



SEISMIC REHABILITATION OF RIVIÈRE-DU-LOUP HOSPITAL USING FRICTION DAMPERS

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ABSTRACT: Built in 1968, the Rivière-du-Loup Hospital is located in a high seismic zone in Eastern Canada on the south shore of the St. Lawrence River, 200 km east from Quebec City. On March 6th 2005, a magnitude 5.4 earthquake occurred in the Charlevoix seismic zone: it was strongly felt in Rivière-du-Loup. This earthquake, and the release of a new seismic hazard map prescribed by NBCC 2005, convinced the authorities to take action. The structural irregularities and the building's seismic forces resistance system (SFRS), composed of non-ductile moment-frames, made the 8-stories hospital very likely to perform poorly under large seismic displacement demand. Using a performance-based approach, some SFRS options were studied to upgrade the seismic performance of the hospital. The main performance criterias considered were inter-story drift ratios and floor accelerations. Non-Linear Time History Analysis and Pushover Analysis were used to adequately capture the behavior of both the existing and retrofitted structures. Seismic rehabilitation using friction dampers was selected as the optimal solution. A sensibility study was then conducted to select the optimal value of the performance parameters. The tight construction schedule and numerous particularities of the structure brought the engineers to design and draw in 3-D the 27 types of connections needed. The work on site began in May, 2013, and will be completed in September, 2015. The project's main challenge was to keep the Hospital fully operational during the construction work, which was successfully achieved thanks to a great collaboration between the client, the project's architect, the mechanical engineer and the structural engineers.

1. Introduction

The Rivière-du-Loup hospital provides health services to a 85 000 person population spread along the south shore of the St. Lawrence River. The hospital's facilities include four buildings identified as wings A, B, C and D. Amongst those four buildings, wings A and B are classified as "post-disaster" buildings with respect to their importance category, which implies that the dispensed health services must remain operational after the NBCC-prescribed 2500 years design earthquake. The building formed by wings A and B includes more than 15 000 m² of floor area providing health services.

In 1988, the magnitude 5.9 Saguenay earthquake first caused the authorities to worry about the hospital safety in the event of a major earthquake. Following a structural analysis conducted in the 90's, wings A and B were fixed together with floor anchors, in an effort to improve their seismic behaviour by preventing the two buildings from colliding together.

Then, on March 6th of 2005, a magnitude 5.4 earthquake occurred in Rivière-du-Loup, which is located inside the Charlevoix seismic zone. It was strongly felt in the city. Those two earthquakes, and the release of a new seismic hazard map prescribed by NBCC 2005, brought the authorities to request proposals for the seismic retrofit of wings A and B. The non-structural components were excluded from the proposals: only the building's structure had to be studied and retrofitted.

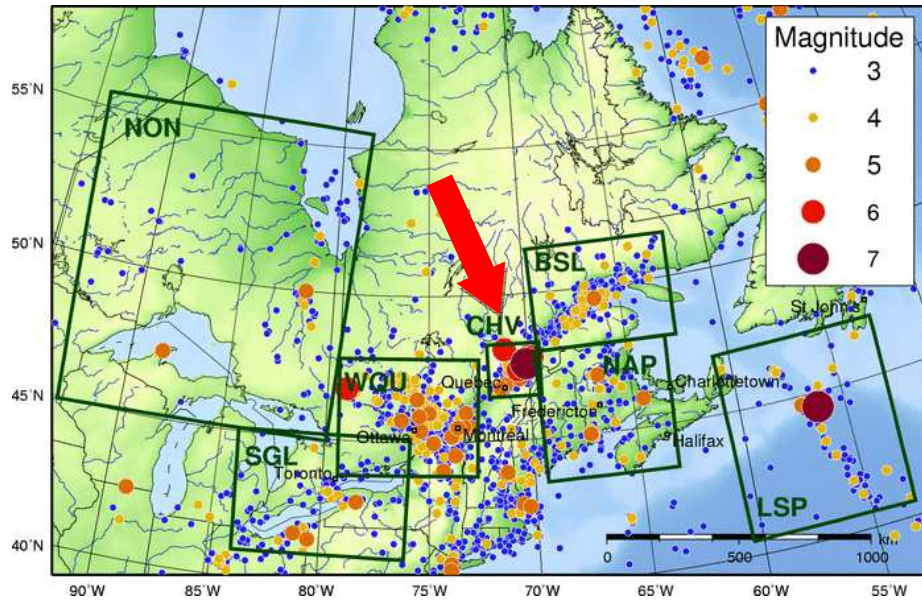


Figure 1 - Charlevoix seismic zone (CHV)

2. Structural System of the Hospital Buildings

Wings A and B of the Rivière-du-Loup hospital were built in 1968. Wing A is made of a 7-stories steel tower built on a 3-level concrete podium (including one level below the soil grade line). Wing B is composed of a 5-stories concrete structure built on a 2-levels concrete podium, including one level below the soil grade line. Also, wing A's steel tower is oriented with a 45 degrees angle from wing B, as shown on Figure 2.

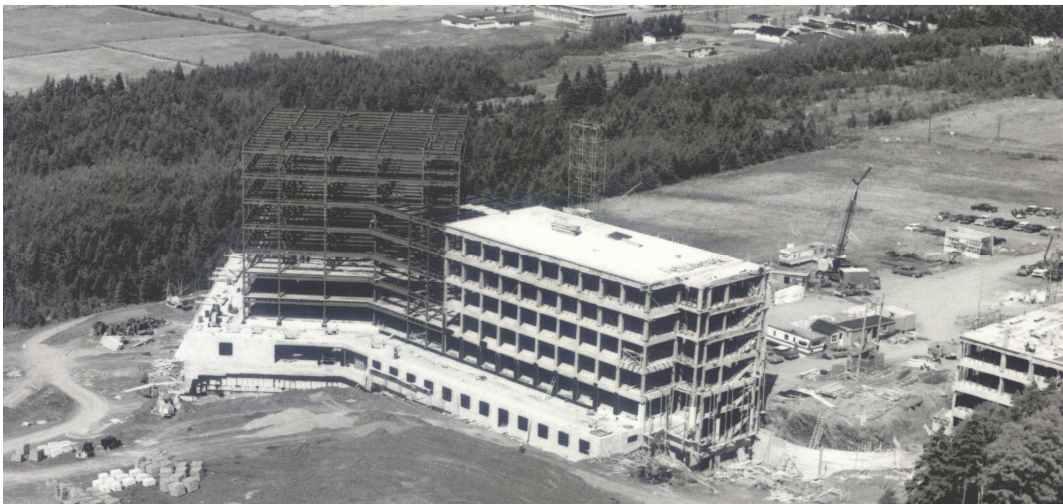


Figure 2 – Aerial view of wings A and B under construction (1968)

The existing seismic forces resistance system (SFRS) of both wings is the moment-frames formed by the rigid connections between the beams and the columns. Because the 7-stories steel tower and the 5-levels concrete structure connect together with a 45 degrees angle, the SFRS of the structural building formed by wings A and B tied together is irregular, as per NBCC 2010 definition. The detailed SFRS characteristics for wings A and B are summarized in table 1.

Table 1 – SFRS OF WINGS a AND b

	Wing A	Wing B
Vertical SFRS	Steel non-ductile* moment-frames	Concrete non-ductile** moment-frames
Diaphragm	4 inches thick concrete slab on steel beams with studs	2.5 inches thick slab on concrete joists and diaphragms, concrete beams

* Non-ductile according to ASCE 31 and FEMA 355f provisions.

** Non-ductile according to ASCE 31 and the push-over analysis.

The structural irregularities and the building's SFRS composed of non-ductile moment-frames made the 8-stories hospital very likely to perform poorly under large seismic displacement demand. In addition, the moment frame's structural members have limited capacity, and includes many joints with beams having higher capacities than the column (*strong beam – weak column* type of deficiency). Also, the shear reinforcement steel in the columns, at the joint, is composed of #3 stirrups spaced 12 in. c/c, which does not allow ductile behavior.

3. Seismic Retrofit Objectives

The objective of the seismic retrofit was to make the hospital safe in the event of a major earthquake. Some restrictions were imposed by the project's nature, namely: allow the hospital services to remain operational during construction work, and minimize the impacts on the hospital users. The structural engineers also looked for a solution that would minimize the floor accelerations, especially since the non-structural components were not to be retrofitted. This would allow for mitigating the earthquake impacts on the non-structural components and hospital services in the event of a strong earthquake.

From these objectives and constraints were derived the following requirements for the seismic retrofit solution to encounter:

- Minimize the construction work inside the hospital (including structural reinforcements);
- Control the displacement demand to acceptable values;
- Provide sufficient damping to reduce the floor accelerations and the forces in the structure;
- Give flexibility to the owner and the architect for current and future space planning;
- Do not hinder windows or modify the global architectural aspect of the hospital.

These requirements allowed for the comparison and selection of a preferred retrofit solution.

4. Considered Seismic Retrofit Options

Several seismic retrofit schemes were considered in order to meet the above-mentioned requirements. A solution with concrete shear walls (figure 3) offered a good performance, despite not providing the most damping, but had unacceptable impacts on the hospital operations, both after and during the construction work. The seismic retrofit using concrete shear walls also caused stresses concentrations in the floor diaphragms, which would have required parts of the diaphragms to be strengthened.

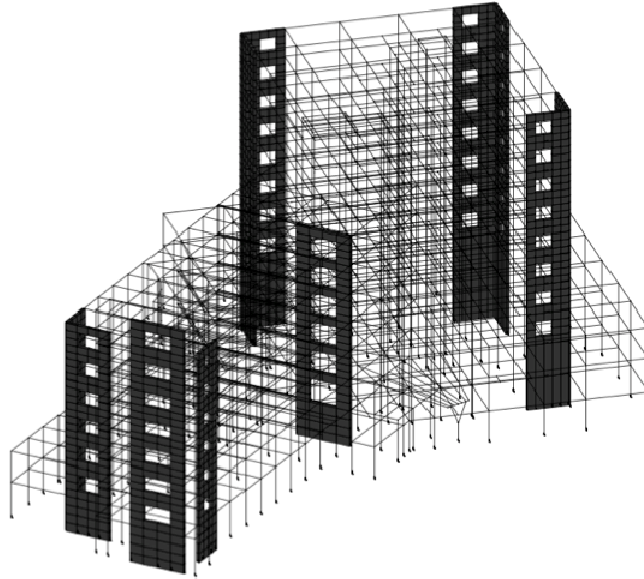


Figure 3 – Seismic retrofit scheme using concrete shear walls

At some point in the design process, it was determined by the architect that the building's envelope had major performance issues and needed to be replaced. Retrofit schemes with braces disposed along the exterior walls therefore became the preferred solution. Various steel brace types were then considered, including eccentrically braced frames (EBF), buckling-restrained braces (BRB), viscous dampers and braces equipped with friction dampers.

5. Implemented Retrofit Solution

Seismic rehabilitation using friction dampers was selected as the optimal solution to control the displacement and force demands experienced by the structure of wings A and B under strong seismic motions. The friction dampers, working together with the existing moment-frames, provide the best combination of stiffness and supplemental damping, and a great overall dynamic behavior.

Three analysis methods were used to assess the performance of the retrofitted structure in an efficient and reliable way: **Multi-modal response spectrum analysis** allowed for the assessment of the general dynamic behavior of the structure and establishing reference values for the expected forces associated with various ductility levels. **Push-over analyses** were used for a comprehensive study of the stresses generated in the reinforced concrete members under different drift ratios. **Nonlinear time history analysis** allowed for assessing the dynamic response of the structure integrating the inelastic behavior of the dampers.

A literature review on the performance of both concrete and steel non-ductile moment frames, relative to their strain level, was conducted. The target performance criteria on the inter-story drift was selected as 0,70%, based on the literature review and the pushover analysis realized on finite element fibre models and finite element shell 2D models of the concrete moment-frames. The pushover analysis allowed for a good estimate of the stresses associated with various drift levels, for the particular geometry of the moment-frames.

5.1. Non-linear Time-history Analysis

The slippage mechanism of the friction damper mounted on an elastic brace constitutes nonlinearity. In order to quantify the amount of energy dissipated by this mechanism, which is a function of the displacement, nonlinear response history analysis were conducted on a 3D numerical model in ETABS (figure 4) with friction dampers modeled as a yield force. The numerical modeling of braces equipped with friction dampers is simple: since the hysteretic loop of the damper corresponds to the rectangular loop of an ideal elasto-plastic material, the slip load of the damper can be modeled as a fictitious yield force, using, in this case, a *Plastic(Wen)* type of nonlinear link.

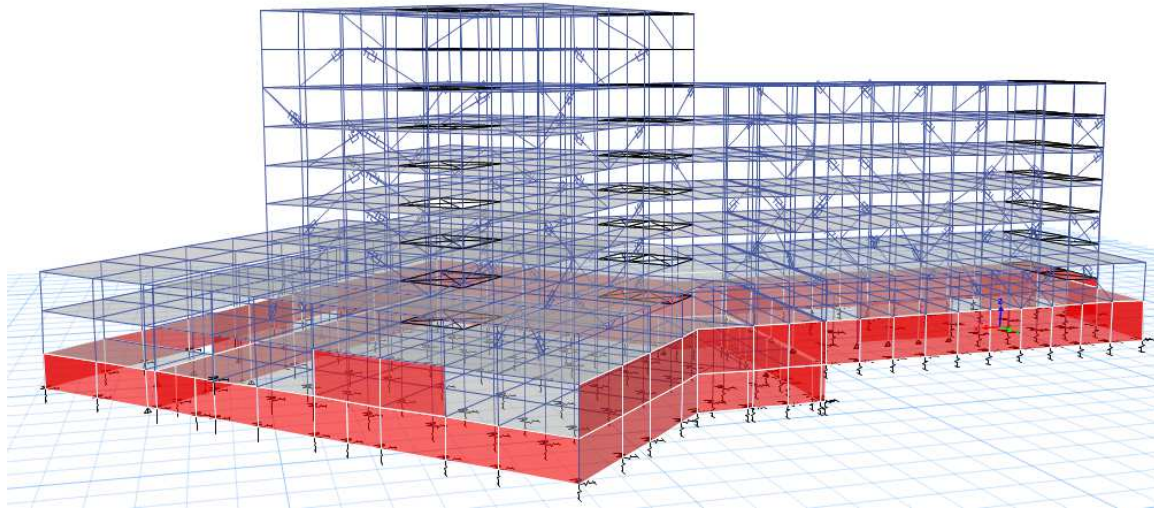


Figure 4 – Numerical model of the structure (Etabs nonlinear)

To adequately capture the non-linear dynamic response of the structure retrofitted with friction dampers, nine (9) pairs of orthogonal ground motions were used for the nonlinear time history analysis. In order to properly select the ground motions, the design team requested the deaggregation of the seismic hazard at the hospital site (figure 5). This was performed by the Geological Survey of Canada for a probability of exceedance of 2% in 50 years. The assessment of the contribution to the seismic hazard of both the Historical model and Regional model for each of the spectral ordinates ($S_a(0.2)$, $S_a(0.5)$, $S_a(1.0)$, $S_a(2.0)$) revealed a strong contribution from the Historical model. The choice of earthquake scenarios (magnitude-distance, M-R) was therefore based primarily on the deaggregation of the Historical model, but M-R scenarios from the Regional model were also considered to incorporate the potential adverse effects produced by the differences in the duration and frequency of the dominant seismic motions. The M-R scenarios retained for the ground motions selection were : M6.5-7.0 for $R = 10-80$ km, and M7.0-7.5 for $R = 20-100$ km.

The use of 9 pairs of motions was deemed appropriate considering the mass contribution that was found in several vibration modes to the dynamic response of the structure. The ground motions were spectrally matched to the 2010 NBCC, 1:2475 years response spectrum (figure 6). The motion histories were selected and adjusted following the 2010 NBCC provisions, completed with the ASCE 7-10 and ASCE 41 provisions. Those references also served as guidelines for the nonlinear time history analyses.

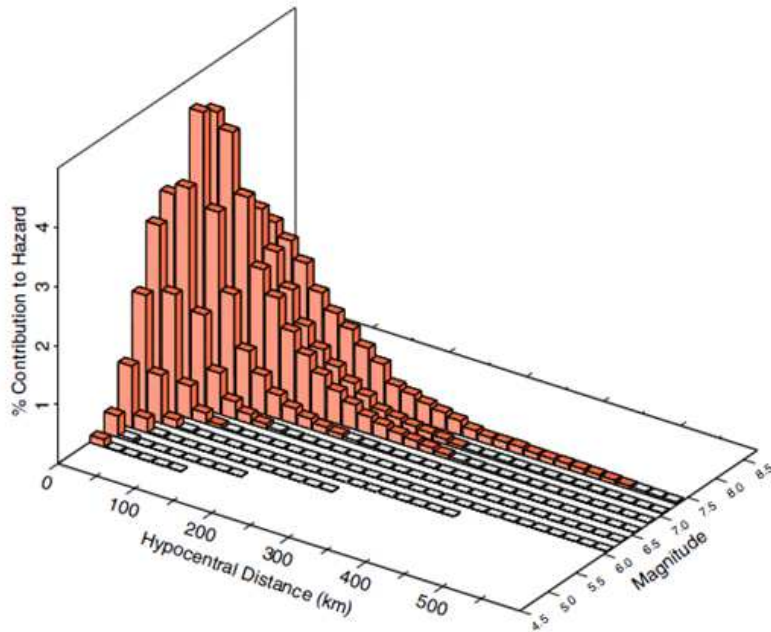


Figure 5 – Deaggregation of the seismic hazard (Regional model, Sa(2.0))

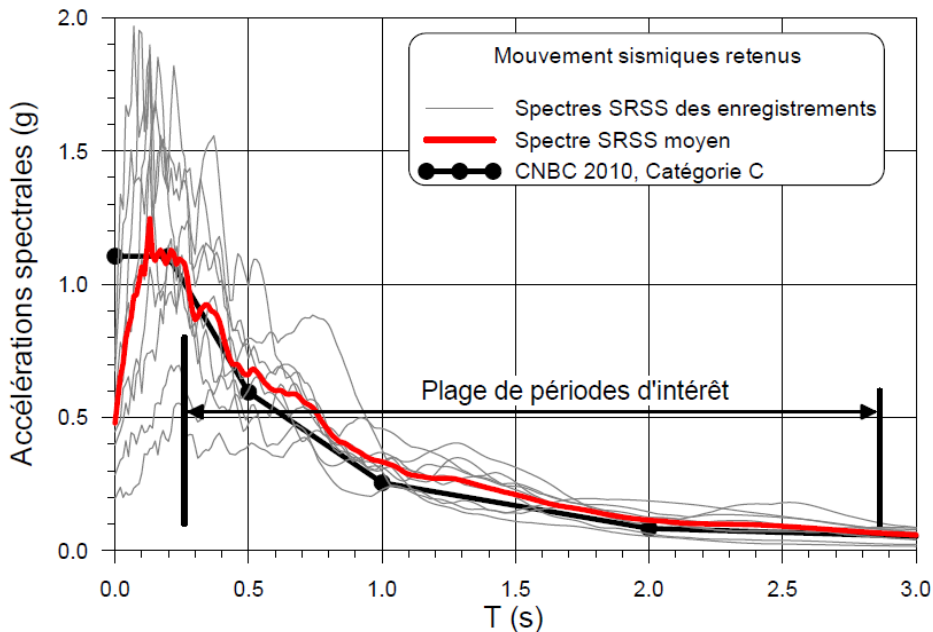


Figure 6 – Response spectrums of the ground motions used for the analyses

A series of analyses were completed to determine the optimal slip load values for each story in order to meet the performance criteria on the drift ratio while minimizing the forces in the columns and foundations. The selected dampers slip loads are 700 kN at the ground and 1st levels, 600 kN at the 2nd and 3rd levels, 500 kN at the 4th and 5th levels, and then 400 kN up to the higher level of the steel tower. The number of braces required per story is also reduced as the story number increases.

The use of friction dampers allowed for a retrofit solution offering good balance between the control of the seismic displacements (0,70% inter-story drifts) and the forces generated in the members by the seismic motions. The importance of controlling inter-story drift ratios needs to be emphasized here, as it allows

the moment-frames to remain essentially elastic, and therefore be able to provide the required centering capacity to the SFRS. The contribution of the existing moment-frames is key to the good behavior of the retrofitted SFRS. Also, keeping the stresses inside the resistance envelope of the columns and foundations was important, in order to minimize the construction work (structural reinforcements) inside the hospital.

The diagonal and chevron steel braces with in-line friction dampers were mainly disposed along the exterior walls in order to minimize the work inside the hospital. The seismic control of the transverse direction of wing B, and the first level of the steel tower however required that some braces be placed inside the hospital. On a total of 118 friction dampers required for the seismic control of the hospital, 30 were located inside the building.

The nonlinear time history analyses allowed for a comprehensive study of several key features of the implemented retrofit scheme, starting with quantifying the amount of dissipated energy (figure 7). For example, figure 8 shows the hysteresis loop of a 400 kN friction damper under one given earthquake motion. The area of this loop represents the amount of energy dissipated in one full cycle of movement. Nonlinear time history analyses also allowed for the assessment of the floor accelerations at different levels, and the global behavior of the retrofit scheme with respect to the ability of the existing moment frames to provide sufficient centering capacity for various dampers slip load values. In order to study the latter, the displacement history of joints located at various levels were drawn to make sure their permanent offset was reasonable (figure 9).

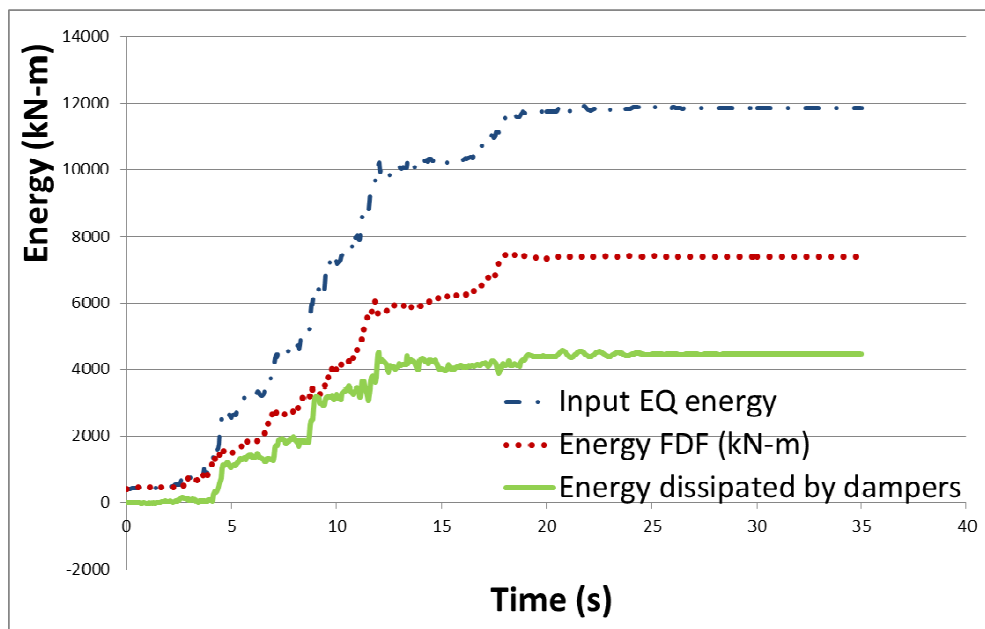


Figure 7 – Time histories of earthquake energy (input and dissipated) for one given ground motion

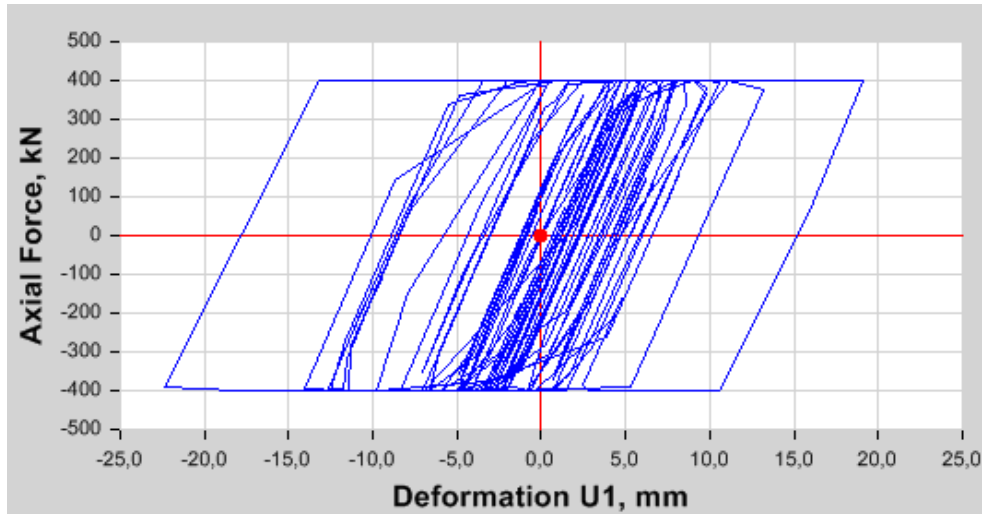


Figure 8 – Hysteresis loop of a 400 kN friction damper under one selected ground motion

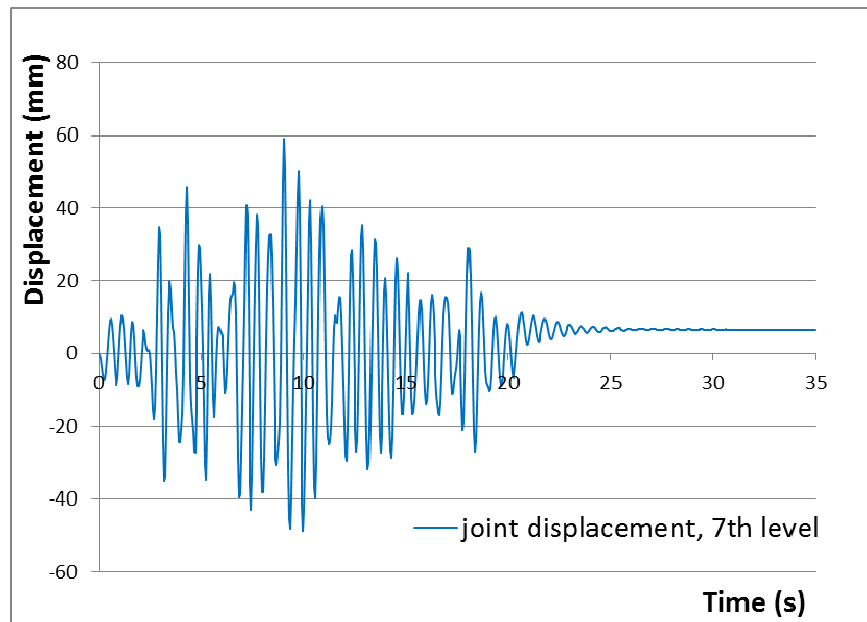


Figure 9 – Time histories of displacement at 7th level of the steel tower

The implemented retrofit solution had the following advantages over the considered alternatives:

- Limited structural reinforcements in the existing frame
- Limited quantity of rock anchors
- Limited disturbance of the hospital activities
- Good overall seismic behavior
- Limited floor acceleration

5.2. Design and modeling of the connections

The construction schedule would not allow for the design of the connections to be made by the contractor, as it would usually be the case. Because of that, and because the concrete frame had many specificities to take into account, the structural engineer designed and modeled all the 27 needed types of connections (figure 10).

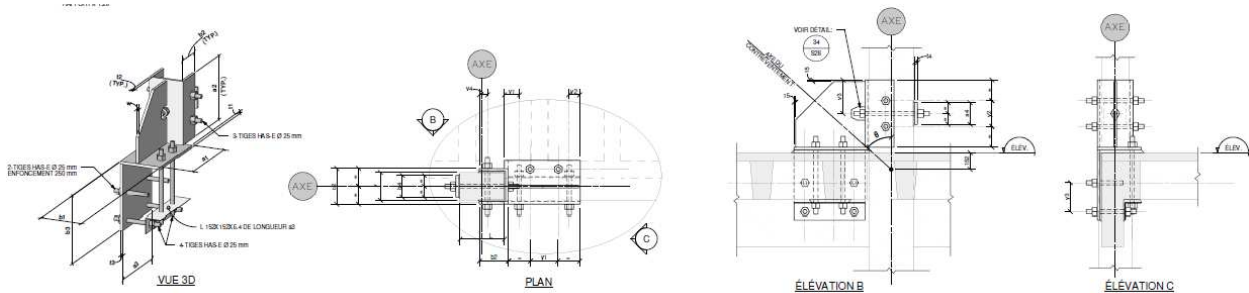


Figure 10 – One of 27 types of brace connection

5.3. On-site Construction Work

Working in an hospital that keeps on providing health services definitely comes with its challenges. Amongst those figures the users comfort, the infection prevention, and minimizing the disturbance on the general hospital activities. These challenges were accounted for by different measures put in place in collaboration with the CSSS experts, including keeping negative pressure in the work areas to make sure no dust or contaminated air would get out of the sealed workspaces (figures J and P).



Figure 11 – East side of wing B under retrofit work



Figure 12 – Installed friction damper

Some lessons were learnt from this project. Since no specialized equipment with severe vibrations tolerances were located close to the construction work, the structural engineer opted for connections with percussion-drilled concrete anchors in wing B of the hospital. However, it turned out that the percussion-drilling of some of these connections was too disturbing for the services given in some areas of the hospital. Therefore, the contractor had to work in the evening, outside the work schedule of those services. Also, the hospital being located on the top of a hill, the dominant N-W winds made it difficult for the contractor to maintain the negative pressure for some of the construction phases.

6. Acknowledgements

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7. References

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FEMA 355c State of the Art Report on Systems Performance of Steel Moment Frames Subject to Earthquake Ground Shaking

FEMA 355e State of the Art Report on Past Performance of Steel Moment-Frame Buildings in Earthquake

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