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Seismic Rehabilitation of the Hydro-Quebec Head Office

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ABSTRACT

The complete seismic evaluation and analysis of a Montreal office building belonging to Hydro-Quebec is documented. The 30-year-old structure displayed excessive flexibility in the East-West direction combined with an appreciable mass to stiffness ratio. A seismic retrofit was implemented and constructed in accordance with newly revised design standards while taking into account some relaxation of stringent code requirements for existing structures. Special attention was demanded with regard to stability and with the overall provision of ductility.

INTRODUCTION

The head office building of Hydro-Quebec, located at 75, René-Lévesque West in Montreal, was designed in 1959 to resist 1,44 kPa wind loads. It was decided in 1990, along with a major renovation undertaking, to verify the structure's capability to withstand seismic forces in accordance with design standard CAN/CSA-S16.1-M89 (1989) and the National Building Code of Canada (N.B.C.C.) 1990. The complete seismic study was deemed necessary after a comparison of the design wind base shear with the earthquake shear calculated using the above-mentioned standards.

BUILDING DESCRIPTION

The edifice in question is a 26-story steel framed office tower (Figs. 1 & 2). Lateral loads are resisted entirely by ductile moment resisting frames, in both directions. Also taken into account are the five (5) story parking sub-structure and the adjacent "Poste Dorchester" electrical sub-station. For the purpose of base shear calculation, only the weights of the superstructure components are considered; it is these elements that effectively oscillate in event of seismic activity.

STRUCTURAL MODEL

Analysis of the structural response was carried out in three dimensions (3D) using the SAP90 finite element software package (Wilson and Habibullah 1989). Beams and columns were modelled with "Frame" elements and were effectively linked at each level to correctly

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simulate diaphragm behaviour. Rigid end offsets were included at nodal regions where flexural deformations were negligible. Foundation walls were included using "Frame" elements braced with very stiff diagonal members to account for the large in-plane stiffness usually exhibited by such components. Geometry, member sizes and gravity loads were extracted from the existing structural and architectural drawings.

Secondary effects, resulting from the gravity load acting on the displaced structure, were treated with precaution for the following reasons: (i) the 26 story moment resisting frames constitute a very flexible system (ii) the steel design standard S16.1-M89 (1989) requires that amplified deflections be used for such calculations; i.e. the deflections obtained from a linear elastic frame analysis must be multiplied by the force modification factor "R" to account for the inelastic action. To accurately evaluate this phenomenon, referred to as the P-delta effect, two methods were verified and compared using SAP90; the iterative gravity load method and the negative inertia fictitious column method (Gaiotti and Stafford Smith 1989). These were examined in a 2D frame and results obtained were identical in both cases. Subsequently, the 3D version of the negative inertia fictitious column method was chosen for inclusion in the 3D model.

STATIC ANALYSES

Original Building

Seismic loads corresponding to $R = 3.0$ were applied to the model. This level of ductility was chosen because the moment frame joints displayed details of nominal ductility. It should be noted that the top concentrated force F_t was distributed equally to the three (3) penthouse levels. This assumption was later verified through dynamic analysis. Displacement verifications subsequent to these runs on the original structure indicated a high degree of flexibility in the E-W direction. Interstory drifts, at the inelastic level were 175 mm on average, as compared with the story height / 50 limit of 76 mm. Analysis in the N-S direction yielded average drifts of 63 mm, thus indicating an acceptable result. The large displacements in the E-W analysis were expected because this orientation corresponds to the moment resisting frames' weak direction.

In order to verify the model's precision, wind loads as per the original design were applied and results obtained were in agreement with the frame forces indicated on the original drawings.

It is important to note that the lack of lateral stiffness was not only reflected by exceedance of prescribed drift limits. P-delta analyses of the iterative type displayed lack of convergence indicating potential instability.

Reinforced building

Several retrofit schemes were studied in order to add lateral rigidity and effectively increase the stiffness to weight ratio. Three options considered additional X-bracing at various bays, and one scheme attempted to stiffen the existing beams of exterior moment frames.

The tentative retrofit choice, similar to that in Fig.5, consisted of adding two X-braced bays at the building's centreline (column line E, Fig. 1) linked with HSS trusses at each floor. The link beams (or trusses) enabled the two ductile braced frames to function as a unit, employing a lever arm of 21.72 m. This system proved to be much more rigid than the cumulative effect of two independent braced bays, each having 7.239 m length (Fig. 3).

Displacement results showed favourable behaviour, with an average interstory drift of 70 mm at the inelastic level. In order to refine the analysis and to obtain accurate member forces for execution of code checks, it was decided to perform a dynamic analysis.

DYNAMIC ANALYSIS

An eigenvalue analysis was performed, based on 26 masses times three degrees of freedom (3 D.O.F.) per mass yielding 78 dynamic D.O.F. Resulting fundamental periods indicated an E-W translational mode and read as follows:

original bldg.	$T_{dyn.}$	=	12.4 sec.	
	$T_{code.}$	=	2.6	(0.1 N)
reinforced bldg.	$T_{dyn.}$	=	6.1	
	$T_{code.}$	=	2.1	($0.09\, h_n/\sqrt{21,72}$)

One can note that the differences between the calculated periods and the code periods are probably due to the contribution of non-structural elements in building performance, observed during the development of code formulas. It is also clear, from the above figures, that the building in question demonstrates a notable amount of lateral flexibility, as can be expected in moment resisting framed structures greater than ten stories high (Picard and Beaulieu 1991).

After the eigenvalue analysis, SAP90 combines the results of each mode using the complete quadratic combination (C.Q.C.) technique in the mode-superposition method. In order to simplify the comprehension of structural effects, calculation of story shears and subsequent static lateral loads was performed. Finally the obtained lateral loads were calibrated to produce an equal base shear to that stipulated by N.B.C.C.(1990). The resulting model consisted of a 3D frame subjected to static loads that were coherent with the lateral load distribution calculated by dynamic analysis. At this point, comparison of the overturning moments about ground level between the original code distribution (with $F_t/3$) and the new static distribution (derived from dynamic analyses) showed a 7% difference, thus validating the adequacy of the $F_t/3$ assumption.

MEMBER VERIFICATION

Next, two static analyses were executed; one with seismic load applied at the centre of mass, and the other with an eccentricity of 10% of the normal plan dimension (2.837 m). Insofar as the members away from the reinforced zone are concerned, beams were adequate, and columns on the average were stressed to 70% of their maximum allowable stress. Loading combinations considered are listed below:

1)	1,25 D + 1,0 Q	D : dead load
2)	1,25 D + 0,7 (1,5 L + 1,0 Q)	L : live load
3)	1,0 D - 1,0 Q	Q : earthquake

Beams and columns in the reinforced bays displayed stress ratios ranging from 0,6 to 1,03 upon combination of flexure and axial load. These results are satisfactory considering the beneficial effects of non-structural elements and the overall redundancy of the surrounding moment frames.

Modifications to the existing structure, excluding the proposed retrofit, were limited to the provision of anchorage devices at columns E8 and E14. Uplift resistance was required due to the small proportion of dead load attributed to these columns. In combination 3, the use of a 1.0 dead load factor was justified by the conservative tabulation of counteractive dead load in uplift computations. The existing foundations, however, were adequate with respect to downward forces, exhibiting a safety factor of 1.7 against bearing capacity failure (acceptable, given the temporary nature of seismic loading).

FINAL REVISION OF RETROFIT

Upon evaluation of construction costs related to implementation of the aforementioned retrofit, it was decided by the building owners to verify the impact of a reduction of seismic load on the amount of required reinforcement, due to budget restrictions. Because limited guidance was available in Canadian design standards regarding the earthquake resistance of existing buildings, an approach suggested by Allen (1991) was employed. The procedure, based on a life safety criterion, yielded a load factor of 0.75. Consequently, the static lateral load was reduced to 75% of its original value, while conserving the vertical distribution based on dynamic analyses.

The selected retrofit, illustrated in Fig 5, combines the two (2) vertically continuous braced bays via link beams at every second floor. This system, although downsized from previous solutions, still manages to produce composite action of the braced frames, thereby effectively providing the required increase in lateral stiffness. Resulting interstory drifts, considering plastic deformation, were 60 mm on average, clearly inferior to the 76 mm limit (Fig.4).

The load factor revision also produced a substantial reduction (40%) in the amount of anchorage required at columns E8 and E14.

DUCTILITY

Recent Canadian standards now include specific detailing requirements in order to justify the use of a force modification factor in calculating seismic base shear. For energy to be dissipated, cyclic plastic deformations must occur without brittle failure of members and connections. For the building in question, comparable force reduction was considered throughout, due to the use of ductile braced frames surrounded by existing moment frames possesing, at the very least, nominal ductility. NBCC 1990 assigns an $R=3.0$ value to both these categories.

The retrofit at column line E achieves provision of ductility through buckling and yielding of diagonal brace members while the HSS link beams are designed to behave elastically. The surrounding moment frames can sustain nominal amounts of cyclic deformation, mostly by plastic hinge mechanisms in the beams, after verification of connection details and resistance hierarchy at typical nodes.

The inherent reduncancy of the moment resisting frames combined with the stabilizing effect of the implemented seismic retrofit provided feasible and effective resistance to lateral loads (Fig. 4).

CONCLUSION

The building studied herein has shown satisfactory performance for over 30 years of service. It has been demonstrated, however, that in event of a strong earthquake in the East-West direction (with a 476 year recurrance interval), the present structural system is incapable of providing the needed strength, stiffness and stability. The original calculation of lateral load, based on wind, was dependant on the building's vertical face dimensions. Consequently the frames were designed for predomintantly North-South effects. Because of its high mass to stiffness ratio in the other direction, severe ground motion could induce large deflections and potential damage to structure and occupants.

For the above reasons, the structural reinforcements illustrated in Fig. 5 have been implemented in a seismic retrofit operation at a cost of $700 000,00. These modifications have been realized in accordance with the relevant Canadian design standards and procedures, in conformance with accepted engineering practice in order to insure the safety of the edifice and its occupants.

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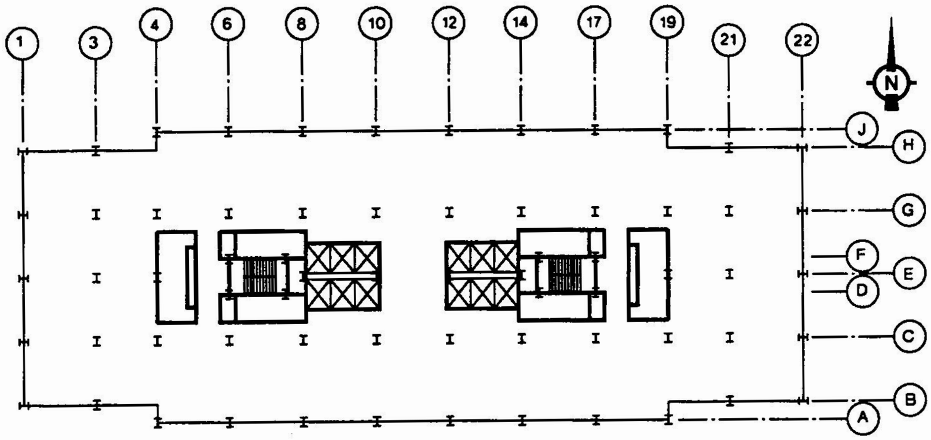


Figure 1 - Typical floor plan

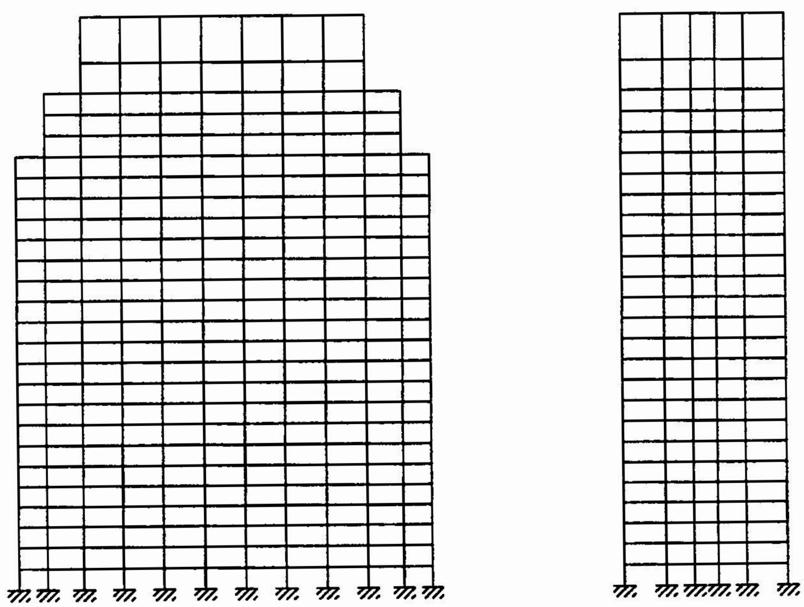
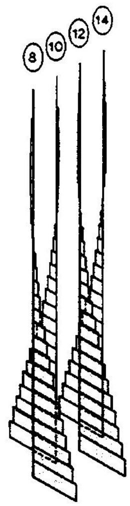
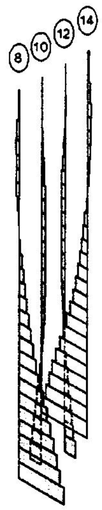


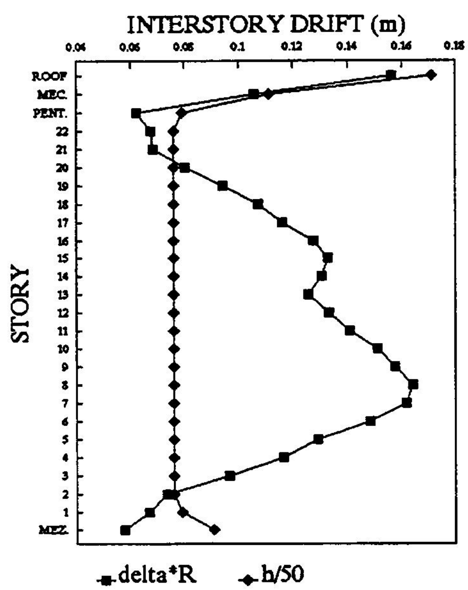
Figure 2 - Elevations (sub-structure not shown)



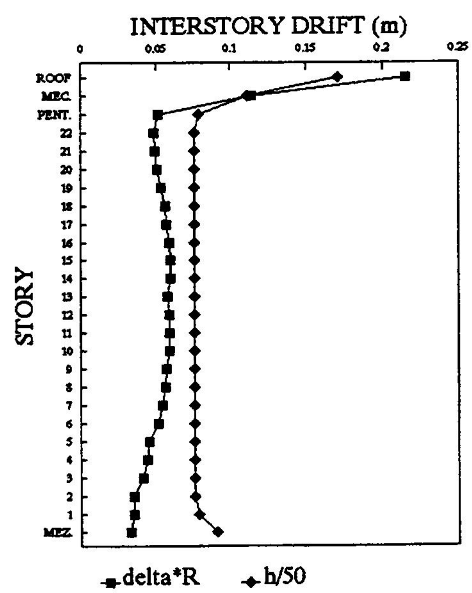


BRACING ALONE BRACING WITH LINK BEAMS

Figure 3 - Axial load diagram for columns at column line 'E'



ORIGINAL BUILDING



REINFORCED BUILDING

Figure 4 – Deflection checks for 75% of NBCC 1990 base shear (E–W)

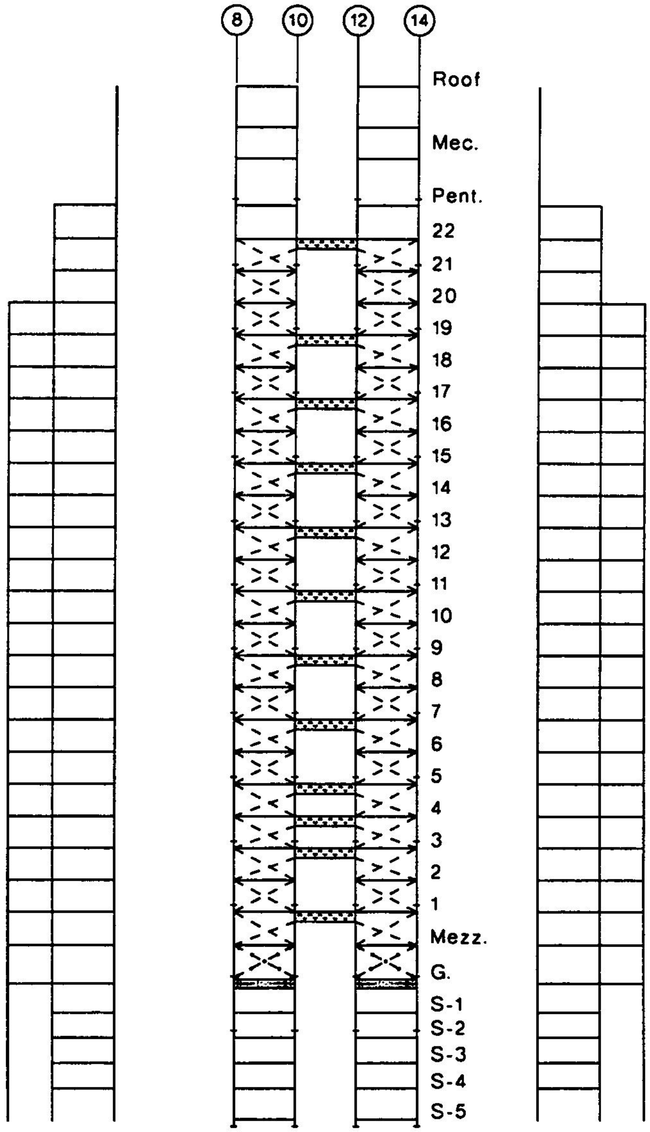


Figure 5 - Elevation of retrofit at column line 'E'