Performance-Based Seismic Design of a Four-Story Building Using Friction Dampers

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Abstract

This paper presents the design of a four-story braced frame structure which utilizes friction connections to dissipate seismic energy. The design conforms to a performance criteria established by Stanford University to minimize structural damage from a large earthquake and avoid collapse during an extremely large earthquake.

Introduction

The Escondido Village Graduate Student Housing Project consists of the construction of four four-story residential buildings for Stanford University. After the Loma Prieta earthquake in 1989 Stanford developed a comprehensive seismic performance criteria with three levels: A (the most rigorous), B and C (reserved for retrofit projects). Stanford required that this project meet the class B performance level. As such, under the rare earthquake EQ-I (10% probability event in 50 years), structural and architectural damage was to be limited so that repairs could be performed in several weeks to then allow full occupancy. Collapse prevention was required under the larger EQ-II event (10% probability in 100 years).

To design such a building we considered various types of traditional building construction. The system that best suited the budget and seismic performance goals consisted of an energy-dissipating steel frame building with braced frames and wood floors and diaphragms. The braced frame dissipates energy with friction dampers located at the base of the columns.

The detailed design of the structure relied on a parametric study comprising of non-linear time history analyses using NonlinPro, a two-dimentional non-linear analysis program based on Drain 2DX. The design team worked in conjunction with Stanford's Seismic Peer Review Committee[1] to arrive at a design that met Stanford's seismic performance objectives.

Building Description

To design a building that meets Stanford's seismic performance criteria we considered various structural systems. Because a traditional four-story wood frame shear wall building would sustain significant damage under a design level earthquake we decided against a wood frame system. A steel braced frame with composite concrete diaphragm system was ruled out due to the large brace members which would be required to remain elastic during a large earthquake. A braced frame system designed to behave inelastically, such as a SCBF, an EBF or a braced frame with compression-yielding braces, would have difficulty meeting the performance criteria due to the repairing time associated with replacing brace members.

We ultimately arrived at a hybrid energy-dissipating steel braced frame-wood diaphragm system, thereby taking advantage of the reduced-weight of the wood diaphragm and the strength of the braced frame. To allow the braced frame to remain elastic we used vertical slip friction connections at the base of the braced frame columns (see Fig. 2) to dissipate seismic energy.

Design Strategy

Using a performance-based design approach, we were able to design the structure with a clearly defined seismic failure sequence. The underlying premise of our design was that the diaphragm and braced frame were to remain elastic during large ground shaking. The fuse elements were limited to structural members that could be repaired relatively quickly with minimal disruption to the

[1] Stanford's Peer Review Committee for this project consisted of: Evan Reis - Comartin/Reis Consulting Engineers, Jim Malley - Degenkolb Engineers, Brett Lizundia - Rutherford & Chekene.

building occupants after a moderate-sized earthquake.

Under low level ground shaking, the structure is designed to behave as a traditional braced frame system. Frame members remain elastic and no architectural damage occurs.

During larger ground shaking, the friction connection at the base of each braced frame column (see Fig. 2) slips and dissipates energy as the frame rocks. The braced frame rocking at the column bases is analogous to a reinforced concrete shear wall with plastic hinges formed at its base. As the frame rocks back and forth, the friction connection serves as a hysteretic friction damper (see Fig. 8). This level of shaking coincides with a base shear of 0.4g. The frame would remain elastic and the building would suffer only minor structural damage. After an earthquake of this size the frame could have a residual offset (see Fig. 5) which would be repaired by unbolting the slip connection, thereby allowing the gravity load to re-plumb the frame.

If the stroke of the slotted connection is exhausted, the connection locks up and the full tension force resulting from frame overturning is transmitted into the concrete pier. As the overturning force increases, the reinforcing in the concrete pier will then yield in tension, thereby creating a fuse to protect the friction connection from fracture. Pier yielding is set to occur at a base shear of 0.64g. As the reinforcing bars in the pier yield and strain harden brace buckling in the frame will eventually occur at 0.75g. The braces are configured and the connections and columns are proportioned such that after the compression braces buckle, the post-buckled residual strength of the frame corresponds to 0.62g. Braced frame columns are designed to support loads coinciding with 1.14g to preclude failure of the columns, which would jeopardize the gravity load resistance of the system.

Analysis

To study the behavior of the building during an earthquake, we modeled the structure in NonlinPro (see Fig. 6) and performed non-linear time history analyses using ground motions from Kobe (Takotori record), Northridge (Sylmar record) and synthesized design ground motions EQ-1 and EQ-2 provided by Stanford. The model incorporated the friction slip connection properties, the expected behavior of the concrete pier and the diaphragm stiffness.

At the base of each braced frame column a series of spring, hook and gap elements were used to replicate the behavior of the friction slip connection and the concrete pier (see Fig. 7). A linear elasto-plastic spring (S1) represents the elastic column stiffness (k1) until the friction slip force is reached (Fs). The post-yield zero stiffness of the spring coincides with connection slip. The spring unloads inelastically to capture the hysteretic energy dissipated in the system. The hook and gap elements in parallel with the spring correspond to the length of travel of the slotted connection.

Once the hook or gap is engaged the connection is locked-up and a secondary pier spring (S2) is activated. The pier spring is also elasto-plastic yielding in tension to represent the yielding of the pier reinforcement. To account for the uncertainty in estimating pier stiffness and possible pier movement, we modeled lower and upper-bound pier stiffnesses. The lower-bound pier stiffness assumes a 1/4" pier slip in tension and a full effective pier length while the upper-bound stiffness assumed no pier slip and a 2/3 effective pier length. Because the pier stiffness is different in tension and compression we used the average stiffness for the spring in each direction. Using the average pier stiffness at each column base produces the same drift as two varying stiffness springs would. The difference between the two models is that the point of rotation of a rigid frame using average spring values would be at the centerline of the frame, as opposed to being about the compression pier if the stiffnesses were different.

To take into account the effect of diaphragm behavior in the model we connected the floor masses to the braced frame with linear elastic springs with the stiffness of the diaphragm (see Fig. 6). Different diaphragm springs were used for the stiffer longitudinal and the softer transverse direction braced frames.

An envelope of structural responses was obtained for the different ground motions by NonlinPro runs of two primary models: a soft diaphragm, lower-bound pier stiffness model and a stiff diaphragm, upper-bound pier stiffness model. Additional parameters, such as the connection slip force and slot length, were also varied to bound the results.

Analytical Results

By studying the behavior of the stiff and the soft models, trends were observed in drift and force demands. As expected, the softer model (lower bound pier stiffness with a transverse direction diaphragm spring) underwent greater drifts with lower base shear and member forces than the stiffer model (upper bound pier stiffness with a longitudinal direction spring). Increasing the slip force also tended to generate larger base shears and member forces with a reduced drift.

Using models with different parametric combinations we were able to generate design force and displacement envelopes. The member force envelopes were obtained from runs of the stiffer model with a slip force of 125% the target design slip force. The increase in slip force beyond the target value was to take into account the possibility of bolt over-tightening or connection corrosion and the resulting force increase on the members. A slip force of 80% the target value was used with the soft model to determine the design displacement envelope.

The length of travel in the friction connection was a parameter that we studied extensively. When the connection reaches the end of its stroke the braced frame is in effect pounded, resulting in a spike in member forces and base shear. The locked up braced frame behaves like a rigid frame with an increase in member forces due to the impact loading. Due to the difficulty in quantifying the impact force on the connection and the importance of avoiding a fracture at the slip connection, we decided to design the slot to avoid lock-up during the design earthquake. We found that increasing the slot length to avoid lock-up kept the forces low with only a minimal increase in lateral drift from the locked case.

To compare the dynamic behavior of the model and the idealized static behavior with the design response spectra, we superimposed the dynamic force-displacement, the static push-over and the response spectra curves (see Fig.__). In doing so we noticed that the dynamic response was resulting in spectral accelerations in excess of the spectral accelerations calculated from the static push over case for a model that was slipping without locking up. This increased response in the dynamic system is a result of higher mode effects which remain unseen in static behavior. Because story shears in a dynamic response don't necessarily all go in the same direction, the effective height of the horizontal force resultant is reduced, resulting in an increased base shear in a structure for a given overturning moment fuse strength. The dynamic higher mode effects also increase diaphragm demands from those that would be expected from a static push-over force distribution.

Design Details

For the structure to behave as intended during an earthquake, special attention was paid to the design of critical members and connections. It was critical that the slip connection at the base of the braced frame columns functioned properly. To assure a predictable friction slip surface, we used brass shims between the connection plates consistent with research by Grigorian, Yang and Popov[1] (see Fig. 2). The inside face of the plates in contact with the brass slip surface were sandblasted to remove loose mill scale, further reassuring a consistent slip plane. For slip to occur at the target load, the bolt tension and bearing pressure on the slip surface must be uniform. We used Belleville washers to accurately gauge the bolt tension with the washer deformation in the field (see Fig. 2B). To sandwich the center slip plate and brass shims we used heavy $1^1/_4$" plates to achieve a relatively uniform bearing surface on the slip plane without the use of washer plates. To reduce the possibility of corrosion between the slip surfaces, which could increase the slip force, all possible ways for moisture to reach the slip plane were caulked and sealed with sheet metal strips (see Fig. 2B). To verify that the connection responded as designed, a series of tests were conducted at a private testing laboratory.

Additionally, the friction connection had to be able to elastically undergo the out-of-plane displacements caused by the rocking of a perpendicular braced frame. Using spherical bearings to connect the column's gusset plate to the slip plate we were able to reduce the out-of-plane displacement demand on the slip plate while avoiding possible binding of a bent center plate during slip. A kinematic study was also done to determine the maximum expected translation of the bolts due to frame rotation to size the width of the slots. For maximum ductility in the event of connection lock up, we used Bethlehem Steel's high-toughness T-Star plate with an CVN toughness in excess of 150 foot pounds for the center slip plate.

The central connection of the V-configuration connecting the braces to the foundation had similar design concerns. Because the central connection transfers the base shear to the foundation, it was designed to displace vertically without any horizontal slot oversizing (see Fig.__). The connection is not intended to dissipate friction energy so brass shims were not used. To allow for out of plane frame displacement, we used Belleville Springs between the center slip plate and outer plates.

To design the braced frame members we used member forces from the time history force envelope and followed the recommendations of AISC's "Seismic Provisions for Structural Steel Buildings," April 1997. The frame was additionally designed for a two-story collapse mechanism with brace buckling for loads in excess of the design earthquake (see Fig. 8). Gusset plates and columns were designed to fully develop the member's tension capacity while incorporating material overstrength factors. To assure out-of-plane rotation capacity at the brace connections, gusset plates we designed with the fold-line criteria per Astaneh et al.[2]

For the piers to act as a fuse after bolt lock-up, it is critical that pier yield occurs at the intended load. To achieve this we specified a range of upper and lower yield strengths for the tension rebar and called for 4' of rebar to be encased in a PVC bond-breaker. With known rebar yield strength and unbonded length we were able to obtain a predictable pier tensile fuse without degrading the pier's compression capacity.

Desirable seismic response of the system is ultimately dependent on the ability of the collector members to effectively transfer the inertial loads into the braced frame. The kinematic demands on the beam-to-column connection, due to the rocking of the braced frame, required specialized details to ensure that the collector connections would not be compromised. The connection design utilized a central bolt at the mid-height of the beam's web with a series of horizontal and vertical slotted holes in an elongated tab plate extending through the TS column (see Fig. 4). The horizontally slotted holes above and below the central bolt carry gravity loads while the vertically slotted holes carry the lateral collector loads with the beam free to rotate about the central bolt. To obtain a more realistic estimate of the design demand loads on the collectors, the time history analysis story forces were used, which accounts for higher mode effects.

Because of the expected building drifts, special attention had to be paid to non-structural items, such as mechanical risers and exterior stairs. An envelope of expected story drifts was provided to the plumbing and fire protection contractors to assure serviceability after an earthquake.

Conclusions

Friction dampers are powerful design tools, capable of dissipating seismic energy without inelastic behavior for a relatively low cost. The true advantage of such systems comes to light when evaluating post-earthquake repair scenarios.

When designing slip connections, the possibility of bolt impact should be minimized at service-level ground shaking due to the uncertainty involved with predicting impact forces.

Using capacity-based design, a structural fuse hierarchy can be developed to ensure that multiple mechanisms are available to dissipate seismic energy. For example, if the friction connections are too strong or exhaust the slot length, the pier fuses are activated to protect the frame and connections. Additionally, if the piers are too strong or if they strain harden, the frame is designed to dissipate energy during activation of the collapse mechanism. The simple details provide three plateau collapse responses which are reliable and effective.

Easily constructable, inexpensive structural fuses can be designed to yield at predictable loads.

When designing structures with a static equivalent lateral force method, higher mode effects seen in dynamic analysis on diaphragms should be taken into consideration.

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1. "Slotted Bolted Connection Energy Dissipators," Grigorian, Yang & Popov, *EERC* Report No. UCB/EERC-92/10, July 1992.

2."Seismic Behavior and Design of Gusset Plates," Astaneh, *Structural Steel Educational Council,* December, 1986.

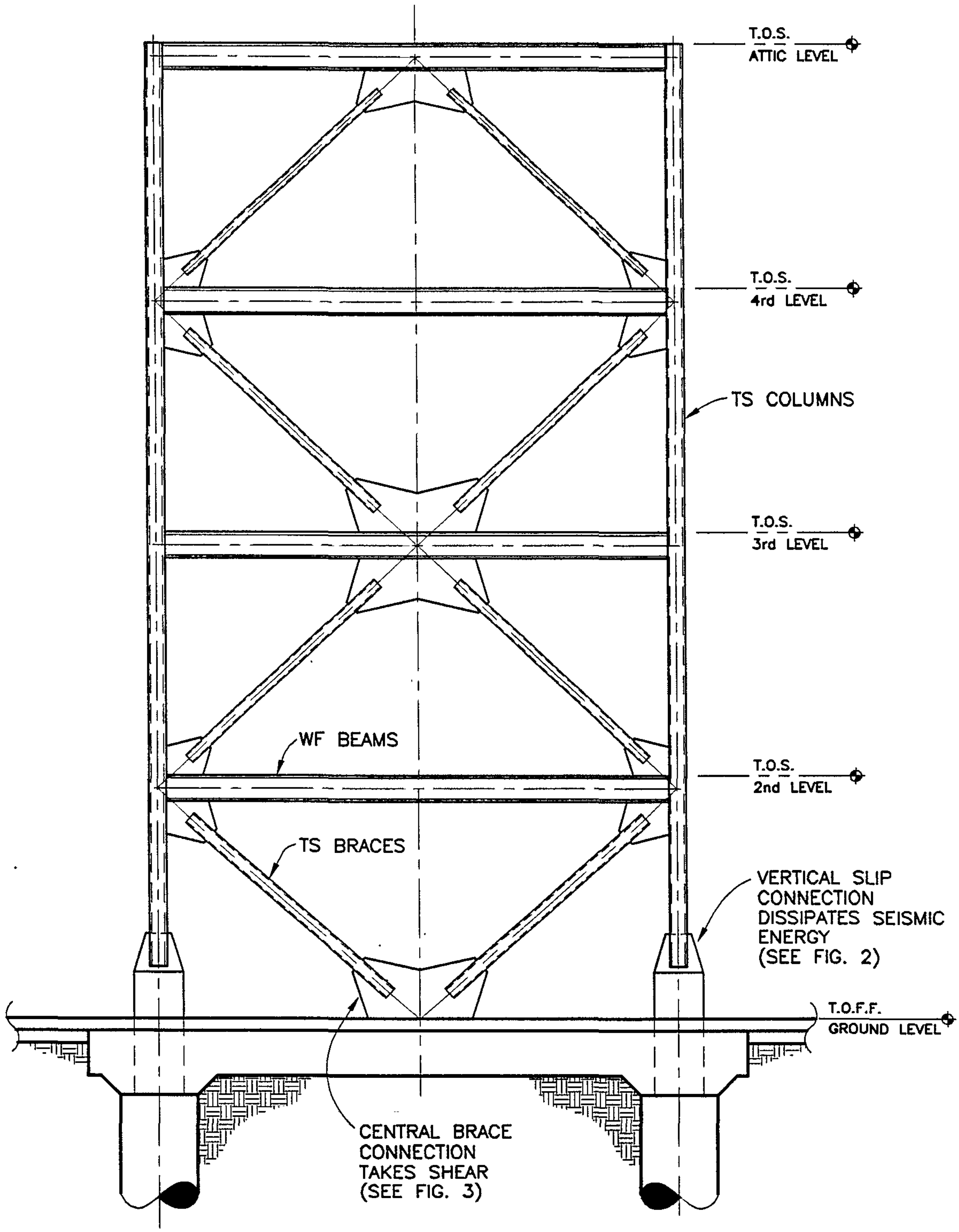


Figure 1 - Typical Braced Frame

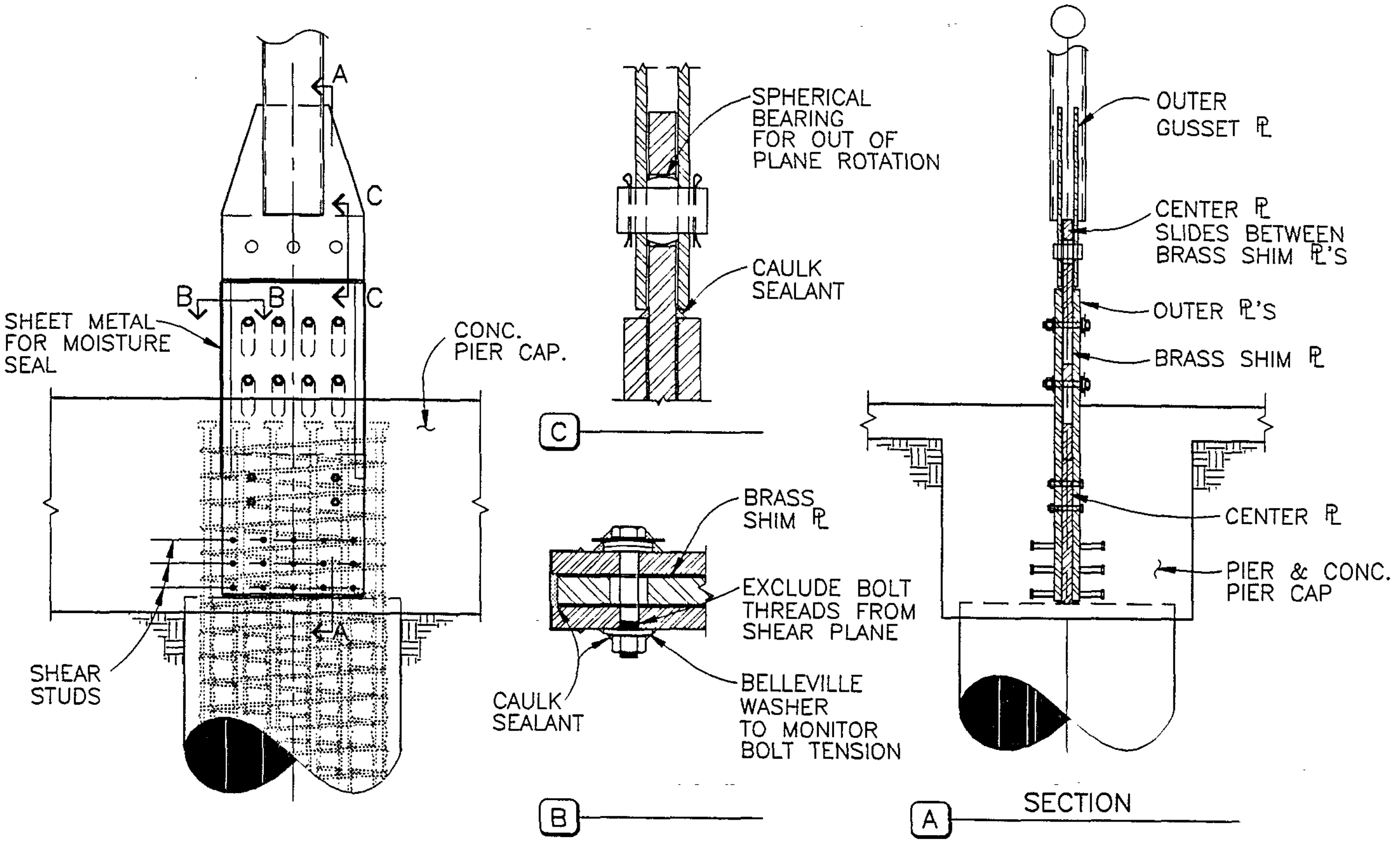


Figure 2 - Braced Frame Friction Connection

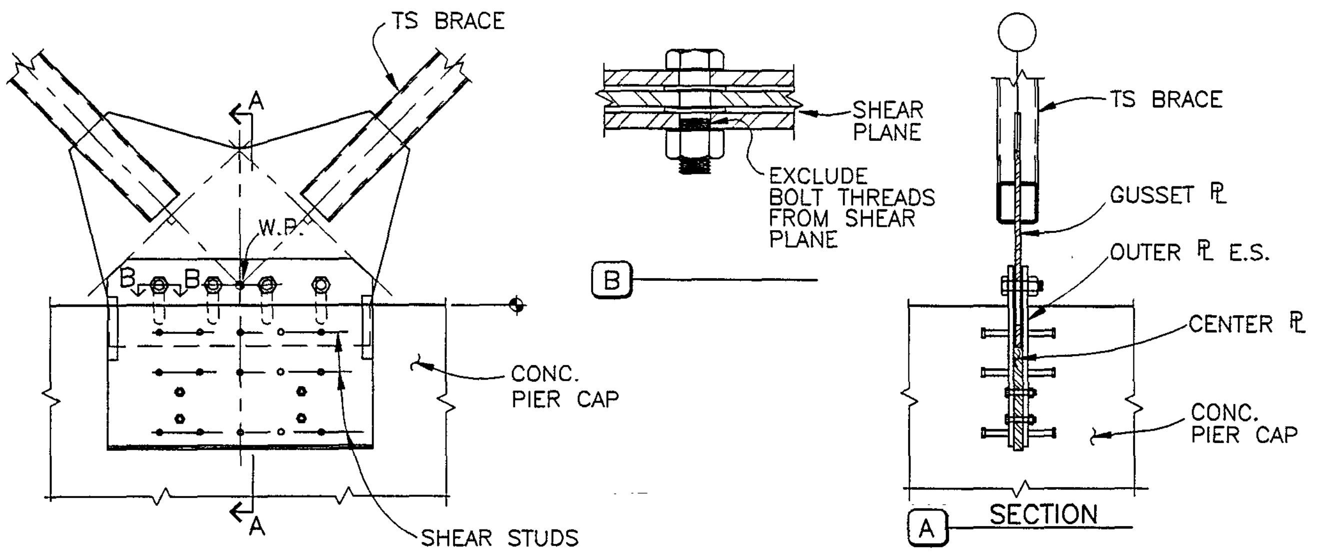


Figure 3 - Central Brace Connection

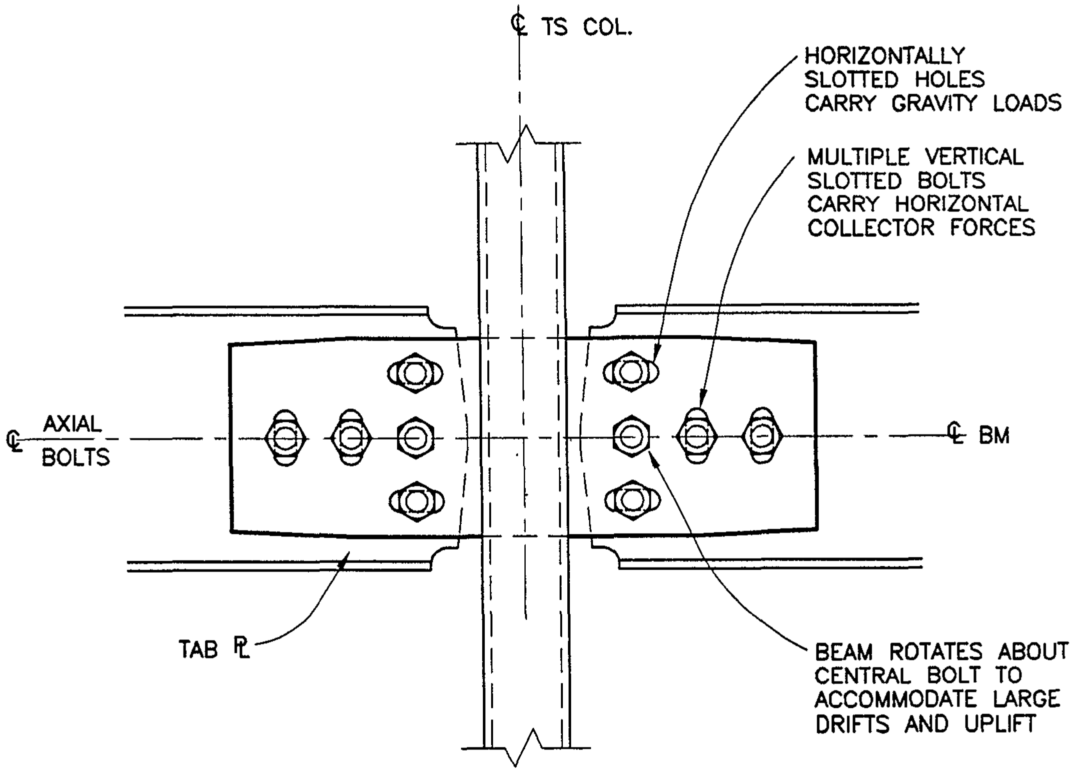


Figure 4 - Beam-to-Column Collector Connection

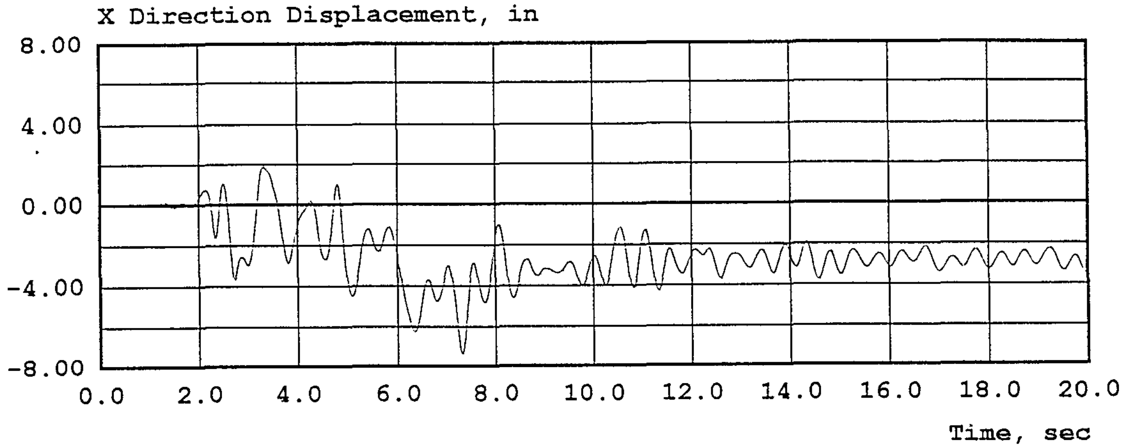


Figure 5 - Dynamic Roof Drift

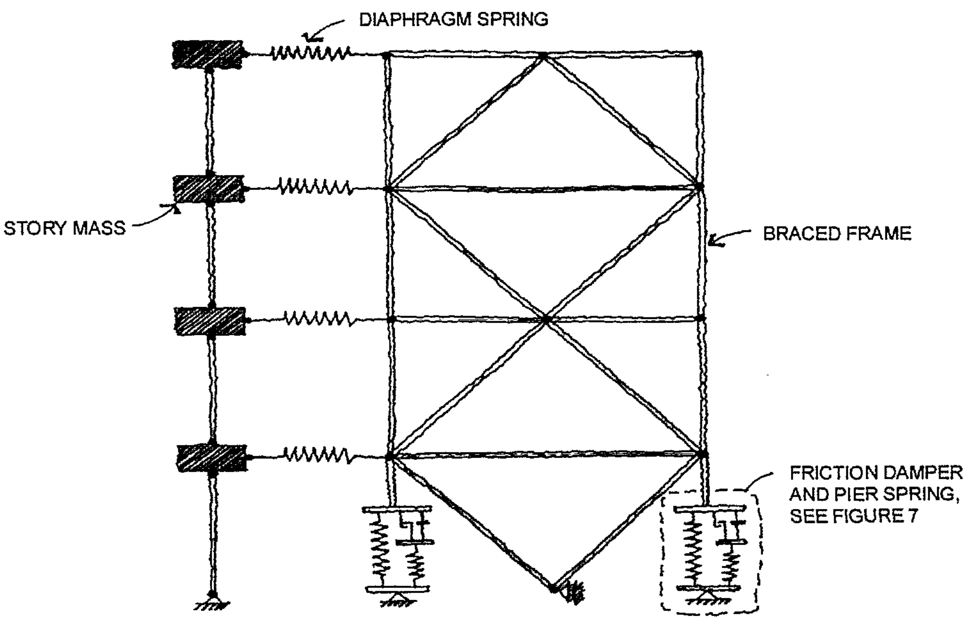
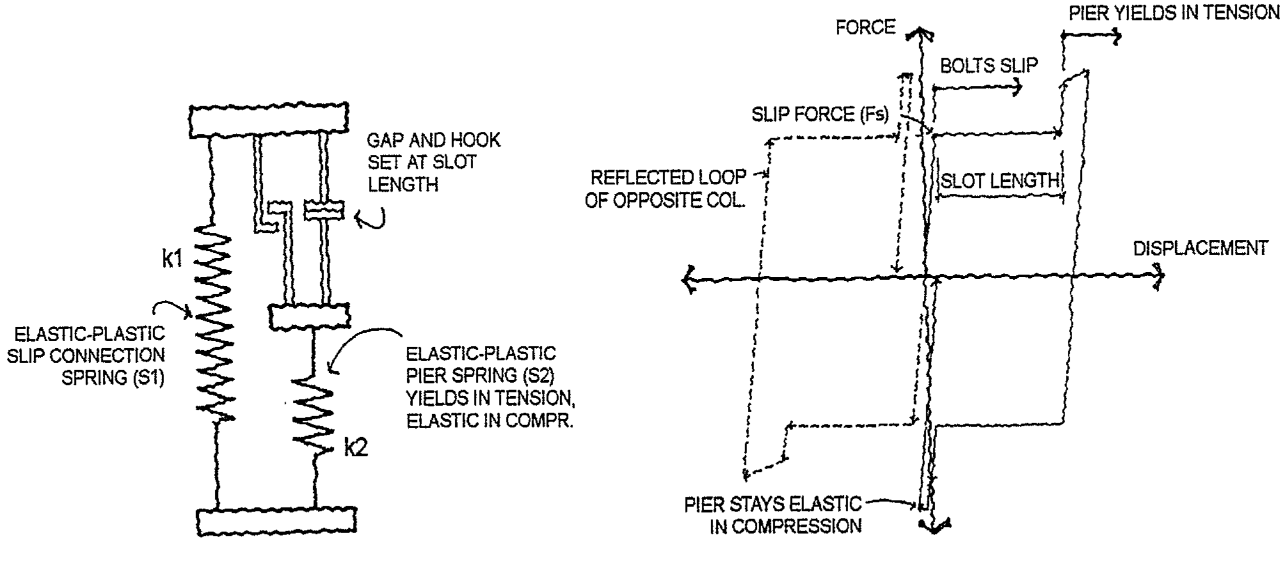


Figure 6 - Building Analytical Model



A. Spring Model

B. Hysteretic Behavior

Figure 7 - Slip Connection and Pier Analytical Model

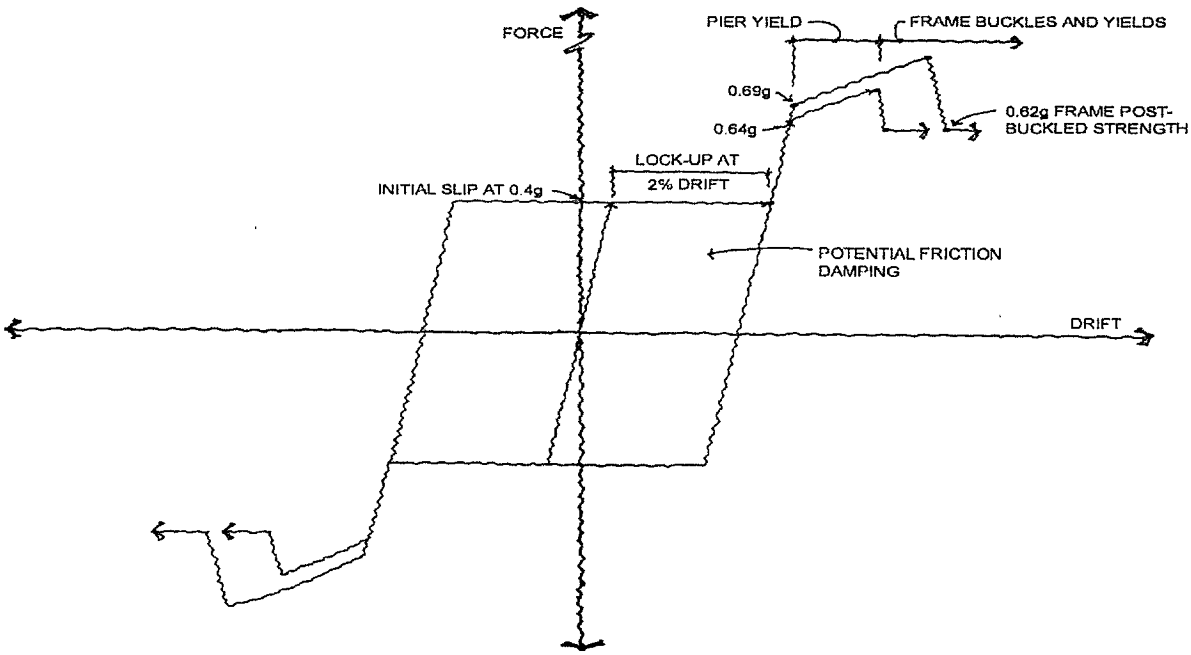


Figure 8 - Idealized Global Force-Displacement Relationship

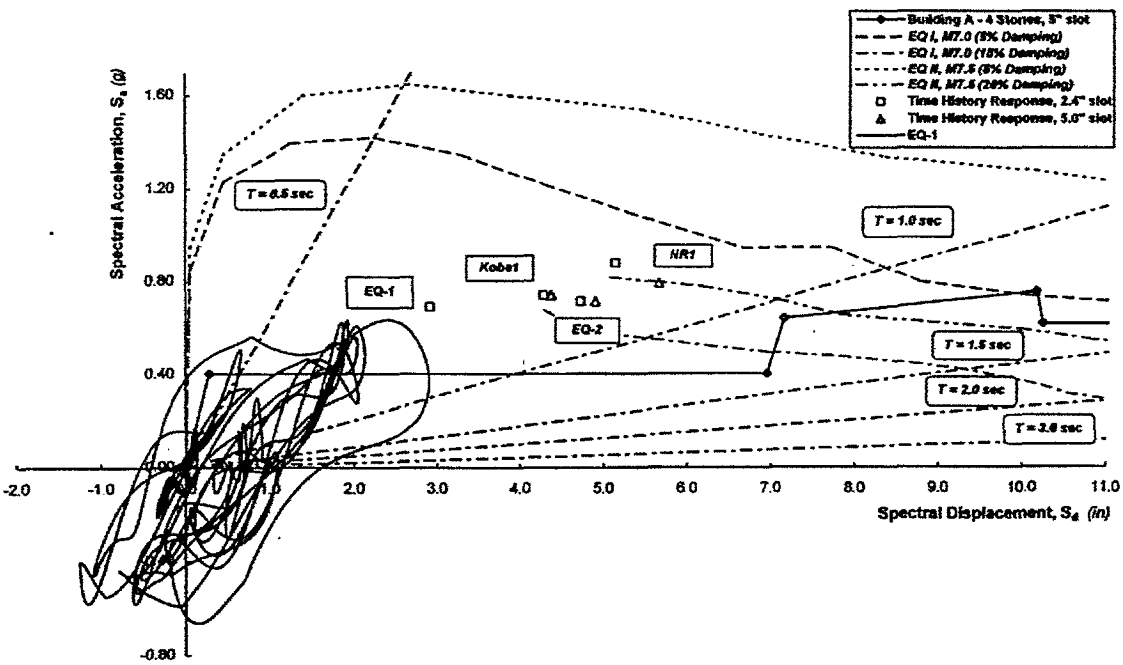


Figure 9 - Structural Response Spectrum