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FULL-SCALE TESTING OF CONCENTRICALLY BRACED AND FRICTION-DAMPED BRACED STEEL FRAMES UNDER SIMULATED SEISMIC LOADING

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ABSTRACT

This paper describes the results of full-scale cyclic and earthquake simulation tests of a cross-braced and a friction damped storey panel of a typical medium-rise steel frame building. Dynamic analyses were conducted on a finite element model of two six storey steel buildings: one with steel HSS cross bracing and one with friction damped bracing; the seismic deformations predicted by the analyses were applied as quasi-static displacements in the panel tests. The results of the cross bracing tests indicate that the cross bracing is very susceptible to fatigue fracture and is capable of significantly fewer cycles in the post yield range than was expected. This appears to be due to the coplanar interaction of the braces. The results of the friction damped bracing tests indicate that the friction damper provides a nearly rectangular and repeatable hysteresis curve. However, these dampers may produce significant secondary bending moments and axial forces in the bracing, which may have to be considered in the design of the system.

KEYWORDS

Full-scale tests; cyclic loading; earthquake simulation; cross-bracing; friction-damped braces.

INTRODUCTION

The Canadian steel design code (CAN/CSA-S16.1-M89) recently introduced new seismic design provisions which are intended to improve the earthquake performance of conventional steel building systems. In particular, ductile braced frame systems have strict requirements for the slenderness and width-to-thickness ratios of the braces. However, the adequacy of some of these specifications need to be verified experimentally under full-scale test conditions.

Recent analytical and experimental studies by numerous investigators have shown that the earthquake performance of conventional building systems can be significantly improved when fitted with energy dissipating devices. One such damping system is the Pall damper (Pall and Marsh, 1982), which introduces a slotted friction joint at the intersection of the cross bracing in a structural frame. Fig. 1 schematically illustrates one form of this system. Although the analytically predicted performance of the Pall damper has been verified in laboratory qualification tests on scale model structures (Filiatrault and Cherry, 1987), full-scale experimental validation of these devices is lacking.

The objective of the study presented in this paper is to evaluate experimentally the performance of a conventional concentrically cross-braced frame (CBF) and of a friction damped braced frame (FDBF) under full-scale testing conditions, involving both cyclic and simulated seismic loads. The results summarized here represent only a part of the overall study undertaken in this investigation.

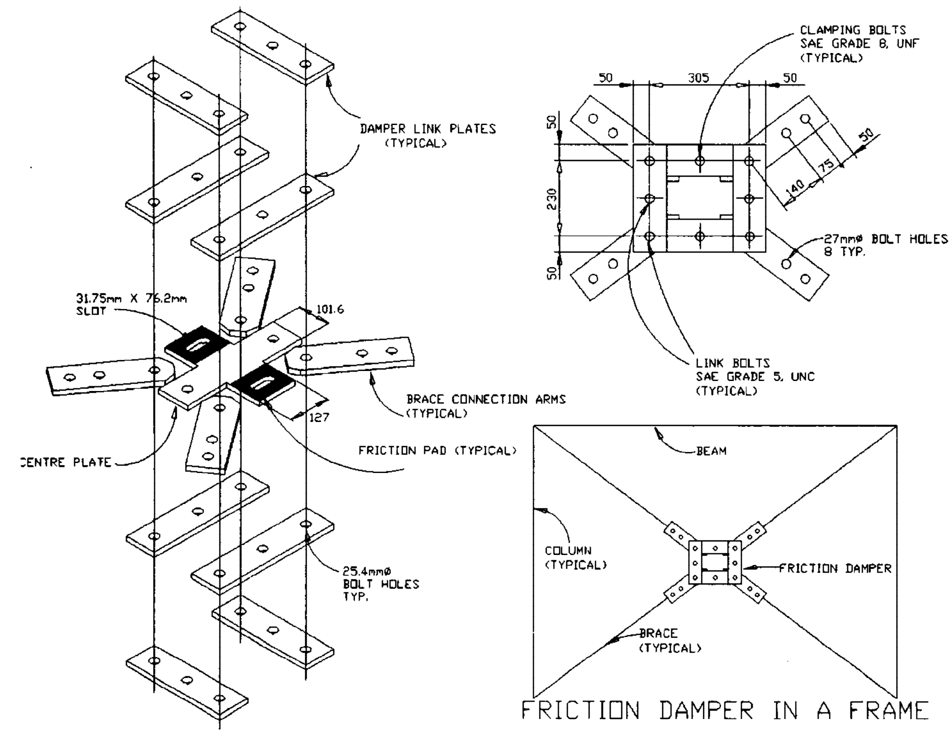

Fig. 1 - Pall Friction Damper.

To accomplish these objectives, a typical steel office building was designed in accordance with the seismic provisions of the Canadian steel design code, CAN/CSA-S16.1-M89, for ductile braced frames. The same design was used for the friction-damped building, except that the cross-braces were fitted with the Pall friction damper. Dynamic analyses were performed on both building frames using the design earthquake specified for a building site in Vancouver, Canada. Full-scale tests on the bracing in the most distressed panel of each frame type were conducted under reverse cyclic conditions and under the seismic deformations predicted by the computer analysis, but applied quasi-statically.

ANALYTICAL STUDY

The structure investigated is a typical medium-rise office building in the downtown core of Vancouver. The elevation and floor plan of the structure is illustrated in Fig. 2. Firm ground soil conditions were assumed at the building site.

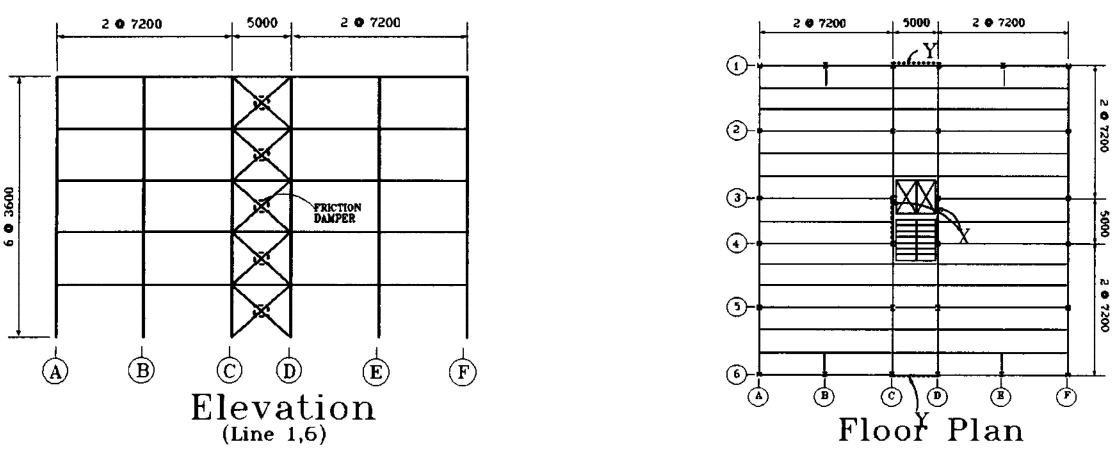

Fig. 2 - Six Storey Office Building.

Three sections were considered in the design of the cross bracing: a double-angle, wide flange, and hollow structural section. The double-angle was found to be very inefficient weight-wise. The HSS proved to be slightly more efficient than the W-shape. Since Black *et al* (1980) suggest that the HSS provides better energy dissipation than the W-shape, this section was chosen for the brace section.

The optimum slip-load for the friction-damped equivalent of the concentrically braced frame was predicted to be 100 kN using the method proposed by Filiatrault and Cherry (1990). This translates into a storey shear resistance of 80 kN for the panel configuration under consideration. The slip-loads used in the analysis were taken as uniform throughout the structure. Therefore, the shear strength of the building was the same in each storey.

Two-dimensional nonlinear dynamic analyses of the two frames were performed. The ground motion selected for the analyses was the first 20 seconds of the N-S component of the El Centro 1940 earthquake, since it reflects the acceleration/velocity ratio specified in the code for the Vancouver region. The earthquake accelerogram was factored to meet the specified peak ground acceleration of 0.21g, corresponding to the local 1 in 475 yr return period design earthquake.

A study of the interstorey drift is of prime importance since it is the factor controlling damage in the structure. Fig. 3 shows a graph of peak drifts for each storey of the CBF and FDBF under the design excitation. The CBF exhibits higher interstorey drifts in the upper floors whereas the FDBF had higher drifts in the bottom floors. This suggests that higher mode response was more significant in the CBF than in the FDBF. No damage (yielding) occurred in the FDBF; however, in the CBF the braces in all the storeys and one column on the fifth storey yielded.

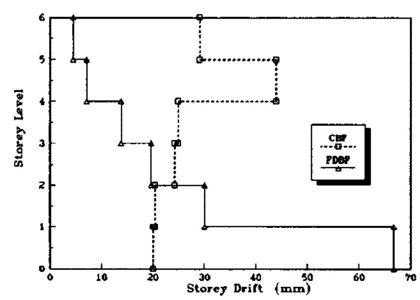

Fig. 3 - Peak Drift Comparison of CBF and FDBF.

Fig. 4 shows the envelopes of the beam moments, column moments, and column axial forces of the two different frames. The FDBF developed very high beam and column bending moments at the first storey. This results from the FDBF operating as a moment resisting frame during slip. Since the FDBF peak drifts are higher than those in the CBF at this location, it seems reasonable that the FDBF moments are also higher. The column axial forces are, however, higher in the CBF; this is due to the high stiffness and strength of the CBF braces.

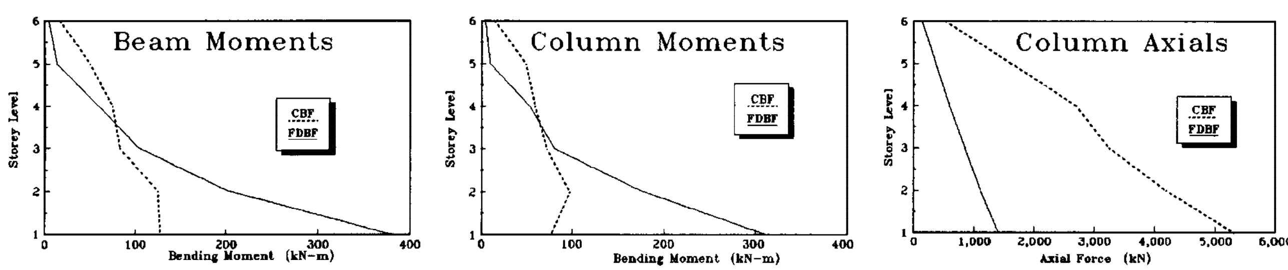

Fig. 4 - Comparisons of Bending Moments and Axials.

Comparisons between the response of the friction damped structure and the conventional braced frame structure under the design event indicate that, for the six storey building under consideration, friction damping does not necessarily lead to an improved response in all storeys. However, the friction dampers protect all the basic structural elements from yielding. It is expected that the overall FDBF response would be superior to the CBF response if both systems have similar modal contributions.

EXPERIMENTAL STUDY

The full-scale panel tests were conducted in the quasi-static horizontal displacement test frame shown in Fig. 5. The test frame is a truly pinned portal frame with steel pins at the four node points. The frame dimensions were chosen to match the building panel size. Displacements were imposed at the cross beam with a 400 kip double acting hydraulic actuator.

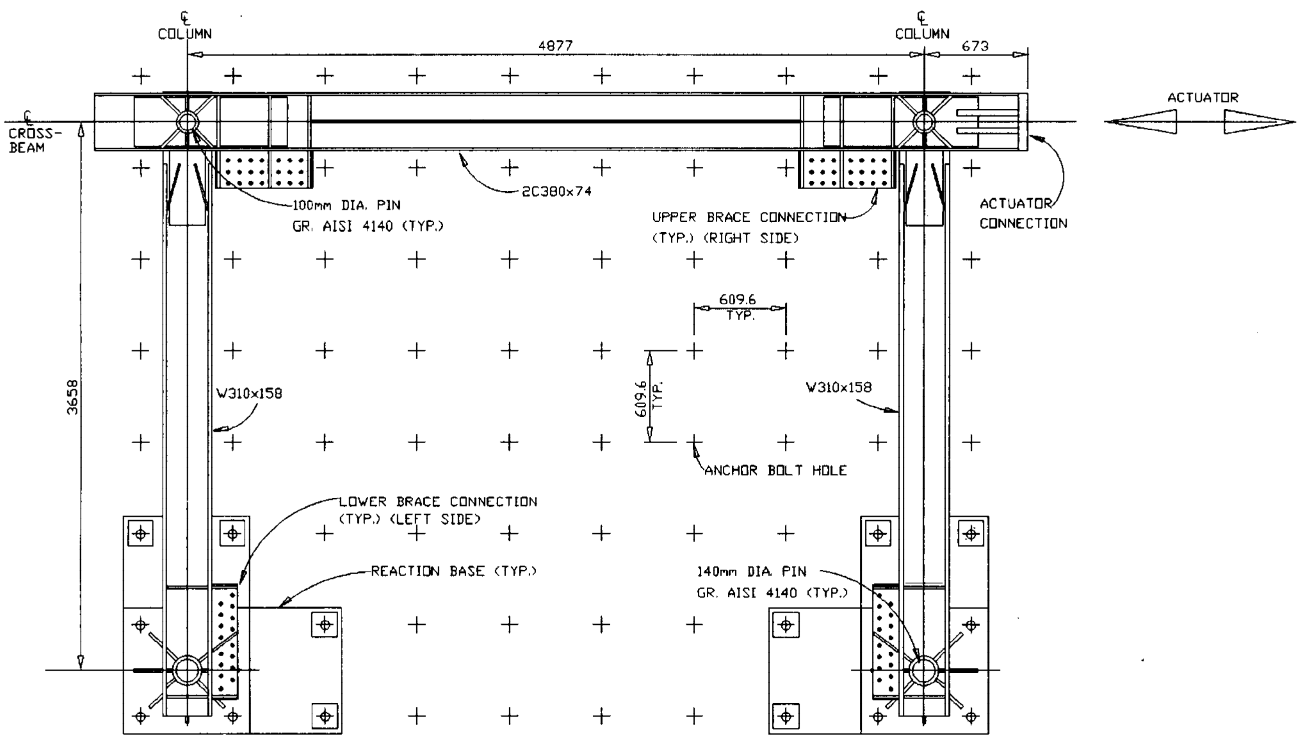

Fig. 5 - Quasi-static Test Frame.

Concentrically Braced Frame

The fifth storey panel of the six storey building used in the analytical investigation was chosen for the cyclic and the earthquake simulation tests. The analysis showed that this panel had the largest inter-storey displacements. The brace sections in this panel were HSS 127x76.2x6.35mm. One brace was continuous through the centre connection; the other brace was discontinuous but the brace force was transferred across the centre connection by cover plates.

Reverse Cyclic Test: The first specimen was tested under controlled reverse cyclic displacements. The cyclic displacements were increased by multiples of the yield displacement (full plastic shear capacity) of the panel. The specimen was subjected to two cycles at each displacement amplitude. Cyclic deformations were carried out until the failure of a brace occurred.

The applied cyclic frame deformations and the resulting hysteresis curves obtained during the test are shown in Fig. 6. The hysteresis curve exhibits very full loops from cycle to cycle, indicating good energy dissipation. Although the pinched behaviour typically associated with buckled braces is evident, it is not significant.

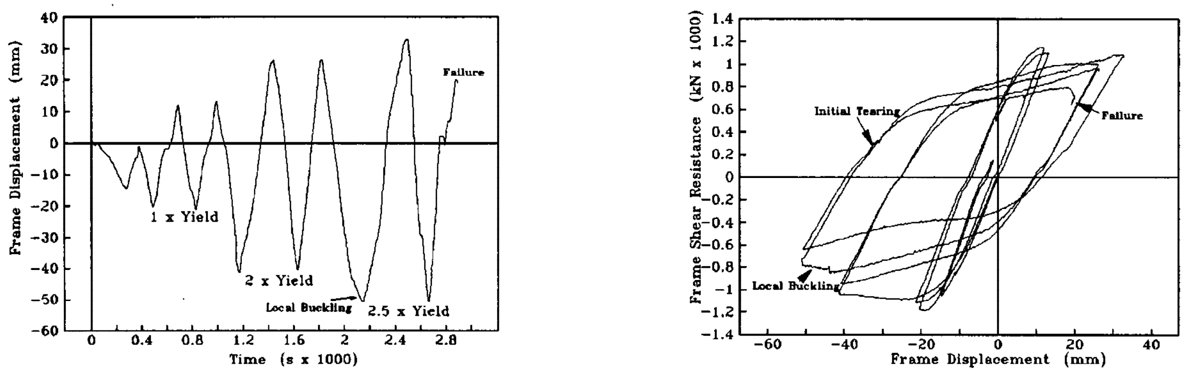

Fig. 6 - CBF Cyclic Response.

The measured maximum out-of-plane deflection of the buckling braces during this test was 170mm. The corresponding twist in the tension brace caused by the buckled brace was approximately 0.1 radians.

Local buckling was initiated in the lower left brace at the plastic hinge location (approximately midway between the end gusset and brace intersection) during Cycle 5. Upon restraightening of the brace, during the later part of Cycle 6, tearing began to occur at the corners of the HSS tube at the location of local buckling. The tears continued to open during increased displacements. Complete rupture of the section took place before the full displacement (2.5 times yield) of the second half of Cycle 6 was attained.

Earthquake Simulation Test: The applied drift time-history used for the earthquake simulation test and the resulting hysteresis curve obtained are shown in Fig. 7, along with the hysteresis curve predicted in the analysis. It is noted that since the test was conducted in a quasi-static manner due to equipment limitations, the time rate for the test differs from that of the analytical response. It was assumed that the small high frequency displacement cycles of the actual drift time-history would not significantly affect the test results and could therefore be neglected.

Local buckling was initiated in the lower left brace during Peak 17 at 11.0 seconds real time. Fatigue tearing began in the lower left brace during Peak 22, at 15.4 seconds. At 20 seconds into the seismic event, corresponding to Peak 27, approximately half of the HSS cross-section was torn.

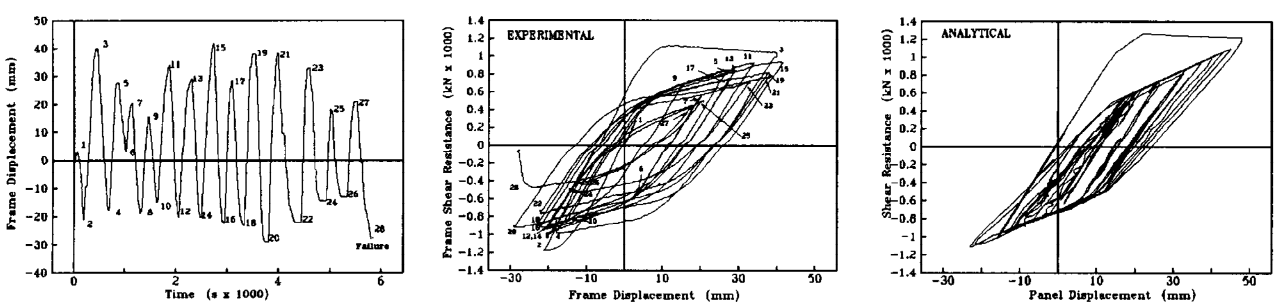

Fig. 7 - CBF Seismic Response.

The peak out-of-plane deflection during the simulation test was 150mm. This deflection is equivalent to 6% of the brace span.

Discussion: The earthquake simulation test was found to dissipate 2.5 times the energy predicted by the computer analysis. Thus, since more energy was dissipated than was expected, improvements in the computer model to account for this discrepancy could be expected to lead to a somewhat better (smaller) response prediction.

As other researchers have discovered, fatigue failure as a result of local buckling is particularly significant when using rectangular tubes as bracing members. The results of the tests conducted in this study are in agreement with those observations. However, especially in the cyclic test, significantly fewer cycles to failure than expected were obtained. It has been shown (Black *et al*, 1980) that single strut tubes having similar size and slenderness as the tubes used in the present study can achieve ten cycles to seven times yield displacement without failure. In the present cyclic test, the specimen failed within six cycles at displacements which reached only two and one half times yield displacement. This is thought to be the result of the coplanar interaction of the braces, whereby the buckled brace induced a twist induced in the tension brace that was well in excess of the torsional capacity of the HSS section.

Tang *et al* (1987) developed an empirical equation to predict the fracture life of rectangular HSS bracing members having similar proportions to those used in the present study, albeit with some variability. The equation is derived from tests on bracing arranged in a chevron configuration rather than X-bracing. When using the lower bound value of the empirical coefficient of the equation, Tang's formula predicts that the cyclic test in the present study should sustain 40 to 50 'standard' cycles. However, the loading history to failure is equivalent to only 11 'standard' cycles, as defined by Tang. Thus, only one quarter of the number of predicted cycles to produce fracture were realized. This suggests that the effects of the coplanar system in the 'X' configuration reduces the fracture life of the bracing.

The energy dissipated by the cyclic test and the earthquake simulation test at the first occurrence of local buckling was 180 and 313kN-m, respectively. The energy at failure was 274 and 360kN-m, respectively. Clearly, the loading history has a significant influence on the fracture life of the brace.

Friction Damped Braced Frame

The brace sections used in the tests were double-angle 65x65x5mm and were friction connected to the damper and to the frame.

Reverse Cyclic Test: The hysteresis curves resulting from two 30 cycle tests are shown in Fig. 8. These curves correspond to slip-load settings resulting in frame shear resistances of 45kN and 90kN, respectively. It can be seen that a fade in the slip-load of approximately 15% occurred during the 30 cycles for the higher slip-load; negligible fade occurred in the other test. The 90kN damper slip-load was experimentally checked within a few hours after completion of the cyclic test and no measurable change in this value was noted.

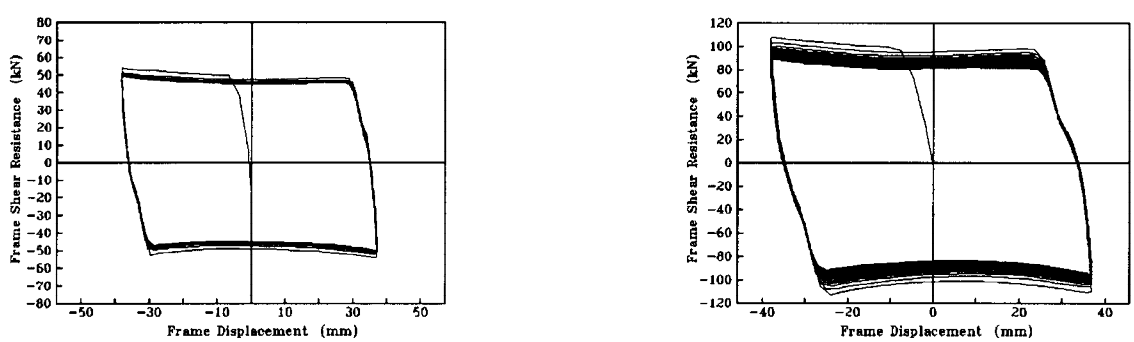

Fig. 8 - 45kN and 90kN Slip-load Cyclic Response.

Earthquake Simulation Test: Earthquake simulation tests were conducted to compare the analytically predicted response with the actual measured response. The slip-load setting was 100kN, resulting in an 80kN frame shear resistance. As in the CBF test, the small high frequency cycles in the calculated drift were assumed to have negligible effect on the response, and the simplified drift time-history presented in Fig. 9 was used for the full-scale simulation test. Also, the time rate used in the FDBF test differs from the actual rate of the analytically predicted response.

The simplified drift time-history and the associated predicted and experimental panel shear hysteresis curves of the first storey in the six storey building are shown in Fig. 9. No evidence of yielding in the braces was observed.

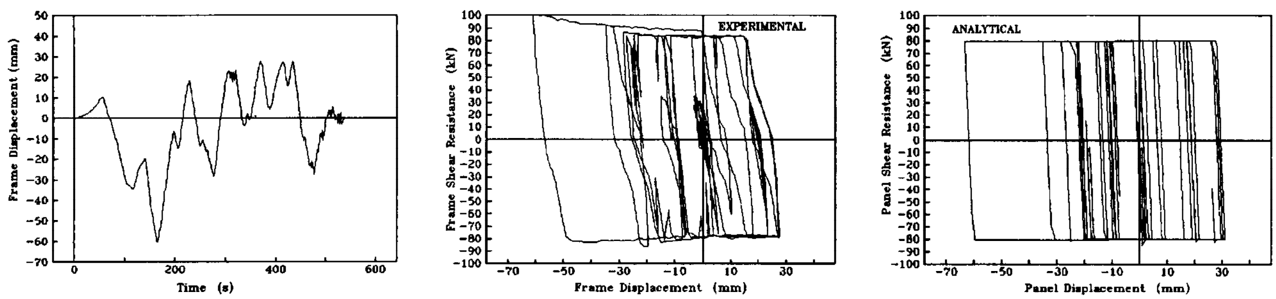

Fig. 9 - FDBF Response.

Discussion: Very nearly rectangular and very repeatable hysteresis curves were developed during the cyclic tests. The 90kN slip-load test was found to have only 15% fade in the slip-load over 30 cycles. The energy dissipated in the earthquake simulation test was only 17% lower than the predicted value.

The test data show that although the shear resistance of the panel is relatively constant with slip, the brace force is not. As seen in Fig. 10, the compression force varies from 50% to 25% of the total brace force. This appears to be the result of a prestress induced into the system as the frame deforms. If one compares the diagonal lengths of the frame in the deformed position with those of the damper in the deformed position, it becomes evident that tensile forces must be developed to maintain compatibility within the system. These forces become more significant the smaller the ratio of the damper to the frame. In this study the damper dimensions were one sixteenth of the frame dimensions. The level of prestress is also expected to be a function of the brace section size; larger braces are expected to develop higher tensile forces. The tests also indicate that the braces must undergo bending deflections if the brace-to-frame connection can develop bending moments. This brace bending appears to increase as the frame displacement increases and appears to be a forced constraint. Therefore, it may be possible that very high bending moments and corresponding shear forces can be developed in the bracing and in the friction damper bolts. Space limitations in this paper prevent a full discussion (Kullmann, 1995) of these matters.

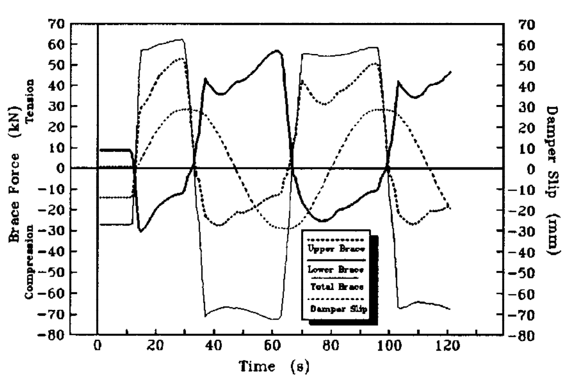

Fig. 10 - Typical Brace Force Time-History.

CONCLUSIONS

Conventional Concentric Cross-Bracing

The full-scale tests carried out in this study indicate that the HSS cross-bracing designed according to the Canadian steel code requirements for ductile braced frames possess very good energy dissipation properties with very little "pinching" behaviour. However, this may be at the expense of early fatigue failure. The tests show that compared to other bracing configurations, significantly fewer load cycles can be developed by such braces prior to fracture.

The coplanar interaction of the braces leads to torsional strains in the tension brace which may be in excess of the yield strains. This may contribute to the high energy dissipation and the low fracture life exhibited by this type of bracing configuration.

Friction-Damped Bracing

The Pall friction damper develops a very rectangular and repeatable hysteresis shape which closely approximates the hysteresis properties of its analytical model.

A mathematical analysis of the friction braced panel fitted with the Pall damper indicates that the system develops secondary tensile forces as the panel deflects laterally. The tests indicate that significant brace bending can occur if the brace connections are capable of developing bending moments. The brace and damper design may have to take these forces into account.

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