7th Canadian Conference on Earthquake Engineering / Montreal / 1995
7ième Conférence canadienne sur le génie paraséismique / Montréal / 1995

Upgrading the Lateral Load Resisting System of the 330 University Avenue Building, Toronto, Using Response Spectra Analysis

T. Kokai[1]

ABSTRACT

This report discusses most of the issues occurring when retrofitting an existing building, highlighting the issues of the period of the structure and the "top force" of buildings with setbacks which are not clearly covered in the National Building Code of Canada, 1990. An engineering solution is presented for resisting the earthquake forces, analyzing the existing and the proposed structural systems for different loading conditions.

INTRODUCTION

The headquarters of Canada Life Assurance Company, 330 University Avenue, Toronto, is undergoing a general architectural,structural, mechanical and electrical retrofit with the purpose of bringing this downtown landmark building to the current code levels in all aspects of building performance.

Description of the Building Structure

The 18 storey building was constructed in 1929 (for the typical floor plan and elevation see sketches S1, S2) and designed for 20 psf (1.0 kPa) uniform wind load. In the major wind direction (east-west) the lateral load resisting system is comprised of 26 moment resisting frames (MRF) with riveted beam-column joints. These joints were completed after the dead load was in place, thus only live and lateral load moments are acting on them. In the minor, north-south, wind direction the only three MRF-s are located on the west building face (see S1). The cladding system consists of limestone (10in/0.25m thick) bonded to the masonry backup (8in/0.2m thick) with no deflection gap at typical floors. The building has two basements utilizing reinforced concrete beam - one-way slab floor construction and square caisson foundations bearing on shale bedrock.

Evaluation of the Existing Structure for the Design Wind Load

Using space frame analysis (ETABS) we concluded that the major wind is resisted by the MRF-s and the minor wind by the masonry-limestone cladding system in combination with the three MRF-s on the west face. Calculations show that the existing structure was adequately designed to resist the assumed wind loads.

[1] Senior Associate, Halsall Associates Limited, 188 Eglinton Avenue East, Suite 600, Toronto, Ontario M4P 2X7

Evaluation of the Existing Structure for Current Code Loads

It was the task of the design team to achieve the most cost effective structural retrofit. To achieve this goal all the documentation was collected, reviewed and all the feasible tests were performed focusing on structural elements and material properties relevant for resisting the lateral loads.

Foundations

Two caissons were investigated by core drilling along the long axis to bedrock to asses the capacity of caissons and the foundation conditions. Additional loads are imposed on existing columns thus additional capacity was required from the bearing strata and the unreinforced caissons. The concrete in caissons was found to be in the range of 35 to 40 MPa. The geotechnical consultant advised that the additional loads calculated could safely be supported by the bedrock.

Wind Loads

Wind tunnel tests were performed to assess the realistic wind loads. The results, as expected, gave wind force levels below those specified in the National Building Code of Canada, 1990 (N.B.C.C.). Wind forces are found to be less severe than earthquake forces in both north-south and east-west directions.

Structural Steel

Tests were performed to determine the composition, weldability and brittleness of the existing structural steel. The structural steel is found to be weldable without any special welding procedure with the limit stress of f_s=206 MPa.

Structural Analysis for Earthquake Forces

The ductility factor "R" of the Code's equivalent static load procedure varies according to the type of material and structural system used to resist the earthquake loads. On the assumption that the masonry may be able to resist the north-south static code loads (R=1), a masonry strut structural system was analyzed on the east face of the structure, and the existing three MRF-s were used on the west face (Paulay and Priestly 1992, Chidiac 1993). As expected, it was found that the masonry and the edge beams would be overstressed by a factor of 2.0 and at the same time the three west frames would also be overstressed by a factor of 3.0. This result lead us to the conclusion that some kind of a bracing system would have to be introduced to stabilize the structure. The investigation of the effects of masonry in resisting the earthquake loads for an 18 storey structure was not followed beyond the range of the linear three-dimensional frame analysis, since the required non-linear step-by-step dynamic analysis was too complex for the given time limit. Also it was felt that it would be unrealistic to assume that the masonry was built tightly to the steel columns and beams and the probability of out of plain failure of the masonry under earthquake conditions could not be ignored (Chidiac 1993).
Based on the above facts the masonry-limestone cladding was accepted as a secondary structural element with no calculated contribution to the earthquake resistance in the north-south direction.

The earthquake forces were resisted by the MRF-s in east-west direction with overstressed riveted joints mostly above the 8th floor level.

Accepted Method of the Structural Retrofit

The structure had to be analyzed for two conditions: 1) the present building configuration with east bracings, 2) the condition with the present courtyards enclosed to form atria, with the future west bracings. The upgradings in the current phase include the complete mechanical, electrical and part of architectural and structural work. In this phase, structurally, the east bracing system is built fulfilling all the requirements of phase II (future phase) when the atria will be completed. To provide flexibility for future architectural atria design the east (phase I) bracings have to be compatible with three future west bracing locations. Therefore no such lateral load resisting system can be accepted for phase I (east) bracing system that requires the finalization of the future phase bracings' location at this stage of the design. Namely, depending on the future bracing locations, (see S1 and S2) the east bracing system's forces will vary, thus an "optimum design slip load" for a friction damped braced frame could not be defined (Pall and Pall 1991, Cherry and Filiatrault 1991). Phasing the construction in the described way and providing the required architectural design flexibility for the future prevented the use of friction damped devices which have advantages in dynamic behaviour over the braced frames with nominal ductility.

As a conclusion of the above analysis braced frames with nominal ductility were accepted for both construction phases to stabilize the structure in north-south direction. The stability of the structure in east-west direction could be achieved by reinforcing the overstressed riveted joints of the MRF-s.

DESIGN RESULTS OF THE PROPOSED RETROFIT

The static and dynamic analysis of the structure was carried out using ETABS (version 5.14).

Establishing the static base shear based on NBCC represented a theoretical problem since the NBCC does not define the periods of buildings with setbacks. Using the recently published article by Wong and Tso, 1994, we could establish the period of the building that defined the static base shear and therefore the results of the response spectra analysis (RSA) could be calibrated. Table 1 summarizes the effects of different periods for the static base shear. Case 2 is the most conservative approach since the period is reduced by neglecting the height of the tower while Case 1 still underestimates the earthquake forces by 16.6% using the proper code values and the full building height. A scale factor of 1.35 was used to establish the total probable design base shear (based on Wong and Tso 1994).

An additional issue is the definition of the "top force" F_t defined in the NBCC- 4.1.9. Applying this force on buildings with setbacks means unnecessary overdesign of the upper portion of the structure. To overcome this problem the use of RSA (or other dynamic method) is unavoidable to achieve a better force redistribution along the building height.

As previously stated, the north-south direction is stabilized with a current phase, east, bracing system (three bays, see S1 and S2) and a future, west, bracing system with three possible locations (see S1, option 1, 2 and 3). To minimize the eccentricity of the earthquake forces acting it was necessary to match the centre of mass of the building with the shear centre in all three cases (options 1, 2 and 3) respectively - this was achieved by proportioning the east and west bracings' stiffnesses. Based on the layout of the bracings and knowing that the shear centre coincides with the centre of mass the "amplification" of masses was reduced to $\pm$ 0.1 D, where D is the building dimension.

The torsional effect was not significant due to the relatively small value of D and the large number of MRF-s in the east-west direction.

CASE	METHOD	HEIGHT (m)	D_s (m)	PERIOD (s)	S	BASE SHEAR (kN)	SCALE FACTOR
1	NBC-Static	78.6	7.2	3.16	0.85	4800	1.16
2	NBC-Static	48.9	7.2	1.97	1.07	6300	1.52
3	Wong and Tso 1994	78.6	7.2	2.4	0.97	5600	1.35
4	NBC - R.S.A. using 5 modes - S.R.S.S.					4140	----

D_s is the dimension of the lateral load resisting structure,
S is the seismic response factor.
S.R.S.S. is the square root of the sum of the square

Table 1

The forces in the east bracing members vary by 37% from Option 1 to Option 3 as a result of their relative distance from the centre of mass. The average diagonal bracing size is W200x59 and the existing columns that are part of the bracing system need not be reinforced. The uplifts at the columns were handled by engaging the foundation walls using structural steel members.

In the east-west direction the existing MRF-s performed well. Some of the riveted joints were overstressed - this problem was solved by reinforcing the joints to achieve the proper joint capacity using the new welds only. Since the mass is not eccentric for east-west analysis (see S1) the mass "amplification" was reduced to $\pm$ 0.1 D.

CONCLUSION

When upgrading an existing structure for the current code requirements it is important to study all existing documentation and to perform the necessary laboratory tests for better prediction of the structure's behaviour. Knowing the structural behaviour, the method of construction and using dynamic analysis, one can achieve the most cost effective and safe structural upgrading. Sometimes, as in this case, phasing the structural retrofit in time, and accommodating for future design flexibility, will decide the type of structure used for stiffening the existing structure.
As presented in this report, the building period of the structures with setbacks are not defined in the NBCC, therefore some structures may be under or overdesigned for earthquake forces by as much as 16%.

ACKNOWLEDGEMENTS

The author would like to extend his sincere appreciation to Canada Life Assurance Company for entrusting us with the challenge of the upgrading of this landmark building and to Michael Jelicic, P.Eng. and Robert Halsall, P.Eng. of the Halsall Associates design team.

REFERENCES

Clough, R.W., Penzien, J. 1975. Dynamics of Structures.

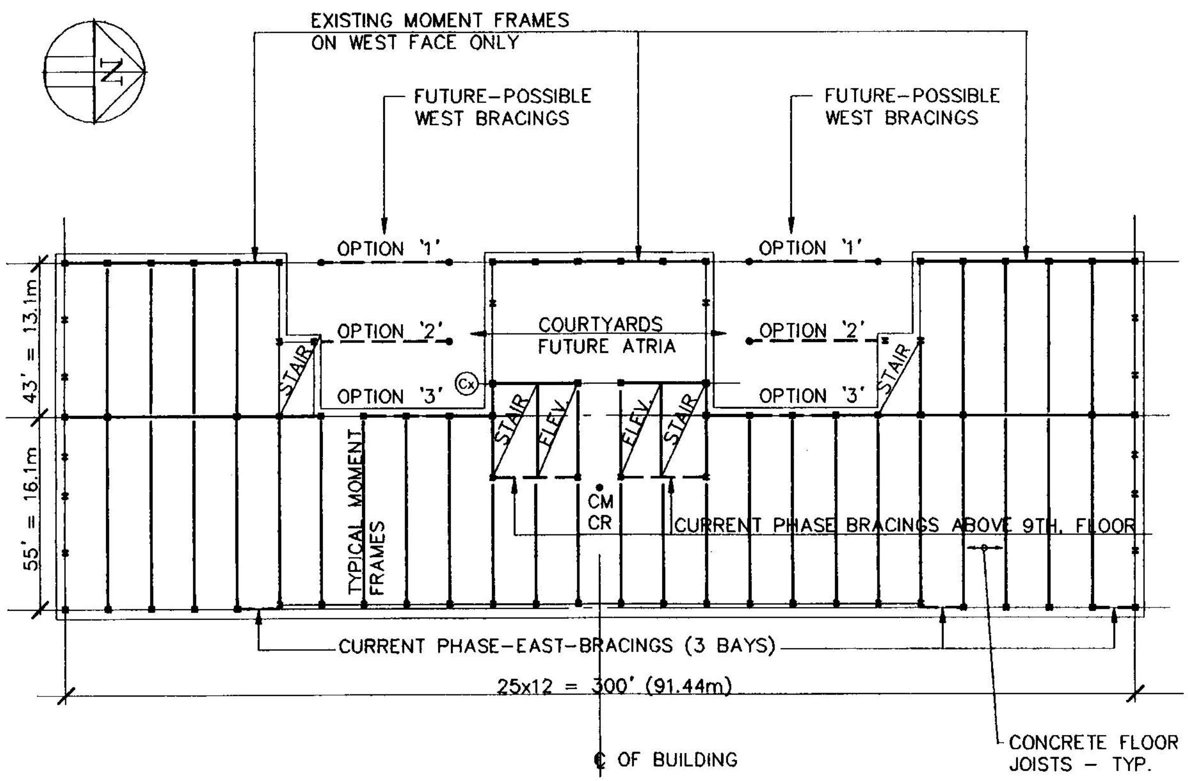
Paulay, T., Priestley, M.J.N. 1992. Seismic Design of Reinforced Concrete and Masonry Buildings.

Wong, C.M., Tso, W.K. 1994. Seismic Loading for Buildings with Setbacks. Canadian Journal of Civil Engineering. Vol. 21. 863-871.

Chidiac., S.E. 1993. Seismic Adequacy of Old Stone Masonry Buildings. I.A.B.S.E. Symposium. Vol. 70. 321-328.

Cherry, S., Filiatrault, A. 1991. The Use of Friction Dampers in the Seismic Design and Retrofit of Buildings. Seminar. Innovative Techniques for Earthquake Resistant Design of Buildings. 6-25.

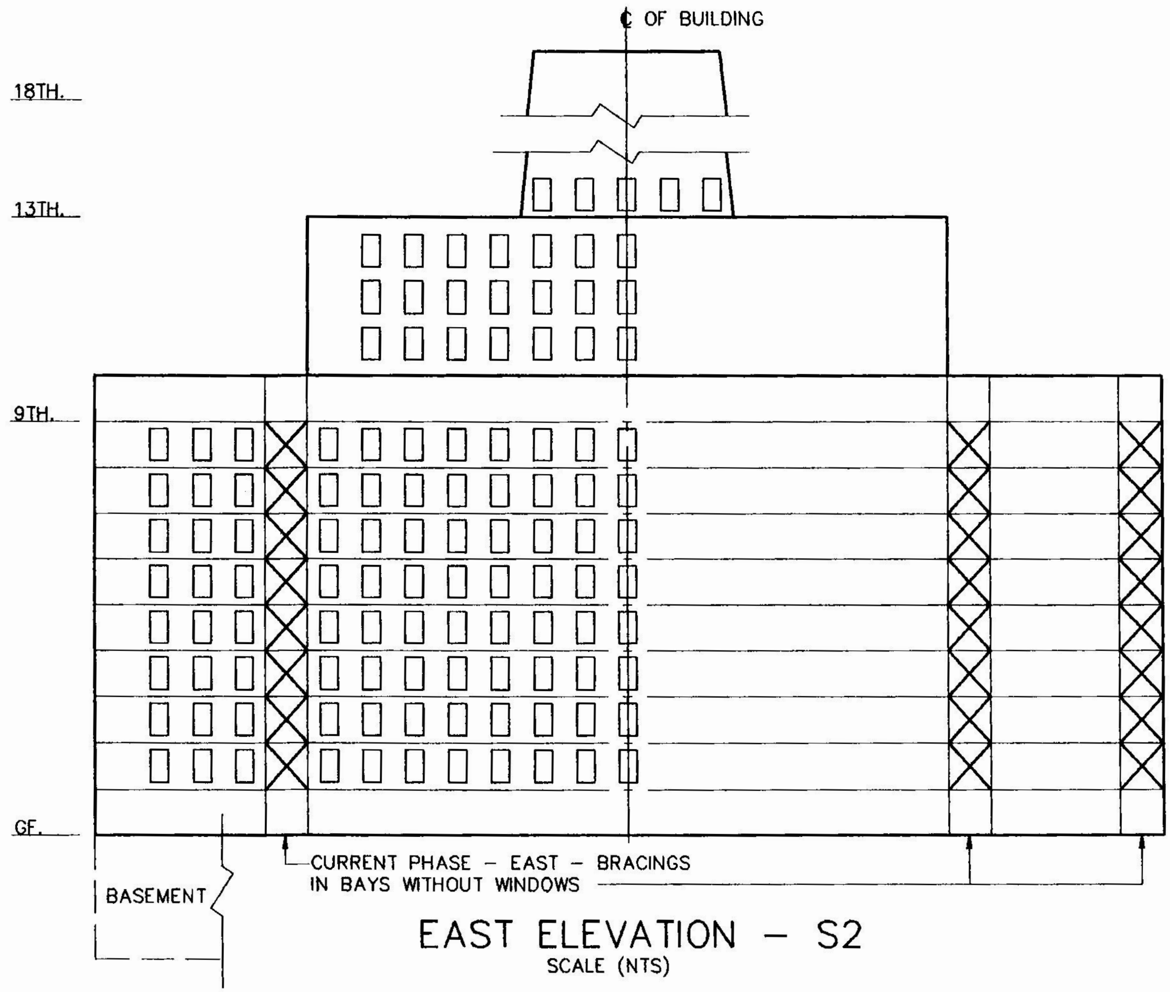
Pall, A.S. Pall, R. 1991. Friction-Dampers for Seismic Control of Buildings. Seminar. Innovative Techniques for Earthquake Resistant Design of Buildings. 26-35.



NOTES: - CM - CENTRE OF MASS
- CR - SHEAR CENTRE

PLAN - S1

SCALE (NTS)



EAST ELEVATION – S2
SCALE (NTS)