Kajima - CUREe Research Project

Innovative Techniques of Response Control

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1993.2

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USE OF SPECIAL CONNECTIONS TO REDUCE THE SEISMIC RESPONSE
OF STEEL STRUCTURES

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January 15, 1992 - January 14, 1993
PROJECT SUMMARY

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1. Introduction

The seismic response of a steel structure is heavily affected by the behavior of the connections between the different members. A semi-rigid frame is a steel frame that contains semi-rigid connections. These semi-rigid connections are expected to provide the inelastic yielding and ductility that a building will need during a major earthquake. In the past, semi-rigid frames have been used to provide lateral stiffness against wind-loads, but not against seismic loads in highly seismic areas.

This project investigates the use of semi-rigid-frame seismic loads. This final report explains the research done over the past year of a multi-year project. This past year had two independent but inter-related studies. The larger study was a parametric analysis of the influence of different connection parameters on the overall global response of a structure. The smaller study was a laboratory experiment to determine the behavior of a single feature of the semi-rigid frame. The feature chosen was the steel welded shear stud between the steel beam and the concrete slab.

2. Analytical Parametric Study

A parametric study was performed on four different steel frames using the DRAIN-2DX non-linear structural analysis program. One-story, two-story, four-story and twenty-four story buildings were studied. The study focused primarily on the four-story and twenty-four story buildings. Three different ground motions were used for a time-history analysis allowing different connection parameters to vary. The three ground motions were scaled to both an ultimate level earthquake and a service level earthquake. More than 100 non-linear time history dynamic analyses of the structures were performed and the results studied.
Most steel connections behave similarly. During loading, most steel connections initially exhibit a relatively linear elastic portion, reach a point of significant yield, then continue to rotate with some amount of strain-hardening, which allows it to carry larger loads. When the load is reduced, the connection begins to rotate in the opposite direction with much the same behavior.

Three parameters generally define the overall response of a semi-rigid steel connection. An elastic stiffness for the connection can be calculated based upon the point of significant yield. For this project, m is defined as this stiffness normalized by dividing by EI/L of the beam. Corresponding to the rotation at significant yield will be a flexural bending moment. This moment is normalized by dividing by the plastic moment capacity of the steel beam and given the symbol \( \alpha \). The strain-hardening stiffness is defined as a percentage of the initial elastic stiffness, and is assigned the symbol s. A bilinear representation of the connection's cyclical moment-rotation curve is defined by these three parameters.

Other parameters also influence the seismic response of a structure: the beam and column sizes, the story height, the bay lengths, the number and location of the semi-rigid connections, the mass supported by the frame and damping in the structure. These parameters were selected to represent typical values for commercial steel structures. However, the location of the semi-rigid connection was placed at a distance from the column face represent the column-tree style of construction that has been used extensively in Japan and has become more common in recent years in tall buildings in the United States. This distance normally ranges between 15 percent and 25 percent of the bay length.

This study supported several findings by earlier research done on seismic behavior and design of steel semi-rigid frames at the University of California at Berkeley by the first author and his research associates. The fundamental advantage of semi-rigid frames is the ability to control the frame's period of vibration, allowing the seismic acceleration of the structure's first mode to be controlled. This may result in a reduction in the overall base shear that a building must resist.

In general, the base shear is significantly reduced from the infinitely rigid case to the frames with lower values of rigidity of the connections, represented by the parameter m.

Excessive building drift is one of the major concerns some engineers have about semi-rigid frames, but this study showed that the overall roof drift increases only slightly as the connection stiffness is reduced to a certain point. However, after this point, drift begins to increase. This level of stiffness is considered the optimal behavior because it corresponds to a minimal base shear and an acceptable drift.

This study also showed that small changes in the strain-hardening ratio of the connection did not impact the overall response. However, as the strain-hardening ratio is
increased, more bending moment is developed in the beam-to-column connection. Recent research has questioned the reliability of some currently used welded beam-to-column connections, and it may be desirable to reduce the magnitude of moment developed in this joint.

Another characteristic of concern is the profile of the building deflection with different levels of connection stiffness. The variation in interstory drift from top to bottom is notable. In the 24-story structure that was studied, many of the upper stories had very small drifts while some stories had very high drifts. The maximum rotation and the total accumulated plastic rotation are both important factors in the connection's reliability. These values were distributed along the height of a building. In general, the more flexible connections had larger maximum rotation although they had lower accumulated plastic rotation. Flexible connections normally had fewer inelastic cycles.

3. Shear Stud Experiments

Shear studs are commonly used to create composite bending strength between a steel beam and the concrete slab. Limited information is available about the cyclic behavior of these studs. The two primary objectives for these tests were to determine the cyclic load versus deflection response and to explore possible methods of controlling this response. In the newly proposed semi-rigid frame system, floor beams are expected to experience localized curvatures near the connections. These sharp curvatures might result in undesirable behavior of the concrete slab. The need for knowledge about the cyclic behavior of shear studs is to allow for study and possible improvement of the floor beams to provide optimum use of composite behavior.

Four specimens were fabricated in the laboratory. All fabrication was performed to best match the actual conditions of a shear stud in a building. Specimens A and B were constructed to represent the standard welded shear stud currently used in Japan and the United States. Specimens C and D were designed to behave in a more ductile behavior. This was done by utilizing an innovative concept of "semi-rigid shear studs" developed by the first author. In this concept, by placing a small metal skirt around the base of the shear stud the concrete is prevented from completely bearing against the bottom portion of the studs. The skirt creates an air void allowing the stud to flex without crushing the concrete.

The first two specimens tested were the standard shear stud. It was found that this system is stiff but has little ductility. A longitudinal crack along the length of the specimen parallel to the application of force created the failure mode of tests A and B. This failure was designated a split tensile failure since it appears to be similar to the cracking of a concrete cylinder in a Split Tensile Test. To prevent this failure mode requires large amounts of reinforcing steel in the concrete, higher than is commonly used.
The next two tests were the semi-rigid shear stud specimens. In these tests, the maximum strength was controlled and was set at the desired level. The ductility that the system showed was much higher, the hysteresis loops were much more consistent and more cycles of deformation were tolerated by the shear studs.

4. Conclusions

Semi-rigid connections can be designed for a steel frame to control the base shear and roof drift during an earthquake. The maximum bending moments developed in the beams and columns can be controlled and limited. Fabrication and construction costs are saved by reducing the cost of the field connections and by reducing the size of the required superstructure as well as the foundation. The cyclic behavior of shear studs is now established and can be altered to create a more ductile behavior. The innovative semi-rigid shear stud showed high cyclic ductility and desirable behavior. Further integration of these studs into the semi-rigid steel frame needs to be considered in future research.

Before developing design procedures and implementing semi-rigid frames, more information is necessary about the cyclic behavior of connections. This project's future goal is to provide that information and to provide the engineering design procedures to implement the concepts to achieve an economical building system.

5. Acknowledgments

The cooperative research summarized herein is part of a joint CUREe/Kajima earthquake research project. The CUREe team consisted of Associate Professors Abolhassan Astaneh-Asl (Team Leader), Gregory Fenves and Mr. Kurt McMullin. The Kajima Team consisted of Dr. Eiji Fukuzawa (Team Leader), Mr. Mitsuo Sakamoto, Mr. Toshikazu Yamada, Mr. Seiichi Muramatsu, and Mr. Naoki Tanaka. The team leaders would like to thank Professor W. Iwan of CUREe and Dr. Akira Endoh of Kajima for their support.

6. References


SUMMARY REPORT

CUREe–Kajima Research Project
California Institute of Technology

Innovative Techniques of Response Control —
Optimum Use of Panel Zones in
Seismic Design of Tall Steel Frames

John F. Hall, Team Leader

SUMMARY

The aim of this research is to evaluate the role of panel zones in the seismic response of tall steel frame buildings using very strong ground motions capable of causing building collapse. The collapse mode considered is that of $P-\Delta$ instability from excessive lateral sway. One question which is addressed is: Can the excellent energy dissipation property of panel zones be exploited to improve seismic performance as a method of passive control? At the same time, an attempt is made to quantify the factor of safety of tall steel frame buildings, as currently design, against earthquake collapse. It is of considerable importance to develop some sense of what this "number" is.

A panel zone is the portion of the column at the beam-column junction. The primary mode of deformation of this zone is one of shear produced by the beam and column moments existing under lateral loads. Yielding of panel zones generally precedes hinging of beams and columns unless doubler plates are added to the column webs in order to strengthen them. The desirability of using doubler plates is still an open question because strong panel zones cause greater ductility demands on the beams and columns, and column hinging is especially detrimental to lateral stability. Frames whose panel zones have been designed to participate in the yielding should have superior performance.

An accurate hysteretic model for panel zone behavior has been developed which is based on test data from the U.S. and Japanese literature. Figure 1c shows the type of agreement that is attained; the plot is panel zone moment vs. shear. As seen, the ultimate strength of a panel zone considerably exceeds its yield level. This is less true for a beam, and so the ductility participation between panel zones and the beams, and the columns as well, is sensitive to the nonlinear models used for the various elements, especially the representation of strain hardening. Therefore, to be used in conjunction with the panel zone element, accurate beam-column elements of the "fiber" type have also been developed. In a fiber element, the member is divided into segments and a cross-section is divided into fibers. An accurate hysteretic model for axial stress-strain is used for
each fiber. Figures 1a and 1b show typical agreement with test data for axial tests on bars. Figure 2 presents a comparison with experimental data for a cantilever beam subjected to a cyclic shear force. The computations were done on a fiber beam element and the agreement is quite good except for the last few cycles when local flange buckling occurs in the experiment. The fiber element is quite versatile, and it is shown in the research that even the complicated buckling and post-buckling of a brace under cyclic loading can be accurately represented.

A computer program has been written for planar response of steel frames under gravity and earthquake loads. The frame model consists of nonlinear panel zone elements and fiber beam-column-brace elements. Finite spatial extents of the joints are represented, and geometric nonlinear effects are included through geometric stiffness and configuration updating. Thus, building collapse can be simulated. $P-\Delta$ effects from the interior part of a building supported by simply-connected frames is also included. Even though using fiber elements is generally regarded as computationally prohibitive, this program is efficient enough to be a practical analysis tool.

Some example results are presented here for a typical U.S. designed 20-story steel frame building (Figure 3). The ground motion employed is a record from the 1971 San Fernando earthquake which contains a significant long-period component (S90W from 8244 Orion Blvd. on the ground floor). The record was scaled amplitude-wise to a peak acceleration of 0.5 g. Design D3, which uses doubler plates to prevent panel zone yielding, has a fundamental period of 3.77 sec., and the pseudo-acceleration ordinate of the ground motion at this period is 0.40 g. Design D1, which has no doubler plates, has a fundamental period of 4.06 sec., and, for this case, the time axis of the ground motion is expanded by a factor of $\frac{4.06}{3.77}$ in order to shift the 0.40 g pseudo-acceleration ordinate to D1's fundamental period. Time history results are shown in Figure 4: horizontal displacements relative to the base of floors 2, 3, 4, 5 and roof. As can be seen, D3 collapses but D1 does not. The reason for this difference in behavior can be seen in the ductility plots of Figure 5 which show a greater amount of column hinging for design D3. The weaker panel zones in D1 cause smaller moments to be applied to the columns and, thereby, reduce the potential for column hinges to develop. In addition, D1's panel zones act as passive dampers by participating in the energy dissipation. These results indicate that allowing panel zones to yield is preferable to having strong, elastic panel zones with doubler plates. This conclusion is supported by other results contained in the research report.

Incidentally, the frame shown in Figure 3 is designed with column strength exceeding beam strength, but column hinging occurs anyway. Because of the wave propagation nature of the response up and down the building, column moments above and below a joint peak at different times, and this allows column hinging to occur.

An analysis was also carried out on a building model supplied by Kajima Corporation, the OJI Paper Company Building. Tube effects and shear-link energy dissipaters were ignored. This building proved to be exceedingly strong and underwent only a small amount of yielding when subjected to El Centro 1940 and Taft 1952 ground motions scaled to peak acceleration amplitudes of 0.67 g. Most of the yielding which did occur, however, was in the panel zones.
Figure 1. Comparison of experimentally determined hysteretic behavior to model predictions: a) and b) steel bar under axial stress, c) panel zone from a test on a beam-to-column subassemblage.

Figure 2. Lateral force-displacement of a cantilever beam (W18x46, 160 cm length): a) experiment, b) fiber element
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Figure 5. Maximum rotational ductility demands up to the times indicated in Figure 4 for designs D1 and D3. Dashed: column lines A and D. Solid: column lines B and C.
INNOVATIVE SEMI-RIGID STEEL FRAMES FOR CONTROL OF THE SEISMIC RESPONSE OF BUILDINGS

by

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Report to CUREe-Kajima

COLLEGE OF ENGINEERING
UNIVERSITY OF CALIFORNIA AT BERKELEY
INNOVATIVE SEMI-RIGID STEEL FRAMES FOR CONTROL
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Report to CUREe-Kajima

Report No. UCB/EERC-93/02
Earthquake Engineering Research Center
College of Engineering

University of California at Berkeley

February 1993
ABSTRACT

A study of a new type of seismic resistant semi-rigid frames was conducted. A parametric study of low rise and high rise frames was performed using non-linear inelastic dynamic time history computer analysis using the DRAIN-2DX program. Six ground motions were used to represent varied intensities and spectral accelerations. The frame geometry and member sizes were selected to represent commercial steel buildings constructed in U.S., Japan and elsewhere using the column-tree construction method. Semi-rigid connections were placed at strategic locations where column tree is attached to floor composite beams. The global behavior of the frames was controlled by changing the elastic stiffness coefficient of the semi-rigid connection. The important ductility parameters of semi-rigid connections were found to be the maximum rotation and the accumulated plastic rotation. The high rise frame required much lower connection ductilities.

Four laboratory experiments were performed to determine the behavior of steel shear studs which are typically used in composite floor beam construction. Standard shear studs were tested to obtain data defining their cyclic behavior. The Split-Tensile failure mode of composite slabs was identified for the first time. An innovative idea proposed by second author was tested to develop a more ductile and controllable composite beam behavior for the floors. This semi-rigid stud provided large ductilities and displacements with consistent hysteresis loops.

This study resulted in clearly establishing the potential of the proposed innovative semi-rigid system in controlling and reducing seismic response. The construction of the proposed semi-rigid system is easier and less costly than the similar rigid system while the forces and displacements are comparatively less than those in the rigid frame.

The future research on the subject should include finalizing properties of semi-rigid connections used in the system, testing a few actual size connections, conducting shaking table tests and developing design office procedures for these versatile systems.
ACKNOWLEDGEMENTS

The study described in this report was supported by the CUREe/Kajima research program. The financial support is appreciated along with the technical support provided by Dr. A. Endoh and E. Fukuzawa of the Kajima Corporation and Professor W.D. Iwan of CUREe. Any opinions, findings, and conclusions are solely those of the authors and do not necessarily reflect the views of the sponsors.

J. von der Lieth of TRW Nelson Stud Welding Division provided the shear studs and fabrication for the experimental testing. A. Kaufman of RMC Lonestar provided the concrete to build the test specimens. The assistance of the University of California staff, especially M. Troxler, B. Mac Cracken and M. Blondet was invaluable.
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EXECUTIVE SUMMARY

1. Background

During 1992 the CUREe/Kajima project sponsored a research project at the University of California at Berkeley to study the innovative use of steel semi-rigid frames to reduce the seismic response of tall buildings. The proposed study has been completed and the findings are presented in this report.

2. Introduction

Currently seismic design is based upon one major philosophy which is to expect some amount of yielding to occur in a structure. This yielding will require a certain amount of ductility so the building will not reach a collapse limit state during a major earthquake. Several new ideas have developed due to advances in computational and construction technologies, so buildings can withstand a major earthquake. One idea is the use of semi-rigid joints between the steel members that make up a structure.

The seismic response of a steel structure is heavily affected by the behavior of the connections between the different members. A semi-rigid frame is a steel frame that contains semi-rigid connections. These semi-rigid connections are expected to provide the inelastic yielding and the ductility that a building will need during a major earthquake. In the past semi-rigid frames have been used to provide lateral stiffness against wind loads, but not seismic.

This project investigates the use of semi-rigid frames for seismic loads. This paper explains the research done over the past year of a multi-year project. This past year had two
independent but inter-related studies. The larger study was a parametric analysis of the influence of different connection parameters on the overall global response of a structure. The smaller study was a laboratory experiment to determine the behavior of a single component of the semi-rigid frame. The component chosen was the steel welded shear stud between the steel beam and the concrete slab.

3. Analytical Parametric Study

A parametric study was performed on four different steel frames using the DRAIN-2DX non-linear structural analysis program. A one story, two story, four story and twenty-four story building were studied. The study primarily focused on the four and twenty-four story buildings based on recommendations from engineers at Kajima Corporation. Three different ground motions were used for a time-history analysis allowing different connection parameters to vary. The three ground motions were scaled to both an ultimate level earthquake and a service level earthquake. More than 100 non-linear time history dynamic analyses of the structures were performed and the results studied.

Most steel connections behave similarly. During loading, most steel connections initially exhibit a relatively linear elastic portion, reach a point of significant yield, then continue to rotate with some amount of strain-hardening, which allows it to carry larger loads. When the load is reduced, the connection begins to rotate in the opposite direction with much the same behavior.

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This study supported several findings by earlier research done at the University of California at Berkeley. The fundamental advantage of semi-rigid frames is the ability to control the frame's period of vibration, allowing the seismic acceleration of the structure's first mode to be controlled. This may result in a reduction in the overall base shear that a building must resist. In general, the base shear is significantly reduced from the infinitely rigid case to the frames with lower values of rigidity of the connections, represented by the parameter m.

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5 Conclusions

Semi-rigid connections can be designed for a steel frame to control the base shear and roof drift during an earthquake. The maximum bending moments developed in the beams and columns can be controlled and limited. Fabrication and construction costs are saved by reducing the cost of the field connections and by reducing the size of the required foundation. The cyclical behavior of a shear stud is now known and can be altered to create a more ductile behavior. The innovative semi-rigid shear stud showed high ductility. Further integration of these studs into the semi-rigid steel frame needs to be considered in future research.

Before developing design procedures and implementing semi-rigid frames more information is necessary about the reliability and cyclical behavior of connections. This project’s long-term goal is to provide that information and to provide the engineering design procedures to implement the concepts to achieve an economical building system.

6. Acknowledgements

The project was supported by the CUREe/Kajima project during 1992. The support of Dr. A. Endoh of Kajima Corporation and Professor W.D. Iwan of CUREe is appreciated. The technical involvement and input by E. Fukuzawa and other Kajima researchers were invaluable.
INTRODUCTION

Current seismic design of structures is based upon one major philosophy; some inelasticity is expected to occur in a structure during a major earthquake. This is accepted because the cost of constructing buildings to remain elastic throughout a major seismic event is prohibitively high. However, this allowance of inelastic behavior during an earthquake also requires a certain amount of ductility to prevent collapse. In the past design engineers have allowed different portions of the structure to develop inelastic behavior: loss of non-structural elements, yielding of steel beams or columns, or axial yielding of diagonal brace members.

In the past few decades engineers have increasingly looked at new ideas and technologies to dissipate energy created during an earthquake. Mechanical, friction, or other forms of energy-dissipating dampers have been included in building construction. Base isolation techniques have been used to reduce the total energy developed in a structure. The primary objectives of these new technologies have generally concentrated on three ideas.

First is the attempt to control the period of vibration of a structure. By adjusting the period of vibration, the threat of resonance between the ground motion frequency and the building's frequency can be reduced. The acceleration of a building during an earthquake is dependent upon the ground motion's spectral acceleration, which is determined by the base rock and local site effects. A seismic response of a structure can be reduced by controlling the structure's period in relation to the local effects. This can reduce the building acceleration, which will be accompanied by a decrease in the base shear of the structure.

Second is the attempt has been made to isolate portions of the structure from the ground motion. Normally this is done by placing an isolation device between the structure and the foundation. Isolation increases a structure's fundamental period of vibration and reduces the base shear. Unfortunately,
isolation has not been feasible for high rise buildings or buildings located on soft soil sites.

A third seismic resistant design technique is to place damping units in the structure. Many of the devices use friction, cyclic yielding of metals, or visco-elastic material. These damping devices are normally installed between diagonal bracing members.

A fourth technique recently being considered is the use of active control elements for a structure. Active control is provided by special structural elements which are activated by sensing the movement of a building during an earthquake. These elements introduce control forces to improve the building response. Maintenance and reliability are problems with active control and the technique has not yet seen general acceptance in the building industry.

1.1 Semi-Rigid Steel Frames

In the United States semi-rigid frames have been used for years for wind bracing of structures (Swenson 1992). Analysis strategies for semi-rigid frames have been well developed (Chen and Lui 1991). AISC has recognized the use of semi-rigid frames for several years. In allowable stress design semi-rigid connections are designated as Type III. Ultimate design methods have designated all non-rigid connections as partially rigid, with the designation of Type PR (AISC 1989, AISC 1986). Engineers have long recognized that most steel connections have semi-rigid behavior. The complexity of semi-rigid connections and the lack of computational resources has limited engineers in their use of inelastic non-linear behavior. Instead of quantitative analysis engineers have relied upon empirical formulas such as the bending and P-delta values listed in building codes (AISC 1989, AISC 1986). With today's computers and mathematical models these empirical procedures can be replaced and the complex behavior of a semi-rigid frame can be analyzed.

The proper use of semi-rigid frame behavior has many applications in today's construction industry. A new structure's seismic behavior can be altered to a more desirable form.
Structure periods, overall deflections and story accelerations can be controlled. Existing structures often require re-evaluation for their seismic response. Many older structures contain a steel frame intended to carry gravity load. This frame was generally constructed with connections that provide some semi-rigid characteristics. Considering the lateral strength of this semi-rigid frame may allow simpler upgrades for existing structures.

Also new construction techniques locate the connections in areas that do not receive the highest loadings. Column-tree construction is one such technique. A schematic drawing is shown in Fig. 1. This style of construction allows for shop fabrication of a steel column with stub beams attached to each face. This allows for shop assembly of the most critically loaded connections. The field connections are often built for full moment capacity and stiffness even though it is recognized that they will never experience such loads. By substituting semi-rigid connections a more efficient and cheaper structure can be built. Column-tree construction has been used successfully in Japan and in the eastern United States but is not used in the highly seismic areas of the United States.

1.1.1 Effects of Connection on Structural Response

The seismic response of a steel structure is affected by the behavior of the connections between the elements. Connections transfer service loads as well as provide passive inelastic energy dissipation during an earthquake. In this role the connection can dissipate energy or isolate portions of the structure. The ability to dissipate energy in the connections has been shown in recent studies of semi-rigid steel structures (Astaneh and Nader 1991, Guh et al 1991, Nader and Astaneh 1992, Astaneh et al 1991). Past research on steel connections under cyclical loading does not provide sufficient data to allow for reliable use in steel frames. Currently physical tests of prototype connections must be performed before application in an actual building. This lack of documented data on semi-rigid connections has limited their application in general

Limited testing is being performed by various researchers into cyclic testing of semi-rigid connections. A study at the University of California (Nader and Astaneh 1989) determined the cyclical behavior of double angle web connections. These double angle connections are often considered simple connections without the ability to transfer moment. But it has been shown that connections with several bolts can provide significant moment capacity.

Recent shaking table tests indicated the parameters influencing the hysteretic behavior of double angle connections (Nader and Astaneh 1992). The rotational stiffness of the angle, the bolt behavior and the gap formed as the angle pulled away from the column were all documented. This study also included a comparison between the actual testing results and the use of a non-linear analysis. The numerical results simulate the global response of the experimental but fail to replicate the precise load path. Research is needed to improve the correlation between the experimental results and the computer models.

Researchers at the University of Minnesota (Leon 1992) have tested the use of composite semi-rigid connections at the beam-to-column joint. These connections use typical steel beam to steel column simple connections but include reinforcement in the concrete slab to create a couple between the slab reinforcement and the beam web connection. This couple allows for bending moment to transfer into the column.

1.1.2 Design Philosophy for Semi-Rigid Frames

A detailed philosophy and procedure for the design of semi-rigid frames has been developed (Nader and Astaneh 1992). This section summarizes these major requirements. There are three primary limit states for a semi-rigid frame: collapse of structure, damageability, and serviceability. The collapse of structures during a major earthquake is unacceptable. Damageability refers to the amount of damage a building receives after a major earthquake. Serviceability refers to a structure's ability to withstand a moderate earthquake without significant
disruption of function. Semi-rigid connections may be damaged during a moderate earthquake. However, this seismic design philosophy limits the damage to evenly distributed yielding of steel rather than fracture of the bolts or welds. Such minor yielding damage can easily be tolerated. As a result, the replacement or repair of different connections will most likely not be required after a moderate earthquake. Although minor damage is acceptable in the connection, a semi-rigid frame design should not allow damage to occur in beams or columns after moderate earthquakes.

A semi-rigid connection must accommodate the demanded rotation during a major earthquake. Connections should be expected to reach a predetermined maximum rotation and should be able to undergo several cycles of large deformation. The connection's desired failure mechanism should be for ductile elements to provide the necessary inelastic rotation. These ductile elements would include plate yielding, bolt bearing or slippage, and in extreme cases weld yielding.

Although the use of semi-rigid frames for seismic loads is still being evaluated, these frames have often been used for wind loading of structures. Chen and Lui (1991) have provided a summary of several design philosophies that have been used for semi-rigid frames under wind loads. The simple framing method assumes a simple support for the design of the beams and girders and assumes fully rigid supports for the design of the columns. The semi-rigid framing method requires that the connection's moment capacity is known. Then the limit for the moment at the beam's supports and at the column are predetermined. This method has not been commonly used due to the lack of available data on connection strengths.

The Disque method was proposed for frames that allow for sidesway. It assumes that a connection maintains a constant moment capacity after yielding. The windward connection unloads elastically while the leeward connection loads up to its yield point and then maintains its load. The beams are designed as if they are simply supported and the columns are designed for inflection points at midheight. The effective length for the
The Liu method assumes that after initial loading the connection will follow the loading and unloading paths with a stiffness that is equal to the connection's initial stiffness. This method assumes two parameters to characterize the connection behavior: initial stiffness and ultimate capacity.

The Barakat and Chen method is based upon the LRFD method for steel frame design (AISC 1986). It requires both an initial and a strain-hardened stiffness for the connection. A first order analysis is performed using these connection stiffness values. An effective length for each column is found based upon the first order analysis and then the moment coefficients for gravity and lateral bending moments are adjusted for the connection's non-linearity. Using these coefficients a column design moment can be calculated.

Chen also indicates that the column stability of a semi-rigid frame must be considered. Interior columns will normally be restrained by one of the connections that has not yielded. In wind loading it is imperative that both connections in a span not yield as this will force the columns to carry the post-yielding wind load as cantilevers. During severe seismic loading both connections in a span are likely to yield. At this point the span no longer has significant additional strength. This creates an upper bound for the acceleration of the building. When both connections yield the restraint of the column to buckling may be lost. A similar situation arises for the exterior columns. These columns have only one connection that will receive loads up to its yield point during a major earthquake. Column-tree construction techniques should reduce some of these concerns about column stability because the semi-rigid connection is not located at the column. Further studies into the stability of the column after connection yielding are necessary.

1.2 Seismic Resistant Semi-Rigid Frame Research Project

Since 1987 several research projects have been completed or are currently underway at the University of California at Berkeley to study semi-rigid steel frames. The main goal of
these projects is to investigate the potential of using semi-rigid connections throughout a structure to control and reduce the global seismic demands. Secondary goals are to identify the parameters of interest, to improve the constructibility of the frames, and to formulate design criteria for future applications. In the past few years the projects have covered analytical studies of typical moment frames, shaking table tests of prototype semi-rigid frames, laboratory tests of semi-rigid steel connections, and developed preliminary guidelines for seismic resistant semi-rigid frames.

This report covers one of the projects sponsored by CUREe-Kajima. This project's goal is to create a design methodology to be used for the engineering design of seismic resistant semi-rigid frames. The research was divided into two independent areas of study.

One area of study was to analyze several building models to define the effect of different structure parameters. Chapter Two describes this analytical study. The second area of study was to experimentally measure the seismic response of a portion of the semi-rigid frame. This was done by building and testing prototypes in a laboratory and collecting the data for a mathematical formulation of the physical behavior. The component chosen was the welded steel stud which connects the steel beam to the concrete slab above. The cyclical behavior of the shear stud is required to determine if the concrete slab impacts the behavior of the steel frame. Traditionally any interaction between the concrete and steel has been ignored. This interaction may significantly affect the seismic behavior of semi-rigid frames. Chapter Three describes the laboratory testing. No attempt was made at this time to incorporate the data obtained from the laboratory tests to the modeling of the frames by computer simulation.
Chapter 2

NON-LINEAR ANALYSIS OF SEMI-RIGID FRAMES

A parametric study of the semi-rigid frame was performed using a non-linear computer analysis. Chen and Lui (1991) lists six types of analysis that can be performed on semi-rigid frames: elastic bifurcation, post-bifurcation behavior, small displacement second order, large displacement second order, second order elastic-plastic hinge, and complete elasto-plastic analysis. For this study a second order elasto-plastic analysis was performed with the DRAIN-2DX (Allahabadi 1987) computer program. This program performs a time-history analysis of non-linear, inelastic models of two-dimensional frames.

Preliminary studies were done to gain insight into the general behavior of semi-rigid frames and to identify important parameters. The first frame considered was a single bay with one story and the parameter studied was the location of the semi-rigid connection. This simple study showed that a frame's seismic bending moment envelope is best controlled if the semi-rigid connection is located between the beam's gravity loaded inflection point and the column. This connection should be near the inflection point to allow for less variation of the moment at the column face. A preliminary study was also performed on a four bay, two story frame. This frame was assumed to have rigid column bases and was excited by the El Centro 1940 ground motion. This frame was tested with various types of semi-rigid connection elements and allowed for selection of the ranges for the connection's parameters.

The main study of the computer analysis was for a four story and twenty-four story frame. These were selected to represent typical commercial low rise and high rise steel buildings.

2.1 Parameters of Study

Several parameters affect the behavior of a structure during a severe earthquake. Many are poorly defined, such as architectural elements, other non-structural components, site
effects, and variation in gravity loading patterns. The global response of structures, however, can be characterized by a few key parameters. The parameters considered in this study are ground motion records and the properties of semi-rigid connections.

2.1.1 Ground Motion Records

A primary parameter for computer analysis is the selection of relevant ground motions. Ground motion records from past earthquakes are the traditional method for simulation. Recently seismologists have been able to develop predicted ground motion records for future earthquakes (Astaneh et al 1993). This prediction is still at a limited state; for this project records from past earthquakes were used.

Much research has tried to quantify the characteristics of an earthquake's ground motion. These characteristics would allow for a set of ground motions to be selected that would cover a range of limit states that a structure would need to satisfy. An outstanding summary of the past research into these parameters has been done (Uang and Bertero 1988). They note that the most commonly used characteristic, the peak ground acceleration, does not correlate well with the damage from past earthquakes. Instead they state that energy intensity and duration of shaking are the most important characteristics. Another aspect of the overall problem is the inaccuracy of using linear ground motion response spectra to predict non-linear damage.

Recorded ground motions from recent earthquakes were used in performing the computer analysis. These earthquake records were selected after consultation with researchers at the Kajima Corporation. Three different earthquake ground motions were used at two levels of peak ground acceleration. This allowed six ground motions to be considered. The ground motion durations varied between 20 and 40 seconds.

The three ground motions used were: 1940 El Centro, 1952 Taft Lincoln School, and 1978 Miyagiken-Oki. The first two ground motions are from southern California and have been used for several years by researchers. El Centro is the first
recorded ground motion of a near source earthquake. Taft has been used because of the large range of frequencies produced by the earthquake. The third ground motion is from Japan and contains a large range of frequencies. This earthquake damaged steel buildings in Japan to a level that was unprecedented (Watabe 1979). These three ground motions cover a diverse earthquake sample. However, all three are recorded from firm soil sites. No attempt has been made in this study to consider the effects of soft soils on semi-rigid frames. It is planned that the future phase of this semi-rigid frame project will include analysis using ground motion records from various soil conditions.

The joint research team of the authors and the researchers from the Kajima Corporation decided that two levels of ground motion were considered in the parametric study. The first represented the shaking during an extreme earthquake which a structure may experience once in its lifetime. This ultimate limit state should not result in collapse of the structure. The second level of shaking is that of a moderate earthquake which can occur several times during the life of a structure. This service limit state should not impede the functionality of the building. Researchers at the Kajima Corporation provided values for these two levels. A peak ground velocity of 500 mm/second was used as the ultimate level of shaking. A peak ground velocity of 250 mm/second was chosen as the service level earthquake. The three recorded ground motions were scaled to these two levels of peak velocity. Table 1 lists the ground motions, their peak ground acceleration and velocity, and the scaling factors used to change the original motion to the desired scale.

The Japanese building industry developed its two level earthquake resistant design procedure based upon the peak velocities from the EL Centro and Taft earthquakes. The same peak velocity for the Miyagiken-Oki earthquake creates a significantly higher acceleration spectra. The impact of this difference may be seen when the behavior of the frames are compared.
2.1.2 Parameters of Semi-Rigid Connections

Beside the ground motion the other dominant parameters cover the connection behavior. All steel connections tend to show non-linear behavior even at low loads (Astaneh et al 1989). This is especially true when very flexible connections are used. To allow for convenient modeling in DRAIN-2DX however, a linear relationship must be used over a small range of loading. In traditional research the complex non-linear response of connections has been represented by a bi-linear response curve. This significantly reduces the amount of computation without reducing the reliability of the analytical results of the global behavior. For the use of a bi-linear connection a value must be selected for the moment at yielding, the initial stiffness, and the ratio of the strain-hardening stiffness to the initial stiffness. Fig. 2 shows a sketch of these parameters.

Nader and Astaneh (1992) show how to determine these parameters. A point of significant yield must be determined for a semi-rigid connection. This point refers to the moment at which a connection changes from one dominant stiffness value to a less stiff value. The yield point in steel connections is vague but can be estimated from the published values of laboratory tests of prototype connections. For the modeling and analysis this value is assigned as the yield point. It is generally agreed that the gravity loading of a beam should not exceed the yield point of the connection.

The first parameter is the level of bending moment at which the connection reaches its yield point. This value is divided by the plastic moment capacity of the beam and creates a ratio designated $\alpha$.

$$\alpha = \frac{M_{p\text{-cons}}}{M_{p\text{-beam}}} \quad (1)$$

A second major connection parameter defined by Nader and Astaneh (1992) is $m$, the relative stiffness of the connection to the beam. They compare the elastic stiffness of the connection to the $EI/L$ value for the beam.
In the study, the connections were located at the beam-to-column joint and the length is the distance between the connections. For a building using the column-tree construction method it is difficult to compare distances between connections. Therefore the distance between column centerlines was selected as the standard for semi-rigid steel frame design.

A third connection parameter is the ratio between the connection's initial stiffness and its stiffness after the point of significant yield. This value will differ greatly, depending upon the design of the connection. In this study, the ratio, s varied between 1% and 10%.

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    s = \frac{E_{\text{strain-hardening}}}{E_{\text{elastic}}}
\]

2.1.3 Location of Semi-Rigid Connections

Another parameter considered in this study is the location of the connection in the bay. In the United States, steel frames are traditionally built with field connections at the face of the column to support the beams. This places the field connection at a point of maximum loading. It is often expected that the field connection will be the weakest part of a system. Therefore the column-tree style of construction has become more popular, especially in Japan.

For this study the location is considered to be the ratio of the distance from column to connection divided by the column centerline distance. The ratio, e, varies from 0.1 to 0.3. This ratio can vary from almost 0.0 to 0.5, but the 0.1 to 0.3 range covers the practical limits. In a building it might be desirable to not use semi-rigid connections at all joints or story levels. For this study all beams were connected to the column-tree by semi-rigid connections.
2.2 Low Rise Building Models

Low rise buildings are the most common form of building structures in the world. They generally have floor plan dimensions that are greater than or about equal to the height of the building. These buildings have often been designed using semi-rigid frames for lateral wind loads in the eastern and central United States. For these reasons it was determined that low rise buildings should be an acceptable structure for the use of seismic resistant semi-rigid frames.

A four bay, four story frame shown in Fig. 3 was selected for the low rise study. This study allowed variation of the parameters for a total of 74 different cases. The range of initial stiffness, strain-hardening ratio and ground motion is listed in Table 2. The gravity load, mass, beam size and column size were the same for all cases.

The steel frame was designed according to the Uniform Building Code (UBC 1991). This frame design was based upon an assumption of a rigid connection at all joints. These beam and column sizes were used for the semi-rigid frame. In some cases it might be desirable to use different sizes of members for a semi-rigid frame than those that would be used in a rigid moment resisting frame.

Specially designed semi-rigid connections were used in the bays of each frame to create a semi-rigid frame behavior. Similar to column-tree construction, a rigid connection was assumed at the beam-to-column joint.

The amount of flexural bending moment that is applied to a beam-to-column joint can be controlled by the yield point of the semi-rigid connection. Determining the semi-rigid connection's moment capacity for the four story model was based upon limiting the flexural moment at the face of the column. A linear bending moment curve was assumed to develop between the point of inflection of the beam and the maximum negative moment at the column face. The inflection point moves during the lifetime of a beam, depending upon the lateral loading and the softening of the connection. However, to size the connection it was assumed this inflection point is located at 0.25 of the span. A limit was set
for the moment at the face of the column to reduce the amount of yielding at the beam-to-column joint. By using these limits a simple formula was derived to determine the moment capacity for each connection, as shown in Fig. 4. Initial stiffness and strain-hardening ratios were varied to allow for comparison between different cases.

2.3 High Rise Building Models

Although high rise buildings are less common than low rise there are inherent advantages to using semi-rigid frames in high rise structures. Normally high rise structures have more connections with larger connectors thus providing a large cost savings if the connection size can be reduced. Also high rise buildings tend to require damping to reduce lateral sway. This damping can be provided by semi-rigid connections.

The member size and geometry of the frame was obtained from a design provided by the Kajima Corporation (Fig. 5). This design of a twenty-four story frame will be used in an upcoming construction project in Japan. The Japanese design is a moment resisting frame with fully rigid joints at all locations. In consultation with Kajima Corp. it was decided that it would be beneficial to investigate how this frame would behave with semi-rigid connections. The column-tree style frame is designed with joints located at 25% of the span. The model of the frame incorporated semi-rigid connections for this analysis project. Similar to the four story frame this frame was analyzed for 37 different cases of connection parameters and ground motions.

As with low rise building models, a bi-linear connection was used to represent the semi-rigid behavior. However, for taller buildings wind also has influence on the behavior of the frame, and the sizing of the connection yield moment was based upon the 1991 Uniform Building Code's wind design criteria. By applying horizontal wind force the bending moment at the connection could be obtained using a linear analysis. The allowable moments obtained from the wind analysis were doubled to account for a factor of safety.
2.4 Discussion of Results

This study indicates that there are several advantages to using semi-rigid connections in steel frames. Many of these aspects have been noted in previous studies (Nader and Astaneh 1992, Shen and Astaneh 1990, Leon 1992, Chen and Lui 1991).

2.4.1 Shift of Building Period

The fundamental advantage of semi-rigid frames is the ability to control the natural periods of a structure. Fig. 6 shows the change of the model's fundamental period of vibration as the connection's initial stiffness is varied. As the connection becomes more flexible the period of the building increases. It should be noted that a building's initial elastic period is independent of the gravity load, the connection's post-yielding behavior or the expected ground motion. The period shift is due entirely to the reduction of the frame's stiffness by introducing flexibility at the semi-rigid connections.

All structures are susceptible to ground motion that contains a dominant period close to the building period, which would create a resonant system. By shifting the fundamental building period away from a ground motion's dominant period, less resonance is expected. One unfortunate aspect of this period shift is the moving of the second period of vibration close to the dominant ground motion period. This may create an undesirable effect due to the higher modes of vibration.

2.4.2 Reduction in Base Shear

By changing the connection's initial stiffness and lengthening the structure's period, the maximum base shear during an earthquake can be changed. Fig. 7 and 8 show this trend for the earthquakes in this study. A rigid connection creates a very high base shear. As the connection is made flexible, the building's base shear decreases. At a certain level of connection flexibility, the trend in the base shear becomes unpredictable. For the four story and the twenty-four story building this level occurs near the stiffness ratio of three. It does not seem to depend upon the number of stories; rather, it is
most heavily impacted by bay geometry. This reduction in base shear was measured experimentally on a shaking table (Nader and Astaneh 1992).

2.4.3 Change in Building Lateral Drift

A major concern of engineers about seismic resistant semi-rigid frames is that a building's drift will increase to unacceptable levels. This is a serious consideration since large drifts create large P-delta loads on columns, can break glass or other brittle facades, and create discomfort for people inside the building. This concern with large drifts is normally founded upon the static design method for seismic loading. Static lateral loads would cause frames with flexible connections to deflect more than frames with stiffer connections. However earthquakes do not create static horizontal forces. They cause accelerations of mass at each story level.

The response of a semi-rigid frame under seismic loading is complex. The energy a building attracts is a function of many variables, including structure stiffness, mass, damping, and ground motion. Semi-rigid frames exhibit different levels of stiffness and damping as they deflect laterally during an earthquake. Because of this complex behavior it is difficult to compare a large semi-rigid frame to a simple one-degree of freedom model implied in the equivalent lateral force procedure. This inability to visualize the behavior is one reason engineers are uncomfortable with seismic resistant semi-rigid frame design.

Fig. 9 and 10 show that there is little change in the roof drift as the connection's stiffness is reduced from infinity to a certain level. As the connection's stiffness is reduced farther the building's drift increases unacceptably. By looking at the combined effects of connection stiffness on base shear and lateral deflection, connections can be designed to minimize the base shear without significantly increasing the drift. It appears that a narrow range of connection stiffnesses allows for this optimal design.

In addition to the roof drift, interstory drift is also important. Fig. 11 shows the lateral deflection of the twenty-
four story frame for different values of connection flexibility. The plot is for a single ground motion, which generally produced the largest deflections. This graph results from the envelope of the maximum deflections of each story during the time-history analysis. The maximum interstory drift may be slightly larger because two neighboring stories may have a larger relative deflection than the difference of their largest deflections. It is generally assumed that most maximum story deflections will occur at nearly the same time, and that the maximum interstory drift will be near the drift calculated from the envelope of the maximum story deflections.

It is apparent that interstory drift changes significantly along the height of the frame. Therefore the building's roof drift should not be considered representative of the interstory drift. In this case, the lower floors should be designed for very large drifts to account for possible P-delta effects. The graph also shows that the deflection of the linear elastic rigid frame is substantially different than the non-linear plastic rigid frame. This is contrary to the assumption often used in design procedures.

2.4.4 Demand on Semi-Rigid Connection

Two criteria are generally used to determine the design requirement for connections. First is the maximum rotation that occurs during an earthquake. A minimum acceptable capacity of 0.030 radians is normally expected from a well detailed steel connection. Second is the accumulated plastic deformation in a connection during an earthquake. These values can be altered according to the connection's stiffness. For this project the connection's moment strength at significant yield is kept constant throughout all the cases. This allows a much higher rotation at yield point for a connection with a low initial stiffness.

Fig. 12 through 23 show how the rotation demands vary for different parameters. These values are the average of the rotations of the connections at each story level. All connections of each story were used for the four story building.
The averages for the twenty-four story building were calculated using only the connections from the four interior bays because the exterior bays had very small rotations. By excluding the small exterior bay rotations the figures show a more accurate value for the required rotation of the interior bay connections. The maximum rotation at a floor level varies along the height of the building. The twenty-four story building has higher connection rotation demands between the 15th and 23rd stories. The profile of the distribution is rather constant for all values of connection stiffness ratio. However the demand values are higher for the semi-rigid frames with more flexible connections.

Fig. 12 through 15 show the difference between the service level earthquake and the ultimate level earthquake. The required maximum rotation for the service level earthquake is half that for the ultimate earthquake. For stiff semi-rigid frames it is below 0.01 radians, an indication that there would be little permanent damage to the structure.

The difference in accumulated plastic connection rotation is more dramatic. These values are the sums of all cycles that went beyond the yield point of the connection. For the high rise frame almost all the accumulated plastic rotation occurs above the 15th floor. From the original building design the beam's stiffness and hence the semi-rigid connection's stiffness decreases significantly at the 15th floor. There is higher accumulated plastic rotation for the stiff semi-rigid frame than for the flexible one. The graphs show that the service level earthquake causes very little accumulated plastic rotation.

Fig. 24 and 25 show the moment versus rotation plot of a connection during the earthquake. The connection plotted is one of the heavier loaded connections and is plotted for the Taft earthquake, which caused some of the largest rotations. Connection 14 is on the second floor of the four story frame. Connection 110 is on the 15th floor of the twenty-four story frame. The more flexible connections have larger rotations, but generally have smaller accumulated plastic deformation. Stiffer connections will yield at a lower rotation, but will be expected to yield more times during an earthquake.
2.4.5 Effect of Strain-Hardening of Connection

Most connection parameters have significant effects on the global behavior of the building; however, the strain-hardening ratio has very little effect as shown in Fig. 26 through 30. The effect upon the building's base shear, roof drift, and connection demand is negligible.

The importance of the connection's strain-hardening ratio is in the possible yielding of the beam-to-column joint. Low values of strain-hardening limit how much moment can be applied to the joint. This joint is often seen as a possible weak link in a steel frame. It may be advantageous to reduce the amount of bending moment this joint will receive.

2.5 Future Research Goals

The global behavior of a seismic resistant semi-rigid frame has been investigated in this preliminary study. The primary parameters have been defined and the correlation between these parameters and the global behavior has been explored. It is now necessary to develop quantifiable limits to these parameters.

A more extensive data base of the cyclic behavior of connections is necessary. This information could be obtained from laboratory testing or from non-linear finite element modeling.

More research of actual structures shaken by real-time ground accelerations is needed. This research should include comparisons between the output from analysis of computer models and the recorded response of actual structures.
This research project also included an experimental study. This study was to collect data from a series of prototype tests to increase the knowledge about the behavior of semi-rigid connections. One area where little research has been done is the behavior of the welded shear stud, which connects a steel beam to the concrete floor slab above. These shear studs create a composite behavior between the beam and slab. This style of construction has typically been used for the beams which are not part of the lateral force system. In the past studs have been tested in a monotonic one-directional loading sequence until they fractured clear from the steel beam (Thurlimann 1959, Ollgaard et al 1971).

Information is needed about the cyclic behavior of shear studs to allow analytical modeling of semi-rigid frames. Shear studs are usually distributed along a steel beam to resist static gravity loads, which cause small curvatures along the length of the beam. However, there is interest in utilizing composite frame behavior to resist seismic loads. This requires the shear studs to resist cyclic loads during an earthquake.

In addition, semi-rigid connections allow rotations to occur in the connections of a steel frame. These rotations cause local areas of the composite beam to develop large curvatures. A concrete slab might not be able to develop large curvatures without cracking. The relative movement between a beam and a slab might reduce the curvature of the concrete slab. This experimental study is intended to provide information about these aspects of composite behavior. In addition, an attempt was made to use the semi-rigid shear stud between the concrete slab and the steel beam to improve the seismic behavior of steel semi-rigid frames.

For these reasons an experimental study was developed with two primary objectives. First was the desire to collect data that would establish the behavior of a standard shear stud due to
cyclic loading. This data should allow for the analytical modeling of a shear stud as it transfers force between the steel beam and the concrete slab. Second was to learn how this behavior might be altered to provide a more ductile behavior that is consistent and predictable.

Shear studs are normally welded to the top of the steel beam. Concrete is then poured to make the floor slab. Some stud manufacturers have developed installation techniques to simplify the stud placement. One such company in the United States is the TRW Nelson Stud Welding Division. It has a mechanical system that locates, holds, and Welds the stud into place. This method is common in the United States and was used for these tests. Most stud testing has used a concrete slab that is much thicker than the typical slab in a steel commercial building (Thurlimann 1959). For this test it was desired to simulate a commercial structure as closely as possible. Therefore the concrete was poured onto a steel deck and with a thickness similar to commercial construction.

The semi-rigid shear stud was developed and tested to improve the seismic behavior of a stud. This semi-rigid stud was fabricated by forming a small skirt around the base of the stud using 22 gauge sheet metal. This skirt was to keep concrete from being cast around the base of the stud. Fig. 31 is a drawing of the semi-rigid stud. Intuitively, a steel stud will crush the concrete around its base at very low loads. This crushing is difficult to quantify and tends to be non-ductile. Also, the stud tends to fail through the weld in a shear mode (Ollgaard et al 1971). The idea behind the semi-rigid stud is to allow the stud to move without excessively crushing the concrete. The stud will then tend to fail in a double curvature bending mode, which would be more ductile and more predictable.

3.1 Parameters of Study

Four full scale steel beam to concrete slab specimens were constructed and tested. Each test consisted of subjecting the specimen to an alternating axial load which would simulate the
dynamic action of a steel beam trying to move relative to the concrete slab.

3.2 Test Specimen Construction

Each specimen was constructed from a W12x40 steel beam sandwiched between two 125 mm (5 inch) concrete slabs of 27.6 MPa (4000 psi) concrete. The steel beam was selected to be large enough to remain elastic throughout the loading sequence. This concept of testing shear studs has been used for many years (Thurlimann 1959). Three studs were welded to each flange of the steel beam to connect the concrete slab to the steel beam. The steel deck was used as a form to hold the concrete.

To reduce the factors contributing to the shear stud behavior, the deck was not welded to the steel beam and no reinforcing steel was placed in the slab. Although these two items are generally done in actual field construction, it is believed that both make the system stronger and more ductile but also more complex. For this limited study it was desired to minimize the factors and so the two items were removed. All specimens were fabricated in the laboratory.

All studs were 100 mm long, 75 mm diameter Nelson studs with yield strength of 444 MPa (64.4 ksi) and ultimate strength of 489 MPa (70.9 ksi). They were welded to the beams using TRW Nelson's proprietary welding system. Test specimens A and B used the stud as it would typically be installed. Test specimens C and D used a new semi-rigid stud construction technique.

Another change from the traditional testing method was to cast all specimens in a manner similar to actual construction. The steel beam was ripped down its length, placed on the floor with the deck on top, and concrete was placed and allowed to cure. Two half specimens were welded together at the beam web to create a symmetric cross section. In some previous research, shear stud specimens were cast sideways, allowing concrete to be poured with the stud in a horizontal position. In actual construction the concrete is almost always poured with the stud standing vertically. It was believed that the casting procedure
used in this project would be more accurate as it would simulate any formation of voids below the head of the shear stud. Since this area below the head is used in creating axial force in the stud it is critical to the overall behavior of the system.

3.3 Test Set-up

The test set-up is shown in Fig. 32. As with previous testing methods a hydraulic actuator was used to apply an axial force to the steel beam. The concrete slabs were placed so that they would react against a fixed base. Unlike previous monotonic tests, this laboratory set-up was required to resist the concrete's movement for either direction of the actuator's loading. When the actuator is retracting, a series of supporting steel angles and post-tensioning bars were used in tension to carry the force from the concrete to the reaction block. It is apparent that the stiffness of the test set-up changes whether the actuator is in extension or in retraction. When in retraction the axial elongation of the steel rods reduces the stiffness of the set-up. Therefore, when using this testing method it is important to record the relative movement between the steel and concrete rather than the overall movement of the actuator. Because of the symmetry of the test specimen, no attempt was made to control out of plane movement. Comparing the measured rotations of the specimen this control was not needed.

Since most steel beams are used to carry gravity loads it is expected that some level of compression in the concrete slab will exist before the cyclical loading of an earthquake occurs. To test the effect due to this compression the pretensioning force on the rods was varied. By applying a pretensioning force the concrete slab is compressed. The rods were tensioned to create a compression of around 700 kPa and 2800 kPa. This level of stress represents the two cases where the shear stud is near the inflection point or near the point of maximum positive bending moment of the beam.

After performing test A it was found necessary to simulate the passive resistance on the edges of the slab parallel to the loading direction. In an actual structure this passive
resistance would be provided by the continuation of the concrete slab. In the laboratory this resistance was provided by placing steel plate along the edge of the slab, steel tubes across these plates and steel threaded rod between the steel tubes on opposite slab edges. These rods were not pretensioned but were installed in a snug position.

3.4 Loading History

The actuator was alternately extended and retracted to create a cyclic displacement between the slab and beam. The actuator was controlled by the relative displacement between the slab and the beam. Cyclic loading was increased by steps of displacement and each level of displacement was used for two cycles. Test A provided information about the displacement required for yielding of the standard shear stud. After performing this test the loading sequence for the remaining specimens was developed.

Often when cyclical testing is performed, a loading sequence is used where the cycles are increased to the point of failure. This basic concept was used for this test but with a slight modification. During an earthquake a member will be loaded and displaced in a random manner. To simulate correctly a shear stud's behavior some information is required as to how partial loops in a cycle perform. It is sometimes presumed that a partial loop will unload linearly and then load linearly until it reaches the cyclic load path which would be obtained from a complete loop. Testing of this hypothesis was desired.

3.5 Instrumentation

The specimens were instrumented according to Fig. 33. The primary information to be obtained was the applied force, the relative displacement of the steel beam to the concrete slab, and the strain developed in the shear studs.

A load cell measured the actuator force. Linear Variable Displacement Transducers (LVDT) measured relative movements and rotations, and a Wire Transducer measured any movement of the reaction block. Strain gauges installed on some of the studs
measured axial strain during the test. These strain gauges were placed 38 mm (1.5 inch) above the steel flange and were on opposite sides of the stud to allow for a comparison of the studs axial and flexural strain. To record data and notes, the studs were numbered from one to six. Stud one was the strain-gauged stud next to the reaction block. Stud two was the strain-gauged stud in the middle of the slab. Stud three was the third stud contained in the same slab as one and two. Studs four, five and six were in the slab on the other side of the specimen.

3.6 Test Procedure

Following is a step-by-step listing of the testing procedure:
1. The steel beam was flame cut along its length. Each half was placed horizontally and studs were welded to the flanges and concrete was placed in the forms.
2. After the concrete cured, two half specimens were connected by welding the web back together. This formed a symmetric specimen which was installed in the test set-up.
3. Instrumentation was installed and a data acquisition system was connected.
4. The actuator was extended to reach the desired relative displacement between the slab and beam.
5. The actuator was retracted to reach the desired relative displacement in the opposite direction.
6. Force was reapplied to repeat the cycle defined by steps 4 and 5.
7. The desired displacement was increased and a dual cycle was performed at each level of relative displacement.
8. At an appropriate point, partial loops of displacement were made to measure the specimens response to erratic loading.
9. Notes and photographs were taken throughout the test to document the testing.
10. Graphs were made to record the data.
3.7 Discussion of Results
Several aspects of shear stud cyclic behavior were discovered during the tests. These are discussed in the following sections.

3.7.1 Load Deflection Curves
The most important information received from the test specimens are the graphs of the cyclic behavior of the load versus the deflection as shown in Fig. 34 through 37. Tests A and B were done using the standard stud construction. The maximum values of shear obtained were nearly 582 kN (130 kips) or 97 kN/stud (22 kips/stud). The loading was stiff and the loops were relatively stable. One important aspect was the dissimilar behavior between the first cycle and the second cycle for a given level of displacement. For the intended displacement the first cycle required a higher force than the second cycle. For the next level of displacement the loading would trace the previous cycle and then continue to a load for the increased deflection. This non-repetitive behavior occurred for all four specimens and indicates that stiffness degradation took place in each specimen.

The maximum force reached tended to be constant once significant yielding had occurred in the system. The force required at zero displacement steadily increased for each cycle throughout the test. This indicates the studs are strain-hardening and requiring more force to deflect.

However, the behavior of the semi-rigid stud tests was different than the standard stud specimens. Tests C and D contained semi-rigid studs where the base of the stud was protected by a skirt, allowing an air void to form around the stem of the stud. These specimens could not develop the loads seen in the standard stud tests. Maximum loads around 224 kN (50 kips) or 37 kN/stud (8.5 kip/stud) were developed before failure. The studs deformed in a much more ductile manner and were able to develop very large displacements. In these tests the studs were actually sheared clear of the steel beam. The testing cycles were more stable with a long plateau of constant maximum force.

In Test C an erratic behavior was seen when displacements of
+5 mm were obtained. Stud fracture was suspected to be the cause of this sudden drop in load. It should be noted that after this occurred the load in the reverse direction was still maintained. Possibly the studs fractured halfway through with little resistance in one direction, but still maintaining their strength when bent the other direction. Test D did not show a sudden drop in force but instead a gradual decrease in maximum load at about the same displacement. Also, Test D had an increase in maximum load at a displacement of 10 mm. This increase in load after an initial drop from the maximum load was unexpected since load deflection graphs normally do not show local minima after reaching the maximum load. The reasons for this are under investigation.

3.7.2 Split Tensile Failure

Specimens A and B both failed by cracking the slab longitudinally, parallel to the direction of the applied force. This is similar to the tension failure of a concrete cylinder during a Split-Tensile test and so this failure mode is designated as a split tensile failure. During the testing the slab is placed in compression by the studs. Due to compression the concrete will develop tensile stresses in the direction perpendicular to the loading. Apparently the tensile stresses became higher than the tensile strength of the concrete and so cracking occurred.

Unlike the test specimens, which did not have shrinkage reinforcement, a floor slab would typically have small amounts of reinforcing steel to control shrinkage cracking, carry negative moments over the beam, or increase the diaphragm shear strength of the slab. This reinforcement ratio varies between 0.0015 and 0.0025 for typical commercial construction. It should be expected that much of this steel will be stressed, possibly up to yield, by gravity loading and concrete curing. If the tensile strength of this steel is compared to the force required to crack the concrete specimen it can be seen that this minimal reinforcement will not stop the split tensile failure mode.

In a typical slab the concrete should be restrained by
continuation of the concrete slab. This should induce a plane strain effect and create compressive stresses rather than tensile stresses perpendicular to the direction of the primary compression. This was simulated by placing the exterior steel restraint around the slab. But comparing the results of Test A and Test B it appears that there is no appreciable difference. In previous studies (Ollgaard et al 1971, Thurlimann 1959) the slabs were much thicker and the reinforcement was extremely high (reinforcement ratios of 0.0148 and 0.0119). This raises a question of the reliability of current composite beams to work adequately as they approach their plastic capacity.

There are possible solutions. First is that the continuation of concrete does indeed provide sufficient restraint to remove tensile stresses. This would still create problems at spandrel beams where the slab is not continuous. Second is the possibility that the steel deck will carry tensile stress at the beam. This deck is normally continuous over most beams and is not in tension at the support. Still this solution is not satisfactory at beams where the deck is not continuous. Third is that the maximum compression and hence the maximum tensile stress occurs only in the midspan region of the beam and that the tensile strength over the entire bay length is sufficient.

This question should be solved before concrete slabs are expected to work compositely to resist seismic loading. Strong earthquakes will load a structure well into the inelastic range and if a member lacks the necessary overstrength and ductility a brittle failure will occur, possibly leading to severe damage or collapse of the structure.

3.7.3 Strain Behavior in Stud

A few studs were instrumented with strain gauges before they were cast in concrete to develop a qualitative understanding of how the stud is stressed as it is cycled back and forth. Strain gauges were placed a distance above the steel flange and so were not expected to read the highest strain in the stud, which should occur at the face of the steel beam. The gauges were mounted so a pair would read the axial and flexural strain in a stud. The
results of these strain gauges shows complex behavior, but a sense can be obtained from their readings.

Fig. 38 through 45 show some of the data collected by the strain gauges. Fig. 38 shows the cyclic readings of a strain gauge for Test B, a standard construction stud. At low loads the gauge picks up strain quickly, then as the load increases the strain begins to decrease and eventually changes sign. This appears to indicate a stud that is behaving in flexure initially, but then begins to carry load axially as it reaches its ultimate strength. In Fig. 40 the axial strain is plotted. It appears that the stud develops very small axial force at low loads and begins to carry more axial force as the load increases.

Fig. 42 is the cyclic graph for Test D. It was not possible to find correlations in this graph, but Fig. 43 shows the time history of the strain readings. At the latter cycles it appears that the strain gauge is recording a permanent offset strain in the stud. This would indicate an inelastic strain producing a permanent axial load in the stud. This is consistent with the expected behavior because kinematic hardening in the stud begins to develop catenary axial force after originally yielding in flexure.

3.7.4 Partial Loops for Load Deflection Curves

Fig. 46 compares the loading cycles that were performed for different specimens at later stages of the testing. The results were surprising and refute the hypothesis of gradual transition from one load cycle to the other. Instead the load dropped linearly to a near zero position. Then as the load was increased in the opposite direction the loading path followed a graph splitting the previous hysteresis cycle in half.

3.7.5 Effect of Gravity Load in Concrete

Comparison of Fig. 34 through 37 show similar results when comparing the change in the compressive stress in the concrete due to the post-tensioning bar tension. Tests B and D, which had higher compressive stress, appeared to have more stable hysteresis loops. The post-tensioning might create better
confinement of the concrete around the stud. These tests do not indicate a significant difference and suggest that the compressive stress in the concrete due to gravity loads may be inconsequential.

3.8 Effects of Shear Stud on Connection Behavior

All of the computer studies were performed with a symmetric connection behavior. Most steel frames are covered with concrete slabs which might cause the connection to behave asymmetrically. This project's experimental study showed that a stud's behavior could be altered significantly. It might be possible to alter the influence of a concrete deck by reducing the shear stud connection locally around the semi-rigid connection.

3.9 Future Research Goals

Many aspects of the cyclic load versus deflection behavior for standard steel shear studs is now known. It is now necessary to determine the demands upon the shear stud. Also a better understanding of the split tensile failure mode is necessary.

For semi-rigid connections it is desirable to allow the connection to rotate without interference from the concrete slab. Design guidelines need to be developed to allow for correct detailing of the semi-rigid stud. A more extensive test allowing for all factors of the semi-rigid connection needs to be performed.
Chapter 4

RECOMMENDATIONS AND CONCLUSIONS

The study demonstrated several important aspects of the seismic behavior of the innovative semi-rigid, column-tree structures. It was shown that semi-rigid connections can be placed in a steel frame to control the period of vibration of a structure and the base shear of the building, reducing the forces in the structure. This decrease in base shear can normally be achieved without allowing unacceptably large roof drifts. Strain-hardening of the connection after reaching the point of "significant yielding" does not affect the global building response but can be utilized to control and limit the bending moment applied to the beam-to-column connection and the column as well.

Column-tree construction is easily adapted to semi-rigid frame design. The location of the connection can be placed in a position which will optimize the global building response.

Semi-rigid connections can be modeled using three parameters, the moment at significant yield, the initial stiffness and the strain-hardening stiffness. Maximum connection rotation and the accumulated plastic connection rotation are two primary demand parameters to be used for the connection detailing. These demand parameters varied according to ground motion, connection initial stiffness, and the building floor which contains the connection. High rise structures showed more potential for this innovative semi-rigid system because the maximum rotations and accumulated cyclic rotations of the connections were lower for high-rise than for low rise structures.

One component of the semi-rigid frame is the welded steel shear stud which creates composite action between the floor concrete slab and the steel beam. The cyclic load behavior for these studs was obtained by experimental testing. The shape of the hysteretic cycles can be manipulated by using a "semi-rigid" stud. This semi-rigid stud is constructed by placing an air void around the base of the stud. The semi-rigid stud showed very ductile and desirable behavior. In addition, by using proposed
innovative "semi-rigid" shear studs, one can efficiently control the stiffness, strength and damping of composite floors.

During the experimental testing a new failure mode of composite beams was found. The split tensile failure develops due to the transverse tensile stresses in a uni-axially compressed concrete slab. This split tensile failure occurred after the shear studs yielded but before they reached their ultimate strength. Additional reinforcing steel might be required to control this failure mode.

4.1 Future Research Needs

The global seismic behavior of an innovative steel semi-rigid frame has been investigated herein. The primary parameters have been defined and the correlation between these parameters and the global seismic behavior has been explored. The study included 4-story and 24-story structures. It is now necessary to develop quantifiable limits to these parameters for a variety of structures with other configurations. Since higher modes appear to be participating more in semi-rigid systems than rigid systems, the effects of soil-structure interaction on the behavior of these semi-rigid systems need to be studied.

A more extensive database of the cyclic behavior of connections is necessary. This information could be obtained from laboratory testing or from non-linear finite element modeling.

More research of actual structures using the proposed semi-rigid system shaken by base accelerations is needed. This research should include comparisons between the output from analysis of computer models and the recorded response of actual structures.

Many aspects of the cyclic load versus deflection behavior for standard steel shear studs is now established by this study. It is now necessary to determine the demands upon the shear stud. Also a better understanding of the split tensile failure mode is necessary.

By conducting above studies, it will be possible to formulate design procedures for actual seismic design of these innovative and efficient steel semi-rigid systems. The final outcome of the project will be code-formatted procedures for seismic design.
REFERENCES


Manual of Steel Construction - LRFD, (1986) American Institute of
Steel Construction (AISC), Chicago.


### Table 1 - Earthquake Ground Motions

<table>
<thead>
<tr>
<th>Ground Motion</th>
<th>Peak Ground Acceleration mm/sec²</th>
<th>Peak Ground Velocity mm/sec</th>
<th>Scale Factor</th>
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<td>3417</td>
<td>334</td>
<td>1.50</td>
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<tr>
<td>Miyagiken-Oki, MO June 12, 1978</td>
<td>2590</td>
<td>358</td>
<td>1.40</td>
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<tr>
<td>Taft School, TF July 21, 1952</td>
<td>1759</td>
<td>177</td>
<td>2.82</td>
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### Table 2 - Model Parameters

<table>
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<tr>
<th>Model</th>
<th>Connection Yield Moment</th>
<th>Conn. Location Span</th>
<th>$E_{\text{strain-hardening}} / E_{\text{elastic}}$</th>
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<td>4 Story</td>
<td>based upon a percentage of the beam’s plastic moment</td>
<td>0.20</td>
<td>0.01, 0.03, 0.05, 0.08, 0.10</td>
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<td></td>
<td>$m = \frac{k_{\text{elastic conn.}}}{(EI/L)_{\text{beam}}}$</td>
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<td></td>
</tr>
<tr>
<td>24 Story</td>
<td>double the moment required for UBC Wind requirements</td>
<td>0.25</td>
<td>0.01, 0.10</td>
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<tr>
<td></td>
<td>$1.0, 3.0, 5.0$</td>
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<td></td>
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</table>
Figure 1  Column-Tree Method of Construction

Figure 2  Parameters of Semi-rigid Connection
Figure 3  Four Story Low Rise Building Model

Column Centerline

\[ \alpha = \frac{M_{p\text{-conn}}}{M_{p\text{-beam}}} = \frac{0.25 L - e}{0.25 L} \frac{0.8 M_{p\text{-beam}}}{M_{p\text{-beam}}} \]

Figure 4  Moment Capacity of Connection for Four Story Frame
Figure 5  Twenty-Four Story High Rise Building Model
Figure 6  Fundamental Building Period vs. Connection Stiffness
Figure 7 Base Shear vs Connection Stiffness - 4 Story Frame
24 STORY BUILDING, $s=10\%$

Figure 8 Base Shear vs Connection Stiffness - 24 Story Frame
Figure 9  Roof Drift vs Connection Stiffness - 4 Story Frame
Figure 10 Roof Drift vs Connection Stiffness - 24 Story Frame
Figure 11  Lateral Deflection due to Taft Ultimate
- 24 Story Frame
Figure 12: Effect of Ground Motion on Maximum Connection Rotation

- m = 3.0, s = 10%
Figure 13 Effect of Ground Motion on Accumulated Plastic Rotation - 4 Story Frame
Figure 14 Effect of Ground Motion on Maximum Connection Rotation - 24 Story Frame
Figure 15 Effect of Ground Motion on Accumulated Plastic Rotation
- 24 Story Frame

AVERAGE PLASTIC ROTATION
m=3.0, s=10%
Figure 16 Effect of Connection Stiffness on Maximum Connection Rotation - 4 Story Frame - Ultimate Level Earthquake
Figure 17 Effect of Connection Stiffness on Accumulated Plastic Rotation - 4 Story Frame - Ultimate Level Earthquake
Figure 18 Effect of Connection Stiffness on Maximum Connection Rotation - 4 Story Frame - Service Level Earthquake
Figure 19  Effect of Connection Stiffness on Accumulated Plastic Rotation - 4 Story Frame
- Service Level Earthquake
Figure 20  Effect of Connection Stiffness on Maximum Connection Rotation - 24 Story Frame
- Ultimate Level Earthquake
Figure 21: Effect of Connection Stiffness on Accumulated Plastic Rotation - 24 Story Frame - Ultimate Level Earthquake
Figure 22  Effect of Connection Stiffness on Maximum Connection Rotation - 24 Story Frame - Service Level Earthquake
Figure 23  Effect of Connection Stiffness on Accumulated Plastic Rotation - 24 Story Frame - Service Level Earthquake
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Figure 25 Bending Moment vs Rotation for Connection 110 in 24 Story Frame - Taft Earthquake
Figure 26  Base Shear and Roof Drift vs. Connection Strain-Hardening - 4 Story Frame
Figure 27  Effect of Strain-Hardening Ratio on Maximum Connection Rotation - 4 Story Frame - Ultimate Level Earthquake
Figure 28 Effect of Strain-Hardening Ratio on Accumulated Plastic Rotation - 4 Story Frame - Ultimate Level Earthquake
Figure 29 Effect of Strain-Hardening Ratio on Maximum Connection Rotation - 24 Story Frame - Ultimate Level Earthquake
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Figure 32 Test Set-up for Shear Stud Experiments
Figure 33 Instrumentation for Shear Stud Experiments
Figure 34 Load vs. Displacement - Test A
Figure 35  Load vs. Displacement - Test B
Figure 36 Load vs. Displacement - Test C
Figure 37 Load vs. Displacement - Test D
Figure 38 Load vs. Strain at Gauge 2A - Test B
Figure 39  Time History of Strain Gauge 2A - Test B
Figure 40  Axial Strain in Stud 2 - Test B
Figure 41 Flexural Strain in Stud 2 - Test B
Figure 42 Load vs. Strain at Gauge 2A - Test D
Figure 43  Time History of Strain Gauge 2A - Test D
Figure 44 Axial Strain in Stud 2 - Test D
Figure 45  Flexural Strain in Stud 2 - Test D
Figure 46 Load vs. Displacement for Partial Loading Cycles
APPENDIX A

Notes from Experiments
LAB NOTES FOR TEST A  
TESTED ON OCTOBER 28, 1992  
KURT MCMULLIN

This specimen had Standard shear studs, with 54 psi compression in the slab before testing.

MAXIMUM FORCE DURING TEST = 128.70 kips

<table>
<thead>
<tr>
<th>DATA POINT</th>
<th>LOAD (kips)</th>
<th>DISPLACEMENT (inches)</th>
<th>NOTES</th>
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<td>716</td>
<td>-34.91</td>
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<td>719</td>
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<td>53.33</td>
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<td>Cycle 4 specimen making sounds like concrete crushing</td>
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<td>Cycle 5 noise like that of steel popping</td>
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<td>2796  119.70 0.05088</td>
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<td>2801  114.10 0.05205</td>
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<td>3015 -118.50 -0.04809</td>
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<td>3137  103.10 0.04800</td>
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<td>3363 -103.70 -0.04827</td>
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**Cycle 9**

---

some of the curves have horizontal spikes like a jump in the displacement

---

**Cycle 10**

---

**Cycle 11**

---

**Cycle 12**

---

**Cycle 13**

---

concrete is spalling off bottom corner due to bearing pressure

---

**Cycle 14**

---

can begin to visually see slab move relative to beam, very much noise at low loads as sliding occurs, sounds like a ticking noise due to friction loss and sand sprinkling, can see through the gap between the hydrastone and the bearing point

---

**Cycle 15**

---

stop to take photos

---

**Cycle 16**

---

specimen seems to have buckled laterally, there is a longitudinal crack along the axis of shear studs 1, 2, 3 and a vertical crack between studs 2 and 3

---

<table>
<thead>
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<td>4783  117.60 0.09971</td>
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<tr>
<td>4943 -123.00 -0.09413</td>
</tr>
<tr>
<td>4949 -118.70 -0.09618</td>
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<table>
<thead>
<tr>
<th>Cycle 16</th>
</tr>
</thead>
<tbody>
<tr>
<td>5048  91.89  0.08751</td>
</tr>
<tr>
<td>5049  56.51  0.12799</td>
</tr>
</tbody>
</table>
LAB NOTES FOR TEST B
TESTED ON DECEMBER 11, 1992
KURT MCMULLIN

This specimen had Standard shear studs, with 413 psi compression in the slab before testing.

MAXIMUM FORCE DURING TEST = 142.5 kips

<table>
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<tr>
<td>30</td>
<td>50.330 0.0055335</td>
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<tr>
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<td>48.760 0.0057175</td>
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<td>98</td>
<td>-42.370 -0.0055335</td>
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<td>-42.190 -0.0055945</td>
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<tr>
<td>104</td>
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<tr>
<td>165</td>
<td>46.820 0.0054725</td>
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<tr>
<td>166</td>
<td>46.190 0.0055340</td>
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<td>71.35 0.010380</td>
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<td>299</td>
<td>67.40 0.010748</td>
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<td>389</td>
<td>-61.53 -0.010130</td>
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<td>442</td>
<td>60.620 0.0100745</td>
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<td>443</td>
<td>60.070 0.0101955</td>
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<td>526</td>
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<td>592</td>
<td>95.16 0.02043</td>
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<td>595</td>
<td>92.85 0.02092</td>
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<td>699</td>
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<tr>
<td>704</td>
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<tr>
<td>795</td>
<td>83.77 0.02031</td>
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<tr>
<td>797</td>
<td>82.52 0.02056</td>
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<tr>
<td>904</td>
<td>-82.73 -0.01981</td>
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<td>906</td>
<td>-81.12 -0.01988</td>
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<tr>
<td>1026</td>
<td>119.60 0.04048</td>
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<td>1029</td>
<td>117.10 0.04115</td>
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<td>106.00 0.04054</td>
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<tr>
<td>1431</td>
<td>-103.60 -0.03998</td>
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<tr>
<td>1435</td>
<td>-101.10 -0.04016</td>
</tr>
</tbody>
</table>

--- specimen is installed backward of other tests, 123 studs face north
--- photos before testing, hydrastone above stud 1 was cracked during pretensioning

--- slight pinging sound from deck, hydrastone is beginning to flake off from steel deck near stud 1
--- hydrastone along edge of deck and slab at stud 6 is cracked
--- this loading cycle has had a flatter and more pinched hysteresis loop

--- loud dull noise
### Cycle 9

<table>
<thead>
<tr>
<th>Time</th>
<th>Load</th>
<th>Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>1558</td>
<td>100.80</td>
<td>0.04023</td>
</tr>
<tr>
<td>1560</td>
<td>100.10</td>
<td>0.04054</td>
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<tr>
<td>1562</td>
<td>-11.500</td>
<td>-0.001193</td>
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<tr>
<td>1687</td>
<td>93.69</td>
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<tr>
<td>1689</td>
<td>93.04</td>
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<tr>
<td>1819</td>
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<tr>
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</tr>
<tr>
<td>1982</td>
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</tr>
</tbody>
</table>

### Cycle 10

<table>
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<tr>
<th>Time</th>
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<tbody>
<tr>
<td>2075</td>
<td>125.80</td>
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<td>2082</td>
<td>123.40</td>
<td>0.05977</td>
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<tr>
<td>2213</td>
<td>-135.8</td>
<td>-0.06020</td>
</tr>
<tr>
<td>2221</td>
<td>-131.2</td>
<td>-0.06186</td>
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</table>

### Cycle 11

<table>
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<tr>
<th>Time</th>
<th>Load</th>
<th>Slope</th>
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</thead>
<tbody>
<tr>
<td>2305</td>
<td>119.30</td>
<td>0.06069</td>
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<tr>
<td>2309</td>
<td>114.40</td>
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<td>-0.06039</td>
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### Cycle 12

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<tr>
<td>2520</td>
<td>132.50</td>
<td>0.07993</td>
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<tr>
<td>2526</td>
<td>127.30</td>
<td>0.08220</td>
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<tr>
<td>2634</td>
<td>-142.5</td>
<td>-0.08005</td>
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<tr>
<td>2644</td>
<td>-136.9</td>
<td>-0.08232</td>
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### Cycle 13

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<tr>
<td>2724</td>
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<td>2738</td>
<td>123.00</td>
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<td>2740</td>
<td>120.70</td>
<td>0.08256</td>
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<td>2857</td>
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<tr>
<td>2861</td>
<td>-126.20</td>
<td>-0.08061</td>
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### Cycle 14

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</thead>
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<tr>
<td>2948</td>
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<td>2955</td>
<td>125.60</td>
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<td>3054</td>
<td>-134.7</td>
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<td>3055</td>
<td>-114.6</td>
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### Cycle 15

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<tr>
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<td>78.72</td>
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<tr>
<td>3170</td>
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<td>3222</td>
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<td>3227</td>
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<td>3352</td>
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<tr>
<td>3361</td>
<td>-108.8</td>
<td>-0.2404</td>
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</tbody>
</table>

---

*tinging sounds while approaching max. load*

---

*loud pinging sound*

---

*slab has split along studs 1, 2, 3, and suddenly displacement has jumped*

---

*a large shear crack starting at stud 2 and extends upward to edge of slab, halfway between stud 1 and stud 2*
Inspection and photos after test
- stud 2 appears to have initiated the final cracking of the slab by trying to move upward (perpendicular to the steel beam) and cracked the slab in flexure
- test cycles did not seem to be symmetric, the maximum load differed (which we have seen in other tests) but more interesting was the fact that the flat region of the hysteresis loop often would not expand but instead would occur at a lower load in retraction and a higher load in extension
- slab 123 moved about 0.5 inch relative to the beam but slab 456 seems to have stayed in the same place
- strain gauges seem to read low values during test, indicating that the stud is doing very little bending but a lot of shear
- does the water-proofing agent over the strain gauges cause this to behave non-symmetrically, is it creating a ductile condition of it's own? does the horizontal rotation always occur toward the gauge side? is there a pattern that relates the movement of one slab's LVDT to the stiffness of that slab's studs?
LAB NOTES FOR TEST C  
TESTED ON NOVEMBER 11, 1992  
KURT MCMULLIN

This specimen had Semi-rigid shear studs, with 48 psi compression in the slab before testing.

MAXIMUM FORCE DURING TEST = 56.63 kips

<table>
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<th>DISPLACEMENT (inches)</th>
<th>NOTES</th>
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<tbody>
<tr>
<td></td>
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<td></td>
<td>--- hydrastone has cracked on southeast and southwest corners due to pretensioning of steel rods</td>
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<tr>
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<td></td>
<td></td>
<td>--- hydrastone between deck and concrete next to south LVDT is cracked</td>
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<td>Cycle 1</td>
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<td>Cycle 3</td>
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<td>237</td>
<td>19.1500</td>
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<td>Cycle 5</td>
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<tr>
<td>Cycle 8</td>
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<td>--- cycles for new loading cycle follow the track of the first cycle from the previous set of cycles</td>
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--- hydrastone is just beginning to flake off from deck, crack in the hydrastone between steel deck and concrete
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<th>Noise</th>
<th>Comments</th>
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<tr>
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<td>1253: 40.98</td>
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<td>1353: -41.54</td>
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<td>1357: -40.59</td>
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<td>1448: 40.030</td>
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<td>1452: 37.610</td>
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<td>1565: -39.20</td>
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<td>1567: -39.01</td>
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<tr>
<td>11</td>
<td>1653: 45.67</td>
<td>0.07867</td>
<td>slight sounds of steel deck tinging or hydrastone flaking off</td>
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<tr>
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<td>1657: 44.54</td>
<td>0.08064</td>
<td>hydrastone flaking off</td>
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<td>1780: -47.33</td>
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<td>1783: -46.56</td>
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<td>1816: -10.190</td>
<td>-0.07034</td>
<td>hydrastone is broken loose between flange and steel deck</td>
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<tr>
<td>12</td>
<td>1853: 20.33</td>
<td>0.01844</td>
<td>dull thump as load is applied</td>
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<tr>
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<td>1892: 44.46</td>
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<td>1894: 43.77</td>
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<td>1975: -46.23</td>
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<tr>
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<td>gap exists between reaction block and concrete</td>
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<td>2159: -50.00</td>
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</tr>
<tr>
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<td>2272: 47.65</td>
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<td>2482: 47.03</td>
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<td>2580: -53.99</td>
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<tr>
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<td>photos taken to show relative movement between steel flange and steel deck</td>
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<tr>
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<td>2681: 44.94</td>
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<tr>
<td>17</td>
<td>2918: 52.01</td>
<td>0.14090</td>
<td>crackling sounds while going through deadband</td>
</tr>
<tr>
<td></td>
<td>2921: 50.61</td>
<td>0.14255</td>
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--- sudden loss of load but no apparent damage to specimen (possibly a stud failed?)

--- zero relative displacement but 0.18" displ. at cylinder

--- at the end of the loading the stiffness has greatly picked up again

--- measure movement out of plane = 0.09"/8kip

--- lateral movement out of plane is 0.09" as cylinder is retracted
Cycle 27

5774 33.510 0.2682
5783 17.400 0.2897
5903 -29.420 -0.2851
5906 -29.160 -0.2880
5926 -3.305 -0.2742 --- stop for photos and to turn video on for real
time recording, at studs 5 and 6 the flange
is apart from the flange far enough to see
the studs, while on side 1 the concrete has
slid 0.25 inches toward the beam from it's
initial position

Cycle 28

6000 23.150 0.3205 --- pop at very end of loading and suddenly
6004 12.750 0.3261 dropped load before opening relief valve
6098 -29.750 -0.3228 --- turn off real-time video recording
6101 -29.020 -0.3294

Cycle 29

6198 22.300 0.3237
6200 21.720 0.3238
6299 -25.240 -0.3277

Cycle 30

6416 25.71 0.3478 --- drop in load before reaching final displ.
6417 16.15 0.3556
6438 21.130 0.5010
6441 20.110 0.5065
6574 -24.800 -0.3477
6580 -19.600 -0.4020
6588 -12.680 -0.4852
6596 -10.520 -0.5131

Cycle 31

6685 13.370 0.4176
6699 9.632 0.5119
6782 -7.810 -0.1530 --- now beam flange is pulling away from concrete
at stud 1, deck at stud 1 is cracked free
from slab

6793 -9.605 -0.3135
6801 -6.382 -0.4482
6805 -6.565 -0.4950
6812 -5.942 -0.5221

Cycle 32

6931 12.090 0.7560
6935 11.680 0.7565
7111 -9.010 -0.7543
7115 -8.140 -0.7581

Cycle 33

7231 2.7100 0.7219 --- start real-time video
7234 2.3810 0.7227
7375 -15.540 -0.9767
7379 -14.590 -0.9775 --- deck snapping away from slab, entire slab
appears to be pulling away from studs

Cycle 34

7640 13.070 2.5400
Inspection and photos after test
- you can see concrete dust that has dropped down through the slot between the face of steel flange and the concrete slab
- one large crack runs vertically across stud 2 and appears to go all the way through the slab
- slab and steel flange are 0.125 inches apart through the length of the beam - is concrete still attached to the beam? if not then what is holding the concrete and what produced the 12 kips of force at the end of the test
- looking down between the slab and the flange you can see a 2 or 3 inch distance between the base of the stud and the tape and sheet metal that had covered the stud
- when the prestressing rods are removed the two slabs fall free from the steel beam, no attachment remains between them
This specimen had Semi-rigid shear studs, with 422 psi compression in the slab before testing.

MAXIMUM FORCE DURING TEST = 52.35 kips

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1650 46.74  -0.05926
1658 44.51  -0.06146

Cycle 10
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1761 48.46  0.06251
1886 44.51  -0.06097
1892 43.45  -0.06153

Cycle 11
2002 44.43  0.08168
2004 43.66  0.08223
2136 48.72  -0.08009
2139 47.99  -0.08076

Cycle 12
2274 42.56  0.08045
2285 40.25  0.08113
2414 47.03  -0.08094
2418 46.52  -0.08156

Cycle 13
2508 46.22  0.10078
2516 44.35  0.10369
2629 -50.88  -0.10028
2638 -48.35  -0.10173

Cycle 14
2769 45.12  0.10472
2771 44.43  0.10533
2875 -49.23  -0.10273
2882 -47.36  -0.10225

Cycle 15
3013 46.81  0.11870
3019 45.71  0.12220
3039 10.440  0.10859
3122 -51.91  -0.12005
3132 -49.86  -0.12225

Cycle 16
3264 45.60  0.12040
3267 44.54  0.12105
3393 -49.64  -0.11975
3401 -48.32  -0.12080

Cycle 17
3505 22.96  0.02558
3551 47.50  0.14040
3556 46.18  0.14220
3693 -52.35  -0.1387
3698 -51.39  -0.1415

Cycle 18
3824 46.26  0.14245
3828 45.41  0.14415
3959 -50.22  -0.13970
3963 -49.64  -0.14205

--- consistently we unload about 10 kips
--- at a linear slope and then require
--- displacement to reduce the load

--- loading to maximum displacement normally
--- requires the same load but not the same
--- path as the first cycle

--- stop to take photos

--- some loud sounds while we move through
--- the zero displacement point
### Cycle 19

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--- start of partial loading loops

--- load sounds going through zero displ.

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--- stop for photos

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--- you can see sand and grit on the floor where it has dropped from between the beam and concrete

### Cycle 24

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### Cycle 26

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--- when moving through the zero displacement it sounds like the deck may be buckling
CUREe–Kajima Research Project

Optimum Use of Panel Zones in Seismic Design of Tall Steel Frames

John F. Hall
California Institute of Technology

Kajima contact: Dr. Eiji Fukuzawa

January 15, 1992 to January 14, 1993
Overview

The aim of this research is to evaluate the role of panel zones in the seismic response of tall steel frame buildings using very strong ground motions capable of causing building collapse. The collapse mode considered is that of $P-\Delta$ instability from excessive lateral sway. One question which is addressed is: Can the excellent energy dissipation property of panel zones be exploited to improve seismic performance as a method of passive control? At the same time, an attempt is made to quantify the factor of safety of tall steel frame buildings, as currently designed, against earthquake collapse. It is of considerable importance to develop some sense of what this "number" is.

A panel zone is the portion of the column at the beam-column junction. The primary mode of deformation of this zone is one of shear produced by the beam and column moments existing under lateral loads. Yielding of panel zones generally precedes hinging of beams and columns unless doubler plates are added to the column webs in order to strengthen them. The desirability of using doubler plates is still an open question because strong panel zones cause greater ductility demands on the beams and columns, and column hinging is especially detrimental to lateral stability. Frames whose panel zones have been designed to participate in the yielding should have superior performance. A panel zone element with accurate hysteretic properties is developed here.

The ductility participation between panel zones, beams and columns is sensitive to the nonlinear models used for the various elements, especially as regards strain hardening. For this reason, accurate beam-column elements of the "fiber" type have also been developed. In a fiber element, the member is divided into segments and a cross-section is divided into fibers. An accurate hysteretic model for axial stress-strain is used for each fiber.

This report is divided into three parts. In Part 1, beam-column elements are formulated and tested against experimental data. The panel zone element is developed in Part 2 as is the complete treatment of a planar frame. Here also are presented results for the effect of panel zone design on frame response, with emphasis on collapse-level ground motions. A building designed by Kajima Corporation is examined in Part 3.
Part 1 — Beam–Column Modelling

Introduction

Many beam-column mathematical models have been described in the literature that incorporate material and geometric nonlinearities. Capabilities of the available models vary widely as do their complexity and computational requirements. Two general classes of nonlinear beam-column models are the plastic hinge type (1,7,11) and the fiber type (6,10). In this study, a plastic hinge element and a fiber element are compared for four problems in structural engineering using steel members. In addition, several improvements to existing formulations are made. In all cases, only two-dimensional planar behavior is considered.
Plastic Hinge Element

The plastic hinge element lumps the material nonlinearity into discrete locations called plastic hinges where kink rotations occur when the bending moment reaches the moment capacity of the section. A plastic hinge exhibits rigid-plastic behavior for the moment-kink rotation relation. The moment capacity of a section is reduced by the presence of axial force such as according to the following relations which are appropriate for an I section bent about its strong axis (sa) or weak axis (wa)

\[
M_{pr} = M_p - M_{pw} \cdot \rho^2 \\
= M_p - M_{pw} \cdot (1 + 4\gamma\lambda(1 + \lambda)) 
\]

for \( \rho \leq 1 \)

where

\[
M_{pw} = \sigma_y t_w d_w^2/4 \text{ (sa)} , \quad \sigma_y d t_w^2/4 \text{ (wa)} \\
\rho = \left(|P|/P_y\right) \cdot \left(A/A_w\right) \\
A_w = d_w t_w \text{ (sa)} , \quad t_w d \text{ (wa)} \\
d_w = d - 2t_f \\
\gamma = b/t_w \text{ (sa)} , \quad 2t_f/d \text{ (wa)} \\
\lambda = (\rho - 1)/2\gamma
\]

and \( M_{pr} \) = reduced moment capacity, \( M_p \) = plastic moment capacity in absence of axial force, \( P \) = axial force, \( P_y = \sigma_y A \), \( \sigma_y \) = yield stress, \( A \) = cross-sectional area, \( d \) = total depth parallel to web, \( b \) = flange width, \( t_w \) = web thickness, \( t_f \) = flange thickness. For simplicity, plastic hinges are allowed to form only at the nodes, and so if plastic hinges within the member are to be accommodated, then the beam-column must be divided into a number of segments with internal nodes (Fig. 1a). The hinge at an internal node is associated with only one of the two adjacent segments.

In order to approximately account for strain hardening, an elastic rotational spring of stiffness \( \kappa \) is connected across each plastic hinge. Considering this spring to be part of the hinge, the plastic part of the moment-kink rotation relation takes on a positive slope equal to \( \kappa \). Since a rotational spring is associated locally with a plastic hinge and since its moment changes only when the kink rotation changes, this may be a more rational approach for incorporating strain hardening than the parallel beam (1).

The following formulation considers geometric nonlinearity through updating of the nodal coordinates and by including the contribution to stiffness from axial load. Bowing
(7,11) and axial buckling are accounted for at the member level through the multiplescgment and configuration updating capabilities. Strains are assumed to be small as are deflections relative to the chord.

In the standard iterative solution process, the nodal displacements \( a_i \) of each segment (Fig. 1b), expressed as \( (u_1, v_1, \theta_1, u_2, v_2, \theta_2) \) in local coordinates (Fig. 1c), are updated from displacement increments computed in each iteration. Iterations continue until convergence is achieved in each load step (or time step). To define the solution process, it is sufficient to state the procedure by which the segment nodal forces \( \{R^s\} = (\cdots R_i \cdots)^T \) (Fig. 1b) are determined and to define the segment tangent stiffness matrix \([K^s_T]\) which is assembled into the tangent stiffness matrix of the member. Current values of \( \{R^s\} \) and \([K^s_T]\) are computed using the cumulative displacement increments from the beginning of the load (time) step in order to avoid problems of artificial unloading.

The forces \( \{R^s\} \) are found from the segment axial force \( P \), shear force \( Q \), and end moments \( M_1 \) and \( M_2 \) (Fig. 1c) from standard equilibrium relations using the current chord orientation angle \( \alpha \):

\[
\{R^s\} = [T]^T (P, -Q, M_1, -P, Q, M_2)^T
\]

(2a)

where

\[
[T] = \begin{bmatrix}
\cos \alpha & \sin \alpha & 0 & 0 & 0 & 0 \\
-\sin \alpha & \cos \alpha & 0 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 & 0 \\
0 & 0 & 0 & \cos \alpha & \sin \alpha & 0 \\
0 & 0 & 0 & -\sin \alpha & \cos \alpha & 0 \\
0 & 0 & 0 & 0 & 0 & 1
\end{bmatrix}
\]

(2b)

Determination of \( P, Q, M_1 \) and \( M_2 \) requires \( \alpha \), the chord elongation \( u \) (negative for shortening), the rotations \( \phi'_1 \) and \( \phi'_2 \) relative to the chord just inside the plastic hinge locations, and similar rotations \( \phi_1 \) and \( \phi_2 \) just outside the plastic hinge locations. Note that \( \phi_1 - \phi'_1 \) and \( \phi_2 - \phi'_2 \) are the kink rotations at nodes 1 and 2. Values of \( \alpha \) and \( u \) are found from the current nodal locations using exact trigonometry relations. Calculation of \( \phi'_1 \) and \( \phi'_2 \) must consider plastic hinges and axial yielding, while \( \phi_1 = \theta_1 - \alpha \) and \( \phi_2 = \theta_2 - \alpha \).

A possible state of axial yielding is checked for first. For this purpose, the amount of plastic axial displacement, denoted \( u_p \), must be tracked. Whenever \( |u - u_p| > \varepsilon_y \ell \) (where \( \ell \) = original segment length, \( \varepsilon_y = \sigma_y/E \) and \( E \) is Young's modulus), then \( P = P_y \) for tensile
yielding or \(-P_y\) for compressive yielding. Rotations \(\phi'_1\) and \(\phi'_2\) are found from eqs. 3b and 3c below after substituting \(M_1 = \kappa(\phi_1 - \phi'_1)\) and \(M_2 = \kappa(\phi_2 - \phi'_2)\), and then \(M_1, M_2\) and \(Q\) are computed from eqs. 3b, 3c and 3d. The segment length should be chosen so that a segment yields in compression before it buckles.

If the axial yield check is false, then

\[
P = EA \frac{u - u_p}{\ell} \quad (3a)
\]

\[
M_1 = c_{11} \phi'_1 + c_{12} \phi'_2 \quad (3b)
\]

\[
M_2 = c_{12} \phi'_1 + c_{11} \phi'_2 \quad (3c)
\]

\[
Q = \frac{(M_1 + M_2)}{\ell + u} \quad (3d)
\]

where \(c_{11}\) and \(c_{12}\) are rotational stiffnesses. Considering both shear deformations (14) and effect of axial load (15), expressions for \(c_{11}\) and \(c_{12}\) are

\[
c_{11} = \frac{EI}{\ell} \frac{A \omega \sin \omega - \omega^2 \cos \omega}{2A - 2A \cos \omega - \omega \sin \omega} \quad \text{for } P < 0 \quad (4a)
\]

\[
= \frac{EI}{\ell} \frac{4 + \beta}{1 + \beta} \quad \text{for } P = 0 \quad (4b)
\]

\[
= \frac{EI}{\ell} \frac{-A \omega \sinh \omega + \omega^2 \cosh \omega}{2A - 2A \cosh \omega + \omega \sinh \omega} \quad \text{for } P > 0 \quad (4c)
\]

\[
c_{12} = \frac{EI}{\ell} \frac{-A \omega \sin \omega + \omega^2}{2A - 2A \cos \omega - \omega \sin \omega} \quad \text{for } P < 0 \quad (4d)
\]

\[
= \frac{EI}{\ell} \frac{2 - \beta}{1 + \beta} \quad \text{for } P = 0 \quad (4e)
\]

\[
= \frac{EI}{\ell} \frac{A \omega \sinh \omega - \omega^2}{2A - 2A \cosh \omega + \omega \sinh \omega} \quad \text{for } P > 0. \quad (4f)
\]

where

\[
\omega = \ell \sqrt{|P|/EI}
\]

\[
A = 1 + \omega^2 \frac{\beta}{12}
\]

\[
\beta = \frac{12 EI}{GA_s \ell^2}
\]

and \(I = \text{cross-sectional moment of inertia, } G = \text{shear modulus, } A_s = \text{effective shear area} = 5/6 \ dt_w \ (sa), \ 5/3 \ bt_f \ (wa)\).
Rotations $\phi'_1$ and $\phi'_2$ depend on the plastic hinge state of which there are nine possibilities: 00, 0+, +0, 0−, −0, ++, −−, ++ and −+, where 0 means not an active hinge, + means active positive hinge, − means active negative hinge, and the first position corresponds to node 1 and the second position corresponds to node 2. The correct state can be found by successively assuming each of the nine states to be the correct one and checking for violations. Only one state will survive. For example, if state +0 is assumed, proceed as follows:

1. Compute $P$ from eq. 3a and then $c_{11}$ and $c_{12}$ from eq. 4.
2. Compute $\phi'_2$ assuming zero increment of kink rotation at node 2.
3. Compute $\phi'_1$ from equation 3b with

$$M_1 = M_{pr} + \kappa \cdot (\phi_1 - \phi'_1)$$

(5)

where $M_{pr}$ is determined from equation 1 using $P$ from equation 3a.

4. Check that the kink rotation for node 1, equal to $\phi_1 - \phi'_1$, is positive.

5. Compute $M_2$ from equation 3c and check that

$$|M_2 - \kappa \cdot (\phi_2 - \phi'_2)| < M_{pr}.$$  

(6)

6. If no violations occur in steps 3 and 4, finish by computing $Q$ from equation 3d. Otherwise, test the next state.

Tangent stiffness matrices for a beam-column segment that relate nodal force increments $\Delta R_i$ to nodal displacement increments $\Delta a_i$ for each of the nine possible hinge states are easily derived. The tangent stiffness matrix for a segment is given by

$$[K^g_T] = [T]^T [B^T AB + K^g_G][T]$$

(7a)

where

$$[B] = \begin{bmatrix} 1 & 0 & 0 & -1 & 0 & 0 \\ 0 & \frac{1}{\ell + u} & 1 & 0 & \frac{1}{\ell + u} & 0 \\ 0 & \frac{1}{\ell + u} & 0 & 0 & \frac{1}{\ell + u} & 1 \end{bmatrix}$$

(7b)
\[ [A] = \begin{bmatrix}
  k_a & 0 & 0 \\
  k_{11} & k_{12} & \text{sym} \\
  0 & 0 & k_{22}
\end{bmatrix} \quad (7c) \]

\[
[K_{GC}^s] = \frac{1}{\ell + u} \begin{bmatrix}
  0 & Q & 0 & 0 & -Q & 0 \\
  P & 0 & -Q & -P & 0 \\
  0 & 0 & 0 & 0 & 0 \\
  \text{sym} & 0 & Q & 0 & P \\
  0 & 0 & 0 & 0 & 0
\end{bmatrix} \quad (7d)
\]

\[ [A] \text{ is the tangent matrix which relates } (\Delta P, \Delta M_1, \Delta M_2)^T \text{ to } (\Delta u, \Delta \phi_1, \Delta \phi_2)^T, \text{ and} \]

\[ k_a = \frac{AE}{\ell} \quad \text{if axial yield is not occurring} \quad (8a) \]

\[ k_{11} = c_{11} \quad \text{for state 00} \]

\[ d \quad \text{for states 0+ and 0–} \quad (8b) \]

\[ e \quad \text{for states +0 and –0} \]

\[ f \quad \text{for states ++, ––, +– and –+} \]

\[ k_{12} = c_{12} \quad \text{for state 00} \]

\[ g \quad \text{for states 0+, +0, 0–, –0} \quad (8c) \]

\[ h \quad \text{for states ++, ––, +– and –+} \]

\[ k_{22} = c_{11} \quad \text{for state 00} \]

\[ e \quad \text{for states 0+ and 0–} \quad (8d) \]

\[ d \quad \text{for states +0 and –0} \]

\[ f \quad \text{for states ++, ––, +– and –+} \]
In eq. 7, the terms involving $Q$ often have little effect and are usually neglected, and, consistent with the small strain assumption, $\ell + u$ can be replaced by $\ell$. 

\begin{align*}
    d &= c_{11}\kappa/b \\
    e &= c_{11} - c_{12}^2/b \\
    f &= \kappa \left( c_{11}b - c_{12}^2 \right) / \left( b^2 - c_{12}^2 \right) \\
    g &= c_{12}\kappa/b \\
    h &= c_{12}\kappa^2 / \left( b^2 - c_{12}^2 \right) \\
    b &= c_{11} + \kappa.
\end{align*}
Fiber Element

The fiber element improves on the plastic hinge element by including residual stresses, representing the spread of yielding within the cross-section and along the length, properly coupling the axial and flexural yielding, and more accurately accounting for strain hardening. A beam-column is divided into segments as before (fig. 1a) with the same degrees of freedom. Further, each segment is divided into a number of fibers over its cross-section (fig. 2) for which nonlinear axial stress-strain behavior is considered.

The formulation is along the lines of that for a two-node beam element with linear shape functions, shear deformations included and center-point integration (2). Because of the linear shape functions, a multi-segment discretization will always have to be used. The segment tangent stiffness matrix which relates nodal force increments \( \{ \Delta F \} \) to nodal displacement increments \( \{ \Delta a \} \) can be derived as (4)

\[
[K^S_T] = [T]^T [K^S_F + K^S_{SH}] [T]
\]  
(9a)

where \([K_F]\) contains the contributions from fiber axial stiffness and geometric stiffness, and \([K_{SH}]\) represents the shear stiffness which is taken to be elastic:

\[
[K^S_F] = \sum_{i=1}^{N_f} \begin{bmatrix} \Lambda_i & -\Lambda_i \\ -\Lambda_i & \Lambda_i \end{bmatrix}, \quad [\Lambda_i] = \frac{A_i}{\ell} \begin{bmatrix} E_{Ti} & 0 & -E_{Ti}h_i \\ 0 & \sigma_i & 0 \\ -E_{Ti}h_i & 0 & E_{Ti}h_i^2 \end{bmatrix}
\]  
(9b)

\[
[K^S_{SH}] = GA_s \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ \frac{1}{\ell} & -\frac{1}{2} & 0 & -\frac{1}{\ell} & -\frac{1}{2} \\ \frac{\ell}{4} & 0 & \frac{1}{2} & \frac{\ell}{4} & \frac{1}{2} \\ \text{sym} & 0 & 0 & 0 & 0 \\ \frac{1}{\ell} & \frac{1}{2} & \frac{\ell}{4} & \text{sym} \end{bmatrix}
\]  
(9c)

where \(N_f = \text{number of fibers} \); \(A_i = \text{area of fiber } i \); \(E_{Ti} = \text{current tangent modulus of fiber } i \); \(\ell = \text{original segment length} \); \(h_i = \text{distance of fiber } i \text{ above centroidal axis} \); \(\sigma_i = \text{current axial stress in fiber } i \); \(G = \text{elastic shear modulus} \); \(A_s = \text{effective shear area} \). Consistent
with an assumption of small strains and the lack of bowing in a segment because of the linear shape functions, only the original length is used in the above.

The member tangent stiffness matrix is assembled from segment tangent stiffness matrices as given by eq. 9 and thus, because of the use of the fiber tangent modulus $E_{Ti}$, includes material nonlinearity. Actually, the tangent matrix depends on the relative magnitudes and signs of the displacement increments which are not known a priori, which is especially true during unloading. Convergence problems owing to the use of an incorrect tangent matrix can occur when the $E_{Ti}$ values are small. Specifying a lower bound for $E_{Ti}$ helps; $E_{sh}$ (defined below) works well, but occasionally a larger value is needed.

An important aspect of the fiber model is the hysteretic unaxial stress-strain relation, and an original one is employed here (4). The stress-strain curve for monotonic loading shown in fig. 3 is used for tension and compression and is defined by elastic modulus $E$, yield stress $\sigma_y$, ultimate stress $\sigma_u$, strain $\varepsilon_{sh}$ at initiation of strain hardening, strain $\varepsilon_u$ at ultimate stress, rupture strain $\varepsilon_r$, and initial strain-hardening tangent modulus $E_{sh}$. The curved strain-hardening part is a cubic ellipse centered at $\varepsilon_u, \sigma_o$ with radii $a$ and $b$:

$$\left(\frac{\varepsilon - \varepsilon_u}{a}\right)^3 + \left(\frac{\sigma - \sigma_o}{b}\right)^3 = 1$$

Parameters $\sigma_o, a$ and $b$ are determined from $\varepsilon_{sh}, \varepsilon_u, E_{sh}, \sigma_y$ and $\sigma_u$. The slope $E_{AB}$ is tangent to the curve at $B$ and plays a role in the hysteretic behavior.

The behavior during loading and unloading, as illustrated in fig. 4, makes use of two backbone curves — one for tensile stress states and one for compressive stress states. These curves have the same shape as the monotonic loading curve, and they shift along the strain axis. The compression backbone curve shifts when the tension backbone curve is being followed, and vice versa. The initial backbone curves coincide with the tension-compression monotonic loading curve except if a residual (initial) stress is to be included, in which case an initial strain shift in the amount $\varepsilon_{res} = \sigma_{res}/E$ is specified, where $\sigma_{res}$ is the residual stress.

When a reversal occurs from a backbone curve, a linear path at slope $E$ is followed to the $\sigma = 0$ axis (segments 2–3, 5–6 and 8–9 in fig. 4) where a shifted backbone curve is encountered. The path is then that of a cubic ellipse (3–4, 6–6A–7, 9–10) with initial slope $E$ and a target point back on the shifted backbone curve, after which this backbone curve is followed (4–5, 7–8, 10–11). The location of the target point (4,7,10) is that of the previous turning point (2,5,8) transferred to the current backbone curve, and the approach
slope to the target point is either $E_{AB}$, the tangent to the backbone curve, or a zero slope depending on whether the target point lies along $AB$, $BC$, or $CD$ in fig. 3, respectively, as do points 4, 7 and 10 in fig. 4, respectively. An exception to the above occurs when the initial yield excursion (segment 1–2) is too short; in which case, the following linear unloading segment (2–3) must be extended beyond the $\sigma = 0$ axis so that the subsequent cubic ellipse can be completed with the desired slope.

The path followed when reversal occurs from other than a backbone curve (point 6A in fig. 4) consists of loops which close fully and have matching halves. Thus each of the half loops 6A–6B–6C, 6C–6D–6E, 6E–6F–6G, 6G–6H–6E is a subsegment of 5–6–6A with point 5 as the start. Once an inner loop is completed (point 6E), then the outer loop is continued until either the outer loop is completed (point 6E) or another reversal occurs. Eventually, a backbone curve is reached (6A to 7) and followed.

The above scheme is convenient because the current fiber stress $\sigma_i$ and tangent modulus $E_T$ can be expressed explicitly in terms of the current fiber strain $\varepsilon_i$. The strain $\varepsilon_i$ is completely determined by the segment nodal displacements $\{a\}$. The local segment forces are found as

$$P = \sum_{i=1}^{N_f} \sigma_i A_i$$  \hspace{1cm} (11a)

$$Q = \frac{1}{2} GA_s (\theta_1 + \theta_2 - 2\alpha)$$  \hspace{1cm} (11b)

$$M_1 = -\sum_{i=1}^{N_f} \sigma_i A_i h_i + \frac{1}{2} Q \ell$$  \hspace{1cm} (11c)

$$M_2 = \sum_{i=1}^{N_f} \sigma_i A_i h_i + \frac{1}{2} Q \ell$$  \hspace{1cm} (11d)

which are then transformed by eq. 2 to $\{F\}$ using the current chord rotation $\alpha$ as determined from updated nodal locations.

A demonstration of the performance of the hysteretic uniaxial stress-strain model is shown in fig. 5 where comparisons to three sets of experimental data are made. These data are for a steel bar under uniaxial stress and include a virgin curve (8) and two cyclic loading paths (8,13). In each case, the model was calibrated with actual or estimated parameters $E$, $E_{sh}$, $\sigma_y$, $\sigma_u$, $\varepsilon_{sh}$ and $\varepsilon_u$ of the experiment material, and it was subjected
to the axial strain history of the experiment. The uniaxial hysteretic model simulates the data very well.
Comparison Studies

Results are presented for four problems using steel members which are solved using the plastic hinge and fiber beam-column elements. The four problems are: 1) moment amplification in a slender column, 2) hysteretic behavior of a brace, 3) cyclic loading of a cantilever beam, and 4) earthquake response of a frame. Problems 2 and 3 simulate laboratory tests, and the results are compared to the data. Emphasis here is on geometric effects and on yielding from axial force and bending moment. Other important features of structural steel behavior such as local plate buckling, connection behavior and joint panel zone yielding have not been included in the models.

The material properties required in the analyses are \( E, G \) and \( \sigma_y \) and, additionally, \( \kappa \) for the plastic hinge model and \( \sigma_u, \varepsilon_{sh}, \varepsilon_u, \varepsilon_r, E_{sh} \) and \( \sigma_{res} \) for the fiber model. To the extent possible, the values of the material properties for problems 2 and 3 are based on the values provided in the laboratory reports. The rotational spring stiffness \( \kappa \) for the plastic hinge elements is always set equal to \( 0.025 \cdot (6E_1/L) \) where \( L \) is the member length, which proved to be a reasonable value. \( G \) is always set to \( E/2.6 \).

Member segmentation varies for the different problems. The designation is \( PHn \) for a plastic hinge element and \( Fn \) for a fiber element, where \( n \) is the number of segments. For the fiber element, the particular fiber patterns shown in Figures 2a and 2b for strong and weak axis bending, respectively, are used.

1. Moment amplification

This problem considers a pin-ended slender steel column constrained to bend about its strong axis (W8x31, length 9.68 m, \( E = 207,000 \) MPa, \( \sigma_y = 228 \) MPa, \( \sigma_u = 366 \) MPa, \( \varepsilon_{sh} = 0.011, \varepsilon_u = 0.14, \varepsilon_r = 0.30, \text{ and } E_{sh} = 4000 \) MPa). The column is initially compressed to \( P = -0.6 \) \( P_y \) and then bent by positive end moments \( M_{top} \) and \( M_{bot} \) (sign convention of fig. 1). These loads are applied by force control. To investigate the effect of residual stress, two fiber elements are used: one including residual stress at \( \sigma_{res} = 34 \) MPa and the other with no residual stress. All elements use 20 equal segments to accommodate a possible plastic hinge at some unknown location between the ends. Such a hinge is likely since the initial axial compression is close to the Euler buckling load \( P_c = \pi^2 EI/L^2 \) (\( P_c = 0.74 P_y \)) which means that moment amplification can be important.
For $M_{bot}/M_{top} = 0.50$, $M_{top}$ reaches $0.55 M_{pr}$ for PH20, $0.45 M_{pr}$ for F20 without residual stress, and $0.29 M_{pr}$ for F20 with residual stress. In none of the cases does $M_{top}$ reach $M_{pr}$ because of the formation of a plastic hinge below the top, immediately after which failure occurs. With the plastic hinge element, the behavior is elastic until the amplified moment reaches $M_{pr}$, while the fiber elements begin to soften earlier, as soon as yield occurs at an extreme fiber. This softening leads to greater deflection and increased moment amplification. A complete hinge forms at only a slightly larger load than that at first yield, which is considerably below that reached by the plastic hinge element. Of the two fiber elements, the one with the residual stress yields first and, consequently, fails first.

For $M_{bot}/M_{top} = 0.99$, the moment amplification is less, and $M_{top}$ reaches $M_{pr}$ for PH20. However, enough interior softening occurs for the fiber elements to cause the column to unwind, i.e., to move into a more one-sided configuration. Unwinding increases the moment amplification, which leads to a plastic hinge below the top and failure at $M_{top}$ less than $M_{pr}$. The value reached by $M_{top}$ is nearly the same with or without residual stresses ($\approx 0.91 M_{pr}$). Unwinding has been investigated in ref. 9 where results on a similar problem analyzed by a different method showed $M_{top}$ to reach $0.85 M_{pr}$ for $M_{bot}/M_{top} = 1.00$. That calculation used an elastic-plastic material (same $E$ and $\sigma_y$ as used here) with residual stresses reaching 68 MPa at the flange tips.

2. Hysteretic buckling of a brace

Buckling, post-buckling, tension yield and hysteretic effects are important features of the behavior of a brace in a structure undergoing severe cyclic loading. A pinned steel member (W6x20, 3.07 m length including 23 cm long rigid mountings at each end) is compressed axially under displacement control past buckling into a mechanism involving a plastic hinge at the center, then stretched into tension, and subsequently cycled between increasing displacement limits. Buckling occurs in the weak direction.

The requirement of having a node at the center location where the plastic hinge forms means at least two segments have to be used for the plastic hinge element. Since two segments are accurate enough to capture geometric effects, two (of equal length) are used (PH2). To investigate the segment discretization necessary to achieve converged results with the fiber element, two elements are employed: F7 with 7 segments (lengths of .25L, .16L, .07L, .04L, .07L, .16L, .25L from one end to the other) and F13 with 13 segments (.14L, .11L, .09L, .07L, .05L, .03L, .02L, .03L, ..., .14L). The axial load is applied with a small eccentricity of 0.25 cm. Material properties used in the analysis are $E = 200,000$
MPa, $\sigma_y = 317$ MPa, $\sigma_u = 510$ MPa, $\varepsilon_{sh} = 0.012$, $\varepsilon_u = 0.16$, $\varepsilon_r = 0.3$, $E_{sh} = 4000$ MPa, and $\sigma_{res} = 48$ MPa.

Results appear in Figure 6 as axial force-displacement curves. The experiment (ref. 3, specimen #3) shows that, under multiple cycles of load, the compressive capacity deteriorates. This has been attributed to residual camber and reduced tangent modulus (5). The fiber element is able to capture this behavior quite well, but not the plastic hinge element. Also, the shapes of the tensile loading portions of the axial force-displacement curves are better reproduced by the fiber element. Overall, the fiber element results are quite satisfactory, and the 7-segment discretization is adequate.

3. Cyclic loading of a cantilever beam

Experimental results are available for this problem also (ref. 16, specimen #9). A cantilever beam (W18x46, 1.60 m length) is cycled between increasing displacement limits by a lateral shear load applied at the free end. The direction of bending is the strong direction.

A single segment is sufficient for the plastic hinge element (PH1), while, again to investigate the segmentation requirements of the fiber element, two elements are used: F4 (lengths of .04L, .12L, .34L, .50L from the fixed end to the free end) and F8 (.02L, .04L, .06L, .09L, .12L, .17L, .22L, .28L). Material properties used in the analysis are $E = 200,000$ MPa, $\sigma_y = 262$ MPa, $\sigma_u = 421$ MPa, $\varepsilon_{sh} = 0.012$, $\varepsilon_u = 0.16$, $\varepsilon_r = 0.3$, $E_{sh} = 4000$ MPa and $\sigma_{res} = 41$ MPa.

Results appear in Figure 7 as lateral force-displacement curves. The plastic hinge element shows a bilinear hysteresis as expected, while the fiber element’s hysteresis is curved because of this element’s ability to model the gradual spread of yielding within the cross-section and along the length. The latter results closely resemble those of the experiment except during the last few cycles when local flange buckling occurs in the experiment. This behavior is not included in either the plastic hinge or fiber elements, but it would be desirable to do so. Differences between F4 and F8 are attributed to discretization error in the former.

Some further results (computation only) are shown in Figure 8 where an axial compressive load equal to $0.4\, P_y$ is maintained during application of the same lateral displacement history used previously without axial load. Greater differences are now seen between the behaviors of the plastic hinge and fiber elements. These differences
are attributable to the increased effect of strain hardening when axial load is present and the fact that strain hardening is handled differently by the two elements. The fiber element rapidly increases its moment capacity above $M_{pr}$ during the cyclic loading. Such behavior has been observed in laboratory tests of axially loaded columns cycled laterally into the nonlinear range (12).

4. Frame response under strong earthquake

The moment resistant frame of fig. 9 meets the seismic Zone 4 requirements of the 1991 Uniform Building Code and was extensively studied in ref. 16 with regard to behavior of the panel zones in the joints. For the present purpose of comparing beam-column elements, the panel zones are strengthened with doubler plates so that yielding does not occur. A panel zone is represented in the analysis by an elastic rotational spring, and the proper spatial extent of a panel zone is modelled.

The frame is an exterior three-bay frame of a three-bay by four-bay rectangular building (18.3 m × 30.5 m plan). The three parallel interior frames in the short direction support gravity load only. Therefore, the exterior frame provides half the lateral resistance of the building in the short direction. Gravity loads and masses are computed using a dead load for the roof and floors of 4.79 kPa, a reduced floor live load of 0.96 kPa, and a cladding load of 1.67 kPa over the exterior walls. The gravity loads applied to the exterior frame are those tributary to it, and the gravity loads supported by the interior frames produce horizontal $P-\Delta$ loads which are applied during the dynamic response. The horizontal masses are those of half the building while the vertical masses are those tributary to the exterior frame only.

In the analysis, iterations in each time step are performed on an elastic frame model which contains degrees of freedom only at the intersections of the beams and columns. Mass and damping forces are associated only with these degrees of freedom. This allows the segmented members to be treated as separate substructures subjected to end displacement histories which are computed in the solutions at the frame level. Solutions for each member are obtained in each frame level iteration and involve iterations themselves according to the procedure described earlier in this paper. The member solutions produce end forces which are used to compute residual forces in the frame iterations.

Two frames are considered: one modelled entirely with plastic hinge elements and the other entirely with fiber elements. The stockiness of the members and the likelihood that
plastic hinges will form only at the ends justifies the use of a single segment per member for the plastic hinge elements (PH1). Eight segments are used for the fiber elements (lengths of \(0.03L, 0.06L, 0.16L, 0.25L, 0.16L, 0.06L, 0.03L\) from one end to the other). Based on the previous results, such a discretization should be adequate. Note that this F8 discretization differs from the one in the previous problem. Material properties are \(E = 200,000\) MPa, \(\sigma_y = 248\) MPa, \(\sigma_u = 400\) MPa, \(\varepsilon_{sh} = 0.012, \varepsilon_u = 0.16, \varepsilon_r = 0.30, E_{sh} = 4000\) MPa and \(\sigma_{res} = 41\) MPa.

The fundamental period \(T_1\) of the elastic building is 3.82 sec for the PH1 model and 3.77 sec for the F8 model. The base motion is a ramped harmonic displacement \(Ct \cos \frac{2\pi t}{T_1}\) where \(t = \text{time} and C\) is chosen so that the corresponding base acceleration grows by \(0.05g\) per period. This is a long period motion tuned to the natural period of the elastic building. It does not necessarily represent a real ground motion, but it provides a good test of the nonlinear features of the beam-column elements. Damping consists of stiffness and mass proportional parts and is set to 2% of critical at frequencies of 0.25 and 2 Hz.

Time histories of the horizontal deflections of floors 2, 3, 4 and 5 are shown in Figure 10. Both models show increasing amounts of lateral drift with time leading to collapse, with the plastic hinge model having a slightly greater rate of drift. The first significant drift of the plastic hinge model occurs during the negative response peak at 11 seconds, and this predisposes the model to drift in the negative direction. The fiber model experiences its first significant drift during the positive response peak at 13 seconds and subsequently drifts in the positive direction. Drifts in both models are associated with flexural yielding in the columns, and the smaller drift of the fiber model is in line with the greater moment capacity developed in the fiber elements as a result of strain hardening, as was demonstrated in Figure 8.

Shown in Figure 11 are ductility demands for the beams and columns. Ductility demand is defined as the kink rotation normalized by the factor \(6E_1/L\). For the fiber elements, an equivalent kink rotation is computed by subtracting from the actual end rotation that which would occur under elastic behavior with the same end forces and moments. The plotted values are the maxima to occur over the first 17 seconds involving the ends of all beams at a floor level, the tops of all columns at a story, and the bottoms of all columns at a story. Results for the plastic hinge and fiber element models are very similar.
The 20 second earthquake response of the 20 story steel frame was calculated at a time step of 0.05 sec on a DEC station 5000 with a 24 MIPS RISC processor. Required CPU times were 8 minutes for the PH1 model and 50 minutes for the F8 model.
Conclusions

The plastic hinge and fiber elements developed here are intended for steel structures. Both employ segmentation, geometric stiffness, configuration updating and elastic shear deformations. In the plastic hinge element, the yielding occurs in a plastic hinge which includes strain hardening, albeit in a crude way, and moment capacity reduction from axial load. Yielding, strain hardening and moment capacity reduction occur naturally in the fiber element, and a realistic stress-strain law which includes residual stress is used.

Differences in behavior of the plastic hinge and fiber elements are seen in situations where residual stresses and/or details of the yielding process are important. Examples include hysteretic behavior of a brace, collapse of a slender column where moment amplification is important, and cyclic flexure in the presence of large axial load. On the other hand, the two element types lead to similar predictions in the collapse rate and ductility demands for a code-designed high-rise building frame subjected to strong long-period ground motions. It is suspected that the good agreement seen may be fortuitous and further studies are needed. With inclusion of panel zone yielding, the distribution of ductility demand between panel zones and beam-columns may be sensitive to the choice of element type.

It is hoped that the results here will stimulate the use and further improvement of fiber models. One desirable feature to account for is local flange buckling. As the frame earthquake response problem showed, computational demands of fiber models are not unreasonable on today’s workstation computers.
References


1. Beam-column modeling details.
   a) Division of member into segments.
   b) Forces and displacements for a segment in global coordinate system.
   c) Local forces and displacements for a segment.
2. Fiber representation of a beam-column I-section for planar bending about the strong axis (a) or weak axis (b). To include residual stresses, + fibers are assigned an initial stress $+\sigma_{res}$, – fibers are assigned $-\sigma_{res}$, where $\sigma_{res} > 0$. 
3. Monotonic stress-strain loading curve for fiber model.
5. Comparison of experimentally determined hysteretic behavior of a steel bar under axial stress to model predictions.
6. Axial force-displacement of a pin-ended brace (W6×20, 3.07 m length).
   
a) Experiment (3)
b) Plastic hinge element, PH2
c) Fiber elements, F7 and F13
7. Lateral force-displacement of a cantilever beam (W18×46, 1.60 m length) with no axial load.
   a) Experiment (16)
   b) Fiber element, F8
8. Lateral force-displacement of a cantilever beam (W18×46, 1.60 m length) with axial load $P = 0.4 P_y$.
   a) Plastic hinge element, PH1
   b) Fiber element, F8
10. Time histories of floors 2, 3, 4 and 5 of the frame (horizontal displacement relative to base).
11. Ductility demands for the frame
   a) Plastic hinge model, PH1
   b) Fiber model, F8
Part 2 — Earthquake Collapse Analysis of Steel Frames

Introduction

The nonlinear response of tall steel moment resistant frame buildings to earthquake ground motions has been studied for many years with computer analysis (1,2,3). The investigation of ref. 3 which includes behavior of the column panel zone at the beam junction is typical of the state of the art. Such studies have provided confidence that tall steel buildings are safe against collapse during earthquake shaking at the levels anticipated by building codes. However, little information is available on the actual margin of safety, or, in other words, what level of shaking is necessary to cause collapse. The subject is complicated by the many factors which can contribute to collapse, such as member buckling, degradation under cyclic loading, and $P-\Delta$ effects. Cyclic degradation such as caused by local plate buckling is especially difficult to account for. As a start, the investigation reported here examines $P-\Delta$ type collapses initiated by excessive lateral sway. The ground motions employed may seem unrealistically severe, but they enable some sense to be gained of the margin of safety against this type of collapse.

The frame model adopted employs fiber members for the beams and columns and shear elements for the panel zones. In the fiber approach (4,5) a beam-column member is divided into fibers within the cross-section. Each fiber carries an axial stress which obeys a hysteretic stress-strain relation, and the particular hysteretic relation used here is original. Compared to the cheaper and more commonly used plastic hinge beam-column model (6), the fiber model better accounts for strain hardening and can represent axial-flexural yield interaction, residual stresses, and gradual spread of yielding within the cross-section and along the member length. Shear deformation of the panel zone follows a hysteretic relation, and the one used here has similarities to that employed for the fibers. The present frame model (7) includes geometric stiffness effects along with configuration updating. As such, moment amplification and buckling in the columns as well as frame $P-\Delta$ effects are included.

A 20 story steel moment-resistant frame taken from the literature (3) which meets the Zone 4 requirements of the Uniform Building Code is chosen for study. Two types of ground motions are selected: an oscillatory one with a significant long-period component and near-fault displacement pulses. A tall building may be most vulnerable to such motions. Because
of the importance of panel zone deformations, analyses are run with different panel designs (as was also done in ref. 3).
Planar behavior of a moment-resistant frame with nonlinear stiffness is considered. The frame model consists of panel zone elements and segmented beam-column members. A panel zone is the portion of the column within the depth of the connecting beams, and it is assumed to deform only in shear. The panel zone deformation is the shear strain $\gamma^{pz}$ which is given by

$$\gamma^{pz} = \theta^b - \theta^c$$  \hspace{1cm} (1)

where $\theta^b$ and $\theta^c$ are the end rotations of the connecting beams and columns, respectively (fig. 1). The angles $\theta^b$ and $\theta^c$ together with the horizontal translation $U$ and vertical translation $V$ of a frame node (fig. 1) are the degrees of freedom used in the equation of motion of the frame:

$$[M]\{\ddot{a}(t + \Delta t)\} + [C]\{\dot{a}(t + \Delta t)\} + \{R(t + \Delta t)\} = \{f(t + \Delta t)\}$$  \hspace{1cm} (2)

which is written at time $t + \Delta t$ and where $[M] = \text{mass matrix}$, $[C] = \text{damping matrix}$, $\{\dot{a}(t + \Delta t)\}$ and $\{\ddot{a}(t + \Delta t)\} = \text{velocity and acceleration vectors}$, $\{R(t + \Delta t)\} = \text{vector of stiffness forces corresponding to the displacements} \{a(t + \Delta t)\}$, and $\{f(t + \Delta t)\} = \text{load vector containing gravity and earthquake loads}$. The degrees of freedom at the interior nodes of the beams and columns are condensed out prior to assembly and so only the frame degrees of freedom appear in the equations of motion.

Eq. 3 is solved by a time-stepping scheme whereby the solution is advanced in time steps. Because the stiffness forces are nonlinear functions of the displacements, iterations are necessary in each time step. Linearization of the stiffness forces about a current approximation $\{a\}$ to the displacements at time $t + \Delta t$, with corresponding vector $\{p\}$, takes the form

$$\{R(t + \Delta t)\} = [K_T]\{\Delta a\} + \{R\}$$  \hspace{1cm} (3)

where $[K_T]$ is a tangent matrix and $\{\Delta a\}$ is a displacement increment. Substitution into eq. 2 along with time-stepping expressions based on constant average acceleration (8) gives

$$\begin{bmatrix} M + \frac{\Delta t}{2} C + \frac{\Delta t^2}{4} K_T \end{bmatrix} \{\Delta a\} = \frac{\Delta t^2}{4} \{f(t + \Delta t)\} - \begin{bmatrix} M + \frac{\Delta t}{2} C \end{bmatrix} \{a\}$$

$$- \frac{\Delta t^2}{4} \{R\} + [M] \left\{a(t) + \Delta t \dot{a}(t) + \frac{\Delta t^2}{4} \ddot{a}(t)\right\}$$

$$- [C] \left\{\frac{\Delta t}{2} a(t) + \frac{\Delta t^2}{4} \dot{a}(t)\right\}$$  \hspace{1cm} (4)
which is solved for \( \{\Delta a\} \). For the next iteration, \([K_T]\) and \(\{R\}\) are computed at the new approximation \(\{a + \Delta a\}\) by assembling contributions from the panel zones, beams and columns. Complete updating of the frame geometry is also performed and this fully accounts for \(P-\Delta\) effects. Iterations continue until convergence.

Mass and damping are only associated with the frame degrees of freedom; actually, mass is only included for \(U\) and \(V\). The damping matrix \([C]\) is expressed as

\[
[C] = \alpha_0[M] + \alpha_1[K_{EL}]
\]  
(5)

where \([K_{EL}]\) is the elastic stiffness matrix of the frame and \(\alpha_0\) and \(\alpha_1\) are chosen to give desired values of damping as percent of critical at two specified frequencies. The use of the elastic stiffness means that the damping forces are linear.

A panel zone contributes stiffness to \([K_T]\) as a rotational spring, affecting the degrees of freedom \(\theta^b\) and \(\theta^c\) at a frame node. The tangent spring stiffness \(k_T\) relates increments of the panel zone moment \(M^{pz}\) to increments of the shear strain \(\gamma^{pz}\), and is defined as (3)

\[
k_T = G_T d_c d_b t
\]  
(6)

where \(G_T\) = current tangent shear modulus, \(d_c\) = column depth, \(d_b\) = beam depth, and \(t\) = thickness of column web including doubler plates, if any. The tangent panel zone stiffness matrix is

\[
|K_T^{pz}| = \begin{bmatrix} k_T & -k_T \\ -k_T & k_T \end{bmatrix}
\]  
(7)

For assembly into \(\{R\}\), the panel zone vector \(\{R^{pz}\}\) contains the moments \(M^b\) and \(M^c\) which correspond to \(\theta^b\) and \(\theta^c\)

\[
\{R^{pz}\} = \begin{bmatrix} M^b \\ M^c \end{bmatrix}
\]  
(8)

where \(M^b = -M^c = M^{pz}\) and where \(M^{pz}\) depends on the history of \(\gamma^{pz}\) and is found using the hysteretic relation given in the next section. The current value of \(M^{pz}\) is the sum of \(M^{pz}(t)\) at the beginning of the time step and an increment \(\Delta M^{pz}\) computed using the cumulative increment of \(\gamma^{pz}\) since the beginning of the time step. Use of the cumulative increment avoids potential problems associated with artificial unloading.

For the beams and columns, the contributions to \([K_T]\) and \(\{R\}\) in eq. 4 are computed from separate analyses of each member which are performed after each iteration of the
frame analysis. These member analyses themselves involve iterations. The iterative scheme for a member is similar to that used for the frame except that mass and damping terms are not present and the loading comes from the end displacement histories computed in the frame iterations. Tangent stiffness matrices and vectors of stiffness forces for a beam or column are denoted as $\tilde{K}_T$ and $\{\tilde{R}\}$ where the $\tilde{}$ signifies that the degrees of freedom are those of the member. It is convenient to express $\tilde{K}_T$ and $\{\tilde{R}\}$, as well as the member displacements $\{\tilde{a}\}$, in partitioned form:

$$
\begin{bmatrix}
\tilde{K}_{II} & \tilde{K}_{IE} \\
\tilde{K}_{EI} & \tilde{K}_{EE}
\end{bmatrix}, \quad \{\tilde{R}\} = \begin{bmatrix} \tilde{R}_I \\ \tilde{R}_E \end{bmatrix}, \quad \{\tilde{a}\} = \begin{bmatrix} \tilde{a}_I \\ \tilde{a}_E \end{bmatrix}
$$

(9)

where $I$ and $E$ denote interior and end degrees of freedom. In an iteration during a member analysis, displacement increments of the interior nodes are computed from

$$
\tilde{K}_{II} \{\Delta \tilde{a}_I\} = -\tilde{K}_{IE} \{\Delta \tilde{a}_E\} - \{\tilde{R}_E\}
$$

(10)

where the end displacement increments $\{\Delta \tilde{a}_E\}$ resulted from the previous frame iteration and are nonzero for only the first member iteration. The member displacements are updated as $\{\tilde{a} + \Delta \tilde{a}\}$ and the iterations continue until convergence.

The particular segment formulation used here for a beam or column has two nodes, independent interpolation of displacements and rotation, center point Gauss quadrature, and shear deformations included (8). $\tilde{K}_T$ is assembled from the tangent stiffness matrices of the segments

$$
[K^S_T] = [T]^T [K^S_F + K^S_{SH}] [T]
$$

(11)

where $[K^S_F]$ contains contributions from fiber axial stiffness and geometric stiffness, $[K^S_{SH}]$ represents the shear stiffness which is taken to be elastic, and $[T] =$ transformation matrix from local to global degrees of freedom:

$$
[K^S_F] = \sum_{i=1}^{N_f} \begin{bmatrix}
\Lambda_i & -\Lambda_i \\
-\Lambda_i & \Lambda_i
\end{bmatrix}, \quad [\Lambda_i] = \frac{A_i}{\ell} \begin{bmatrix}
E_{Ti} & 0 & -h_i E_{Ti} \\
0 & \sigma_i & 0 \\
-h_i E_{Ti} & 0 & \frac{h_i^2}{2} E_{Ti}
\end{bmatrix}
$$

(12)

$$
[K^S_{SH}] = G A_s
$$

(13)
where the local segment degrees of freedom are ordered as $u_1, v_1, \theta_1, u_2, v_2 \theta_2$; $N_f$ = number of fibers; $A_i$ = area of fiber $i$; $E_{T,i}$ = current tangent axial modulus of fiber $i$; $\ell$ = original segment length; $h_i$ = distance of fiber $i$ above centroidal axis; $\sigma_i$ = current axial stress in fiber $i$; $G$ = elastic shear modulus; $A_s$ = effective shear area; and $\alpha$ = current segment chord rotation. $\{R^s\}$ is assembled from segment force vectors

$$\{R^s\} = [T]^T \begin{pmatrix} P, Q, M_1, -P, Q, M_2 \end{pmatrix}^T$$

where the axial force $P$, shear force $Q$ and end moments $M_1$ and $M_2$ are given by

$$P = \sum_{i=1}^{N_f} \sigma_i A_i \quad (16a)$$

$$Q = \frac{1}{2} \frac{G A_s}{\ell} \left( \theta_1 + \theta_2 - 2\alpha \right) \quad (16b)$$

$$M_1 = -\sum_{i=1}^{N_f} \sigma_i A_i h_i + \frac{1}{2} Q \ell \quad (16c)$$

$$M_2 = \sum_{i=1}^{N_f} \sigma_i A_i h_i + \frac{1}{2} Q \ell \quad (16d)$$

The fiber axial stress $\sigma_i$ depends on the history of fiber axial strain $\varepsilon_i$ and is found using the hysteretic rules of the next section. The strain $\varepsilon_i$ is computed as

$$\varepsilon_i = \frac{u}{\ell} - \frac{h_i}{\ell} \left( \theta_1 - \theta_2 \right) \quad (17)$$

where $u$ = axial elongation of the segment. The current value of $\sigma_i$ is the sum of $\sigma_i(t)$ at the beginning of the time step and an increment $\Delta \sigma_i$ computed using the cumulative increment of $\varepsilon_i$ since the beginning of the time step.
Geometric nonlinearity of a member is accounted for through the stress term in eq. 12 and through configuration updating. In each iteration, the locations of the nodes are updated, and these locations are used to update $u$ and $\alpha$. Thus, moment amplification, buckling, and geometric stiffness are included.

When convergence in a member analysis is achieved, $\hat{K}_T$ and $\{\hat{R}\}$ are computed for this converged state and the interior degrees of freedom are condensed out, producing

$$
[K_T^{bc}] = \begin{bmatrix} \hat{K}_{EE} & \hat{K}_{EI} \end{bmatrix} - \begin{bmatrix} \hat{K}_{II} \end{bmatrix}^{-1} \begin{bmatrix} \hat{K}_{IE} \end{bmatrix}
$$

(18a)

$$
\{R^{bc}\} = \{\hat{R}_E\} - \begin{bmatrix} \hat{K}_{EI} \end{bmatrix} - \begin{bmatrix} \hat{K}_{II} \end{bmatrix}^{-1} \{\hat{R}_I\}
$$

(18b)

which are the tangent stiffness matrix and vector of stiffness forces for a beam or column written for the six end degrees of freedom $a_1, a_2, \ldots, a_6$ (fig. 1). $\{R^{bc}\}$ contains the end forces $(R_1, R_2, \ldots, R_6)$. Before $[K_T^{bc}]$ and $\{R^{bc}\}$ can be assembled into $[K_T]$ and $\{R\}$ of eq. 4, $[K_T^{bc}]$ and $\{R^{bc}\}$ must be transformed from the member degrees of freedom $a_1, a_2, \ldots, a_6$ to the frame degrees of freedom $U_i, V_i, \theta_i^b, \theta_i^p, U_j, V_j, \theta_j^b, \theta_j^p$, which also accounts for the finite dimensions of the panel zone. The transformation process uses a matrix $[S^b]$ for a beam and $[S^c]$ for a column. Taking a beam as an example,

$$
[K_T^b] = [S^b]^T [K_T^{bc}] [S^b]
$$

(19a)

$$
\{R^b\} = [S^b]^T \{R^{bc}\}
$$

(19b)

where

$$
[S^b] = \begin{bmatrix} 1 & 0 & 0 & \frac{da}{2} \sin \theta_i^b & 0 & 0 & 0 & 0 \\
0 & 1 & 0 & -\frac{da}{2} \cos \theta_i^b & 0 & 0 & 0 & 0 \\
0 & 0 & 1 & 0 & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 1 & 0 & 0 & \frac{da}{2} \sin \theta_j^b & 0 \\
0 & 0 & 0 & 0 & 1 & 0 & \frac{da}{2} \cos \theta_j^b & 0 \\
0 & 0 & 0 & 0 & 0 & 1 & 0 & 0 \end{bmatrix}
$$

(19c)

Note that the beams and columns connect to the midpoints of the panel zone edges. It is $[K_T^b]$ and $\{R^b\}$ which are assembled into $[K_T]$ and $\{R\}$.

Use of the tangent stiffness in eqs. 4 and 10 can occasionally lead to convergence difficulties, especially during load reversals which are frequent for earthquake loading. To avoid such difficulties, the first iteration at both the frame and member levels uses the elastic stiffness rather than the tangent stiffness. If divergence is detected during the subsequent tangent iterations, then several more elastic iterations are performed. Also, lower limits on $G_T$ and $E_T$ of about 4% of the elastic values are imposed.
Hysteretic Behavior

A number of hysteretic models exist for steel bars under axial stress, two noteworthy ones being refs. 9 and 10. However, there is still need for improvement, and the present model is an attempt to achieve some optimization among simplicity, efficiency and accuracy. Characteristics include a realistic monotonic loading curve, elliptical unloading and reloading curves, explicit computing of stress and tangent modulus given strain, backbone curves as strain-shifted monotonic loading curves, and absence of any parameters except those used to define the monotonic loading curve. The model is used for the fibers in a beam-column member and, with certain modifications, for the panel zone as well.

The monotonic loading curve for a fiber, assumed the same in tension and compression, is defined by elastic modulus \( E \), yield stress \( \sigma_y \), ultimate stress \( \sigma_u \), strain \( \varepsilon_{sh} \) at initiation of strain hardening, strain \( \varepsilon_u \) at ultimate stress, rupture strain \( \varepsilon_r \), and initial strain hardening modulus \( E_{sh} \) (fig. 2a). The curved part is a cubic ellipse centered at \( \varepsilon_u, \sigma_0 \) with radii \( a \) and \( b \):

\[
\frac{(\varepsilon - \varepsilon_u)^3}{a^3} + \frac{(\sigma - \sigma_0)^3}{b^3} = 1 .
\]

Parameters \( \sigma_0, a \) and \( b \) are determined from \( \varepsilon_{sh}, \varepsilon_u, E_{sh}, \sigma_y \) and \( \sigma_u \). The slope \( E_{AB} \) is tangent to the curve at \( B \) and plays a role in the hysteretic behavior. A fiber can be given a residual stress by strain shifting the monotonic loading curve until the stress at zero strain equals the desired residual stress.

The behavior during loading and unloading makes use of two backbone curves — one for tensile stress states and one for compressive stress states. These curves have the same shape as the monotonic curve, and they shift along the strain axis. The compression backbone curve shifts when the tension one is being followed, and vice versa. The initial backbone curves coincide with the tension-compression monotonic loading curve. Rules for cyclic loading will be demonstrated for the fiber with reference to the stress path in fig. 3.

When a reversal occurs from a backbone curve, a linear path at slope \( E \) is followed to the \( \sigma=0 \) axis (segments 2–3, 5–6 and 8–9 in fig. 3) where a shifted backbone curve is encountered. The path is then that of a cubic ellipse (3–4, 6–6A–7, 9–10) with initial slope \( E \) and a target point back on the shifted backbone curve, after which this backbone curve is followed (4–5, 7–8, 10–11). The location of the target point (4,7,10) is that of the previous turning point (2,5,8) transferred to the current backbone curve, and the approach
slope to the target point is either \( E_{AB} \), the tangent to the backbone curve, or a zero slope depending on whether the target point lies along \( AB \), \( BC \) or \( CD \) in fig. 2a, respectively, as do points 4, 7 and 10, respectively. An exception to the above occurs when the initial yield excursion (segment 1–2) is too short; in which case, the following linear unloading segment (2–3) must be extended beyond the \( \sigma=0 \) axis so that the subsequent cubic ellipse can be completed with the desired slope.

The path followed when reversal occurs from other than a backbone curve (point 6A in fig. 3) consists of loops which close fully and have matching halves. Thus, each of the half loops 6A–6B–6C, 6C–6D–6E, 6E–6F–6G, 6G–6H–6E is a subsegment of 5–6–6A beginning at point 5. Once an inner loop is completed (point 6E), then the outer loop is continued until either the outer loop is completed (point 6A) or another reversal occurs. Eventually, a backbone curve is reached (6A to 7) and followed.

For a panel zone the hysteretic relation is for moment \( M^{pz} \) and shear strain \( \gamma^{pz} \). The monotonic loading curve (fig 2b) begins with a linear segment to 0.8 of the yield moment \( M_y^{pz} \) suggested in ref. 3:

\[
M_y^{pz} = \tau_y d_c d_b t
\]

where the shear yield stress is \( \tau_y = \sigma_y / \sqrt{3} \). Following the linear segment is a quadratic ellipse that is tangent to the preceding linear segment and reaches a zero slope at \( \gamma_u = 100 \gamma_y, M_y^{pz} = 2.35 M_y^{pz} \), where \( \gamma_y \) is the shear yield strain. This form of the monotonic loading curve is chosen to match a set of experimental data summarized in ref. 11, which is reproduced in fig. 2b. Because of a lack of information, the curve is taken to be flat beyond \( \gamma_u \). Rules for loading and unloading of the panel zone are similar to those of a fiber except that linear segments have slope \( Gd_c d_b t \), and the approach slope to a target point is always tangent to the backbone curve. Although a quadratic ellipse is used in the monotonic loading curve, the curved parts of the other curves are cubic ellipses.

The performance of the hysteretic relations is illustrated in fig. 4 where comparisons to three sets of experimental data are made. The data in parts a and b are for a steel bar under axial stress and include two cyclic load paths (12,13). The fiber model was calibrated with actual or estimated parameters \( E, E_{sh}, \sigma_y, \sigma_u, \varepsilon_{sh} \) and \( \varepsilon_u \), and it was subjected to the axial strain history of the experiment. Agreement is very good. The data in part c is a panel \( M^{pz} - \gamma^{pz} \) curve from a test on a beam-to-column subassemblage (14). The panel zone model was calibrated with \( M_y^{pz} \) (e.g., \( \tau_y, d_b, d_c \), and \( t \)) and \( G \) of the experiment and subjected to the \( \gamma^{pz} \) history of the experiment. Again, the agreement is very good.
Results

The building analyzed is a 20 story steel structure that has been previously studied in
the literature (3). The plan of the building is rectangular with three bays in one direction
and four bays in the other (fig. 5). Design gravity loads are 4.79 kPa dead load for the
floors and roof, 3.84 kPa live load for the floors, 0.96 kPa live load for the roof, and 1.67
kPa cladding load over the exterior walls. The steel type is A36.

The building meets the Zone 4 requirements of the 1991 Uniform Building Code.
Lateral resistance is provided by moment resistant frames on the perimeter, all interior
beam-to-column connections being simple. Consideration of earthquake here is only for
the short direction of the building, and this allows attention to focus on the exterior three-
bay frames. In the design of each of these frames, gravity loads are taken from a half-bay
tributary width while the masses used to compute the seismic loads are those of half the
building.

Dimensions and member designations for the exterior three-bay frame are shown in
fig. 6. The code requires that the moment capacities of the columns at a joint, as reduced
by the presence of axial force, exceed the moment capacities of the beams at a joint. For
each joint, fig. 7 plots the sums of the plastic moment capacities at a joint for the columns
($\sum M_p^c$) and the beams ($\sum M_p^b$). Plastic moment capacities are based on $\sigma_y$. The excess
of $\sum M_p^c$ over $\sum M_p^b$ is enough to account for the code reduction in $\sum M_p^c$ from axial
force. Also shown in fig. 7 is the yield moment $M_{yPZ}$ of the panel zones computed from
eq. 21 using the original column web without doubler plates. This design, referred to as
D1, meets the minimum UBC panel strength requirements. Since $M_{yPZ}$ for design D1 at
a joint is significantly less than $\sum M_p^b$, two other panel designs using doubler plates are
considered. In design D2, the panel thickness is computed from eq. 21 with $M_{yPZ}$ taken
as $0.8 \sum M_p^b$, but not less than the D1 thickness. In design D3, $M_{yPZ}$ equal to $1.5 \sum M_p^b$
is used which is sufficient to give elastic panel behavior. Resulting values for $M_{yPZ}$ under
designs D2 and D3 are also shown in fig. 7.

A nonlinear seismic analysis of the frame is conducted by the procedure described
earlier. In this analysis, gravity loads and masses are computed using the full roof, floor
and cladding dead loads, no roof live load, and a reduced floor live load of 0.96 kPa.
Gravity loads and masses for the vertical degrees of freedom are computed with the half-
bay tributary width, while masses for the horizontal degrees of freedom are those of half
the building (fig. 5). The damping matrix is computed from eq. 5 with $\alpha_0$ and $\alpha_1$ chosen to give 2% damping at 0.25 Hz and 2 Hz.

An additional factor which must be included is the $P-\Delta$ effect from the interior building weight. This is accomplished by applying at each floor of the external frame, say floor $i$, a time-varying horizontal force $F_i(t)$ given by

$$F_i(t) = \frac{(W + W_i) \Delta_b(t)}{h_b} - \frac{W \Delta_a(t)}{h_a}$$

(22)

where $W = \text{weight of building above the mid-story level above floor } i \text{ within the } 1 \frac{1}{2} \text{ bay wide interior tributary area shown in fig. 5, } W_i = \text{weight of building within the same tributary area but between the mid-story levels above and below floor } i, \Delta_a(t) \text{ and } \Delta_b(t) = \text{lateral deflections at time } t \text{ in the stories above and below floor } i, \text{ and } h_a \text{ and } h_b = \text{heights of the stories above and below floor } i.$

Material properties are $E = 200,000 \text{ MPa}, \ G = E/2.6, \ E_{sh} = 4000 \text{ MPa, } \sigma_y = 248 \text{ MPa, } \sigma_u = 400 \text{ MPa, } \epsilon_{sh} = 0.012, \epsilon_u = 0.160 \text{ and } \epsilon_r = 0.300$. The particular fiber layout shown in fig. 1 is used. Individual fibers are assigned a residual stress of either $+\sigma_{res}$ or $-\sigma_{res}$ according to the sign indicated, where $\sigma_{res} = 41 \text{ MPa}$. The beams and columns are modeled with eight segments with lengths, from one end to the other, of 0.03L, 0.06L, 0.16L, 2 @ 0.25L, 0.16L, 0.06L, 0.03L. Studies which verified the adequacy of the eight-segment discretization were performed (not presented here).

The ground motions employed in the frame analysis are chosen to define upper limits of what the frame can withstand and, as a result, may be unrealistic. In interpreting the results, it is important to keep in mind that structural degradation is not included, which is unconservative as degradation could cause collapse at less severe ground motions. Having a similar effect are the neglect of torsion in the building and the use of only a single horizontal component of ground motion. On the other hand, the neglect of non-structural components is conservative. Foundation behavior and soil-structure interaction are not included as well. Two types of ground motion are employed: an actual earthquake motion with strong long-period oscillations and simulated displacement pulses intended to represent the fault slip near a strike-slip fault.

The oscillatory earthquake motion selected is the S90W component of that recorded on the ground floor of a building at 8244 Orion Blvd., Los Angeles, during the 1971 San Fernando earthquake, but scaled to a peak acceleration of 0.5g (original peak 0.13g). Figures 8 and 9 show this earthquake motion and its pseudo acceleration response spectrum.
Sa vs. T1, where T1 is the fundamental period. For reference, the fundamental periods of
the elastic building are T1D1 = 4.06 sec and T1D3 = 3.77 sec. In addition to the amplitude
scaling, the time axis of the earthquake motion is expanded by a factor f whose value
depends on the case being analyzed. This multiplies the duration of the loading by f,
and the ground velocity and displacement by f and f2, respectively. A particular case
is defined by the building being analyzed (D1 or D3) and a point on the Sa-T1 curve
which is shifted to either T1D1 and T1D3 through the time axis scaling. Two points on the
Sa-T1 curve are used (Fig. 9): point A at Sa = 0.81g, T1 = 3.26 sec and point B at Sa
= 0.40g, T1 = 3.77 sec. Thus, case D1-SFA is an analysis of building D1 subjected to the
San Fernando ground motion of Fig. 8 but with the time axis scaled by f = \frac{4.06}{3.26} = 1.25
which moves point A in fig. 9 to the period T1D1. Other cases are D3-SFA, D1-SFB and
D3-SFB, and all four are listed in Table 1 together with the ground motion data pertinent
to each.

Pulse ground motions are either parallel to the fault (fig. 10a) or normal to the fault
(fig. 10b). A particular case is defined by the building being analyzed (D1 or D3), the
pulse type (PP or NP for parallel or normal), the fault displacement d_{max} (1.5 m or 3.0
m), and the maximum fault velocity v_{max} (1 m/sec or 2 m/sec). Twelve cases are listed
in Table 1. A fault velocity of 2 m/sec is a high value which is thought to be a reasonable
upper bound for use in engineering evaluation (15).

Results are shown in figs. 11 through 17 as horizontal displacement time histories of
five floors (numbers 2,3,4,5 and roof) and maximum ductility demands. For a panel zone,
the ductility demand is defined as the absolute value of the difference between \gamma P and the
recoverable amount \frac{M^P}{G d_c d_b t}, divided by the yield value \gamma Y = \frac{\sigma_y}{G}. For a beam or
column, the ductility demand is defined as the absolute value of equivalent kink rotations
divided by the value \frac{M_P}{\sigma_y L}, where M_P is the plastic moment capacity based on \sigma_y. The
equivalent kink rotations are expressed as a_3 - a'_3 and a_6 - a'_6 where a_3 and a_6 are the
actual end rotations and a'_3 and a'_6 are corresponding values computed from an elastic
analysis of the member using the same end forces and moments.

Under the SFB ground motions, building D3 collapses while D1 does not (Fig. 11).
In the case of D3, significant yield excursions occur at about t/f = 17 sec and 23 sec;
thereafter, the lateral offset increases with each cycle of response until a P-Ä instability
occurs at about t/f = 48 sec. The story displacements prior to the instability are quite
large, for example, exceeding 50 cm in the first story. The largest story displacements in
the D3 response occur in the lowest few stories where significant flexural yielding of the
columns occurs (fig. 12b). Even though the flexural strengths of the columns at a joint exceed those of the beams, significant column hinging still takes place. The unreinforced panel zones in D1 cause smaller moments to be applied to the columns, reducing the amount of column hinging enough so that no collapse mechanism occurs. The panel zones of D1 also participate in the energy dissipation, especially the interior panel zones, and reduce the ductility demands on the beams and columns (fig. 12a).

The SFA ground motions (results shown in fig. 13) are more severe because of the greater $S_a$ value at point A compared to point B (fig. 9). Building D3 collapses earlier, at about $t/f = 31$ sec, and in the opposite direction as the previous case. Building D1 also fails but it survives longer than D3, until $t/f = 36$ sec. The improved performance is attributable to the column hinges not forming as readily with the weaker panel zones.

Responses to the 1.5 m pulses with $v_{max} = 2$ m/sec appear in fig. 14. No significant differences occur between D1 and D3, and the residual story displacements amount to about 20 cm for the first story and about 15 cm each for the two above. Responses to the 3.0 m pulses with $v_{max} = 2$ m/sec are much more severe (fig. 15). While no $P-\Delta$ instability occurs, large residual displacements develop in the bottom few stories for the parallel pulse, more so for D3, and also for D3 under the normal pulse. The better performance of building D1 can again be attributed to less column hinging compared to that in D3. A sample ductility plot for the 3.0 m normal pulse (fig. 15) shows the greater column hinging for D3 as well as a considerable amount of the yielding occurring in the middle height of the building for both D1 and D3. Decreasing the maximum ground velocity to 1 m/sec for the 3.0 m pulse significantly reduces the story offsets (fig. 17). For these less severe motions, the stronger building D3 behaves better than D1.

The undesirable flexural yielding in the columns observed in this study deserves a comment. While the ground level hinges are expected because of the lateral support condition there, the flexural yielding in the columns at higher elevations may be surprising to some since the code requirement that column strength exceed beam strength at a joint is satisfied. Part of the explanation is that the reduction in column moment strength from axial force can be larger than accounted for in design because the earthquake axial forces can exceed the design ones. A more important effect has to do with the wave propagation nature of the building response. For example, a shear wave propagating up the building causes the column moment below a joint to peak before the column moment above a joint, and the difference between the two from this time delay can be enough for a hinge to form below the joint if the ground motion is strong.
The large ductility demands seen in the figures together with the cyclic nature of the response indicates that structural degradation should be included in analyses for strong earthquake motions. Local flange buckling at the column hinge locations just above the ground floor level may be especially significant. Another factor is that large axial tensions, reaching values considerably in excess of the initial compressions, occur in the exterior columns (fig. 18). Unless special provision has been made in design, these tensions may lead to some foundation pull-out or uplift, which should also be included.
Conclusions

The ground motions required to collapse a typical 20-story code designed steel frame building of regular geometry appear to be quite severe. The question of how realistic are such motions regarding long-period content and near-fault effects for large earthquakes is not an easy one to answer because of the current sparsity of knowledge in this area. Even if the ground motions used here are deemed to be improbable, the results are still useful as they provide some indication of what a tall building is capable of withstanding.

The frame model employed here is state of the art in many ways: fiber beams and columns, panel zone elements, realistic hysteretic relationships, and geometric nonlinearity that includes configuration updating. Yet, structural degradation, effects of nonstructural elements, foundation behavior, soil-structure interaction, multi-component ground motion, and torsion are not included, and it is difficult to assess the effect of these omissions. For the ground motions employed, the large ductility demands and the cyclic nature of the loadings means that structural degradation could be a very important factor. One critical location is just above the ground floor level where column hinging is most pronounced. Also, the large tensions which develop in the exterior columns indicate that foundation uplift should be considered.

Panel zones designed to yield may have two advantages regarding resistance to strong earthquake motions. First, such panel zones reduce the ductility demands on the other structural elements by dissipating energy themselves. Second, being relatively weak, such panel zones act as fuses and limit the column moments, making it more difficult to form a collapse mechanism involving column hinges. Of course, this could also be accomplished by reducing the strengths of the beams, but at the expense of a greater reduction in the lateral stiffness of the frame. Also, panel zones exhibit a very stable hysteresis, and it is desirable for them to participate in the energy dissipation. Strengthening the columns also makes it more difficult to form a collapse mechanism involving column hinges, and this would be most beneficial in the lower stories which appear to be most vulnerable to such behavior. Just having column strength exceed beam strength is not sufficient to prevent column hinging when the ground motions are strong.

Numerical solutions of the type presented here are practical on modern workstation computers. For example, each of the 2500 time-step analyses for the modified San Fernando earthquake motions took about three hours of CPU time on a DEC station 5000 with a 24 MIPS RISC processor.
<table>
<thead>
<tr>
<th>Case</th>
<th>f</th>
<th>$a_{\text{max}}$ (g)</th>
<th>$v_{\text{max}}$ (m/sec)</th>
<th>$d_{\text{max}}$ (m)</th>
<th>Duration (sec)</th>
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<tbody>
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**Table 1.** Ground motion data for each case analyzed. $a_{\text{max}} = $ peak acceleration; $v_{\text{max}} = $ peak velocity; $d_{\text{max}} = $ maximum peak-to-peak displacement (SF) or fault displacement as shown in Fig. 10 (PP and NP).
References


15. Heaton, T.H., personal communication.
1. Structural model
2. Monotonic loading curves for the fiber (a) and panel zone (b)
3. Hysteretic stress-strain behavior for the fiber model
4. Comparison of experimentally determined hysteretic behavior to model predictions: 
a and b) steel bar under axial stress (fiber model), c) panel zone from a test on a 
beam-to-column subassemblage (panel zone model)
5. Plan view of building
6. Exterior frame
7. Moment strengths of columns, beams and panel zones at the joints of the frame
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9. Pseudo acceleration response spectrum of the ground motion shown in fig. 8. In the analyses, points A and B are both shifted to periods $T_1^{D1}$ and $T_1^{D3}$ through time axis scaling.
10. Simulated near-fault ground motions in directions parallel and normal to a strike-slip fault.
11. Computed time histories of floors 2, 3, 4, 5 and roof (horizontal displacement relative to base) for cases D1-SFB and D3-SFB
12. Maximum ductility demands up to the times indicated in Fig. 11 for cases D1-SFB and D3-SFB. Dashed: column lines A and D (exterior). Solid: column lines B and C (interior)
13. Computed time histories of floors 2, 3, 4, 5 and roof (horizontal displacement relative to base) for cases D1-SFA and D3-SFA
14. Computed time histories of floors 2,3,4,5 and roof (horizontal displacement relative to base) for cases D1-PP1, D3-PP1, D1-NP1 and D3-NP1
15. Computed time histories of floors 2, 3, 4, 5 and roof (horizontal displacement relative to base) for cases D1-PP2, D3-PP2, D1-NP2 and D3-NP2
17. Computed time histories of floors 2, 3, 4, 5 and roof (horizontal displacement relative to base) for cases D1-PP3, D3-PP3, D1-NP3 and D3-NP3
18. Computed time histories for axial force in the first story column on line A (parts a and c) and line D (part b)
Part 3 — Earthquake Analysis of the OJI Paper Co. Building

The OJI Paper Company building is a 15-story steel frame structure designed by Kajima Corporation. Modelling of this building is very similar to that for the building in Part 2 in that lateral loads are assumed to be carried entirely by the two external frames; the one analyzed is shown in fig. 1. Gravity loads and masses for the vertical degrees of freedom are computed from the beam reactions based on the typical floor plan shown in fig. 2. Masses for the horizontal degrees of freedom are from half the building. Vertical $P-\Delta$ loads from the interior building weight are included. Combined dead and live floor loads used for computing the above were supplied by Kajima Corporation: 12.5 kPa for the 2nd floor, 8.5 kPa for the 3rd floor, 8.9 kPa for the 4th floor, 5.2 kPa for floors 5 through 15, and 14.1 kPa for the roof.

The closely spaced columns suggest a "tube effect" but this was ignored in the analysis. A special design feature in the building is the presence of shear links in the beams to dissipate energy. However, in the analysis the beams were treated as continuous without shear links. Also, the braces were taken to be pin-ended.

As seen in fig. 1, the frame contains a great number of members. Because of a large computational effort if fiber elements were used, it was decided to first analyze the frame using plastic hinge elements, and then try a fiber analysis if the situation warranted. As it turned out, the frame proved to be very strong and exhibited only a small amount of yielding, so the fiber analysis was not performed. Material properties are $E = 200,000$ MPa, $\sigma_y = 324$ MPa, $\kappa = 0.025 \cdot (6EI/L)$ for the beams, columns and braces and $G = E/2.6$ and $\tau_y = \sigma_y/\sqrt{3}$ for the panel zones. No doubler plates for the panel zones are present, and this corresponds to design D1 from Part 2.

Two ground motions are employed: the S00E component El Centro 1940 and the S69E component Taft 1952 (fig. 3). These are scaled to peak accelerations of 0.67g, the original peaks being 0.35g and 0.18g for El Centro and Taft, respectively. Pseudo acceleration response spectra are shown in figs. 4 and 5 where the fundamental period of the elastic building, $T_1^{D1} = 1.72$ sec, is plotted for reference.

Results are shown in Figures 6 and 7 as displacement time histories relative to the base for floors 2, 4, 6, 8, 10, 12, 14 and roof and ductility demands for the beams, columns and panel zones. The ductilities are as defined in Part 1. From fig. 6, any permanent lateral offset over the height of the building is small, less than five centimeters. Yielding is
confined mostly to the panel zones above the braced stories, and the maximum ductility demand is less than three (fig. 7). The braces behave elastically with axial compressions (not shown) reaching only about half of the Euler buckling loads. Thus, the building is very strong.
1. External frame.
2. Typical floor plan.
3. Ground motions used in the analyses, scaled to a peak acceleration of 0.67g.
4. Pseudo acceleration spectrum for the scaled El Centro ground motion.
5. Pseudo acceleration spectrum for the scaled Taft ground motion.
6. Computed time histories of floors 2, 4, 6, 8, 10, 12, 14 and roof (horizontal displacement relative to base).