Seismic Evaluation of the Historic East-Memorial Building Retrofitted with Friction Dampers, Ottawa, Canada

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SUMMARY:
A seismic evaluation under the seismic forces required by the current National Building Code of Canada, NBCC 2005, was conducted on the historic 8-story East-Memorial Building in Ottawa. The 1955 reinforced concrete structure was retrofitted in 1995 with steel bracing with Pall friction dampers. The Earthquake loads were determined using non-linear time history analysis of simulated records obtained from the University of Western Ontario. Five records matching the site location were scaled for the correct site class. The displacements obtained in this analysis are comparable or slightly higher than the published 1995 results of the original design. However, all the lateral drifts remained within the tolerances of the current building code. It was concluded that the structure remains elastic under the time-history earthquake loads which allows the building to go back to its initial position after the earthquake ground motion is over. No further retrofit was recommended.

Keywords: Non-Linear Time-History Friction Dampers Modelling Seismic Drifts

1. INTRODUCTION

On August, 2010, Halsall Associates was engaged by SNC-LAVALIN Operations and Maintenance Inc. to complete a Seismic Assessment of the East Memorial Building on 284 Wellington Street in Ottawa, Ontario, Canada. Our services included: (i) Conducting a complete seismic study/ impact assessment to determine, if the building meets current Real Property Services (RPS) as per Public Works Government Services Canada policy on seismic resistance; (ii) Executing a detailed seismic assessment of the building's structural elements and the degree to which they meet the current National Building Code of Canada (NBCC 2005) seismic resistance requirements; and (iii) Performing a detailed 3-D seismic analysis of the building and prepare a report of the findings with options and recommendations for repair, if required.

RPS policy on seismic resistance states that seismic requirements are to be in full compliance with current local by-laws and provincial/territorial building codes, where such requirements exist. There are no such requirements for upgrading the existing buildings in Ottawa. The Policy further states that seismic upgrading of a building structure is not mandatory, if the building structure meets or exceeds 60% of seismic load requirements for new construction as specified by current NBCC.

1.1. General Description of the Building

The original East Memorial Building built in 1955, is an eight storey building with one basement. The plan dimensions are approximately 91mx54m. Major renovations to the building began in 1995 and were completed in 1997. The structural work included a seismic upgrade to NBCC 1995 which was the building code at the time. Based on the information contained in the existing drawings, the structural system of the building consists of cast-in-place reinforced concrete floor joists, beams and slabs supported on columns. The structure is founded on spread footings bearing directly on bedrock with an allowable bearing capacity of 2400 kPa (50 kips/ft^2). The roof structure is framed with structural steel and light weight concrete panels.

The lateral load carrying system of the original building consisted of frame action between concrete columns and beams, masonry infill and nominally reinforced concrete walls around elevators. As a part of the 1997 renovation, structural steel braces, with friction dampers, were added to upgrade the seismic performance of the building to the NBCC 1995 seismic requirements.

1.2. General Description of the Friction Dampers

Pall friction-dampers manufactured by Pall Dynamic Ltd, consist of series of steel plates specially treated and clamped together with high strength steel bolts to develop breaking style friction that slow down the motion of vibrating buildings and dissipate energy in friction. During severe seismic excitations, friction-dampers slip at a predetermined optimum load before yielding occurs in other structural members and dissipate a major portion of the seismic energy. This usually allows the building to remain elastic during moderate earthquakes while yielding is delayed to be available during maximum design earthquakes forces. After the earthquake, the building returns to its near original alignment under the spring action of an elastic structure.

Pall friction-dampers possess rectangular hysteresis loops, similar to ideal elasto-plastic behaviour, with negligible fade over several cycles of reversals (Balazic et al. 2000). See Fig. 1.1 below.

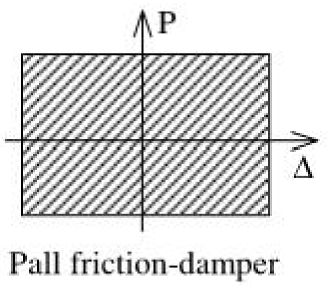

Figure 1.1. Elasto-plastic behaviour of the friction dampers

2. STRUCTURAL ANALYSIS

2.1. Seismic Hazard

According to NBCC 2005, strong ground motion is defined as having a probability of exceedance of 2% in 50 years at the median confidence level. This corresponds to a 0.04% annual probability of exceedance. This ground motion is termed as the maximum earthquake ground motion to be considered and therefore referred to as the design ground motion (DGM). Knowing that the original building is founded directly on bedrock, a Site Class "B" was assumed for this site. The 5% Damped Spectral Response Acceleration values for a Site Class B in Ottawa are shown in Table 3.1. below. The plot of the design response spectrum is also shown in Fig. 2.1. below.

Table 2.1. 5% Damped Spectral Response Acceleration Values for Ottawa Site Class "B"

NBCC 2005 Values for 2%/50 Years in decimal percentages of g, accelerations due to gravity				
S(0.2)	S(0.5)	S(1.0)	S(2.0)	S(4.0)
0.5702	0.2016	0.0819	0.0277	0.0139

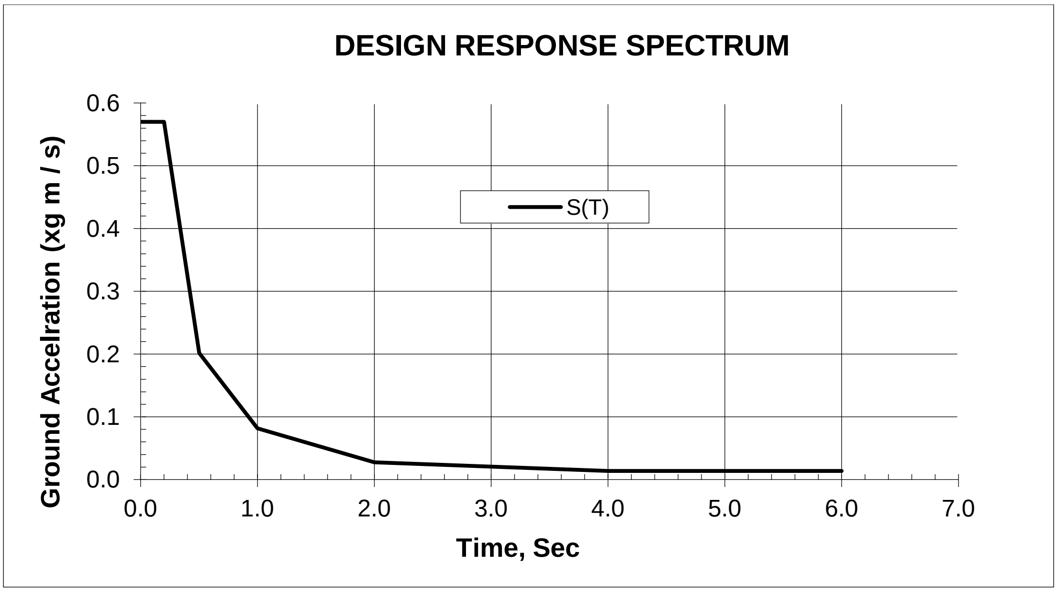

Figure 2.1. Design Response Spectrum

2.2. Structural Modelling

A 3D finite element analysis model of the building was created using ETABS 9.7.1 as shown in Fig. 2.2. All braces, columns, beams, and floor structure were included in the model. The floors were modeled as rigid diaphragms. The Pall friction dampers were modeled as non-linear link elements. For the purpose of determining stiffness and deflections in the structure, cracked section properties are modeled by specifying stiffness modifiers for slabs, beams, and columns in accordance with CSA A23.3-04 (Concrete Design Handbook 2006). Fig. 2.3. shows an example braced bay elevation. The concrete floors of the building were modeled as finite element shell objects and the braces, columns, and beams were modeled as finite frame elements. Appropriate reduction factors to the inertias and cross sections were applied to the elements in the model to account for reduced stiffness due to cracked concrete elements. A rigid diaphragm constraint was applied to each of the concrete floors Wilson, E., L., (2000).

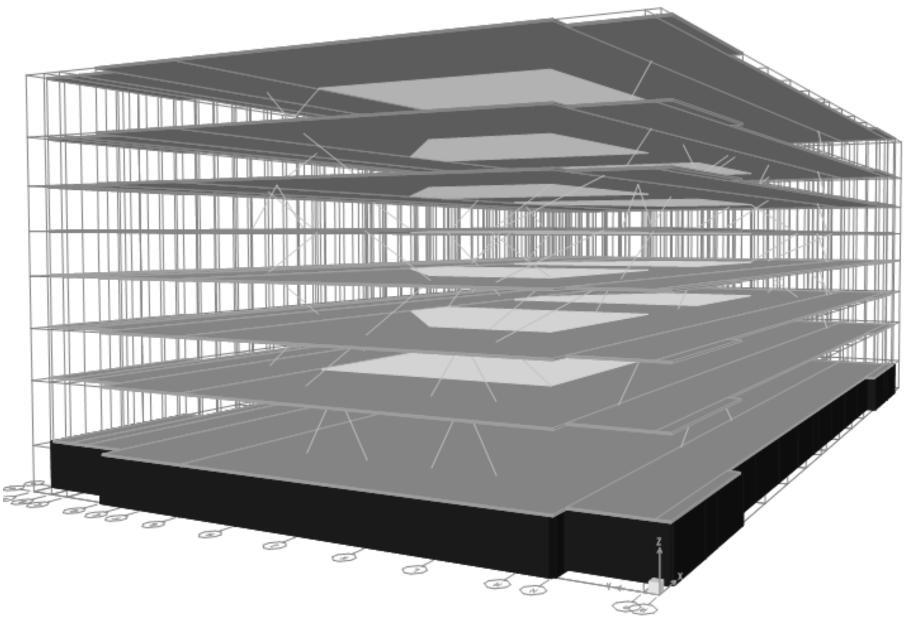

Figure 2.2. ETABS Model

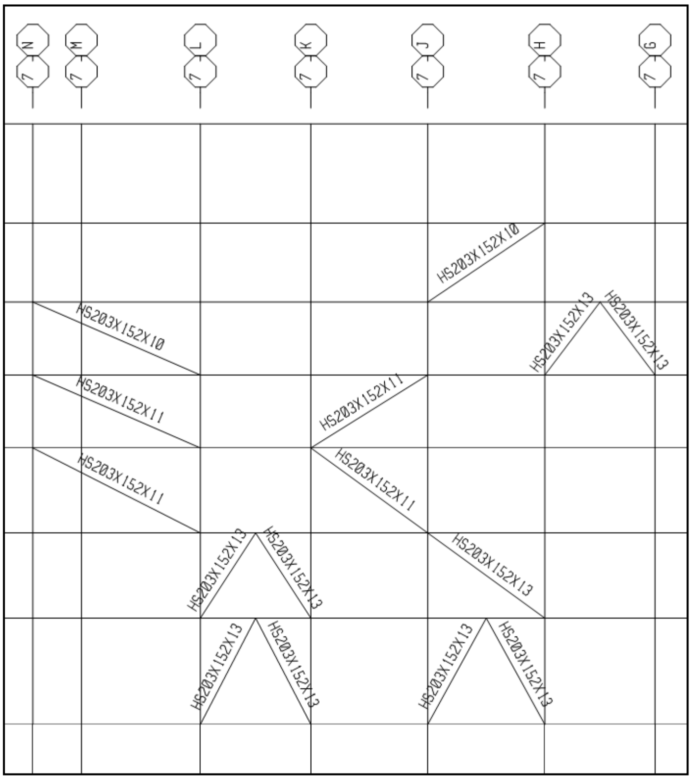

Figure 2.3. ETABS Brace Elevation

The friction dampers are modeled as non-linear link elements with a hysteretic loop assumed to be almost rectangular, simulating an elasto-plastic material. This was achieved by specifying a fictitious plastic element for these links, having a yield force equal to the slip load of the friction dampers. The link elements were assigned an effective stiffness calculated with respect to the properties of the brace and the total length of the brace plus the damper. For example, in the case of diagonal braces, the brace plus friction damper (damped brace) is modeled as a link element. See Fig. 2.4. below.

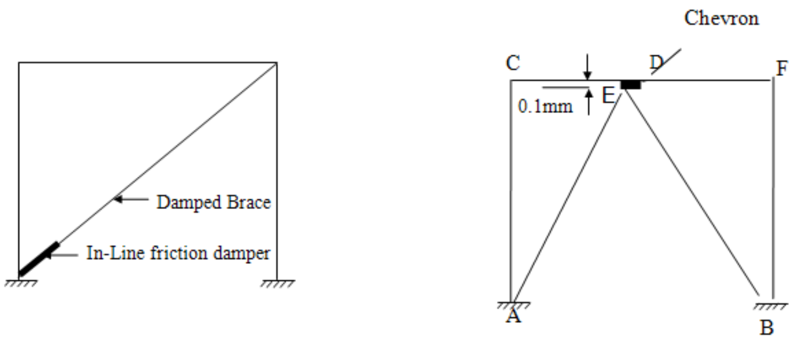

Figure 2.4. Link Modelling

There are three different brace sizes used in diagonal bracings and chevron bracing retrofits in the building: HSS 203×152×12.7, HSS 203×152×11, and HSS 203×152×9.5. These are connected to Type 1, Type 2, and Type 3 friction dampers respectively. Tables 2.2 and 2.3 below show the brace capacities excluding and including the friction dampers respectively. The ultimate approximate compressive strengths of the brace calculated in accordance with S16-09 based on the average un-supported length of the brace in Table 2.2., and the ultimate compressive strength of the brace is based on the pre-determined slip load of the friction damper in Table 2.3 (Handbook of Steel Construction, 2011).

Table 2.2. Brace Properties excluding dampers

Type	Brace Size	Approximate Compressive Strength (kN)
1 (X-brace)	HSS 203×152×12.7	822
2 (X-brace)	HSS 203×152×11	744
3 (X-brace)	HSS 203×152×9.5	661
4 (Chevron)	HSS 203×152×12.7	1130
5 (Chevron)	HSS 203×152×11	1047
6 (Chevron)	HSS 203×152×9.5	905

Table 2.3. Friction Dampers Slip Loads

Friction Damper	Brace Size	Damper Slip Load (kN)
Type 1	HSS 203×152×12.7	700
Type 2	HSS 203×152×11	600
Type 3	HSS 203×152×9.5	500

2.3. Equivalent Static Method

For comparative purposes, the equivalent static force procedure was applied to obtain the base and story shears. The total building weight was estimated to be: W_T = 220,000 kN including the weight of walls, columns and beams. The code fundamental period of the structure was calculated to be: T_a = 0.85 sec. Based on the ETABS model, including modelling the HSS braces as the main SFRS elements without the friction dampers, the fundamental period of the structure in the North South was found to be: T_y = 1.74 sec. In the East West direction, T_x was found to be 1.79 sec. These lateral periods obtained from the model are larger than 2 times T_a which is the maximum permissible period of NBCC 2005. Therefore, 2×T_a = 2×0.85 = 1.7 sec was used in the Equivalent Static Force Procedure. The equivalent static base shear is calculated as follows:

$$V_{\text{Equiv.Static}}\,(T_a = 1.7) = \frac{S(T_a)M_v I_e W_T}{R_d R_o} = \frac{0.044 \times 1.0 \times 1.0 \times 220{,}000}{1.5 \times 1.3} = 5000\ kN \qquad (2.1)$$

Under the Equivalent Static Method procedure, 90 braces are modeled with their HSS section properties, 35 out of 90 braces appear to have their compressive yield strength exceeded. Table 2.4. below summarizes the drifts and storey shears from Equivalent Static Procedure.

Table 2.4. Story Drifts under Equivalent Static Procedure

Storey	Drift (mm)	Drift×R_dR_o (mm)	Inter-storey (mm)	Storey Height (m)	% Drift	Storey Shears (kN)
7	54.90	107.06	19.31	4.99	0.39%	1051
6	45.00	87.75	21.45	3.96	0.54%	1956
5	34.00	66.30	16.19	3.66	0.44%	2737
4	25.70	50.12	16.58	3.66	0.45%	3404
3	17.20	33.54	19.21	4.27	0.45%	3957
2	7.35	14.33	14.17	4.27	0.33%	4376
1	0.08	0.16	0.10	5.34	0.00%	4789
ground	0.03	0.07	0.07	3.81	0.00%	4945
Base	0.00	0.00	0.00	0.00	0.00%	1051

The maximum lateral drift obtained at the top of the building using the Equivalent Static modelling was 107mm at the roof, and the maximum inter-storey drift obtained was 0.54%. These values represent a theoretical model that assumes that the Seismic Force Resisting Elements remain elastic which is not the case.

2.4. Time-History Records

Simulated time-history records with a probability of exceedance of 2% in 50 years were obtained from seismologist Professor Gail Atkinson at the University of Western Ontario. The raw data could be found on the website: www.seismotoolbox.ca. There are 45 ground acceleration data for each site class range, available for Western Canada and for Eastern Canada. The response spectrum associated with each data is also provided. (Atkinson 2009). Fig. 2.5. below shows the response spectrum accelerations of one of the simulated records and the design NBCC response spectrum accelerations for Ottawa, Site class B (Set as the "target" response spectrum).

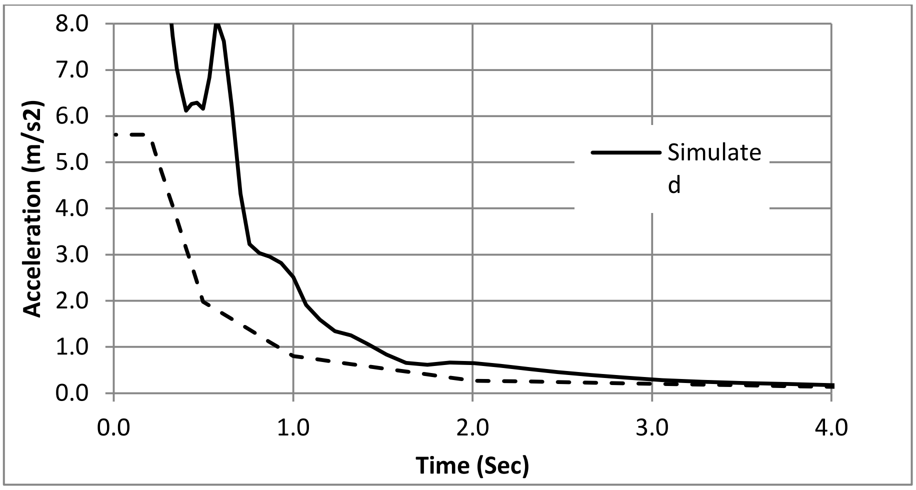

Figure 2.5. Simulated Response Spectrum v/s Design Response Spectrum

Fig. 2.6. below shows the same simulated record but scaled to match the NBCC target spectrum. The scale factor was calculated as the average of ratios of the target acceleration to simulated acceleration, S_{target} / S_{sim}, for the time range T = 0.2 to T = 1.0 sec. The standard deviation was also calculated for the ratios S_{target} / S_{sim} between 0.2 and 1.0 sec. The simulated record of Figs. 2.5. and 2.6. is an example of a record with a standard deviation of 0.1. This procedure was repeated for all the time history records available for Eastern Canada. 4 records with the smallest standard deviations were considered.

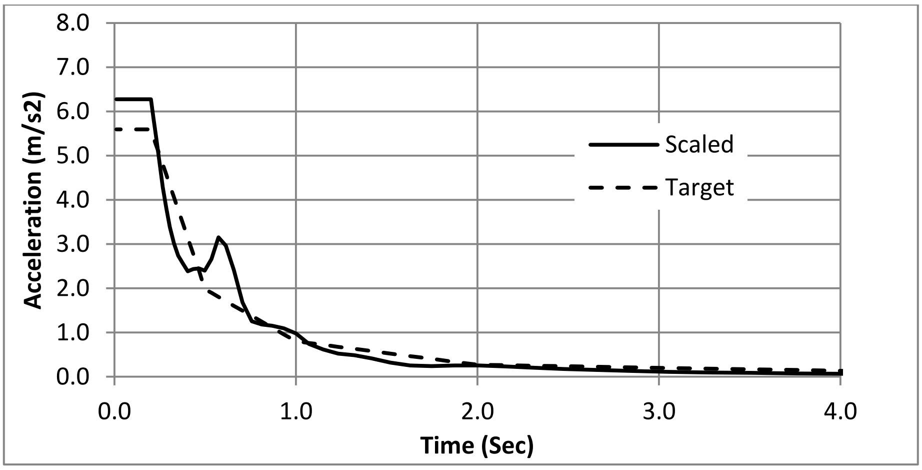

Figure 2.6. Scaled Response Spectrum v/s Design Response Spectrum

The ground accelerations of the corresponding record are then multiplied by the scale factored obtained. Fig. 2.7. shows the ground acceleration scaled record that corresponds to the response spectrum described above. This record is one of four time history functions considered in the analysis. These records are applied in both North-South and East-West directions.

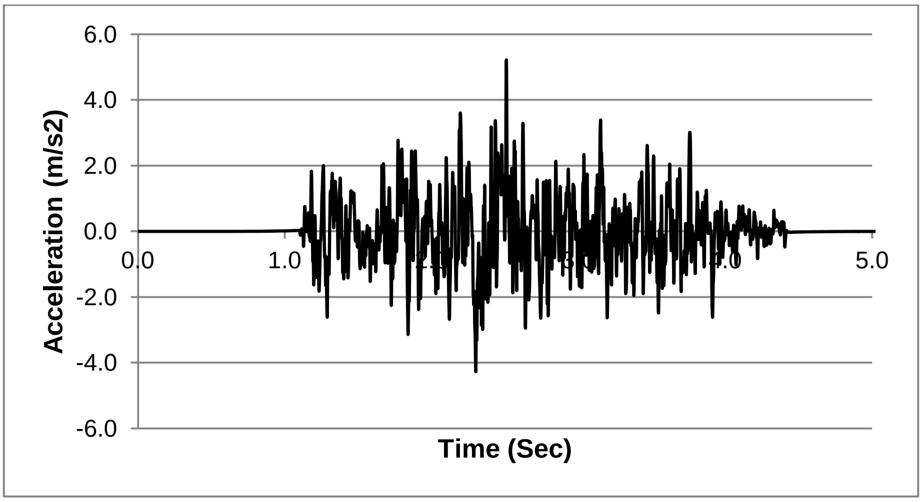

Figure 2.7. Scaled Simulated Ground Acceleration Record

3. RESULTS

3.1. Displacements

Figs. 3.1. and 3.2. show the axial forces time histories in (kN), and the displacements time histories in (mm), obtained for one of the link elements for (Type 2). It can be seen that the axial force in the damped brace does not exceed the slip load of the friction damper. The displacements obtained are in the range of 4 to 5mm. For that, about 3mm displacements should be added as an initial elastic deformation in the brace element itself. It can also be seen from the charts that most of the axial forces and displacements occur up to about 5.0 seconds, which is the considered duration of the ground accelerations. Beyond that time frame, the 5% modal damping of the structural ensures a damped response of the braces back down to zero by the end of the time histories at about 26 sec.

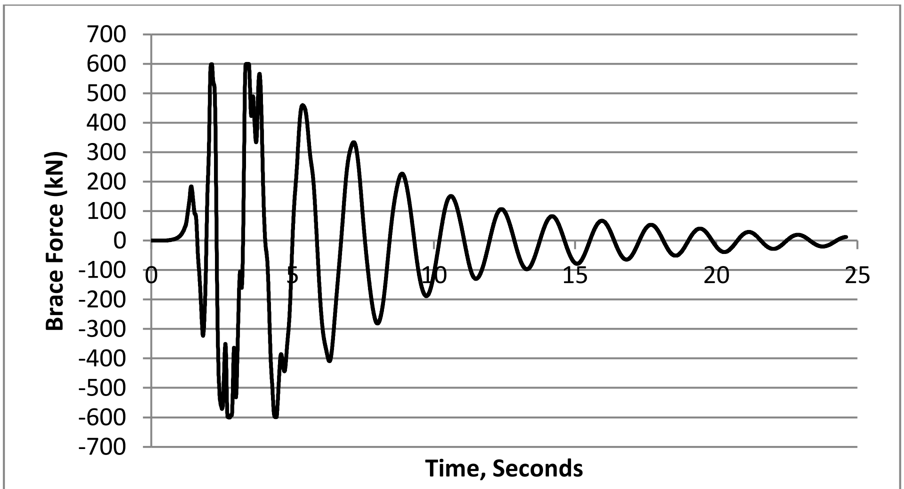

Figure 3.1. Axial force for brace with Type 2 friction damper

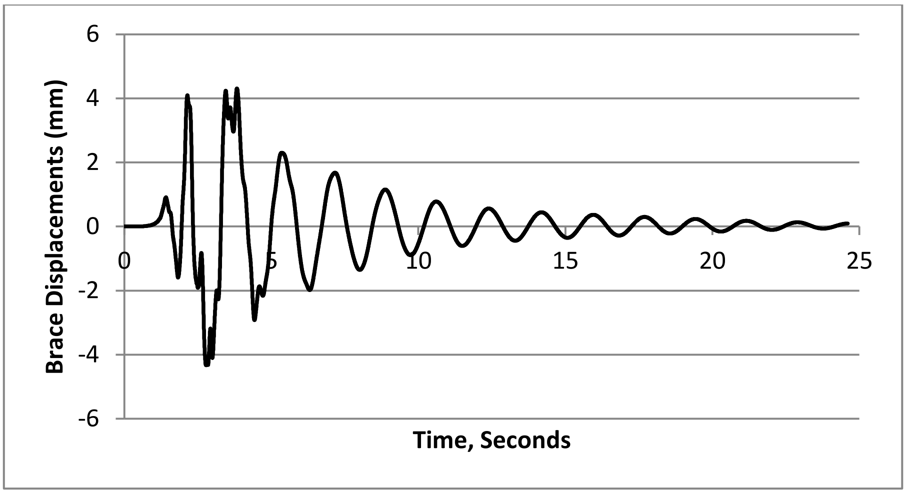

Figure 3.2. Displacement for brace with Type 2 friction damper

In comparison, Fig. 3.3. below shows a theoretical the axial force time history for an un-damped Type 2 brace. This is the same brace with the histories shown in Figs. 3.1 and 3.2 but no friction dampers in the model. That time history, however, will not occur since the brace would yield before reaching axial forces of such high magnitude. When the model was run without friction dampers under the same ground motion time histories, 72 out of 90 braces yielded (80%).

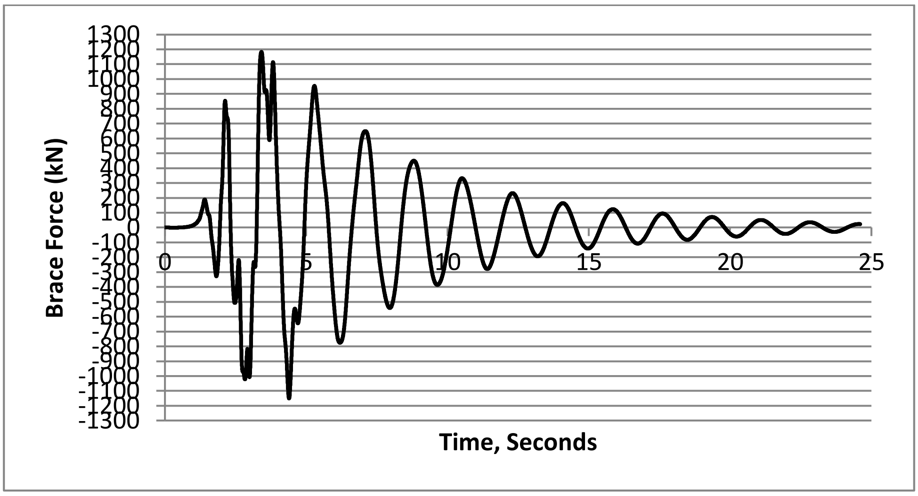

Figure 3.3. Theoretical axial force in Type 2 brace with no damper

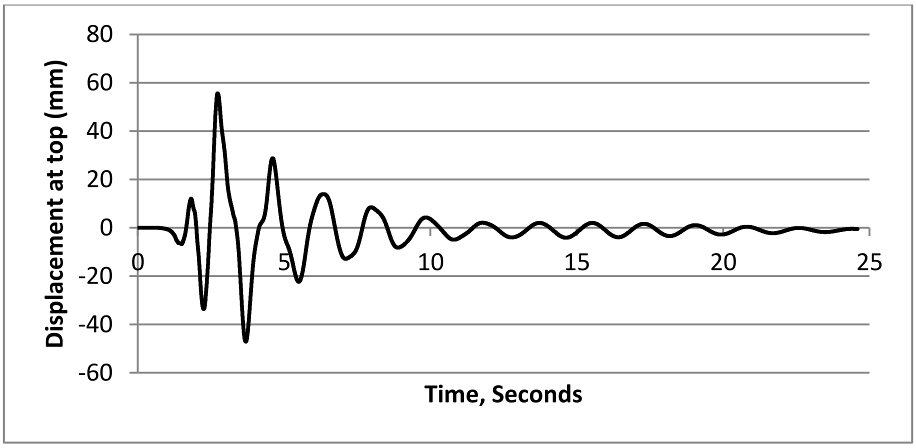

Figure 3.4. Roof Displacement in the N-S Direction

Fig. 3.4 shows the roof displacement time histories in the North South direction. The maximum displacement at the roof in the North South direction obtained was 57 mm. This is significantly less than those obtained from the Equivalent Static Force Procedure as shown in Table 2.4.

3.2. Story Shears

Table 3.1. below summarises the story shears obtained. For comparison purposes, the story shears obtained from the model with no friction dampers, and the storey shears obtained from the model with no basement are also shown.

Table 3.1. Storey Shears Summary

Storey	Model with Friction Dampers		Model with no Dampers		Model With no Basement		Equiv. Static	Equiv. Static ×RdRo
	Vx	Vy	Vx	Vy	Vx	Vy	V	Ve
7	4415	3611	8335	5655	4116	2628	1051	2049
6	5194	5246	9228	7266	4537	3717	1956	3814
5	5669	5247	7845	6773	4051	4857	2737	5338
4	6055	5357	8061	7447	4706	5196	3404	6638
3	5422	5231	7133	7260	4128	6610	3957	7716
2	4966	4797	6757	6810	3274	5559	4376	8534
1	5960	6004	10162	8957	3311	8928	4789	9338
Ground	43215	64024	44485	64421	-	-	4945	9643

It could be seen from these results that the story shears in the model with no dampers are significantly higher than those with the friction dampers. However, the story shears in the model with no dampers, are comparable to the elastic shears obtained from the Equivalent Static Procedure. This indicates the dampening effects of the friction dampers on the structural response to the time history functions.
The storey shear at the ground level obtained is very large. This is likely due to the foundation walls that constitute very stiff elements attracting large shears in time history modelling. There are no braces in the ground level, but the walls are likely able to resist the large shears that they attract by their own stiffness. It could be seen that the magnitude of the story shears obtained in a model where no basement walls are included is similar to the story shears obtained in levels 1 and up in the model that includes friction dampers. This indicates that the large storey shears obtained at the lower level are due to the presence of basement walls.

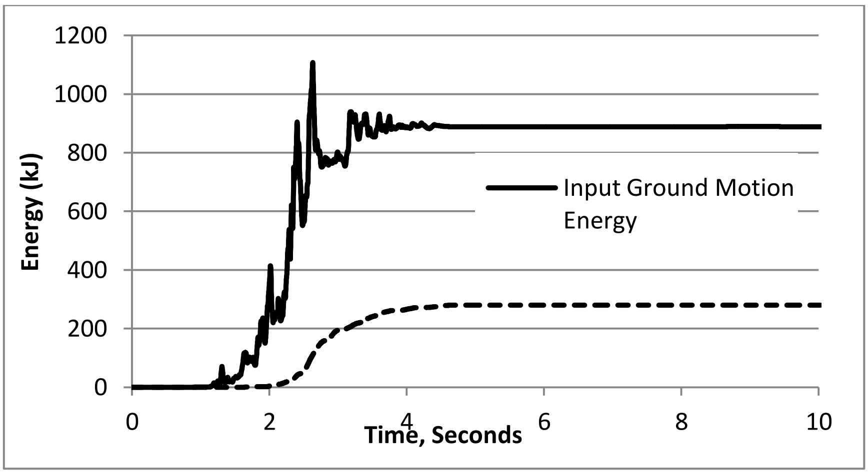

Fig. 3.5. Energy Dissipation

Fig. 3.5. shows the amount of energy dissipated by the friction dampers versus the input energy of the ground motion as obtained from the ETABS model. This also explains the lower story shears obtained in the model with friction dampers as discussed above.

4. CONCLUSION

Gravity and earthquake load combinations were checked for several columns in the building. The load combination for earthquake includes the maximum forces obtained from the time histories plus the specified dead load forces. The analysis shows that the earthquake maximum time-history moments and axial forces combined with the dead loads moments and axial forces dot yield the columns. The maximum utilization ratio of some of the worst case columns was 0.35 (MacGregor et al. 2000). Therefore, it could be concluded that the main structural elements remain elastic during the ground motion time history analysis of this study, and that the columns remain elastic at all times.

No further analysis was performed in regards to the connections. The adequacy of the connection details remains within the requirements of the original design, which is mostly affected by the slip loads of the friction dampers. These remain unchanged in this analysis.

Compared to some of the findings of the initial 1995 retrofit, it could be seen that the displacements obtained in this analysis are slightly higher than the published 1995 results of the original design (Balazic et al., 2000). The earthquake forces obtained from the new time history analysis of this study are higher than those of the original design. However, the new results show that all the lateral drifts are within the tolerances of NBCC2005 and the behaviour of the friction dampers remains adequate to resist the new loads. In addition, the fact that the structure remains elastic under the time-history earthquake loads allows the building to go back to its initial position after the earthquake ground motion is over. Assuming that the connections perform in accordance with what was intended by the original design, and assuming that the friction dampers perform in accordance with the manufacturer’s specifications and with the guidelines and requirements of the 1995 structural drawings, it was concluded that the seismic performance of the structure is adequate under the current provisions of the NBCC code and that no further retrofit is recommended.

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