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Seismic rehabilitation of a four-storey building with a stiffened bracing system

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ABSTRACT

This paper reports on full scale quasi-static testing and nonlinear dynamic analysis of a chevron bracing system with buckling inhibited bracing members. The braces are made of a steel plate embedded in a steel tube filled with mortar. Special provisions are made to prevent friction between the mortar and the steel plate and the tube is sized to develop a buckling capacity greater than the axial yield resistance of the steel plate. The system has been examined for implementation in the seismic retrofit of a four-storey building in Quebec City. The measured response of the tested braced frame was very stable, with very effective energy dissipation capability and beneficial strain hardening response. The results of the analyses indicate that adequate seismic performance of the building can be achieved with the proposed system.

INTRODUCTION

A four-storey steel framed building erected in Quebec City in 1964 had to be seismically upgraded to post-disaster performance level. The existing lateral resistance was essentially provided by unreinforced masonry walls with floor and roof diaphragms made of individual precast concrete slab units. A strengthening scheme has been investigated which included the addition of 14 vertical steel braced frames to be installed along the perimeter of the building, outside of the exterior walls. Horizontal steel trusses were to be also added under the existing floor and roof structures to enhance the in-plane structural integrity of the diaphragms. The vertical braced frames were to be anchored on a new concrete grade beam supported on piles to be driven along the exterior walls.

Vertical bracing complying with the S16.1 (CSA, 1994) provisions for ductile braced frames was originally selected to achieve good seismic performance. In this system, bracing members are sized based on their compressive strength but capacity design provisions require all other elements in the lateral load path, including the horizontal trusses and the foundations, to be designed for the actual tensile resistance of the bracing members. In order to reduce the demand on these elements, an alternative bracing system has been examined in which bracing members are made of steel plates that yield both in tension and compression. Buckling of the plates in compression is prevented by embedding the plates in a steel tube filled with mortar (Maeda et al., 1998; Reina and Normile, 1997). Special detail is provided to minimise the friction between the mortar and the steel plate and the tube has a buckling capacity greater than the axial yield resistance of the steel plate. Because braces yield in compression, a cost effective chevron bracing configuration could be used with no risk of degradation of the storey shear resistance or stiffness due to brace buckling under cyclic inelastic loading.

In this paper, the bracing system is described and the results of a full-scale quasi-static loading test are presented. Results of nonlinear dynamic analyses of a typical braced frame to be used in the existing building are also presented and discussed.

BRACING SYSTEM

The bracing members which were fabricated for the test program are described herein to illustrate the bracing system. These braces are shown in Fig. 1. Yielding is constrained to the narrower segment of the plates. At both ends, the plates have been widened to ensure elastic response and avoid brittle fracture at the connections. A longitudinal stiffener was also added at the ends to prevent buckling of the plates outside the tubes. The reinforced portions of the plate extend 300 mm into the HSS members to achieve a good transition between the supported and unsupported segments of the plates. New Zealand steel code provisions (SANZ, 1989) were applied to minimise stress concentration where the width of the plates changes and the stiffeners did not extend in the narrow portion of the plates to avoid welding in the yielding portion of the plates.

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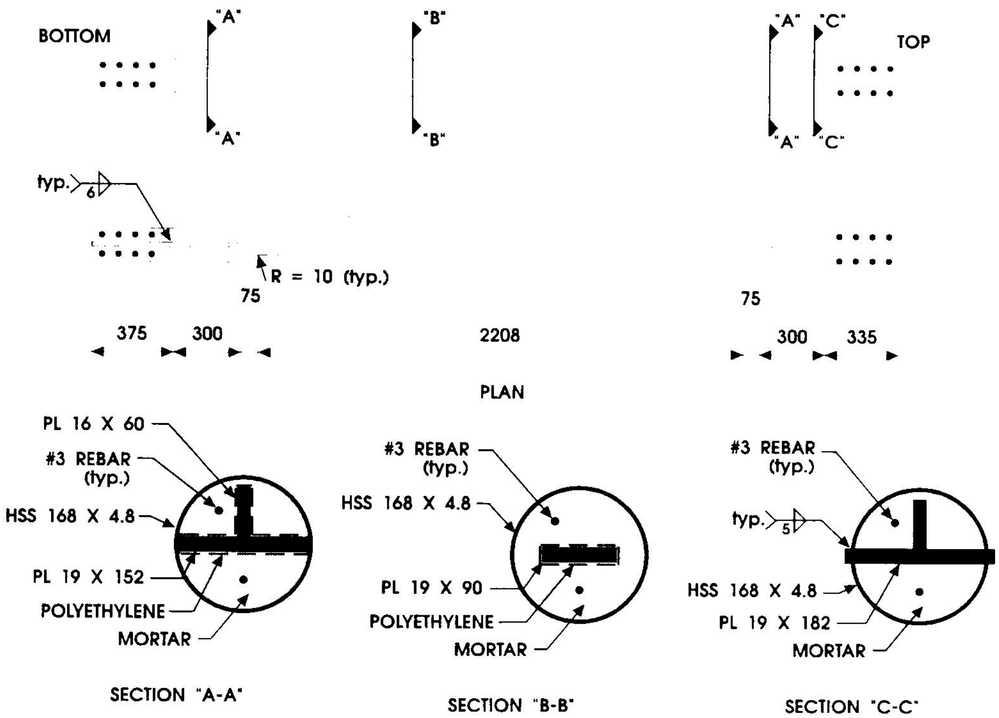

Fig. 1 Bracing member specimen.

The connections were designed to sustain 1.5 times the nominal yield load of the plates and the HSS had an elastic buckling strength greater than twice the yield load of the plates. At the top end of the braces, the tubes were welded to the steel plates to held the two elements together. Type M mortar was used to fill the tubes. Note that a bracing system in which the steel plates just fit in the tubes and for which no mortar is used (Iami et al., 1997; Morino et al., 1996; Shimokawa et al., 1998) was not retained in this study because the tolerances required to develop a proper lateral support for the plates appeared to be too tight to be met in practice. The mortar was poured vertically in the braces before their installation in the test frame. Two reinforcing bars were used to prevent cracking of the mortar during the manipulation of the bracing members.

Fig. 2 Flexible fill material.

The steel plates were wrapped with a membrane made of four plies of polyethylene 0.20 mm thick each prior to their insertion in the tube. The objective was two-folds: *i)* to permit the steel plates to easily slip in the mortar and, *ii)* to accommodate the increase in volume of the plates due to Poisson effects. The use of four independent plies also provided some redundancy to the system in case the outermost layer was damaged during the manipulation. To accommodate the longitudinal movement of the plate relative to the tube/mortar assembly, flexible fill material was used at the changes of plate width (Fig. 2).

TEST PROGRAM

A quasi-static cyclic test was performed at the Structures Laboratory of École Polytechnique of Montreal on the chevron bracing assembly shown in Fig. 3. The test frame is fully pinned at the four corners so that the applied lateral load was entirely resisted by the bracing members. The steel plates of the braces were made of G40.21-350W steel (measured properties: F_y = 357 MPa, F_u = 517 MPa) and the actual cross section area of the narrow segment was 1 737 mm^2. The yield load of the braces, P_y, was then equal to 620 kN, which correspond to a lateral load at yield, V_y, equal to 713 kN. Assuming a Poisson's ratio of 0.3 and 0.5 in the elastic and inelastic ranges, respectively, the width of the narrow portion of the plates was expected to increase by approximately 0.3 mm under an axial deformation equal to 4 times the yield deformation. Such an increase could be easily accommodated by the 0.8 mm thick membrane. At the time of testing, the mortar had a compressive strength of 14.2 MPa.

During the test, the applied lateral load, V, and interstorey drift, Δ, were measured. The longitudinal movement of the steel plate relative to the steel tube was also monitored at the bottom end of the bracing members. Strain gauges were glued at two locations along the steel tube of the left bracing member to measure the bending moment and axial load that developed in the HSS member. The ATC-24 loading sequence (ATC, 1992) was used with a yield displacement, Δ_y, equal to 7.0 mm. The test included 6 elastic cycles, 3 cycles at $\Delta = \pm\Delta_y$, 3 cycles at $\Delta = \pm 2\Delta_y$, 3 cycles at $\Delta = \pm 3\Delta_y$, and 2 cycles at $\Delta = \pm 4\Delta_y$.

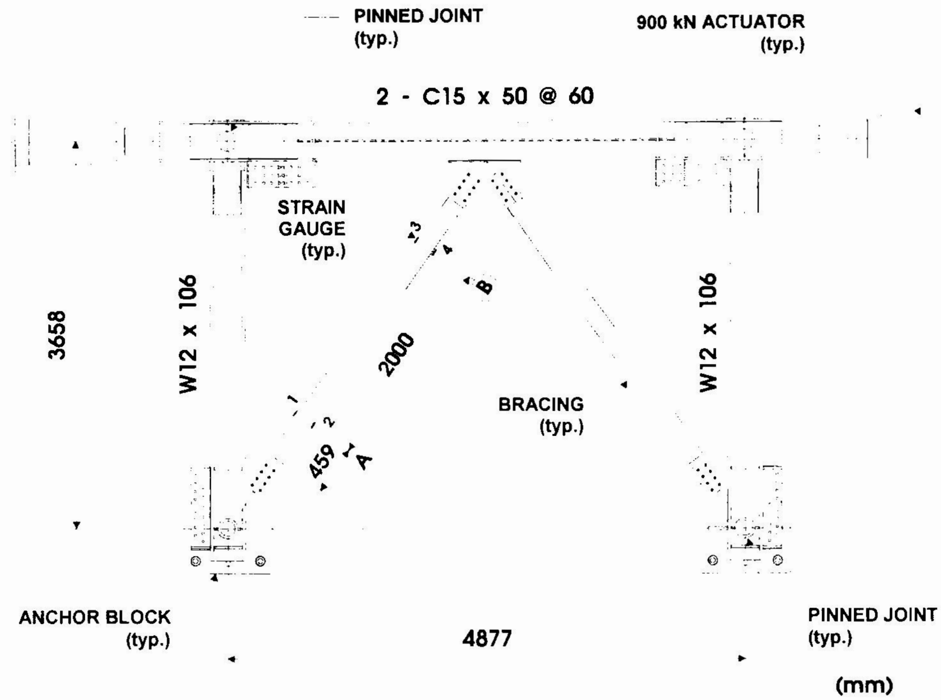

Fig. 3 Test Frame.

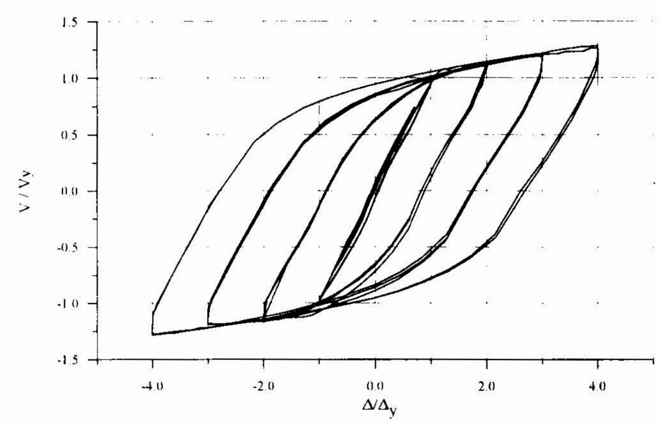

Fig. 4 Load-deformation response of the frame.

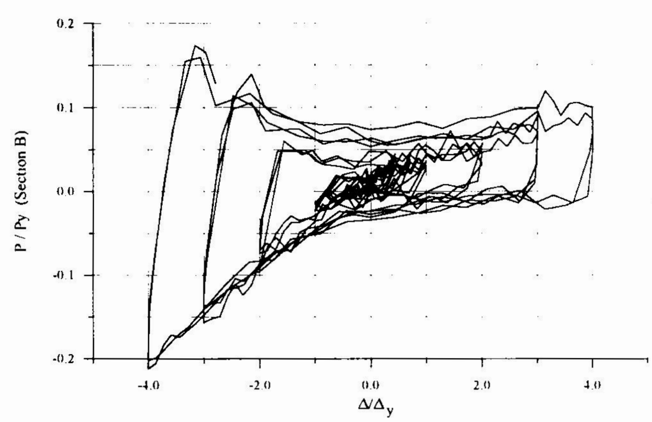

Fig. 5 HSS axial load-frame storey drift (left brace).

During the test, slippage of the steel plates relative to the tubes could be observed at the bottom end of each brace, both in tension and compression. At the end of the test, no damage could be noticed on the specimen, except that approximately 10 mm of membrane had been progressively pulled outside the mortar at the bottom end of both braces. The lateral load-deformation response of the test frame is shown in Fig. 4. The hysteretic behaviour is very stable and symmetrical, with effective energy dissipation capability. The system also exhibited significant strain hardening response. The post-yield stiffness varied between 10% and 15% of the elastic stiffness and the storey shear reached approximately 1.20 V_y at $\Delta = \pm 3\Delta_y$. Figure 5 shows the relationship between the axial load in the steel tube, as obtained from the strain gauge reading at section B of the left bracing member, and the frame drift. In this figure, the axial load is normalised with respect to the plate yield resistance, P_y, and Δ is positive when the frame is deformed to the right. As shown, large compressive axial load developed in the HSS when the frame was pushed towards the left in the inelastic range. At $\Delta = -3\Delta_y$, the HSS load reached approximately 15% of P_y. Upon storey drift reversal, these compressive loads rapidly decrease and tension forces develop in the tube. The tension forces gradually diminish when further deformation is applied to the right. This behaviour suggests that Poisson's effects in the steel plates were sufficient to increase the friction between the plate and the mortar when the plate was pushed into the tube when yielding in compression. Upon load reversal, tension developed until the volume of the volume of the plate diminished again due to Poisson's effects. The axial loads that developed in the HSS most likely contributed to the post-yield resistance observed in Fig. 4. In multi-storey buildings, this strain hardening response helps in distributing inelastic deformation over the building height and, therefore, is highly desirable. On the other hand, forces that develop in the braces must be considered in the design of the surrounding members. The post-yield resistance of the braces could probably be reduced by increasing the thickness of the membrane along the plate edges and/or not welding the tubes to the plates at the top end of the braces.

ANALYSIS

One of the 14 vertical braced frames to be added in the building was analysed when subjected to seismic ground motions. The frame studied is shown in Fig. 6. The dimensions of the cross section of the narrow portion of the steel plates of the bracing members are given in the figure. All plates are made of G40.21-300W steel. All steel plates would be embedded in a HSS 219 x 4.8 member (not shown) filled with mortar and end details similar to those specified in the test specimen would be used.

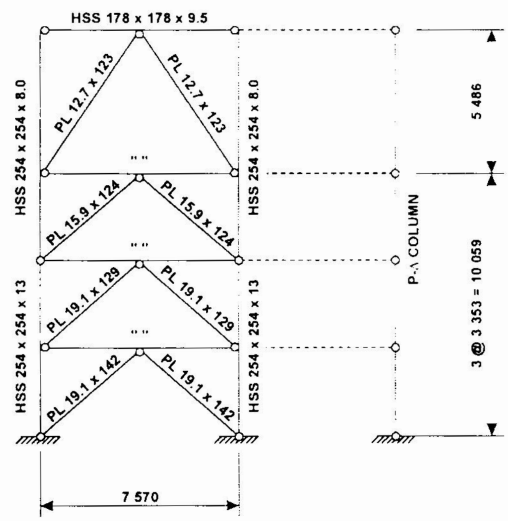

Fig. 6 Frame studied.

The frame was designed using the static loading procedure of the 1995 NBCC (NRCC, 1995). In view of the very stable and ductile inelastic behaviour of the system, a force modification factor, R, of 3.0 was used in the calculation of the seismic lateral loads. An importance factor, I, equal to 1.5 was considered and the inelastic interstorey drifts were kept within 1% of the storey height because the building houses an emergency communication system. The building is founded on a 30 m thick soil deposit made of sand, silt, and till. In the upper 6 m, the soil is loose to very loose while the lower layers are dense to very dense. A foundation factor of 1.5 was then applied in the calculation of the seismic base shear.

Nonlinear dynamic analysis of the frame was performed using the DRAIN-3DX computer program (Prakash et al., 1994). In the model, the frame was attributed its tributary seismic weight and a P-delta column was included which carried the gravity loads (dead load + 50% live load) acting on the building area braced by the frame. The computed period in the first four modes of vibration were 0.87 s, 0.37 s, 0.20 s, and 0.14 s. 3% Rayleigh damping was assumed in the first two modes of vibration. These two modes included 91% of the total mass. The elastic stiffness of the bracing members was adjusted to account for the end reinforcement details. The braces were given a bi-linear hysteretic behaviour with a 10% strain hardening ratio.

The frame was subjected to an ensemble of 9 free field ground motion time histories. that are described in Table 1. Records on rock were first selected and scaled to match the design NBCC spectrum for rock site conditions (Fig. 7). These records and the scaling factors are given in Table 1. The first four time histories are simulations for earthquake scenarios that dominate the seismic hazard in Quebec City (Atkinson and Beresnev, 1998). The other four ground motions are from recent eastern North American earthquakes. The last record is from California but exhibits peak ground acceleration and velocity that are similar to the specified values for Quebec City. The computer program SHAKE (Schnabel et al, 1972) was then used to obtained the free field motion for the 9 records. The N counts from two borings at the site were normalised to $N_{1\text{-}60}$ using the procedure by Seed and Idriss (1970). The corresponding values for K_{2max} were obtained from the relation by Seed et al. (1986) and G_{max} was derived with the equation by Seed and Idriss (1970). The relations proposed by these authors for the variation of damping and shear modulus with strain in sands were used in the calculations. As shown in Fig. 8, the soil deposit was divided into 18 layers and the water table was located at a depth of 3 m. Table 1 gives the peak ground acceleration at the surface and Fig. 7 shows the spectrum of the filtered ground motions. Significant amplification of the ground motion amplitude has been observed and the dominant period of the time histories shifted towards a longer period.

Table 1. Ground motion time histories.

Earthquake event	Station	Comp.	S.F.	PHA Rock (g)	PHA Free field (g)
Ms 5.7 1988 Saguenay	Chicoutimi-Nord (Site 16)	N214°	2.0	0.21	0.28
Ms 5.7 1988 Saguenay	Les Eboulements (Site 20)	0°	2.0	0.25	0.38
Ms 5.7 1988 Saguenay	La Malbaie (Site 8)	N063°	2.0	0.25	0.31
ML 6.2 1985 Nahanni	Battlement Creek (Site 3)	N270°	1.2	0.23	0.24
Artificial Mw 5.5 at 30 km	-	-	1.5	0.27	0.30
Artificial Mw 5.5 at 30 km	-	-	1.5	0.29	0.30
Artificial Mw 7.0 at 150 km	-	-	1.5	0.20	0.24
Artificial Mw 7.0 at 150 km	-	-	1.5	0.19	0.29
ML 6.1 1987 Whittier	Hollywood Storage, L.A.	0°	1.1	0.23	0.24

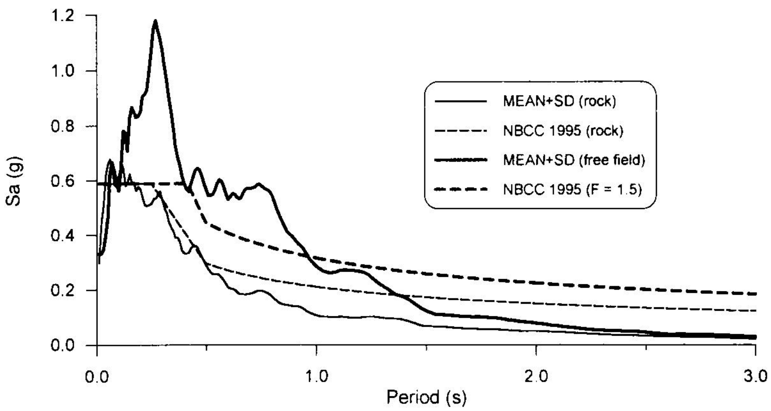

Fig. 7 NBCC and ground motion response spectra.

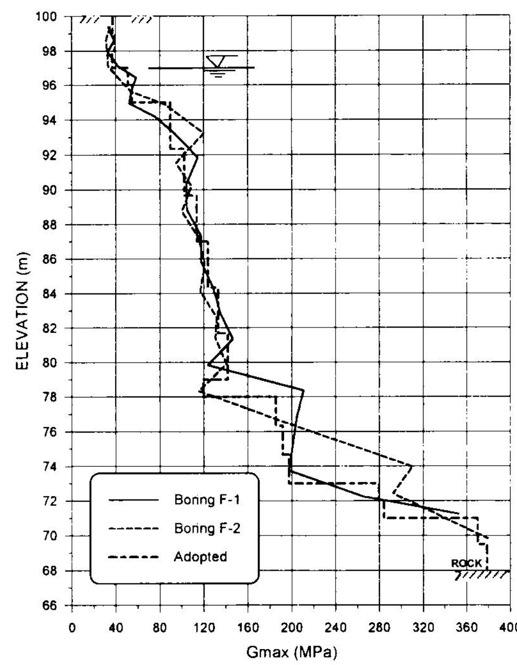

Fig. 8 Shear modulus profile.

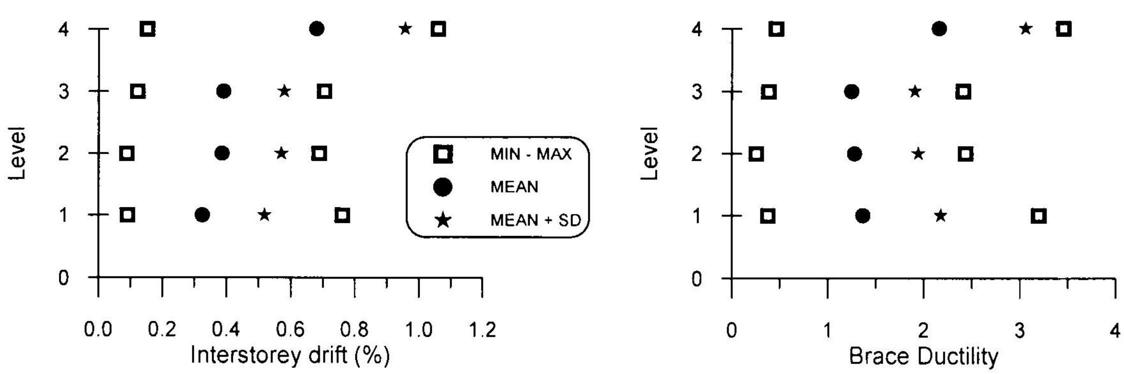

Fig. 9 Results of the analysis: a) peak interstorey drift; b) peak brace ductility demand.

The computed peak interstorey drift and brace ductility demand are given in Fig. 9. Significant scatter was obtained within the record ensemble: the Saguenay record at Chicoutimi-Nord generally produced the minimum demand on the frame while maximum response was observed under the Mw 7.0 simulations and the Whittier record. Nevertheless, the seismic performance of the frame appears to be adequate as the 1% storey height limit for the interstorey drift has been only slightly exceeded under one ground motion at the top floor of the building and, for all ground motions, the maximum ductility demand in the braces remained within the capacity of the system as determined by the test. The maximum force computed in the braces varied between 1.14 and 1.25 times the yield resistance of the steel plates, depending upon the floor level.

CONCLUSION

The seismic performance of a bracing system with the diagonal members made of a steel plate embedded into a steel tube filled with mortar was evaluated through a quasi-static loading test and nonlinear dynamic analysis. The test showed that the bracing system exhibits a very stable hysteretic behaviour with high energy dissipation capacity and beneficial strain hardening response. The strain hardening behaviour is most likely the result of Poisson's effects on the steel plate undergoing large inelastic deformation and could probably be reduced by increasing the thickness of the membrane along the plate edges and by separating the tube from the steel plate at both ends of the bracing members. Further research is needed to investigate this issue. The analysis indicated that the building studied could perform adequately under strong ground motions when strengthened with the proposed bracing system.

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