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FRICTION-DAMPED MOMENT FRAMES

DOUGLAS WAY, EUGENE LOH

Base Isolation Consultants, Inc.

ABSTRACT

Friction-Damped Moment-Resisting Frames are discussed as an alternative to steel ductile moment-resisting frames. The design involves replacing all full penetration welds with bolted friction connections, thereby eliminating the problem of failures at welded connections as exhibited during the Northridge earthquake. Energy dissipation is achieved at the friction interface in lieu of inelastic behavior of the frame. Analytical methods that perform an energy accounting are discussed.

KEY WORDS

Moment frame; friction; damping;

INTRODUCTION

Moment-resisting frames have always been the desired structural lateral resisting elements in buildings. Architects and owners prefer the system because of the lack of bracing elements such as shear walls and braced frames, thereby permitting freedom in space planning and the design of exteriors. Usually vertical load carrying columns are selected as part of the moment-resisting frames. The dimensions of the moment-resisting frame columns are normally increased to meet the building code's drift requirement. Architects customarily consider the increase in column sizes a good exchange for bracing elements.

In areas of high seismicity, building codes mandate that moment-resisting frames be designed for ductility, and categorizes such a design in the lowest base shear group. Building codes use a 475 year return period event as the design basis earthquake (DBE), and factor the actual forces downward according to building flexibility and the selected structural lateral-resisting system. Flexibility translates to a lower fundamental frequency that for stiff soil sites results in attenuation of earthquake energy in the high frequency domain. Ductile detailing allows the structure to dissipate energy while entering the inelastic state. For ductile moment-resisting frame systems, building codes allows the use of a R_w of 12, which is the design base shear reduction factor that takes into account energy dissipation. Using the simplified formulas in the building codes for moment-resisting frames with ductile detailing, one usually arrives at a design base shear of less than 0.1g for non-essential facilities.

Inspections after the Northridge Earthquake of 1994 revealed failures in steel moment-resisting frame connections, some of which were just recently constructed. To date more than 120 buildings have been reported in the Northridge area to exhibit failures in these connections. Data from instrumentation maintained by the California Division of Mines and Geology [1] indicate that horizontal ground accelerations within a 50 km radius of the epicenter and at the sites of these types of buildings ranged, with few exceptions, generally from 0.2g to 0.5g, close to the building codes' prescribed DBE ground motions. Most of the failures occurred in welded connections between the beam flanges and the columns, which are typically achieved by full penetration field welding. The proper performance of such welds depends on many factors, some of which include the metallurgy of the base metal and welding electrodes, the preheating process necessary for such installations, and workmanship. Proper preheating becomes a problem since in most instances welding is performed several stories up with no scaffolding. Field inspection is difficult for the same reason.

In an attempt to develop a retrofit design for these failed welded moment-resisting connections, the steel industry has recently performed testing of different retrofit configurations using full size welded joint specimens [2]. The testing performed to date has focused on the ability of the welded joints to remain intact under 0.035 to 0.050 radians of rotation. Energy dissipated was evidenced by test results in the form of areas enveloped by hysteresis loops while the joints entered the inelastic state. The gage of performance in evaluating the designs has been the degree to which the joints can meet the 0.05 radians criteria without brittle fracture and the amount of energy dissipated during cyclical testing.

It appears that the issues pertaining to this problem relate to both design and installation. First the methodology associated with arriving at the design base shear needs to be revisited, including the use of a R_w of 12. If the factor is used to account for energy dissipated, a more objective approach should be used. Second is that of fabricating the design. The International Conference of Building Officials has strengthened the requirements in the Uniform Building Code by now requiring testing of all steel moment-resisting joints; however, the field conditions and human factors are still of concern to many. In attempting to address these issues, the authors have returned to basics associated with steel moment-resisting frame design.

ENERGY DISSIPATION

For the past 30 years, the concept of ductility has been used in earthquake-resistant design. Since ductility is a measure of the degree a structure can enter the inelastic range and still remain stable, extreme plastic behavior is expected and encouraged. Earthquake energy introduced into the structure is dissipated via the yielding of the structural elements. We as designers rationalize that the building design base shear can be reduced to account for ductile behavior of the structural system.

An improvement would be to dissipate the same degree of energy or more without incurring damage to the structure in the process, which is the by product of ductility. The structural members enter the inelastic state with resulting permanent deformations. Incorporating devices within the structure to dissipate earthquake energy introduced into the structure has been referred to as Passive Energy Dissipation. Few new buildings have been built and existing buildings retrofitted using this strategy. While there are many devices to choose from, the common denominator for all the devices is that they transform earthquake energy into heat which is conducted into the structure.

In moment-resisting frames, energy dissipating features can be incorporated into the beam joints. See Figure A. As the frame deflects, friction dampers that are configured to dissipate energy as the column joints rotate come into play. This is achieved by allowing slippage of the beam flanges. By configuring a friction interface where the slippage takes place, energy dissipation is achieved through joint rotation. The friction interface consists of lead-bronze against stainless steel, and is prestressed with bolts. Strain in the bolts can be measured to check the prestress force. Lead-bronze is desirable in a friction damper design in that the material continually self-lubricates with a mixture of lead and its oxide when rubbing against a metal surface. This phenomenon results in a predictable value for the coefficient of friction between the two surfaces that is independent of velocity, which is desirable from an analytical and performance point of view. The actual value for the coefficient of friction will depend on the roughness and type of material in contact with the lead-bronze and the surface treatment of the lead-bronze. Through proper placement of tension in the clamping bolts, one can configure the slip load in the beam joints. The slip load at the top flange should be the same at the bottom flange at a particular beam joint, but may be different at various locations of the frame. Figure B shows recent force vs displacement test results for a lead/bronze against stainless steel friction interface under a load of 400 kips. A pattern is tooled into the lead-bronze surface to eliminate the slip-stick phenomenon common in most friction assemblages. The slip loads selected should not allow any joint movement under service load conditions with a factor of safety. Configuration of the slip loads will depend on the range of ground motions used for design.

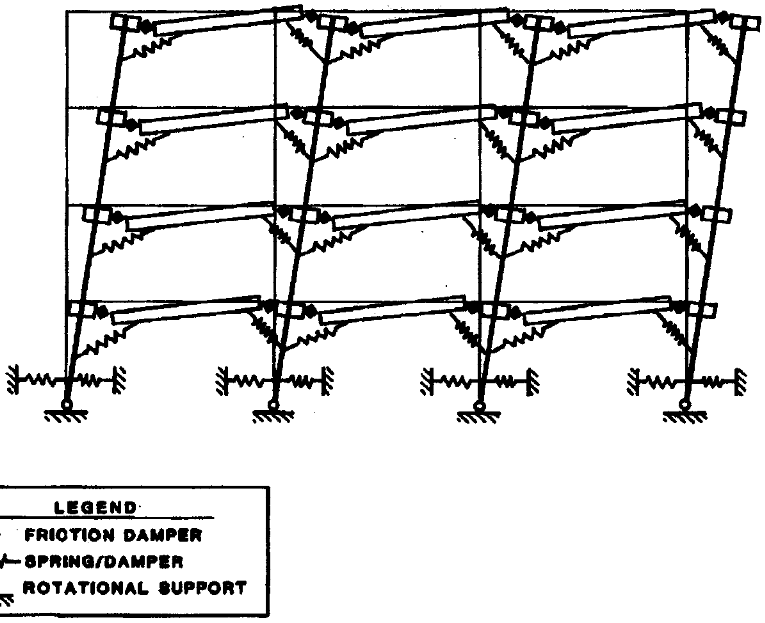

Figure A - Schematic of Friction-Damped Moment Frame

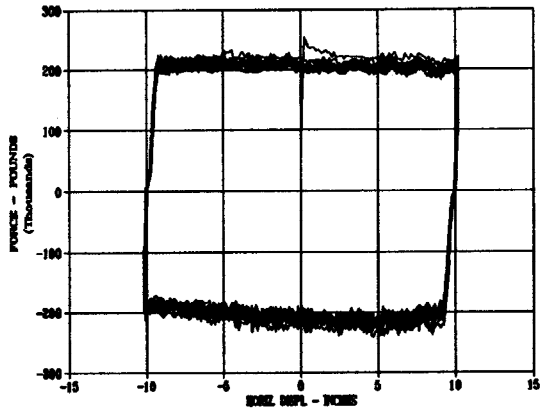

Figure B - Test Results of Lead/Bronze-Stainless Steel Interface Under 400 Kips

Thus Friction-Damped Moment-Resisting Frames involve the placement of friction dampers at strategic locations under controlled conditions within moment-resisting frames for the purpose of energy dissipation in providing earthquake protection for the building. The frame needs to be configured such that the deformations in the structure occur mainly in the assigned beam joints where the friction dampers are located. The columns should be stiff to minimize the flexural distortions in the members. The beam joints can be designed for a rotational value of 0.05 radians. Rotational flexibility also needs to be introduced at the base of the columns. The magnitude of the rotational flexibility of the column bases affects the horizontal period of the structure. The degree of flexibility needs to be designed in concert with the beam joints for maximum effectiveness, and consideration of the capability of the columns to resist moments due to P-delta effects. Drift capabilities of the architectural elements should be considered in addition. Friction dampers can also be added to the column bases. For the purpose of providing restoring features after joint slippage, springs are added at the bases and knees of the frames (See Figures C, D, E and F). Viscous dampers can be added within the springs if velocity induced effects are to be minimized. Such a feature may be desirable for sites within close proximity to earthquake faults where the potential exists for high velocities.

STRUCTURAL CONFIGURATION

One of the more economical designs to achieve column stiffness is in the use of concrete-filled steel pipe columns (See Figure C), but standard wide-flange steel sections can also be used. The steel pipe confines the concrete in hoop tension. This confinement significantly increases the compression strength of the concrete. Flexural capacity exists through composite action of the steel and concrete. A ring connection (See Figure D) is introduced at the column joints to connect the intersecting beams, and to accommodate the friction dampers. The cutting of the rings can be done with programmable torches if the number of rings are high. The pipe column with ring configuration allows the creation of moment connections which is symmetrical in two directions. All field connections of beams to columns are achieved with bolts. Full penetration field welds associated with conventional moment-resisting frame construction have been

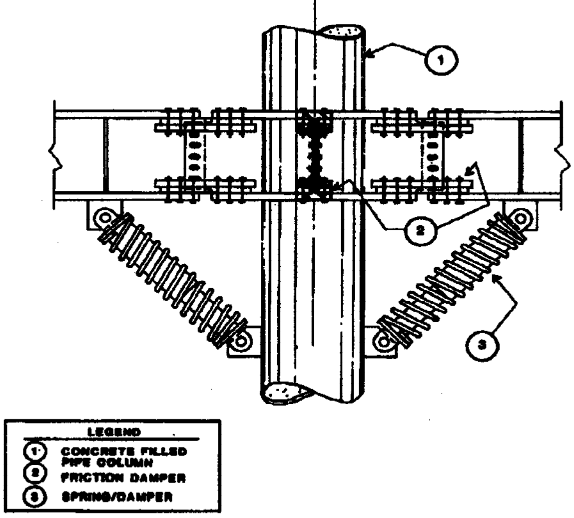

Figure C - Friction-Damped Moment Conn.

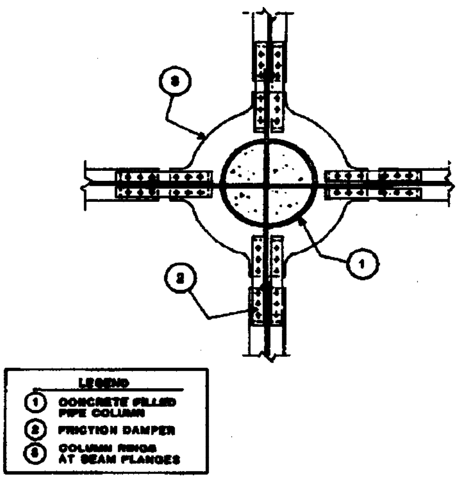

Figure D - Plan Section at Conn.

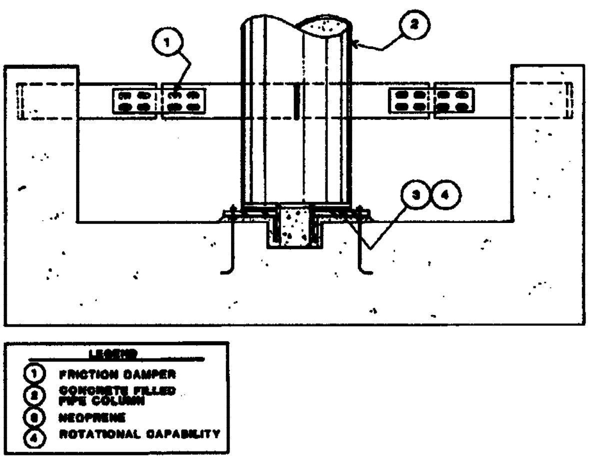

Figure E - Section at Base

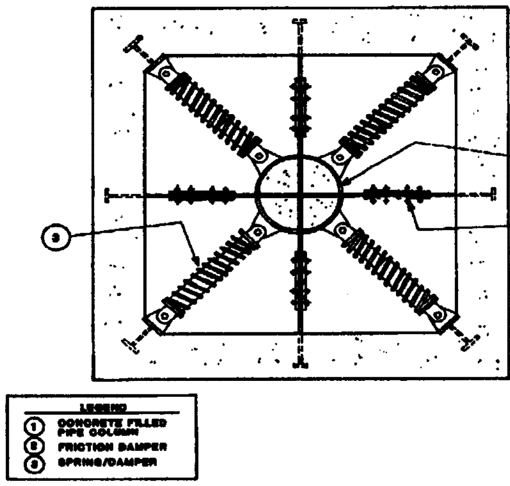

Figure F - Plan Detail of Base

replaced with friction dampers that are bolted. Thus the major contributing factor associated with the experienced failures of moment-resisting frames has been eliminated. A gusset plate is slotted through the pipe to create a shear tab that connects to the beam web with bolts in slotted holes, which allow for rotation of the joint at the beam web. Brass washers separate the shear tab from the beam web to facilitate rotation. With their ends allowed to rotate, the beams will have their maximum moment close to the middle of the span. The connection of the rings and gusset plates to the pipe column is achieved by nominal fillet welds that are shop welded. The only field welding required in the construction of Friction-Damped Moment-Resisting Frames is at the column splices, which are located at column mid-heights where the earthquake induced moments are low.

The design of Friction-Damped Moment-Resisting Frames should involve the use of of nonlinear time history analysis. Analytical tools exist that tract the energy dissipated by each damper along with the incoming earthquake energy and strain energy stored in the structure for every time step. The fact that the dampers do most of the work means that there is less reliance on a high force reduction factor R_w in Friction-Damped Moment-Resisting Frames. The optimum configuration of the slip loads in the frames are dependent upon characteristics of the ground motion; therefore, a series of time histories should be selected to capture the range of potential ground motions, and the dampers designed accordingly.

ANALYTICAL MODEL

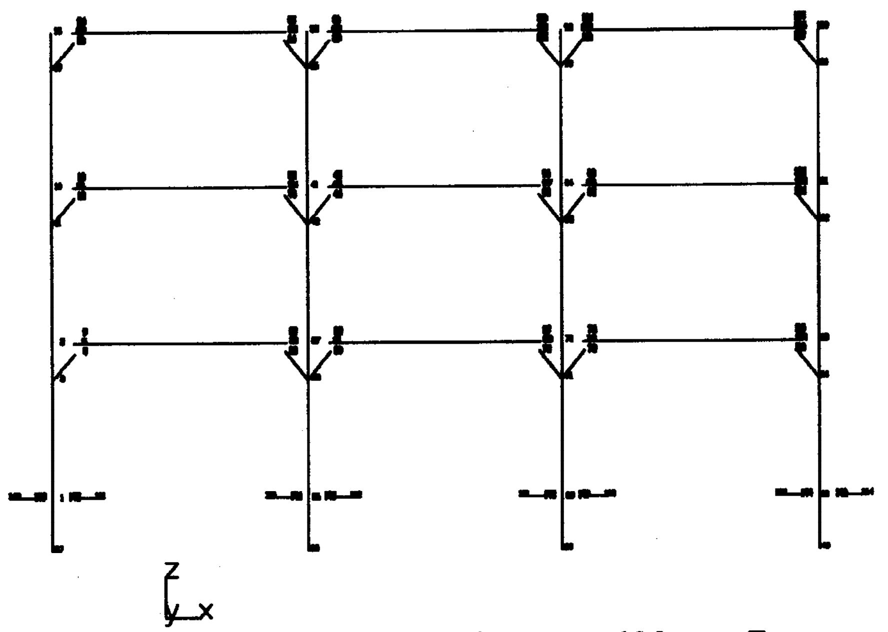

Figure G - Analytical Model of Friction-Damped Moment Frame

In the course of evaluating the performance of Friction-Damped Moment-Resisting Frames, an analytical model was created. Figure G shows the model which was 3 stories with column spaced at 30 feet and typical story heights of 15 feet. To allow for base rotation, the column hinge was placed 5 feet below the ground level with friction dampers and springs located 1 foot below the ground level. Friction dampers and springs were placed at all the column and beam joints. No viscous dampers were added to the springs and since modal damping was neglected, all energy dissipated was through the friction dampers. Plan dimensions of the structure was assumed to be 100 feet by 100 feet. Mass of all the levels was based on 100 pounds per square feet as the unit level weight. Thus total weight of the structure was 3000 kips. Four frames, two in each direction, were assumed to exist symmetrically at the perimeter. Typical composite column section was a 36 inch diameter pipe with a 0.625 inch wall filled with 3000 psi concrete. Beams were typically W24X68. Friction dampers were modelled as bilinear-plastic elements and nonlinear time history analysis was executed.

For ground motion input, the Taft record was scaled to produce a response spectra similar to Uniform Building Code's description of the Design Basis Earthquake, a 475 year return period event for seismic zone 4 with a S1 soil profile, except the acceleration values were not limited at the high frequency range as in the Uniform Building Code. Figure H shows the Taft Response Spectra.

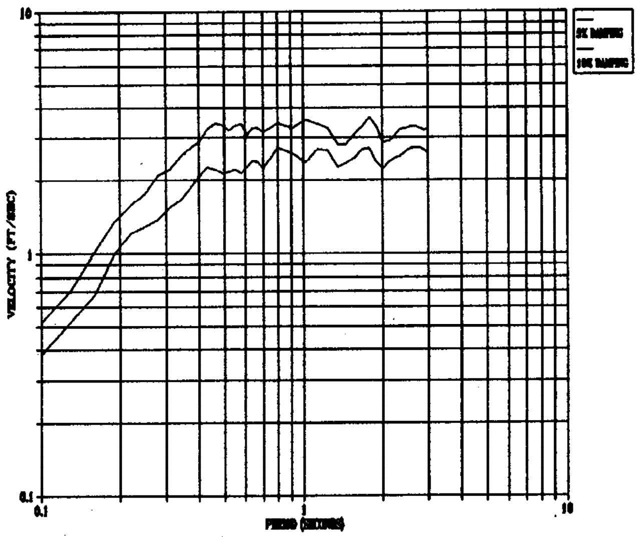

Figure H - Taft Response Spectra

A drift limitation of 0.01 was used to configure the rotational stiffness of the base. Since flexural displacements associated with the composite columns were negligible, most of the structural drift was due to base rotation. The allowable story drift was 0.01x15x12 or 1.8 inches for the Design Basis Earthquake.

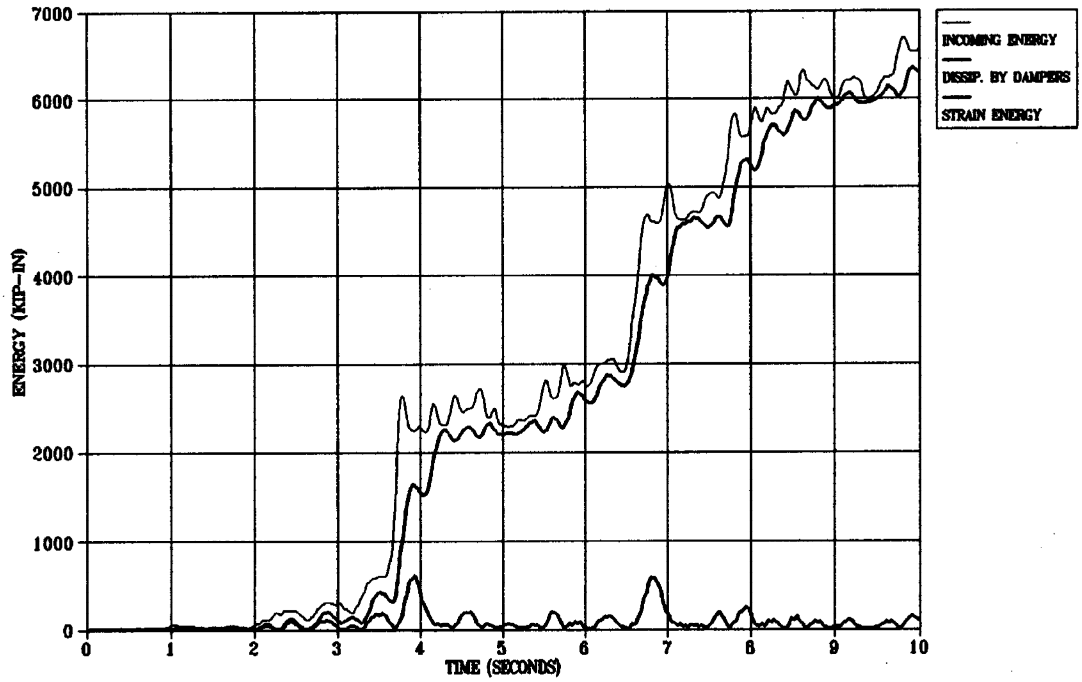

Figure J - Energy Dissipated at Friction Connections

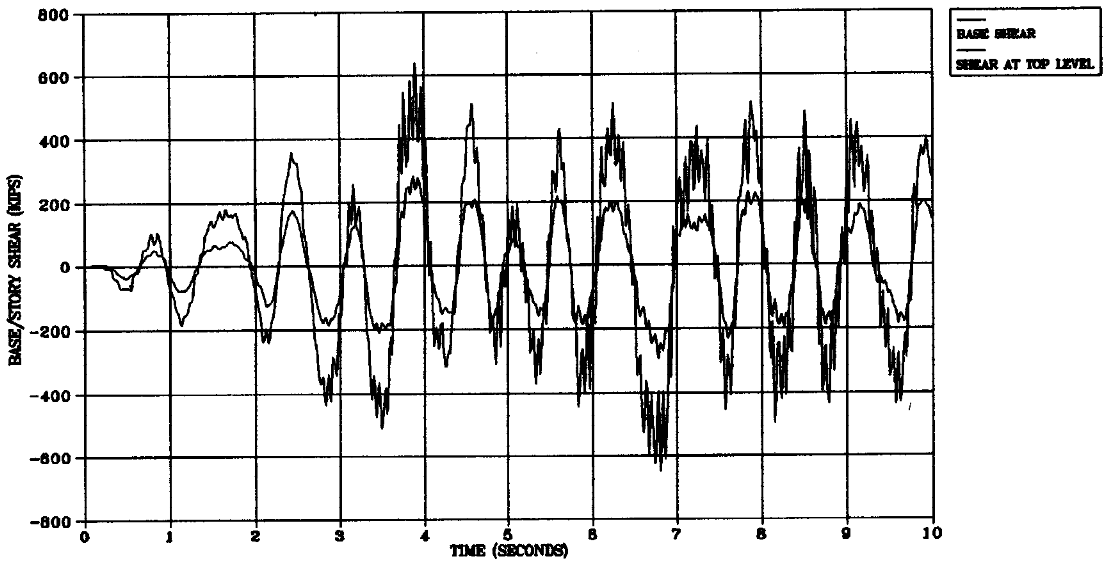

Figure K - Base Shear Compared with Top Level Story Shear

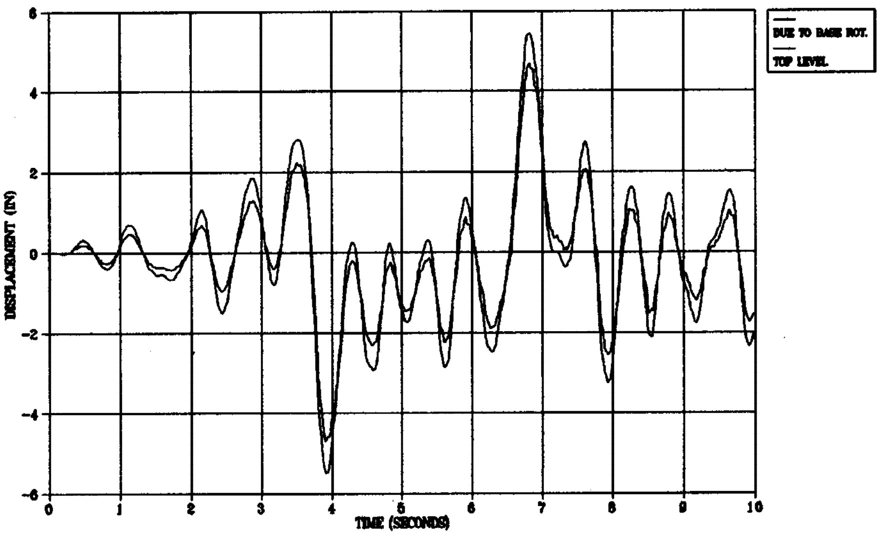

Figure L - Degree of Total Lateral Structural Displacement Due to Base Rotation

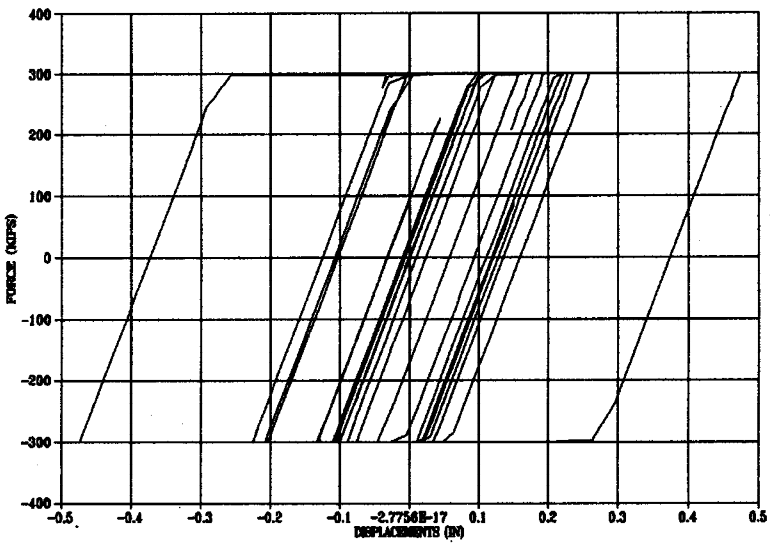

Figure M - Analytical Model of Friction Damper

CONCLUSIONS

Figures J,K,L, and M show the analytical results, which confirm the following:

1. Up to 90% of the incoming energy can be dissipated in the friction dampers.

2. Periods of low to medium rise frames can be controlled by configuring the rotational flexibility of the bases of the columns. Building drift can be similarly controlled.

3. Full penetration welds at column joints can be replaced with bolted friction dampers.

4. Composite columns can be economically designed to remain essentially elastic for the Design Basis Earthquake (475 year return period event).

5. Most of the steel erection can consist of bolting. The only field welding required is at column splices.

The next logical sequence in the development of Friction-Damped Moment-Resisting Frames should involve shake table testing.

REFERENCES

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[2] "AISC Technical Bulletin No. 2: Interim Observations & Recommendations on Steel Moment Resisting Frames", Modern Steel Construction, December 1994.