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Seismic Rehabilitation of the New Hamilton Courthouse

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ABSTRACT

Friction-dampers in steel bracings have been used for the seismic upgrade of an existing seven-storey building to house the new eight-storey Hamilton Courthouse. The existing structure, built in 1934, was not adequate to resist earthquake forces specified in the current building code. The building is of heritage importance and has to be preserved. Conventional methods of stiffening with concrete shearwalls or steel bracings require costly and time consuming work of strengthening existing columns and foundations. Introduction of supplemental damping in conjunction with appropriate stiffness was considered to be the most effective, economical, and hi-tech solution for the seismic rehabilitation of this building. The results of three-dimensional nonlinear time-history dynamic analysis have shown significant reduction in forces on the structure, amplitude of vibrations and floor accelerations. Besides savings in the initial cost of retrofitting, the seismic performance of the friction-damped system will be superior to those with traditional strengthening methods.

INTRODUCTION

The design criteria stipulated in most building codes, including the National Building Code of Canada 1990 (NBC), are based on the philosophy of designing structures to resist moderate earthquakes without significant damage and to resist major earthquakes without structural collapse. The primary emphasis is on life safety with an expectation of substantial structural damage. In general, reliance for survival is placed on the ductility of the structure to dissipate energy while undergoing large inelastic deformations causing bending, twisting and cracking. This assumes permanent damage, repair costs of which could be economically as significant as the collapse of the structure. Recent examples of these are the earthquakes of Northridge-1994 in California and Kobe-1995 in Japan. Although the death tolls were relatively low, damage to the buildings and other associated costs were estimated to be more than US$ 20 billion and US$80 billion, respectively.

While the minimum design provisions of the building codes were adequate in the past for most buildings, safer approaches are desirable for important buildings. In modern buildings, avoidance of structural collapse alone is not enough. The costs of finishes, contents, sensitive instrumentation and electronically stored records can be much higher than the cost of the structure itself and these must be protected. The problems created by the dependence on ductility of a structure can be reduced if a major portion of the seismic energy is dissipated mechanically, independent from the primary structure. With the emergence of friction-dampers, it has become economically feasible to significantly increase the earthquake resistance and damage control potential of a structure.

The existing structure of the new Hamilton Courthouse was built about 60 years ago. A view of the building is shown in Figure 1. It has a carved stone facade of heritage value worth preserving. Preliminary analysis by the structural engineers indicated that the existing structure was not adequate to resist lateral earthquake forces specified in the NBC. The conventional methods of stiffening with

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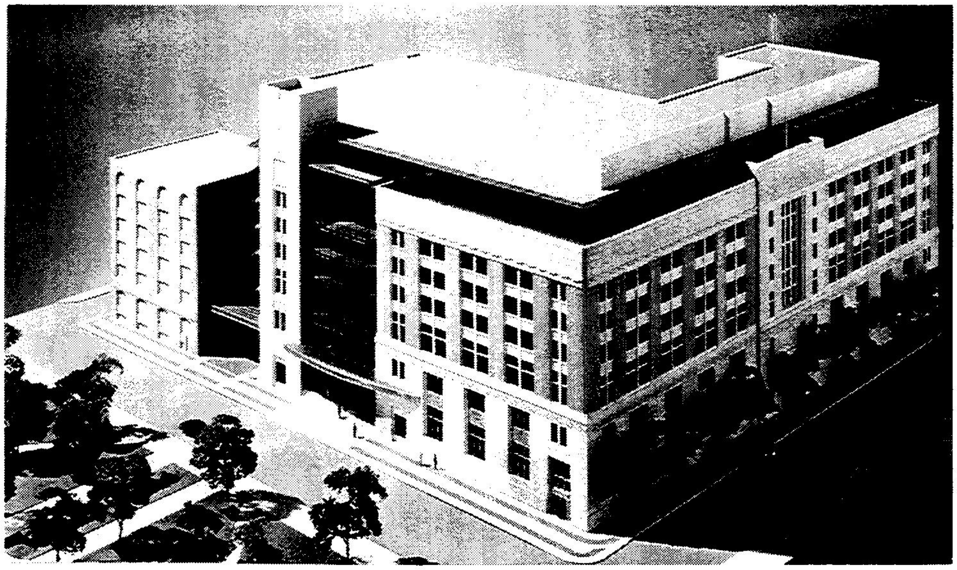

Figure 1. VIEW OF THE NEW HAMILTON COURTHOUSE

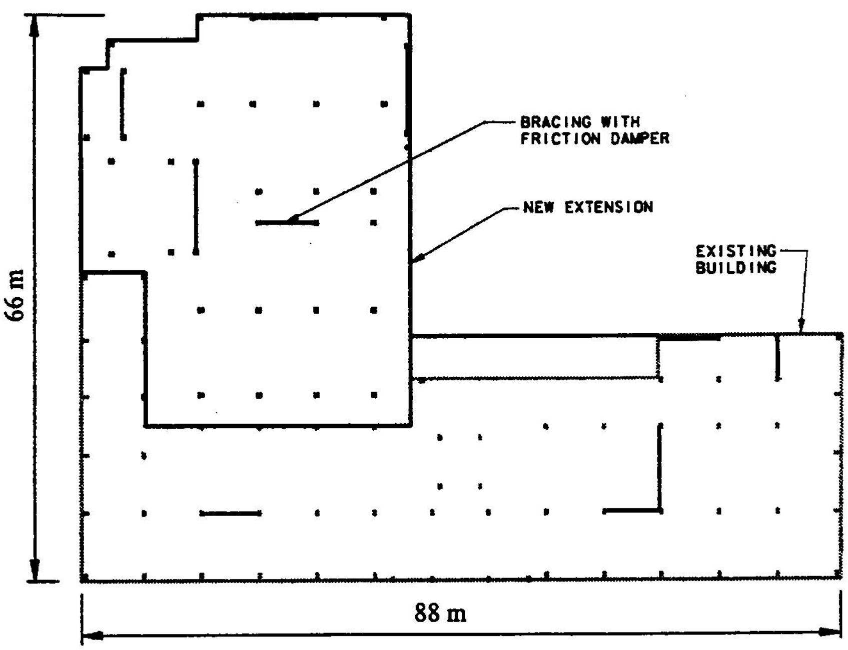

Figure 2. GROUND FLOOR PLAN

concrete shearwalls or steel bracings would require costly and time consuming work of strengthening existing columns and foundations.

The innovative technique of introducing supplemental damping in conjunction with appropriate stiffness was considered to be the most economical, effective and practical hi-tech solution for the seismic rehabilitation of this building. Analytical studies have been made to compare the seismic response with traditional structural systems. This paper will discuss the results of analysis and provide construction details of the chosen structural system. The construction started in early spring of 1995.

HAMILTON COURTHOUSE

The new eight-storey Courthouse is housed in the existing building with an additional storey at the top and a new eight-storey lateral extension. The existing seven-storey Dominion Public Building, constructed in 1934, was built of steel columns and beams with floor slabs of reinforced concrete. The new construction utilizes the same structural system and is well tied to the existing structure. The foundations of the existing building consist of spread footings which are located about 5 m below the basement level. The exterior of the building has carved stone facade of heritage value. The ground floor plan of the structure is shown in Figure 2.

The existing structure derived its lateral rigidity from unreinforced masonry infill of the steel frames. During that period, structures were basically designed to carry gravity loads and wind forces only. There was no criteria for earthquake resistant design. Over a period of sixty years, the building codes have changed drastically, especially with respect to provisions for earthquake resistant design. Masonry infilled structures have performed very well to resist wind forces, but have behaved rather poorly in the event of earthquakes. Unreinforced masonry lacks ductility and is not capable of undergoing inelastic cyclic deformations typically experienced during an earthquake. The preliminary analysis by the structural engineers indicated that the existing structure was not adequate to resist lateral earthquake forces specified in NBC without sustaining extensive structural damage and the loss of the heritage components.

Conventional methods of stiffening with concrete shearwalls or steel bracings were studied. Concrete shearwalls are known to be effective in controlling lateral deflections due to wind and moderate earthquakes. During a major earthquake, these structures tend to attract higher inertial forces on the supporting elements. Therefore, any advantage gained with the added stiffness may be negated by the increased amount of energy input. Moreover, the construction work for shearwalls and its foundations in an existing building is very difficult and expensive. In a conventional braced frame, the energy dissipation capacity of a brace is limited. A brace in tension stretches during severe shock and buckles in compression during reversal of load. On the next application of load in the same direction, this elongated brace is not effective even in tension until it is taut again and is stretched further. As a result, the energy dissipation degrades very quickly and the structure may collapse. Both conventional methods required expensive work of strengthening existing columns and foundations, apart from severely restricting the flexibility of space planning.

The use of friction-dampers in steel bracings was considered to be the most economical, effective, and practical solution for the seismic rehabilitation of this building. The bracings may be in the form of tension-only X-bracings, K-bracings and single diagonal tension/compression bracings. As soon as the structure undergoes small deformations, the friction-dampers go into action and start dissipating energy. Cracking of masonry within repairable limits was considered acceptable. Since a major portion of the seismic energy is dissipated by the dampers, the forces on the structure are considerably reduced. Unlike concrete shearwalls, the friction-damped bracings need not be aligned vertically from bottom to top, thus allowing greater freedom in space planning or construction phasing. The possibility to stagger the braces at different storeys avoids overloading of columns or footings. Hence, the strengthening of columns and foundations is not required.

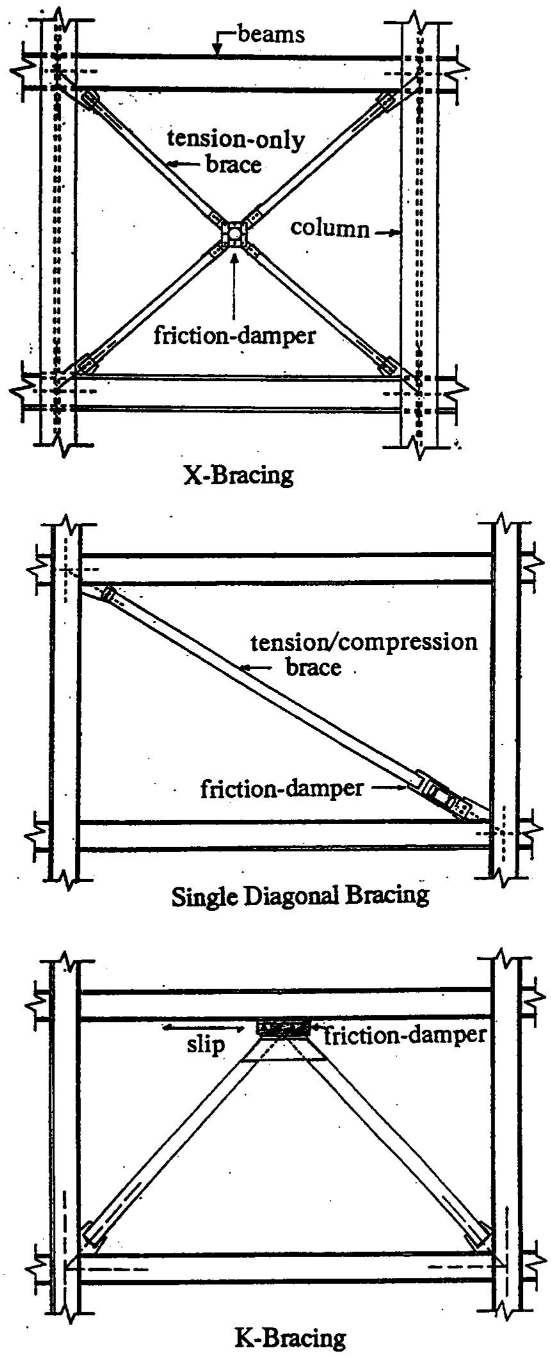

Figure 3. FRICTION-DAMPED BRACED BAYS

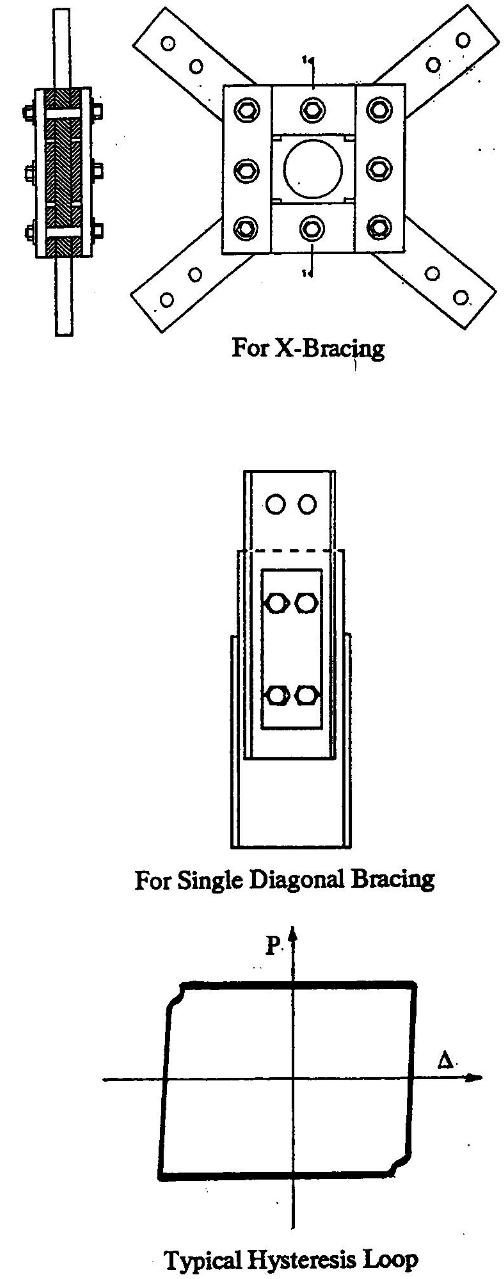

Figure 4. TYPICAL FRICTION-DAMPERS

The location of steel braces with friction-dampers in the lower storey is shown in Figure 2. Architectural planning governed the location and type of bracings. Generally, the bracings at the upper level follow the same arrangement unless the space planning or avoiding the overloading of columns and foundations dictated otherwise. Typical details of braced bays and friction-dampers are shown in Figures 3 and 4, respectively. Patented friction-dampers were designed and supplied by Pall Dynamics Limited.

PALL FRICTION-DAMPERS

Pall friction-dampers are simple and fool-proof in construction (Pall 1982). Basically, these consist of series of steel plates with slotted holes. The plates are specially treated to develop most reliable friction. They are clamped together with high strength bolts and allowed to slip at a predetermined load. Their performance is reliable, repeatable and they possess large rectangular hysteresis loops with negligible fade over several cycles of reversals that can be encountered in successive earthquakes. A much greater quantity of energy can be dissipated in friction than any other method involving the yielding of steel plates or viscoelastic materials. Therefore, fewer damping devices are required to provide the required amount of energy dissipation. Their performance is not affected by temperature, velocity, stiffness degradation due to aging and they need no replacement after the earthquake. These friction-dampers do not require maintenance and are always ready to do their job regardless of how many times they have performed. Friction-dampers are designed not to slip during wind storms or moderate earthquakes. During a major earthquake they slip before yielding of structural members. After the earthquake, the strain energy of the structure brings the dampers back to their near original alignment.

These friction-dampers have successfully gone through rigorous proof-testing on shake tables in Canada and the United States (Filiatrault 1986, Aiken 1988). Patented Pall friction-dampers are available for tension-only X-bracings, single diagonal in tension/compression and chevron or K-bracing systems (Pall 1982). The friction-dampers meet a high standard of quality control. Every damper is load tested to ensure proper slip load before they are shipped to site.

Pall friction-dampers have found several applications for both steel and concrete buildings in new construction and retrofit of existing buildings (Godin 1995, Hale 1995, Pall 1987, Pall 1991, Pall 1993, Pasquin 1993, Savard 1995, Vezina 1992) and several others are under construction.

NONLINEAR TIME-HISTORY DYNAMIC ANALYSIS

Three-dimensional nonlinear time-history dynamic analyses were carried out using the computer program DRAIN-TABS (Guendelman-Israel and Powell 1977), developed at the University of California, Berkeley. This program consists of series of subroutines that carry out a step by step integration of the dynamic equilibrium equations using a constant acceleration within any time step. It is known that different earthquake records, even though of the same intensity, give widely varying structural responses and results obtained using a single record may not be conclusive. As future earthquakes may be erratic in nature, an artificially generated time-history, which includes many earthquake records and covers a wide range of frequency content, has been used (Newmark 1973). For Hamilton, the peak ground accelerations of the artificial earthquake record were scaled to 5.6% g. The duration of the earthquake record was 15 seconds and the integration time step was 0.005 second. Analyses were carried out for earthquake motions applied along the x-axis, the y-axis and in 45 degree direction.

Viscous damping of 5% of critical was assumed in the initial elastic stage to account for the presence of nonstructural elements. Hysteretic damping due to inelastic action of structural elements and slipping of friction-dampers is automatically taken into account by the computer program. Interaction between axial forces and moments for columns and P-Δ effect were taken into account by including geometric stiffness based on axial force under static loads. To account for any accidental eccentricity due to uncertainty in the distribution of mass or possible variation in relative stiffness, etc.,

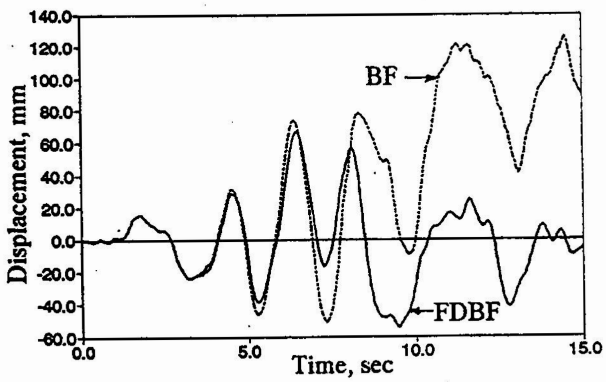

Figure 5. TIME-HISTORIES OF DEFLECTION AT TOP

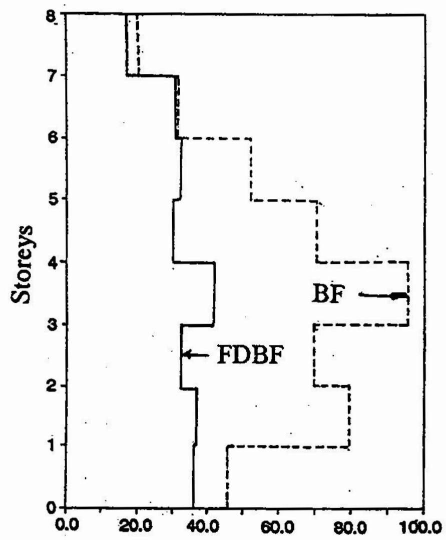

Figure 6. ENVELOPE OF COLUMN SHEAR, kN

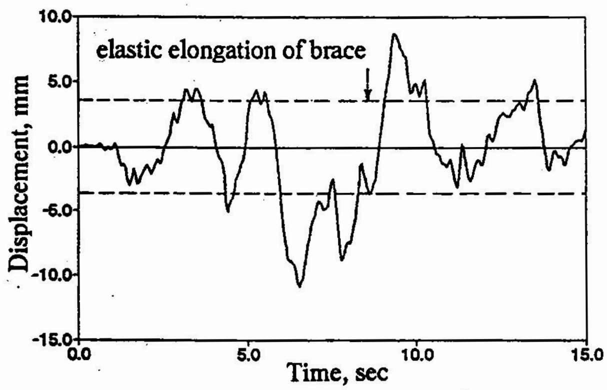

Figure 8. TIME-HISTORIES OF SLIPPAGE IN FRICTION-DAMPER

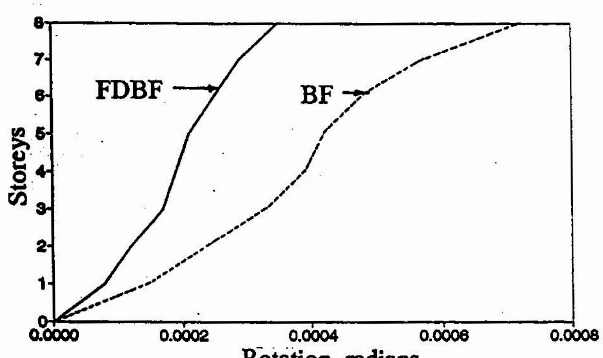

Figure 9. TORSIONAL ROTATION OF BUILDING

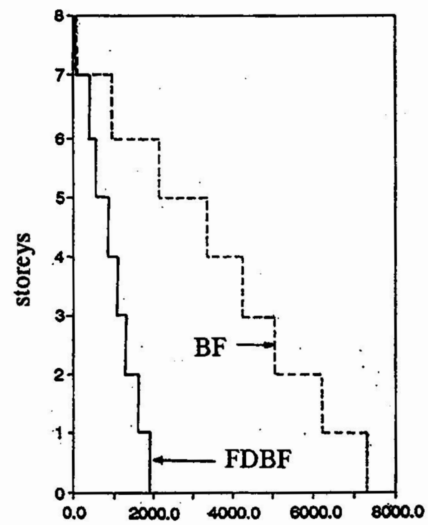

Figure 7. ENVELOPE OF COLUMN AXIAL FORCE, kN

the center of mass was shifted by 10% of the building dimension in both axes. Actual moment capacities of beam / column connections and masonry infill contribution were included in the analyses.

A series of analyses were made to determine the optimum slip load of the friction dampers. The optimization was governed by: minimum response in the new extension; safe load carrying capacity of the existing beam/column connections, columns and foundations. In the existing building, the slip load of friction-dampers was 600 kN in the bottom storey and 500 kN in upper storeys. For new construction, the slip load was 700 kN for the lower five storeys and 600 kN for upper three storeys. A total of 74 friction-dampers were required to extract sufficient seismic energy to safeguard the structure from damage.

The effectiveness of friction-dampers in improving the seismic response is seen when the results of the traditional Braced Frame (BF) system are compared with the ones of the Friction-Damped Braced Frame (FDBF). The braces in BF have an optimum area of 1.5 times the area of FDBF braces. Both smaller and larger areas of brace result in higher responses. Although detailed cost estimates were not prepared for the BF scheme, the studies clearly indicated that significant work was needed for the strengthening of columns and foundations, the cost of which far exceeded the cost of the FDBF scheme.

RESULTS OF ANALYSIS

1 The existing structure was not able to resist even 50% of the code specified forces.

2. The time-histories of deflections at the top of the building are shown in Figure 5. The peak amplitudes are 62 mm and 126 mm for FDBF and BF, respectively. The permanent offset after the earthquake was 4 mm for FDBF and 85 mm for BF.

3. The maximum storey drift was in the lower most storey. These were H/550 and H/150 for FDBF and BF, respectively. The storey drift for FDBF is very small and it is believed that the cracks in the masonry may not be visible or will at least be repairable.

4. The maximum floor accelerations experienced by the FDBF are only 40% of those for BF. Reduction in floor accelerations can significantly reduce the damage to the nonstructural components, finishes and the contents of the building.

5. The maximum envelopes for column shears are shown in Figure 6. The values for FDBF are about 42% of those for BF.

6. The maximum envelopes for column axial forces are shown in Figure 7. The maximum value for FDBF are about 25% of that for BF. In case of BF, all columns and foundations of braced bays would need strengthening. No strenthening was necessary for FDBF.

7. The time-histories of slippage in a typical friction-damper in the lower most storey is shown in Figure 8. The maximum amplitude of slippage is 8 mm. After the earthquake, the friction-dampers returned to their near original alignment.

8. The use of friction-damped braces have significantly improved the torsional response of the structure. The torsional rotation of two types of frames is shown in Figure 9. The rotation for FDBF is only 50% of that for BF.

CONCLUSIONS

The use of friction-dampers has shown to provide a practical, economical and effective new approach to rehabilitate existing structures to resist major earthquakes. Besides savings in the initial cost of retrofit and construction time, the savings in life cycle cost could be significant as damage to the building and its contents is minimized.

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