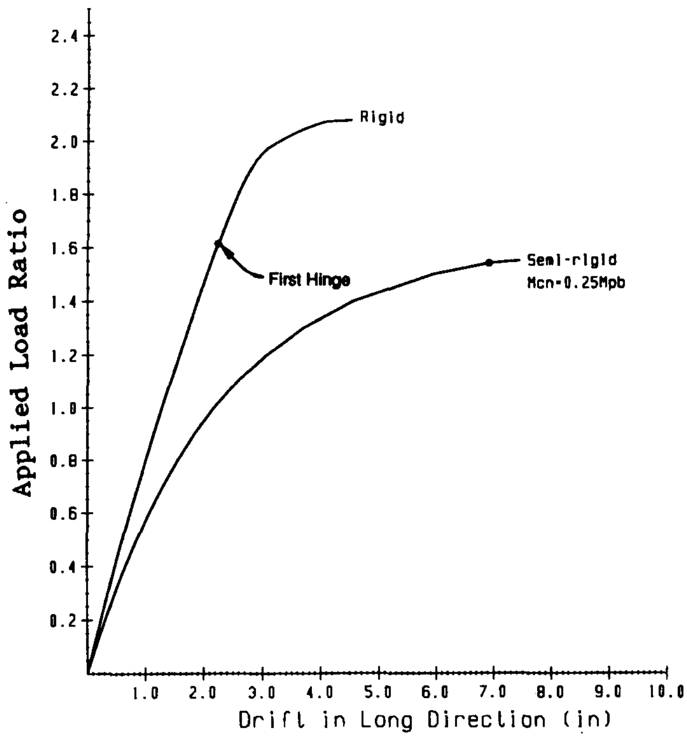


Figure 10. Inelastic Load-Deformation Response of 3-D Frame.

Table 3b - Applied Load Ratios for Factored Load Combinations

Criteria		Applied Load Ratio	
		Rigid	M_{cn}=0.25M_b
1st-Hinge	1.2DL + 1.6LL	1.19	1.28
	1.2DL + 0.5LL + 1.3W-LONG	1.62	1.54
	1.2DL + 0.5LL + 1.3W- SHORT	1.43	1.63
Limit Point	1.2DL + 1.6LL	1.78	1.79
	1.2DL + 0.5LL + 1.3W-LONG	2.08	1.55
	1.2DL + 0.5LL + 1.3W- SHORT	2.33	1.75

For the FR frame, the limit point of 1.78 for the gravity load combination governed the strength and was roughly equal to that of the PR frame for the same load combination. However, the controlling limit point for the PR frame was 1.55 which occurred under wind in the long direction. It is important to note that as in the previous example, although the inelastic limit point for the FR frame was greater, a strength evaluation based on the AISC-LRFD Specification at the full factored load would suggest the opposite. Based on this check, the maximum value for the governing LRFD interaction equations (H1-1a&b) was 0.98 ($\leq$ 1.0) for the FR frame and 0.84 ($\leq$ 1.0) for the PR frame.

Table 3c - Maximum Connection Deformation ($M_{cn} = 0.25M_{pb}$)

Loading		θ (X 10^{-3} radians)
Service	1.0DL + 1.0LL	4
	1.0DL + 0.2LL + 1.0W-LONG	4
	1.0DL + 0.2LL + 1.0W-SHORT	7
Full Factored	1.2DL+1.6LL	12
	1.2DL + 0.5LL + 1.3W-LONG	6
	1.2DL + 0.5LL + 1.3W-SHORT	11
Limit Point	1.2DL+1.6LL	50
	1.2DL + 0.5LL + 1.3W-LONG	20
	1.2DL + 0.5LL + 1.3W-SHORT	32

In this example, the limit points in the PR frames were reached without the connection rotations exceeding 0.05 radians. As shown in Table 3c, the maximum connection rotations for service, factored, and limit point loads were comparable to those in the previous example.

SUMMARY AND CONCLUSIONS

The main purpose of this paper has been to present and demonstrate a computer-aided system which includes connection flexibility in the analysis and design of framed structures. In addition, through two examples the sensitivity of the calculated structural response to several analysis parameters has been investigated.

The method for modeling the semi-rigid connections is based on standardized moment-rotation curves which are obtained using experimental data. For use in design, the curve selection is based on the type of connection. Beyond this, the nominal connection strength is all that is needed to scale the standard curve for a particular structure. In this research, the nominal connection strength is chosen as the moment resisted at a rotation of 0.02 radians. The examples have shown that variations in the shape of the standard connection curve have relatively little influence on the overall structural behavior. However, of greater significance than the precise shape of the moment-rotation curve is the nominal connection strength used in design. Also, in the cases studied, the maximum connection rotations were less than 0.008 radians and 0.013 radians at service and full factored loads. In some instances, the calculated rotations exceeded the limits of measured experimental data at the inelastic limit point for gravity loading.

Results from the two examples also indicate the differences which can arise due to the type of analysis/design procedure used. As indicated by the governing limit points calculated using second-order analyses, the governing strengths for the frames with rigid connections were 15% to 20% larger than for those with semi-rigid connections. The limit points occurred at roughly 1.7 to 1.8 times the full factored load for the FR frames and 1.4 to 1.6 times the full factored load for the PR frames. However, member design checks based on the LRFD Specification using a second-order analysis at the factored load indicated the opposite trend (i.e. that the frames with semi-rigid connections had a larger margin of resistance). This difference is due to the fact that the code based member design checks did not take account of the inelastic force redistribution in the FR frame. It was also shown that the analysis for the PR frames is dependent on the load sequence and the fraction of gravity load applied in combination with the wind load.

The work presented suggests the following areas of needed research and development:

1) Development of accurate models for calculating the nominal resistance (strength) of semi-rigid connections.
2) Investigation and development of guidelines for load combination and load sequence effects in evaluating the service and strength limit state response of frames with semi-rigid connections.
3) Further information is needed regarding the interaction of shear and moment forces on the behavior of semi-rigid connections.

While there is clearly a need for further research, current computer-aided technology can offer significant improvements over methods presently available to handle semi-rigid connections. Analysis and design systems such as that presented herein should make it easier and more convenient to design PR frames. The unavailability of such methods often results in engineers avoiding the design of PR frames where semi-rigid connections could be effectively utilized for providing lateral stability. Moreover, with the development of limit state design methods based on inelastic analysis such as presented by Ziemian et. al. (1990), it will become increasingly important to rationally address joint effects in frames, even frames which at present are considered to have nominally rigid connections.

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AVAILABILITY OF PROGRAMS

The program developed in this research, CU-STAND with semi-rigid connections, is available for distribution to educational institutions through Project SOCRATES at Cornell University. For further information, write to: Project SOCRATES, College of Engineering, Hollister Hall, Cornell University, Ithaca, NY 14853. The programs can also be made available to non-educational affiliates through special arrangement with the authors.

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