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FRICTION CONNECTION FOR SEISMIC CONTROL OF MOMENT RESISTING STEEL FRAME

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ABSTRACT

An analytical study of the seismic performance of steel moment-resisting frames incorporated with a new design for beam to column friction connections is presented. Moment-resisting steel frames have exhibited stable hysteretic behavior during major earthquake events; however, the failures of steel moment-resisting frames during the January 17,1994 Northridge, California earthquake have exposed their weaknesses. The most common failure occurred at the beam to column connections which have been designed to meet the code's requirements, and yet have proved to be lacking in strength and ductility. In this paper, a new connection with energy dissipation mechanism is proposed. During sever earthquake excitations, the proposed friction connection slips to soften the structure and to dissipate the energy. Two prototype moment resisting-frames having eight and twenty stories, were analyzed for several different earthquake excitations. Time-step non-linear dynamic analyses have shown that the frames with friction connections suffer much smaller story drifts, shears, lateral displacements and permanent damages than comparable frames with standard connections.

KEYWORDS

Friction connection; Energy dissipation; Steel moment-resisting frame.

INTRODUCTION

In a seismic environment, designers generally favour moment resisting frames (MRFs) to braced frames for their stable ductile behavior under repeated reversing loads. While being able to meet the code requirements, moment resisting frames have limited inherent damping capacity to cope with an unexpectedly big earthquake. Current methods of aseismic design place reliance on the ductility of the structural elements to dissipate energy while undergoing large inelastic deformations. If the inelastic deformations can be accommodated in the design, permanent damages would occur, otherwise failure would result. Neither alternative is an attractive proposition.

It has been recognized that the dynamic responses of buildings depend highly on its energy dissipation capacity, and thus, the use of energy dissipation devices is a promising approach to the problem. This recognition has stimulated research and implementation of a variety of energy dissipating devices such as viscoelastic dampers (Soong, T.T. and Mahmoodi, P., 1990), brake lining pad dampers (Pall, A.S. and

Marsh, C., 1982) and hysteretic dampers (Scholl, R.E., 1990) (Su Yung-Feng and Hanson, R.D., 1990).

Energy dissipating devices have inherent advantages and disadvantages, and their selection will depend on the specific seismic environment and the requirements of the building. Since the devices have to be tuned so that the entire structure can meet these requirements, the total engineering effort in the design process is dramatically increased (Villaverde, R. and Koyama, L.A., 1993).

The scope and the unprecedented nature of the failures of steel-frame buildings during the January 17, 1994 Northridge, California earthquake have shown the limitation of moment resisting steel frames. The most common type of failures among steel buildings occurred at the welded connections binding the horizontal beams to vertical steel columns (Los Angeles Times, February 27, 1994). The metalalloy welds are supposed to be stronger than the girders they connect and thus are less likely to break. As the welds hold, the beams and columns bend or twist, absorbing the earthquake energy. In the affected buildings, however, some connections became brittle and broke. In some cases, only the welds cracked; in others, the weld fractures continued into the steel, deeply gouging columns or even splitting them.

These unexpected failures raise the new concern to tighten up the codes for the design building subject to greater earthquake forces. Since the use of thicker columns and beams would boost the cost of construction, and the additional stiffness would attract even greater earthquake forces, other approaches are preferable. One approach that is suitable for moment resisting steel frames is to take advantage of the potential for energy dissipation in the connection elements.

In this paper, a new type of beam to column connections with energy dissipation mechanism is proposed. The dynamic responses of MRFs with conventional connections and with the new connections are investigated. Moment resisting frames with friction connections exhibit superior performance during earthquake excitations.

MOMENT RESISTING FRAME WITH FRICTION CONNECTION

T-stub is a commonly-used beam to column connection. It can be designed either as a rigid or semi-rigid connection. A new slipping bolted connection associated with T-stub is show in Fig.1(a). The T-stub may be replaced with seat angle, stiffened angle and welded plate as shown in Fig.1(b) to Fig.1(d).

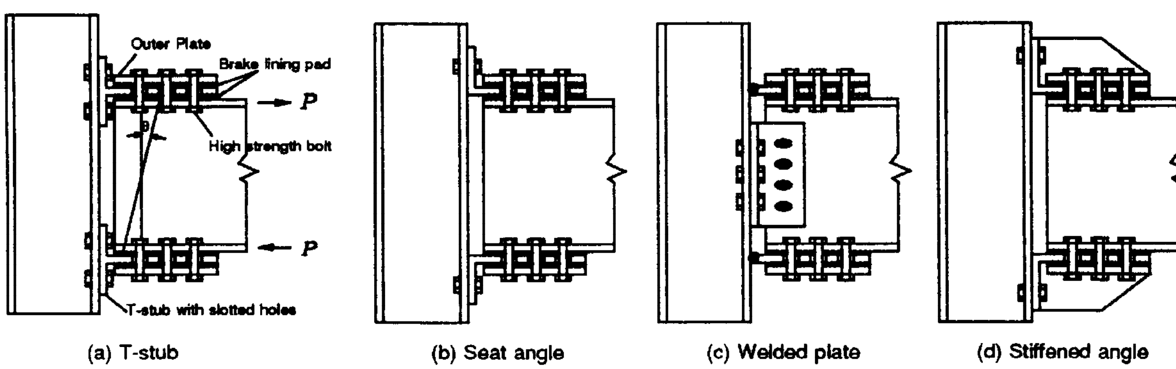

Fig.1 Fiction connection between beam and column

As shown in Fig.1(a), the T-stub has slotted holes in which high strength bolts are inserted. The bolts press the two brake lining pads against the faces of the T-stub. As the T-stubs move within the slotted holes, frictional forces give the desired energy dissipation without causing cracking or yielding of the connecting members. The bending moment in the beam is transmitted to the column by the two tees that are attached to the top and bottom flange of the beam. The force exerted on the column by each tee is P. The idealized relationship between the force P and the displacement of the beam flanges is shown in Fig.2.

The friction joints are designed not to slip under service load, but are expected to slip during severe seismic excitations. Static and dynamic cyclic tests have been conducted on brake lining pads inserted between steel plates with mill scale surfaces (Pall, A.S., 1989) and the results are shown in Fig.3. The friction joint exhibits a constant, repeatable slip load and nearly elastic-plastic behavior, with negligible degradation. The similar hysteresis behavior was obtained from Grigorian, *et al.* by using brass insert plates (Grigorian *et al.*, 1994). The result of the cyclic testing performed at the Earthquake Engineering Research Centre, University of California at Berkeley is shown in Fig.4.

As an alternative, the outer brake lining pad could be removed and the inner pad replaced by ordinary steel plates, resulting in two slipping surfaces: one between the outer plate and the T-stub, and one between the insert plate and the T-stub. The surfaces of the T-stub and the plate may be treated to develop large rectangular hysteresis loops with negligible fade over several cycles of reversals (Savard, G., *et al.*, 1995). The T-stub, high strength bolts and the steel plate could be shop-fabricated and assembled together to minimize the hole clearance and for greater quality control.

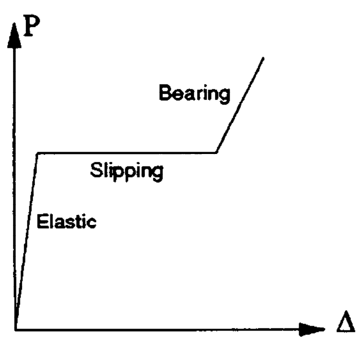

Fig.2 Idealized load-disp. relation of the friction connection

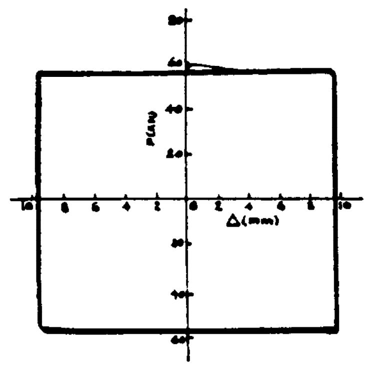

Fig.3 Hysteresis loop of brake lining pad

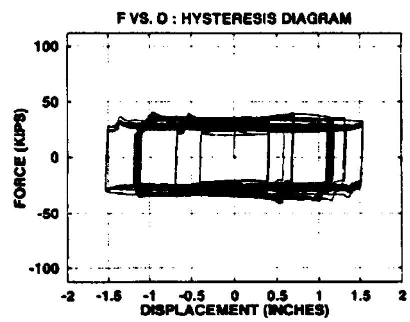

Fig.4 Hysteresis loop of brass insert plate

MODELLING OF THE FRICTION CONNECTION

In the conventional analysis and design of steel frameworks, the connections are assumed to be either ideally pinned or fully rigid. While the modelling of flexible connections is more complex, the essential analytical tools are available provided that the data on joint behavior is available. The computer program DRAIN-TABS (Rafacel Guendelman-Israel and Powell, G.M., 1977), which is widely used for non-linear dynamic analysis of structures, contains an option by which semi-rigid connections can be modeled. The proposed friction connection could be modeled by using a modified semi-rigid connection element. The idealized moment-rotation relationship for the connection elements is shown in Fig.5. The modeled behavior is elasto-plastic until the bolt reaches the end of the slot, after which it becomes elastic again up to bearing failure of the joint. A fictitious yielding moment is specified for the connection element to correspond with the moment in the joint when slippage occurs.

EXAMPLE APPLICATION OF FRICTION CONNECTION

Earthquake response analyses were conducted for two example steel frames having 8 and 20 stories as shown in Fig.6. The dimensions, member sizes, and other properties of the frames are the same as those used by Roeder (Roeder, C.W., *et al.*, 1993). This building was analyzed with and without friction connections, which are located at the two ends of each beam.

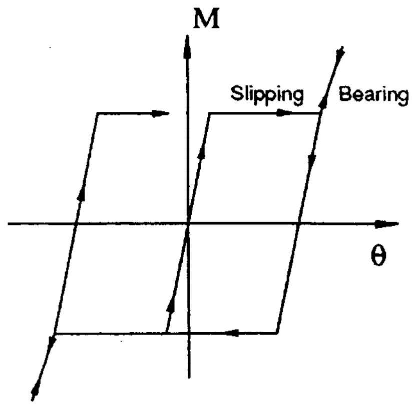

Fig.5 Hysteretic behavior of modified connection element

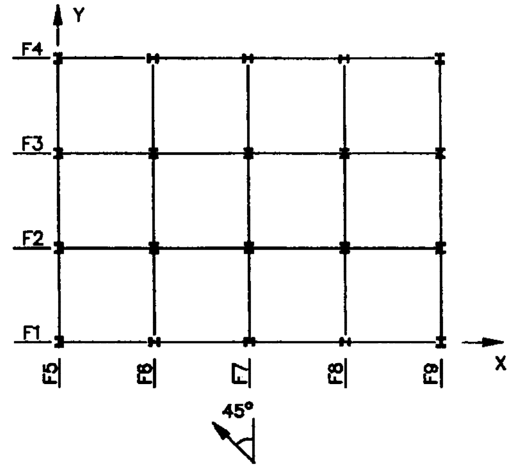

Fig.6 Plan view of the prototype buildings

Four earthquake excitations were chosen as shown in table 1. Earthquake excitation angle is chosen at 45 degrees from the Y-axis as shown in Fig.6. The fictitious yielding moments of the friction connection were specified to be 250 kN·m and 400 kN·m for the eight- and twenty-story frames, respectively. These figures is larger than the maximum bending moments in beams due to service load (combination of dead, live and wind/or quasi-static loads). No viscous damping was considered in the analyses.

Table 1. Earthquake records

1940 EL-Centro Earthquake (NS 0-12 Sec.)
1952 Taft Earthquake (0-15 Sec.)
Newmark-Blume-Kapur Artificial Earthquake (0-15 Sec.)
1985 Mexico City Earthquake (WE 0-70 Sec.)

The dynamic responses of the buildings are summarized in Table 2 and Table 3. In theses tables, only the responses of frame No.1 and frame No.5, which represent the X and Y directions, respectively, are listed. It is clear that the frames with friction connections suffer smaller seismic forces, story drift, base shear and lateral displacements. This phenomenon has been consistently observed for both moderate and severe earthquake excitations, but is more pronounced for the severe one.

The deflection envelopes of the frame No.1 and frame No.5 of the 20-story building are shown in Fig.7. It is seen that for the 0.25g Mexico earthquake, the deflections at top of the frame with standard connections are 63% (X-direction) and 78% (Y-direction) greater than for the frame with friction connections. For the 0.4g El-Centro earthquake, from Table 3, these figures are 46% and 91% for the X and Y direction, respectively.

The maximum story shears of the frame No.1 and frame No.5 are shown in Fig.8. For the 0.25g Mexico earthquake, with friction connections, the frame's base shear is reduced by 48% for frame No.1, and by 70% for frame No.5. For the 0.4g El-Centro earthquake, the reduction in base shear is 43% for frame No.1, and 70% for frame No.5.

Table 2 Dynamic responses of 8-story Moment Resisting-Frame

Earthqu. record	Conn. case	Maximum displacement at top of the frame (m)				Maximum base shear (kN)				Average story drift (%)	
		F1		F5		F1		F5		F1	F5
El-Centro 0.2g	Stand.	.234	-.223	.098	-.111	1002	-1103	342	-365	.978	.439
	Frict.	.146	-.169	.096	-.106	790	-794	318	-346	.796	.424
El-Centro 0.4g	Stand.	.274	-.382	.195	-.220	1111	-1118	632	-752	1.303	.859
	Frict.	.153	-.308	.188	-.135	908	-900	516	-419	1.058	.664
NBK 0.2g	Stand.	.223	-.290	.229	-.237	969	-1100	920	-946	.977	.980
	Frict.	.174	-.218	.154	-.115	745	-757	440	-411	.811	.611
NBK 0.4g	Stand.	.354	-.486	.315	-.332	1116	-1089	1012	-1041	1.606	1.265
	Frict.	.390	-.183	.280	-.147	808	-860	542	-479	1.271	.911
Taft 0.2g	Stand.	.170	-.149	.215	-.234	926	-882	859	-874	.811	.914
	Frict.	.124	-.130	.122	-.155	647	-653	381	-391	.664	.564
Taft 0.4g	Stand.	.248	-.239	.284	-.322	1110	-1055	994	-973	1.161	1.205
	Frict.	.183	-.205	.154	-.202	843	-678	398	-478	.879	.713
Mexico 0.1g	Stand.	.372	-.509	.360	-.376	1110	-1124	1026	-1034	1.521	1.289
	Frict.	.347	-.277	.226	-.211	768	-753	518	-476	1.191	.863
Mexico 0.25g	Stand.	.539	-.670	.575	-.414	1127	-1124	1057	-1061	2.453	1.904
	Frict.	.581	-.610	.490	-.455	834	-817	655	-672	2.111	1.643

Table 3 Dynamic responses of 20-story Moment Resisting-Frame

Earthqu. record	Conn. case	Maximum displacement at top of the frame (m)				Maximum base shear (kN)				Average story drift (%)	
		F1		F5		F1		F5		F1	F5
El-Centro 0.2g	Stand.	.276	-.228	.348	-.375	1083	-1112	886	-851	.537	.595
	Frict.	.267	-.228	.218	-.303	949	-943	454	-520	.485	.424
El-Centro 0.4g	Stand.	.559	-.456	.586	-.651	2008	-2090	1781	-1788	1.024	.999
	Frict.	.243	-.451	.410	-.236	1057	-1282	536	-533	.692	.656
NBK 0.2g	Stand.	.419	-.452	.423	-.445	1466	-1441	1016	-932	.734	.766
	Frict.	.414	-.335	.151	-.333	1213	-1132	527	-556	.621	.450
NBK 0.4g	Stand.	.831	-.830	.654	-.689	2382	-2415	1758	-1891	1.265	1.130
	Frict.	.727	-.603	.287	-.703	1511	-1456	608	-785	1.167	.819
Taft 0.2g	Stand.	.246	-.283	.210	-.203	1209	-1094	585	-659	.563	.486
	Frict.	.244	-.247	.182	-.182	1021	-926	552	-515	.516	.403
Taft 0.4g	Stand.	.502	-.468	.388	-.392	2063	-1884	1307	-154	.970	.895
	Frict.	.351	-.340	.276	-.203	1195	-1108	689	-542	.727	.502
Mexico 0.1g	Stand.	.427	-.364	.563	-.541	1181	-1275	929	-923	.419	.495
	Frict.	.345	-.325	.332	-.351	901	-900	469	-453	.530	.527
Mexico 0.25g	Stand.	.883	-.797	.850	-1.117	2122	-2304	1797	-1741	1.300	1.388
	Frict.	.540	-.489	.645	-.460	1240	-1063	536	-512	.996	.992

The maximum story drifts are shown in Fig.9. For the 0.25g Mexico earthquake, the average maximum story drifts of the frames with standard connections are 30% and 40% greater than the frames with friction connections for the frames No.1 and No.5, respectively. For the 0.4g El-Centro earthquake, the average increase in maximum story drifts are 48% and 52%.

The damage in terms of plastic hinges in columns and beams are shown in Fig.10. It is seen that no columns or beams yielded in the frames with friction connections. For the standard moment resisting frames subject to 0.25g level of the Mexico earthquake, the percentage of yielded columns or beams are, respectively, 41% and 15% for frame No.1, and 20% and 65% yielded for frame No.5. It is also can be seen from Fig.10 that the friction connections at most stories did slip, and thereby dissipating the seismic energy to prevent the beams and columns from yielding.

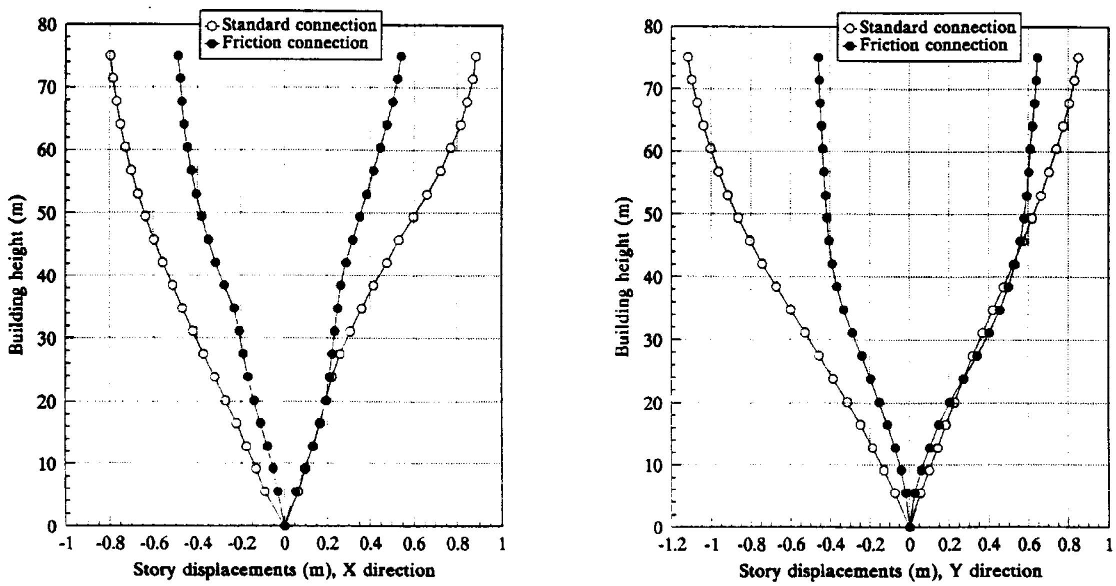

Fig.7 Deflection envelopes of F1 and F5, Mexico 0.25g

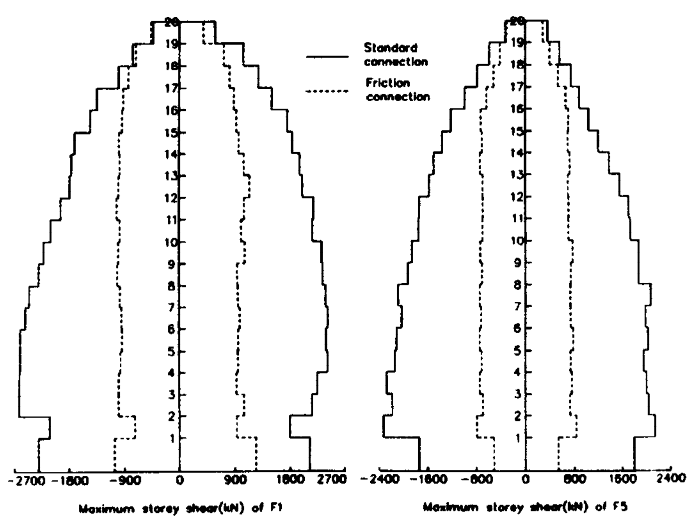

Fig.8 Maximum story shear, Mexico 0.25g

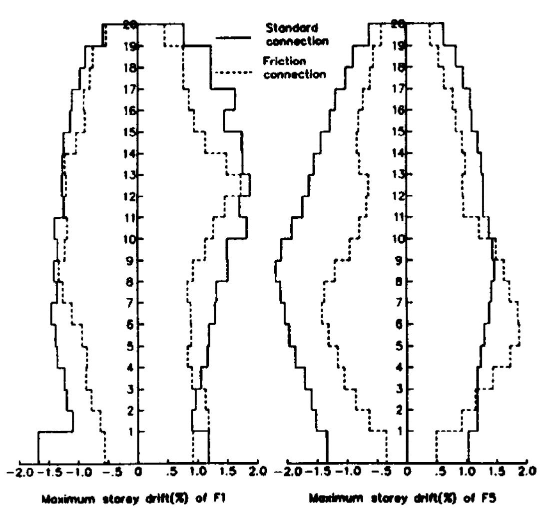

Fig.9 Maximum story drift, Mexico 0.25g

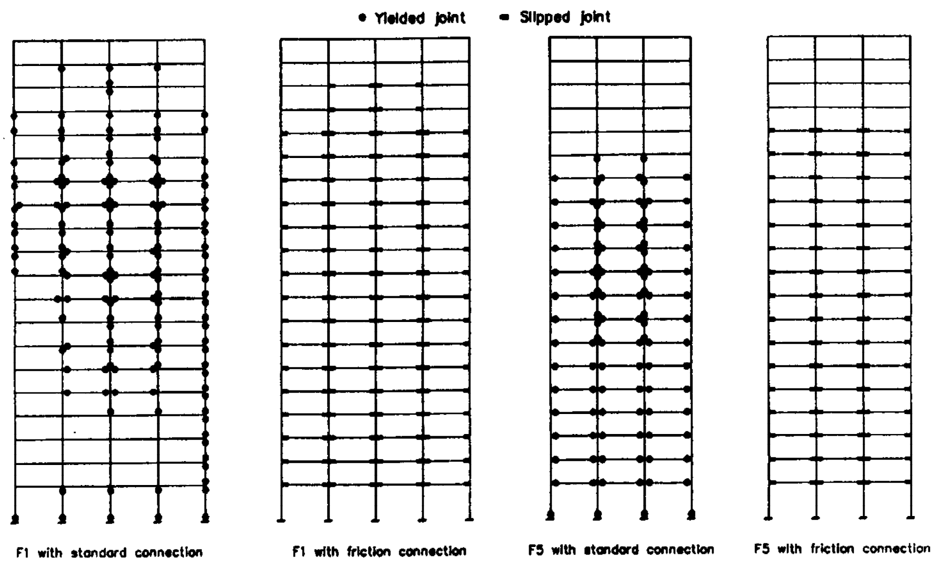

Fig.10 Yielded or slipped joints, Mexico 0.25g

DUCTILITY DEMAND

The maximum rotations in the friction connections are shown in table 4.

Table 4. Maximum rotations of friction connection

Earthquake records	Story of building	Max. Rotation (radians)	Location (story No.)
El-Centro	8	0.00120	2
0.4g	20	0.01218	5
NBK	8	0.01850	2
0.4g	20	0.01687	2
Taft	8	0.00705	1
0.4g	20	0.00516	18
Mexico	8	0.02971	3
0.25g	20	0.01691	6

Referring to Fig.1(a), the connection slippage is found as $\Delta=\theta \cdot d/2$, where θ is the rotation of the connection, and d can be taken to be the total depth of the beam. This equation gives the maximum slippage less than 10mm, and thus the required additional clearance in the slotted holes should be about 20mm.

From the analytical results conducted on these two buildings, it is seen that the use of the friction connection in the moment resisting frame resulted in an upgraded system with a significantly enhanced seismic performance with respect to that of the standard moment resisting frames.

CONCLUSIONS

Two different moment resisting frames having conventional connections and friction connections have been analyzed. Based on this limited investigation, the following conclusions can be drawn:

1. With the friction connections, the rotational capacity of beams can be substantially increased without cracking or yielding of the connections and members.
2. Frames with friction connections distribute the required inelastic excursions in the friction connections throughout the building and therefore, the potential for energy dissipation is much greater than conventional MRFs.
3. Frames with friction connections is softened by friction slipping, attracting much smaller seismic forces. In addition, the earthquake energy is dissipated by friction slipping rather than by yielding of structural elements.

The new type of connections represents a simple modification of standard conventional connections. It is easier to implement and less expensive than other damping devices.

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