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Seismic analysis of structurally interconnected steel frames

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ABSTRACT

This paper evaluates the seismic performance of a pair of closely spaced plane steel frames interconnected by a horizontal structural link to prevent pounding during earthquake excitations. Friction damping capability is incorporated into the modelling of this link in order to also determine its potential for dissipating the seismic energy. The frames, one three-storey, the other eight-storey, are individually designed according to the 1990 edition of the National Building Code of Canada. A parametric study, utilizing nonlinear time-history dynamic analysis, is performed to determine the influence of the slip load for the structural link when the frames are excited by artificial accelerograms representative of an upper bound of the potential ground motion in Vancouver. It is shown that a structural link with friction damping capabilities reduces the ductility demands of both frames compared to a purely elastic link.

INTRODUCTION

Structural damage caused by two buildings, or different parts of the same building, impacting one another during an earthquake has been observed on numerous occasions over the past several decades. For example, during the 1972 Managua earthquake the third floor of the Grant Hotel in downtown Managua, completely collapsed when hit by the roof level of an adjacent two-storey building (Berg and Degenkolb, 1973). Also, the fourteen-storey Westward Hotel suffered damage when it pounded against its low-rise six-storey wing during the 1964 Alaska earthquake (National Academy of Sciences, 1973). More dramatic pounding failures were observed recently in Mexico City in 1985 (EERI, 1985), and in Santa Cruz during the 1989 Loma Prieta earthquake (Asteneh et al., 1989). The problem of pounding is particularly acute in many large cities located in seismically active regions where, due to land usage requirements, buildings are constructed in very close proximity to each other.

Although pounding may constitute one of the primary sources of structural damage during an earthquake, limited research has been conducted on the subject. A literature review reveals that the effect of pounding has been considered in investigations on the response and/or collapse of particular buildings (Mahin et al., 1976; Wada et al., 1984; Wolf and Shrikerud, 1980). Also, some valuable insights on the problem of pounding have been obtained recently from an analytical study (Anagnostopoulos, 1988). In particular, it was shown that for structures aligned in series the response of the end structures can be magnified while the response of the interior structures can be reduced. This result agrees well with observed damage patterns.

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Most modern building codes address the problem of pounding by requiring that adjacent structures be either separated by the sum of their anticipated individual deflections or be connected to each other. Although it seems clear that connecting buildings together will force them to vibrate as a unit, and thereby reduce the pounding potential, very limited information is available on the actual seismic response of interconnected structures. The elastic vibrational response of coupled plane frames was recently investigated (Westermo, 1989). However, the effect of the coupling on the inelastic structural response was not included in this study and therefore the results may not be realistic since most structures will undergo inelastic deformations under severe earthquakes.

This paper attempts to shed some further light on the problem of interconnected buildings by considering a case study. The inelastic earthquake response of a pair (three and eight storey) of closely spaced, code designed, plane steel frames interconnected by a horizontal link is computed using nonlinear time-history dynamic analysis. In addition, the energy dissipating potential of the structural connection is investigated by considering a linkage system that incorporates friction damping capabilities.

STRUCTURAL MODELS AND ASSUMPTIONS

Building Models

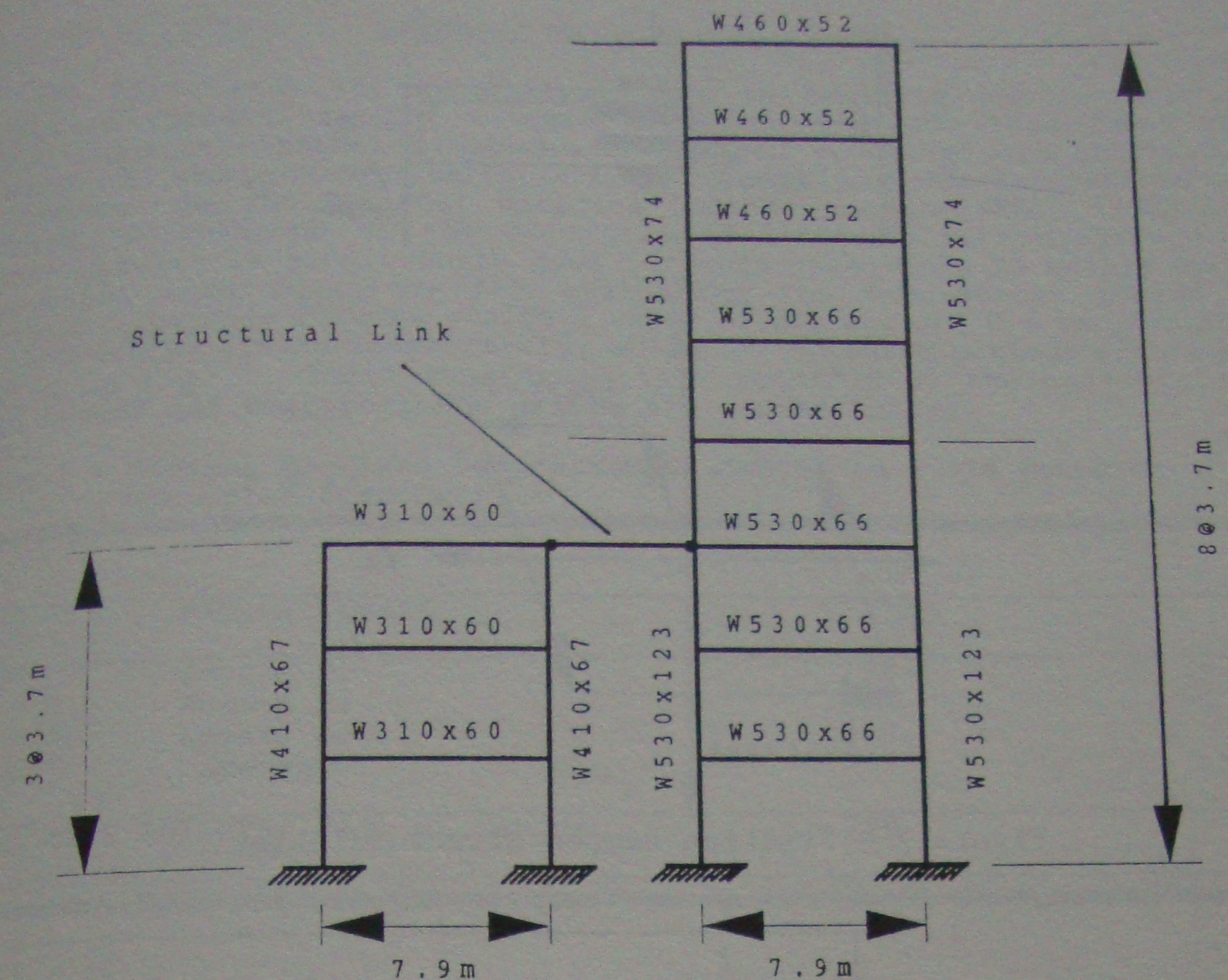
The pair of plane steel framed building models considered in this case study consisted of an eight-storey frame connected to the roof of a three-storey frame. The frames were designed individually as ductile moment resisting frames (R_{μ}=4) according to the static method of the 1990 edition of the National Building Code of Canada (NBCC, 1990). The new seismic detailing requirements of the CAN/CSA-S16.1-M89 (CSA, 1989) Canadian steel code was incorporated into the design. The "weak-beam strong-column philosophy" was utilised by specifying the beams as the critical elements. The resulting structural properties along with some of the design assumptions are detailed in Fig. 1. Note for simplification that the two structures considered have identical floor elevations. For the case where floors of adjacent buildings are not in horizontal alignment, a special vertical beam can be used to span two adjacent floors on one of the buildings (Westermo, 1989). The horizontal connecting link could then be attached to this beam. Horizontal linkage is obviously preferable so as not to introduce any supplementary uplift on the buildings.

Structural Interconnection

For the purpose of this case study the axial stiffness of the linkage system was set equal to the axial stiffness of the more flexible connecting beams (W310x60). In order to investigate the potential of dissipating seismic energy through the linkage system, friction (Coulomb) damping devices were incorporated into their design as illustrated in Fig. 2. In practice, the idealized hysteretic behaviour shown in Fig. 2. could be achieved by inserting heavy duty brake lining pads between clamped steel surfaces. An earlier experimental investigation by the first author has shown that this type of friction damping device produces very stable non-deteriorating hysteresis loops (Filiatrault and Cherry, 1987).

Earthquake Ground Motions

The structural models were subjected to a set of three earthquake records, which represents an upper bound of the potential ground motion in Vancouver originating from a subduction earthquake off the west coast of Vancouver Island. These accelerograms were generated from random vibration theory (Filiatrault and Cherry, 1988) based on a magnitude M_L of 8.5 and an epicentral distance of 150 km. The peak acceleration of each record was 0.35g with a duration of 25 seconds.



Dead Load:	150kN/floor	150kN/floor
Live Load:	300kN (first floor)	300kN (first floor)
	150kN (upper floors)	150kN (upper floors)
Design Base Shear:	75kN	185kN
Uncoupled Period:	0.81s	1.51s
Coupled Period	1.43s	

Figure 1 - Structural Models for Case Study.

The absolute acceleration response spectra corresponding to the seismic events are presented in Fig. 3.

<u>Structural Analysis</u>

All structural responses were obtained from inelastic time-history dynamic analyses using a microcomputer version of the well-known general purpose program DRAIN-2D (Kannan and Powell, 1973). Flexural and axial deformations were monitored in the structural members and the interaction between axial forces and moments at yield were taken into account by means of standardized yield interaction surfaces for steel members. No viscous damping was considered in the

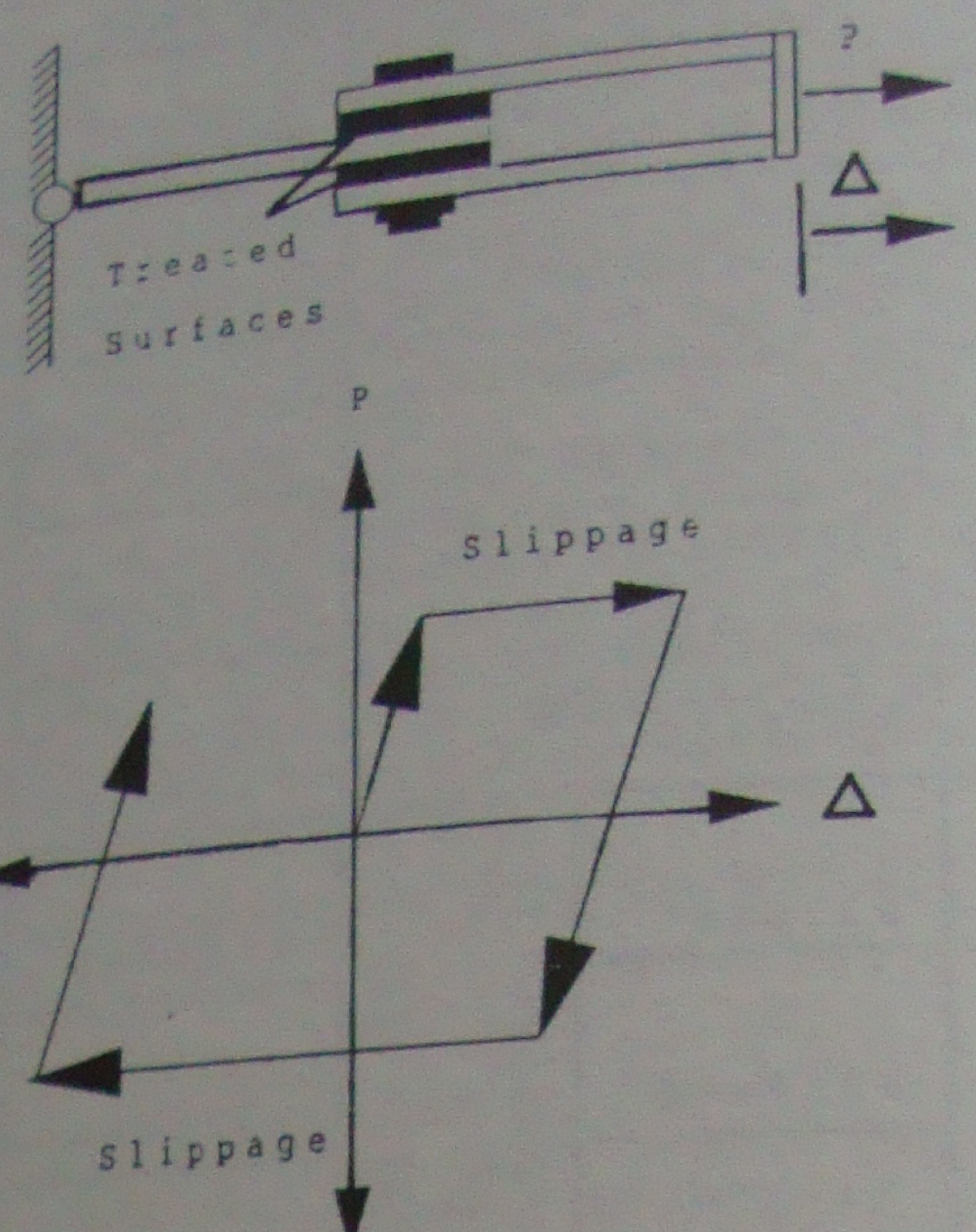


Figure 2 - Friction Damped Structural Link.

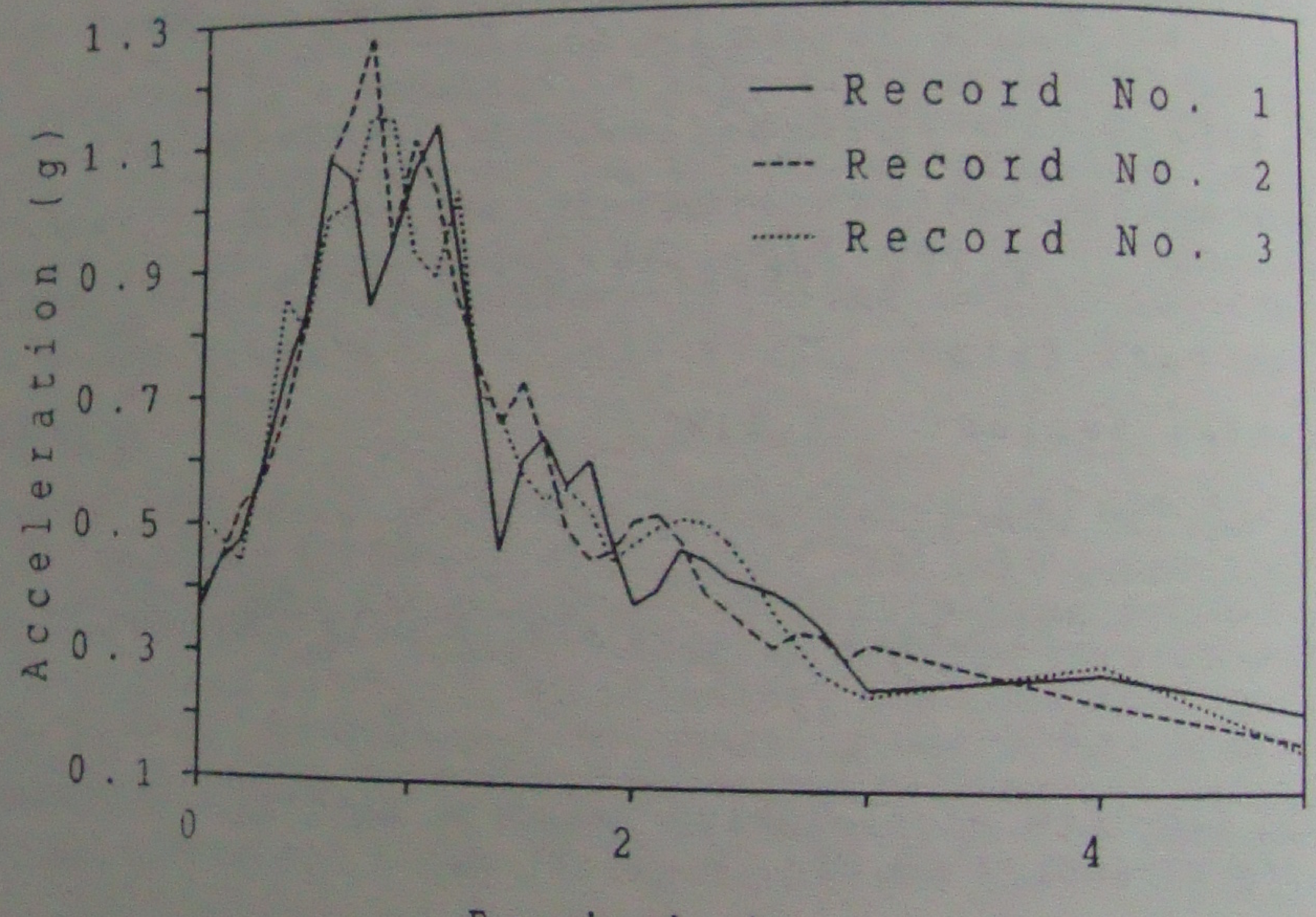


Figure 3 - Acceleration Response Spectra for Seismic Events (5% Damping).

structures so that the proportion of energy dissipated by friction through the linkage system could be more easily identified. Rigid foundations were assumed and soil-structure interaction was neglected. The full dead load was applied to the members prior to the shaking.

RESPONSE OF UNCOUPLED SYSTEM

The first step in the analysis was to evaluate the response of the uncoupled structures to the seismic events considered. From these results, minimum required separation distances to avoid pounding were obtained. These distances (S_r) are presented in Table 1. For comparison, the separations required for Vancouver by the National Building Code of Canada (NBCC, 1990) are also presented. Correlation of the code results with the full dynamic inelastic response results is surprisingly good for this case. This is mainly due to the conservative code equations for evaluating the fundamental period of the structures. The code formula yields a value of 0.3 sec and 0.8 sec for the three and eight-storey frames respectively, while the computed periods are much longer as noted in Fig. 1. These shorter periods required by the code yield larger design forces and therefore the drifts are overestimated.

Table 1 - Minimum Required Separation Distances to Avoid Pounding of the Uncoupled Structures.

Seismic Event	S_r (mm)
Record #1	235
Record #2	290
Record #3	240
Average	255
National Building Code of Canada	290

INFLUENCE OF SLIP LOAD OF LINKAGE SYSTEM

The energy dissipated by the friction damped interconnection is simply equal to the product of the slip load and the total slip travel. For very high slip loads the energy dissipated by friction will be zero, as there will be no slippage. In this situation the buildings will respond as an elastically coupled system. If the slip load is very low, large slip travels will occur but the amount of energy dissipated by the linkage system will again be negligible. In this case, the structures will approach the behaviour of the uncoupled system. Between these extremes, there may be an intermediate value of the slip load which would result in optimum energy dissipation and thereby minimize structural response. This intermediate value is defined as the "Optimum Slip Load". An optimum slip load study was carried out for the building pair excited by the three earthquake records considered in this investigation. A series of inelastic time-history dynamic analyses was performed for different values of slip loads in the elastic range of the linkage system. Figure 4 presents the results of the optimum slip load study. The results are given in terms of the base shear coefficient C and the slip load ratio S, which are defined as

$$C = \frac{\textit{Maximum Base Shear}}{\textit{Weight of Building}} \qquad [1]$$

$$S = \frac{\textit{Slip Load of Linkage System}}{\textit{Yield Load of Linkage System}}$$

where the yield load of the linkage system is simply the product of the cross-sectional area of the link with its yield stress.

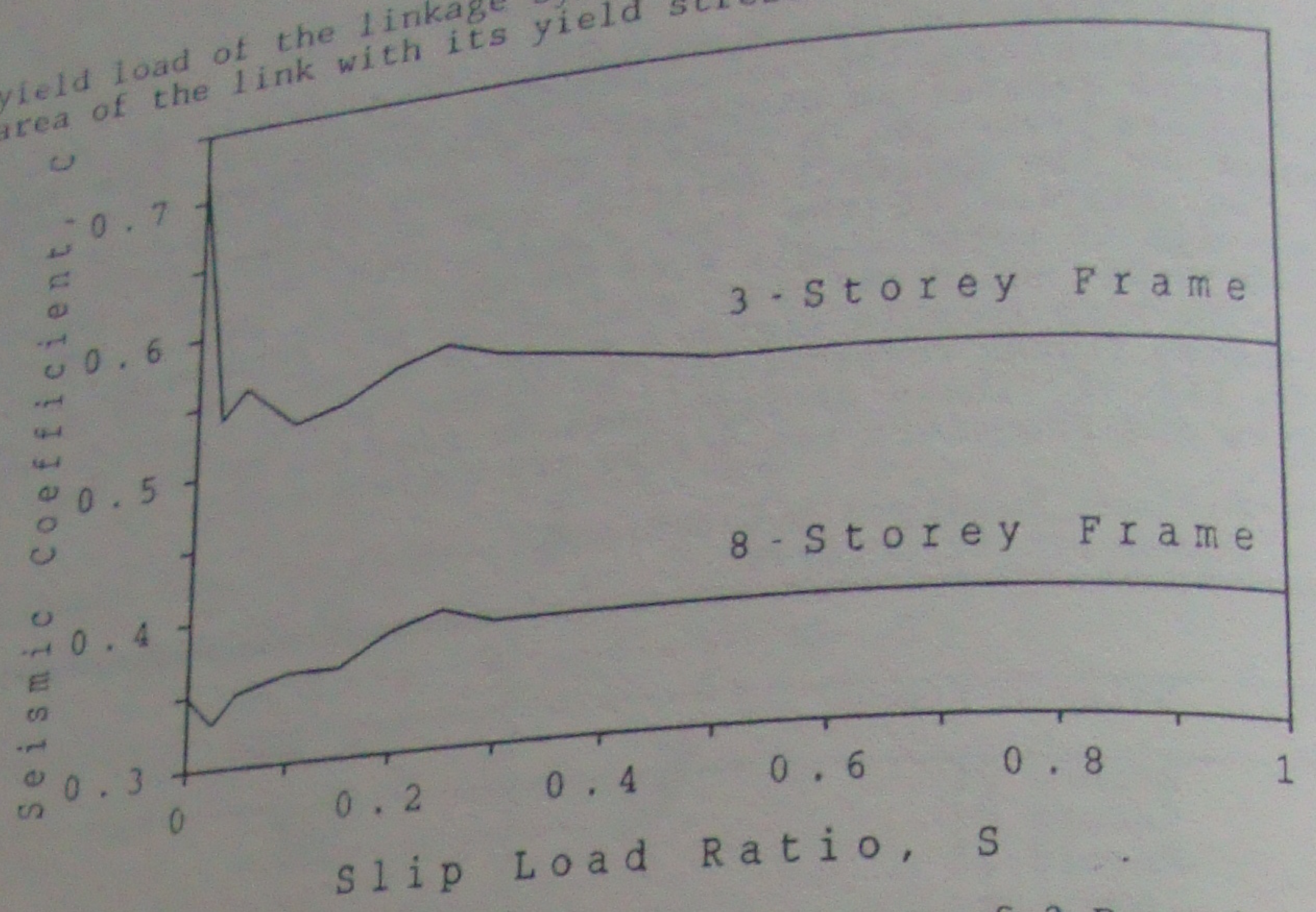


Figure 4 - Optimum Slip Load Study (Average of 3 Records).

For the cases considered the axial loads induced in the linkage system were small and therefore small slip loads were required to cause slippage of the friction devices. Quantitatively for slip loads higher than 50% of the yield load of the link, no slippage was observed. From Fig. 4 it can be seen that the slip load does not significantly influence the base shear developed by the structures. For the three-storey frame, a reduction of base shear of 26% is observed for a slip load ratio S=0.10, compared to 25% for an elastic link. For the eight-storey frame the elastic linkage increases the base shear by 9% while for S=0.025, the base shear is reduced by 6%. While it is not obvious that the use of a friction link is beneficial, a value of S=0.025, which induced a reduction of base shear in both frames, will be used for comparison in the subsequent detailed analyses.

COMPARATIVE DETAILED SEISMIC RESPONSES

In this section the distribution of structural damage in terms of inelastic response in the various members of the two structures after the end of each earthquake is investigated. Three different linkage configurations are compared: 1) the uncoupled system (S=0.0); 2) the elastically coupled system (S>0.50) and 3) the friction damped coupled system (S=0.025).

The plastic hinge distributions are illustrated in Fig. 5 along with the ductility demands. The response quantities presented represent averages over the three earthquake records. A plastic hinge is recorded in Fig. 5 when the section yielded in at least one of the earthquakes.The ductility ratio is based on an assumed plastic hinge length. When an element end moment is in the plastic regime, the ductility ratio is defined as one plus the ratio of the plastic curvature developed during the largest excursion into the plastic range to the yield curvature for the section. The plastic curvature is obtained by assuming that the plastic hinge rotation takes place on a length equal to the depth of the member. It can be seen from Fig. 5 that the uncoupled frames behave according to the code design philosophy, i.e. the plastic hinges are forming in the beams first. However significant ductility demands (>5) occur in the lower beams and base columns of the three-storey frame and in the upper beams of the eight-storey

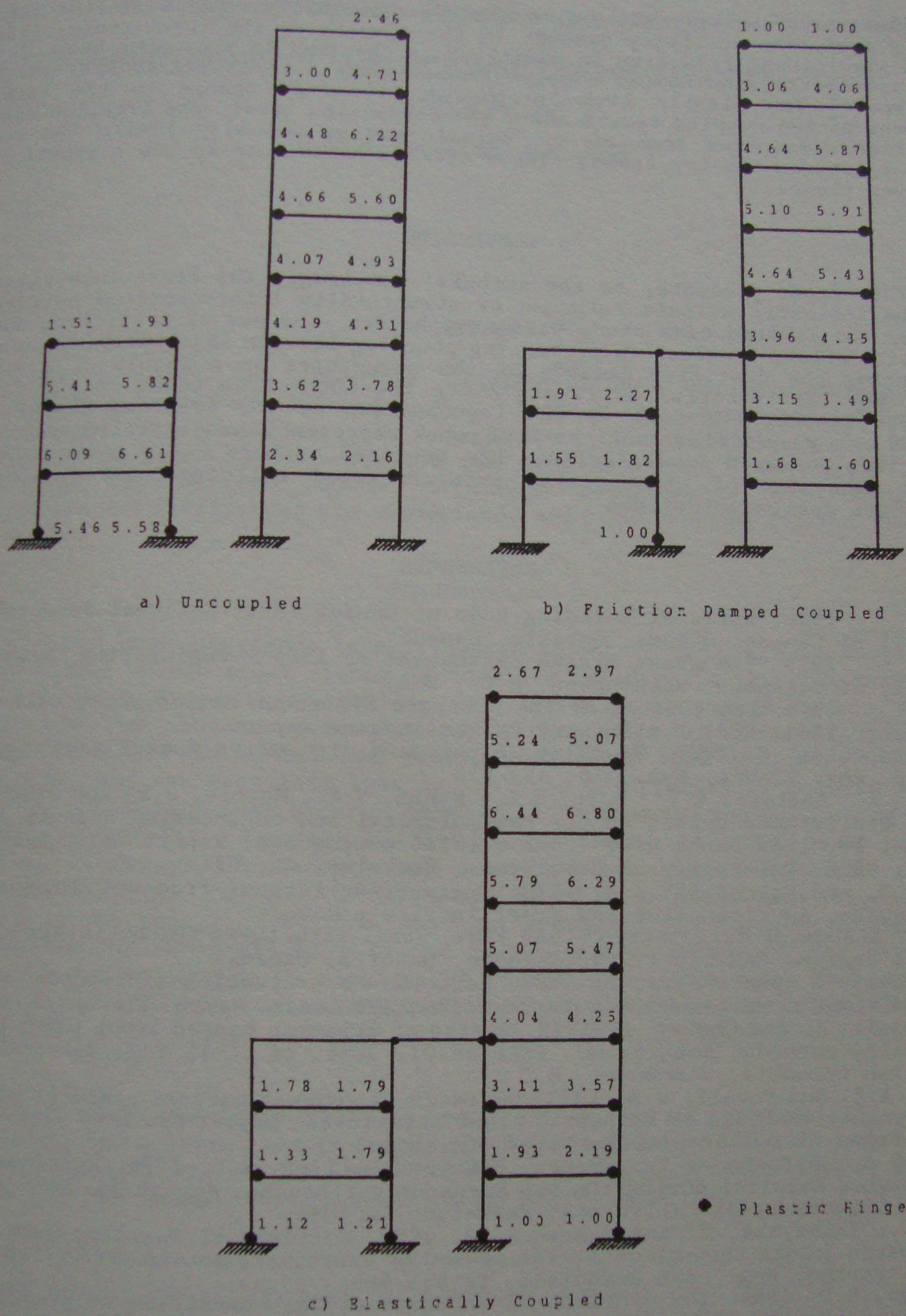


Figure 5 - Plastic Hinge Distributions and Ductility Demands.

structure. Both the friction damped and the elastically coupled systems reduce significantly the ductility demands in the members of the three-storey frame. Also a significant reduction in ductility is observed in the upper beam of the eight-storey friction damped coupled frame compared to it elastically coupled counterpart. The elastic link introduces an abrupt change in the lateral stiffness of the coupled system above the connection level. The friction damped connection dissipates some of the seismic energy and also limits the force transfer between the two frames. These effects contribute to the protection of the upper floors.

CONCLUSION

This paper presents, to the authors' knowledge, the first investigation into the inelastic seismic response of structurally interconnected buildings. Based on the limited case study presented herein, the use of a friction damped connection offers promises in preventing pounding between adjacent buildings and in reducing the ductility demands of the structures as a result of a strong earthquake. Particularly the levels above the connection point in the taller structure benefit the most by the introduction of the friction damped link compared to a purely elastic link. This paper represents only a preliminary study and further research investigations are required to more fully understand the dynamic behaviour of structurally interconnected buildings and to develop appropriate design guidelines.

REFERENCES

__________ 1990. National building code of Canada 1990. National Research Council of Canada, Ottawa, Ontario, Canada.

__________ 1973. The great Alaska earthquake of 1964 - engineering. National Academy of Sciences, Washington, D.C., USA.

__________ 1985. Impressions of the Guerrero-Michoacan Mexico earthquake of 19 September 1985. EERI Preliminary Reconnaissance Report.

Anagnostopoulos, S. 1988. Pounding of buildings in series during earthquakes. Earthq. Eng. Struct. Dyn., 16, 443-456.

Astaneh, A., Bertero, V.V., Bolt, B.A., Mahin, S.A., Moehle, J.P. and Seed, R.B. 1989. Preliminary report on the seismological and engineering aspects of the October 17, 1989 Santa Cruz (Loma Prieta) earthquake. Report No. UCB/EERC-89/14, EERC, University of California, Berkeley, CA, USA.

Berg, G.V. and Degenkolb, J.H. 1973. Engineering lessons from the Managua earthquake, American Iron and Steel Institute Report.

Canadian Standards Association (CSA) 1989. Steel structures for buildings - limit states design. CAN3S16.1-M89, Rexdale, Ontario, Canada.

Filiatrault, A. and Cherry, S. 1987. Performance evaluation of friction damped braced steel frames under simulated earthquake loads. Earth. Spect., 3, 57-78.

Filiatrault, A. and Cherry, S. 1988. Design of friction damped steel plane frames by energy methods. Reports No. EERL-88-01, Dept. of Civil Eng. University of British Columbia, Vancouver, B.C.

Kannan, A.E. and Powell, G.M. 1973. DRAIN-2D, A general purpose computer program for dynamic analysis of inelastic plane structures. Report No. EERC 73-6, EERC, University of California, Berkeley, CA, USA.

Mahin, S.A., Bertero, V.V., Chopra, A.K. and Rollins, R.G. 1976. Response of Olive View hospital during the San Fernando earthquake. Report No. EERC-76/22, EERC, University of California, Berkeley, CA, USA.

Wada, A., Shinoyaki, Y. and Nakamura, N. 1984. Collapse of building with expansion joints through collision caused by earthquake motion. Proc. 8th World Conf. Earthq. Eng., San Francisco, IV,855-862.

Westermo, B.D. 1989. The dynamics of interstructural connection to prevent pounding. Earthq. Eng. Struc. Dyn., 18, 687-699.

Wolf, J.P. and Shrikerud, P.E. 1980. Mutual pounding of adjacent structures during earthquakes. J. Nucl. Eng. Des., ASCE, 57, 253-275.