Seismic Rehabilitation of Les Jardins Westmount, Montreal (Quebec), Canada

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SUMMARY

The existing 4-storey structure that was formerly the Selby Campus of Dawson College in Montreal is being converted into a 7-storey residential complex. The existing building structure did not have any established seismic force resisting system, and derived its lateral stability from partial frame action and unreinforced masonry walls. This structure did not offer adequate resistance to seismic forces specified in the current building code and paraseismic upgrading was consequently required.

The introduction of supplemental damping provided by Pall friction dampers in steel bracing, was found to be the most effective and economical solution. Since a major portion of the energy was dissipated by Pall friction dampers, the seismic forces on the structure and storey drifts were significantly reduced. The lack of ductility was compensated by the mechanical energy dissipation. This paper discusses the design criteria, seismic analysis and results.

Keywords: Pall Friction Dampers, Seismic Retrofit, Time-History Analysis, Storey Drift.

1. INTRODUCTION

The former Selby Campus of Dawson College is located in the city of Westmount, adjacent to Montreal, on St. Antoine Street, Fig 1. The original building, built in 1926, was expanded in several phases over the years, Fig 2. All phases are made of reinforced concrete structure with historic brick façades. As the building was originally a pharmaceutical manufacturing facility, there are various concrete beams and a heavy-duty structure in place to support machinery. The existing structure did not have shear walls and derived its lateral stability from partial frame action in the core of the building. The preliminary analysis indicated that the existing structure was not adequate to resist the lateral seismic forces specified in the National Building Code of Canada 2005.

The new plan was to convert the buildings into a condominium complex (referred to as “Les Jardins Westmount”) by adding three new floors on top of the existing structure. A steel structure was necessary in order to minimize the additional weight to the building and avoid the need for column and foundation reinforcement. The new floor structures would therefore be steel frames with precast pretensioned concrete slabs. The existing building was a complex of several independent structures (construction phases) and in order to facilitate the seismic analysis and reduce the construction costs, it was assumed that all adjacent phases of the building would be structurally attached together so that all phases would behave as one building under seismic excitation. The existing structure needed a seismic retrofit taking into account the new floors on top of the building. Therefore, the seismic rehabilitation work was undertaken along with other renovations for public safety and to protect the new investment. Several rehabilitation techniques alternatives were reviewed and discussed with the client and the architect of the project, in order to find the most optimal retrofit solution.

Figure1. The existing building (left) and the proposed project (right)

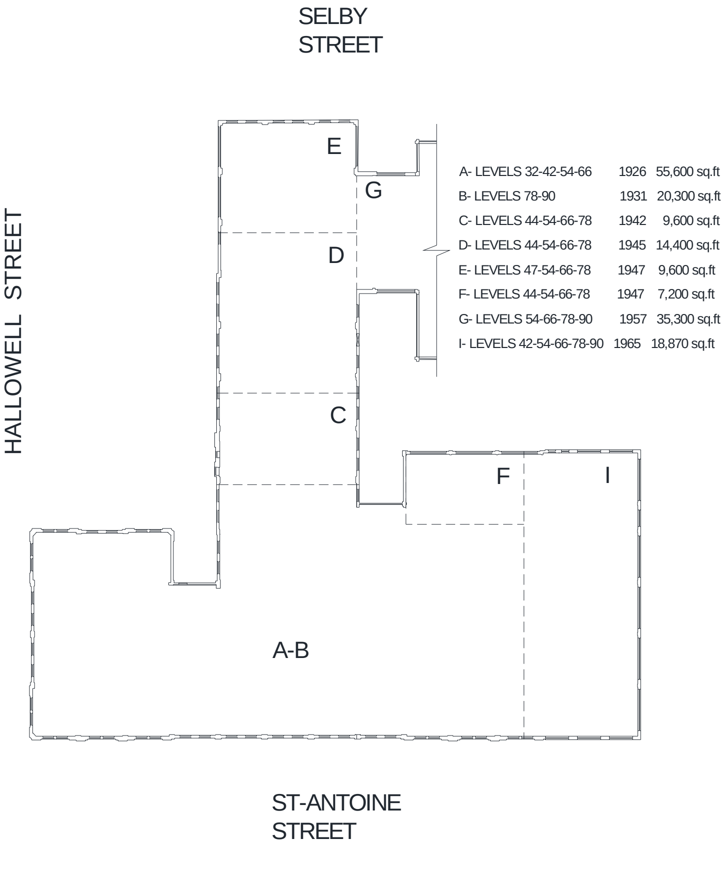

Figure2. Floor plan of the building’s different phases

2. SEISMIC UPGRADE STRATEGY

There were several alternatives for the seismic upgrade of the Jardins Westmount project. The first option - the adding of new concrete shear walls - seemed to be the right decision initially. However, these types of structure attract higher acceleration during a major earthquake, which results in greater force. Furthermore, the shear walls option required strengthening of the existing footings and constructing new seismic pads (very expensive and time-consuming to build). The second option involved adding new steel cross bracing to the existing structure. This option was considered to be unfeasible due to the limited energy dissipation of conventional bracing and massive seismic connections.

A third, more innovative, option – the addition of supplemental damping in conjunction with required stiffness – was found to be an optimal solution to the problem. This was achieved by incorporating Pall friction dampers on the single diagonal bracing installed in the existing building. During a major earthquake, the friction dampers start dissipating energy, considerably reducing the forces acting on the structure. Since the friction damped bracing need not be vertically continuous, it provides great flexibility in terms of architectural restrictions. The lateral seismic force from the bracing is transmitted by the concrete slab's rigid diaphragm to the foundation walls. The friction damped bracings do not carry any gravity load and do not need to go down to the foundation, as their role is merely to dissipate seismic energy and compensate for lack of ductility in the existing structure. Since the bracings are staggered in different locations throughout the existing structure, the overloading of columns and foundation was avoided such that their reinforcing was negligible. It was decided to use conventional bracings on the new floors on top of the building and single diagonal bracing with Pall friction dampers only in the existing building.

Pall friction dampers are very economical, effective, and reliable. They are very simple to install, saving a lot of time during construction. Made of a series of plates specially treated to produce friction, the plates are connected together with high strength bolts. These plates are designed to slip during major earthquakes before other structural members yield. As a result, the structure returns to its original position under the spring action of an elastic structure. Friction dampers have very large rectangular hysteretic loops and their performance is independent of temperature and velocity. Since there is nothing will yield or suffer damage after an earthquake, maintenance or replacement is not required.

3. SEISMIC ANALYSIS

The preliminary design followed the standards laid out in the National Building Code of Canada 2005. The minimum specified lateral seismic load is determined by the static equivalent method defined by the Equation. 3.1:

$$V = S(Ta)M_vI_EW / (R_dR_o) \quad (3.1)$$

where W is the seismic weight of the building, R_d & R_o are ductility and overstrength factors of the structural system and I_E is the importance factor. The value of S is determined from the uniform hazard spectrum for Montreal given in the National Building Code of Canada 2005. The response spectrum method was utilized to design the new floors as a tension-compression cross bracing system. Using formulas provided in the Code, the fundamental period of the building was calculated. This period was then compared with a period obtained from dynamic analysis. The R_d and R_o values chosen were R_d = 2.0, R_o=1.3 for the tension-compression bracing and R_d =4.0, R_o=1.5 for the existing structure, taking into account the energy dissipation capacity of the friction dampers.

Linear and non-linear dynamic analyses were performed to evaluate and estimate the building's behaviour. A three-dimensional model (shown in Fig 3) was built using the ETABS (non-linear version) computer program (Computer and structures Inc). The building was then analyzed using response spectrum and non-linear time-history seismic loading in order to determine the structural response. Since the rectangular hysteretic loops of the friction damper are identical to the rectangular loops of an ideal elasto-plastic material, the slip load of the dampers was considered a fictitious yield forces. Various series of analyses were performed in order to estimate the optimal slip loads. In fact, for the best response, the slip load was determined to be 450kN for the first three floors and 350kN for the fourth. Different time-history records for the Montreal region were used in the analysis to evaluate the structural response in two directions. A rigid diaphragm was assumed for each building floor, as well as a 5% percent viscous damping in order to account for non-structural elements.

Adding the Pall friction dampers to the structure allowed the ductility and stiffness to be increased simultaneously. Two parameters were investigated so as to ensure that the structural behaviour corresponds to Code requirements: 1. a storey drift of <0.02 of the height of each storey; 2. factored forces in the structural members smaller than the member resistance. In order to optimize the amount of bracing with dampers throughout the building, the distribution of stiffness in the existing building was first evaluated. And, in order to optimize the structure's performance and avoid the soft storey mechanism, the stiffness of each storey should be greater than that of the adjacent storey above. Using the method proposed by Paulay and Priestley (1992), the storey stiffness for the existing and retrofitted structure was evaluated in two principal directions. Fig 4 shows the normalized values of storey stiffness prior to and after adding the bracing. It can be observed that the 4^{th} floor in the building prior to the retrofit is subject to the soft storey mechanism.

The following pattern was proposed with respect to all aspects of the retrofit: adding 8 braces with a slip load of 450kN to the ground floor, 14 braces with a slip load of 450kN to the second floor, 12 braces with a slip load of 450kN to the third floor and 10 braces with a slip load of 350kN to the fourth floor. For the new three floors on top, equal amounts of 10 tension-compression cross-bracings were considered.

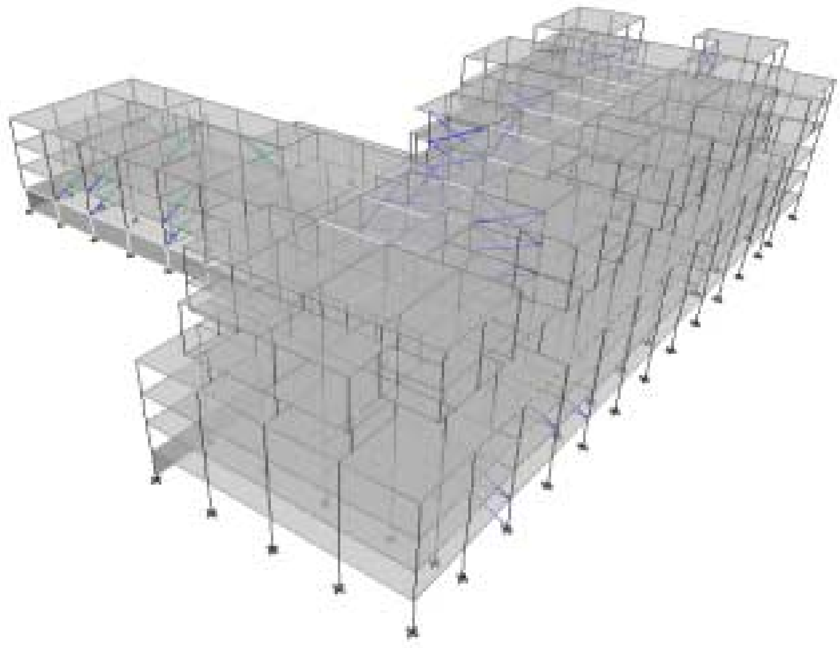

Figure3. Three-dimensional analytical models for Les Jardins Westmount project

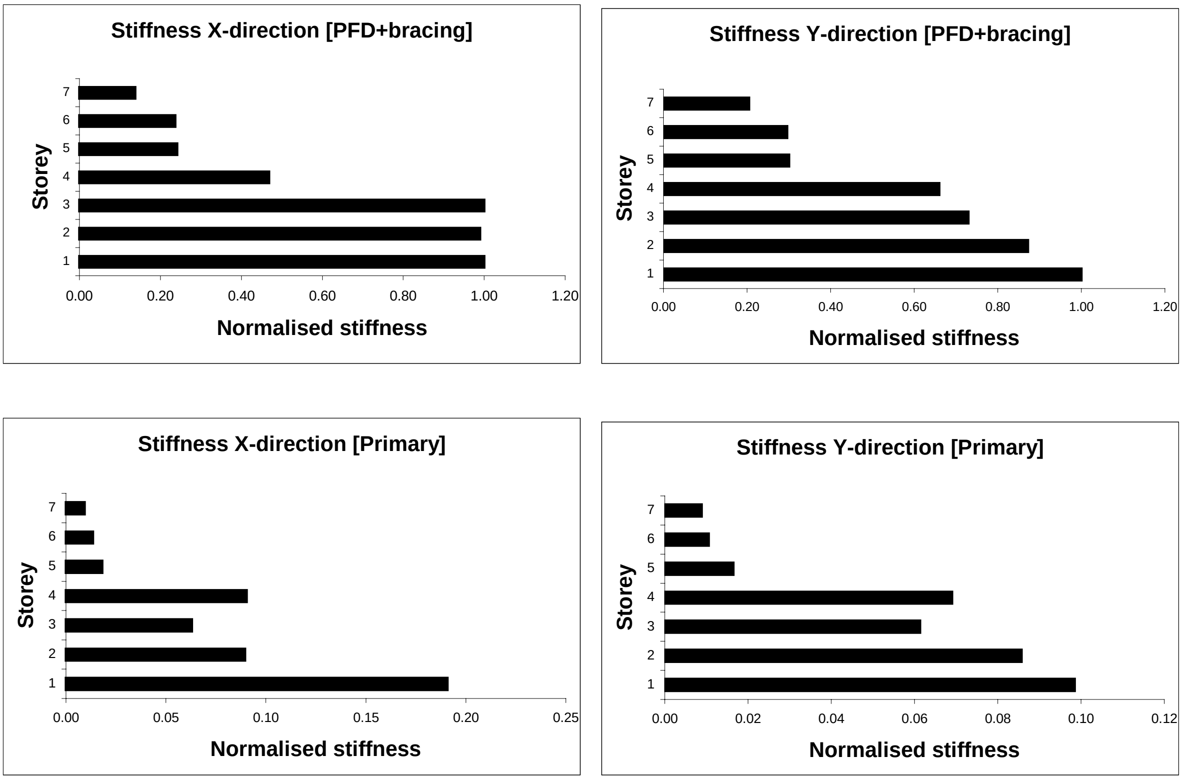

Figure4. Normalized storey stiffness over the structure height for the primary and retrofitted building

4. SEISMIC RESPONSE

Three different three-dimensional computer models were constructed for this project in order to understand and evaluate the precise building response: a primary building model (the building as is, without any particular seismic resisting system); a braced building model (where all the proposed braces were added to the model but without Pall friction dampers); and a damped building model (which is the retrofitted building with Pall friction dampers). The storey drifts were calculated for the three models under 8 scaled ground motions. The building was found to be more flexible in the Y direction and therefore subject to significant storey-drift. Fig 5 shows the global storey drifts for the three models. It can be seen that adding supplemental stiffness to the building using only a rigid steel frame can worsen the structural response in terms of storey drift. However, by adding Pall friction dampers to the braces, the storey drift was dramatically reduced. As is shown in Fig 5, the storey drifts for the retrofitted structure meet the Code requirement (less than 2%). The period of the braced building was found to be T_x =1.07 and T_y =1.26 however, it is found that the period of the damped structure shifts down on the acceleration spectrum curve to T_x =0.97 and T_y =1.05.

Once the axial forces in the brace reach the defined slip loads in the braces, the slipping of the damper begins. The friction dampers follow the complete hysteretic cycles and dissipate the seismic energy. An example of the dampers hysteretic loops with a perfect rectangular shape and a slip load of 450kN is shown in Fig 6.

In order to track the lateral roof displacement of the retrofitted building, a target point was chosen which was susceptible to maximum displacement. Using the ETABS non-linear program, a comparison was made between lateral roof displacements of damped and primary structure (shown in Fig 7 & 8). It is obvious from the graph that adding staggered braces with friction devices to the existing structure does not change the oscillation pattern of the primary structure and the lateral displacements are reduced.

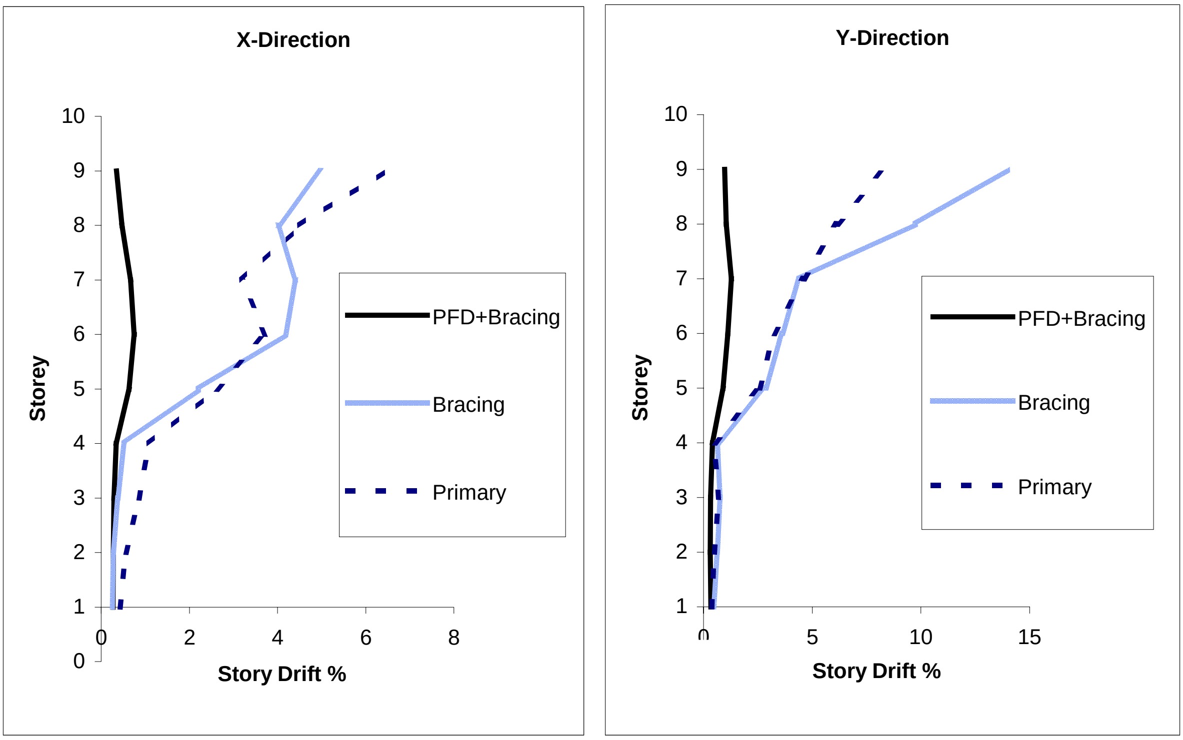

Figure5. Inter-storey drift deflection for primary, un-damped(stiffed) and damped(stiffed and damped) building.

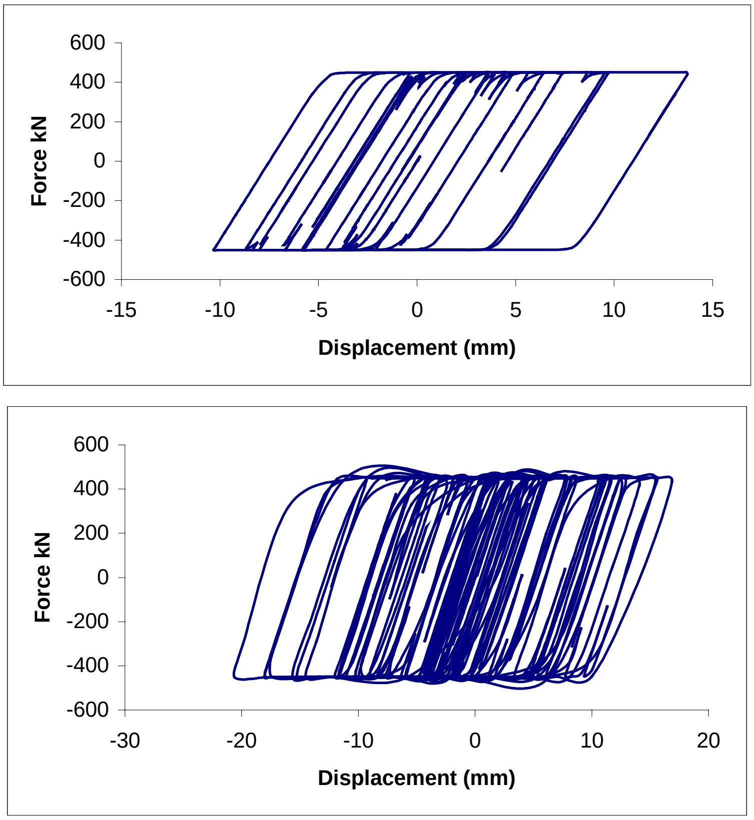

Figure6. Hysteretic loop of a 450kN slip load friction damper in a diagonal brace

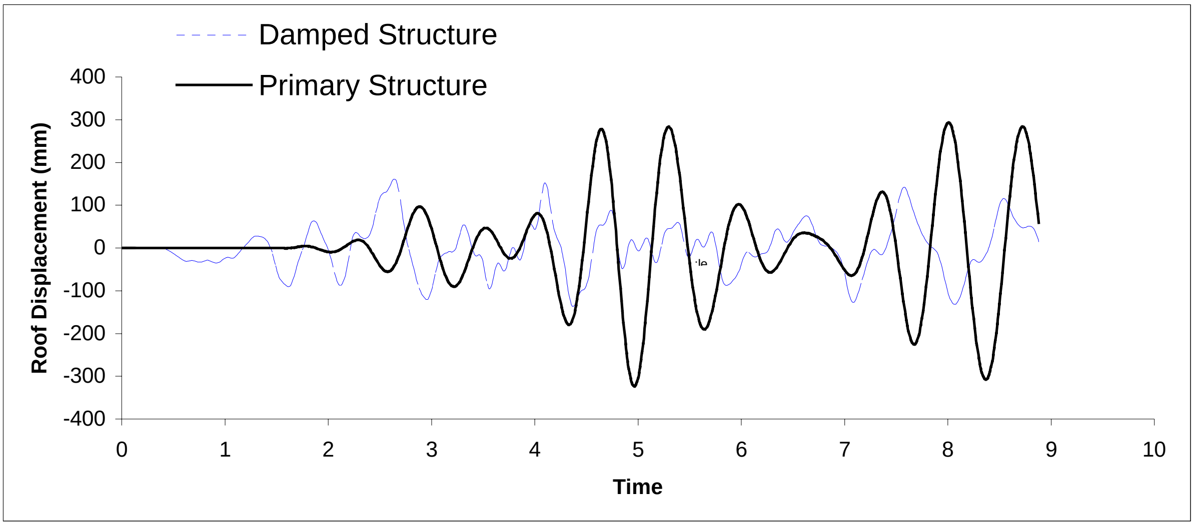

Figure7. Roof displacements of the primary and damped structure versus time in Y direction

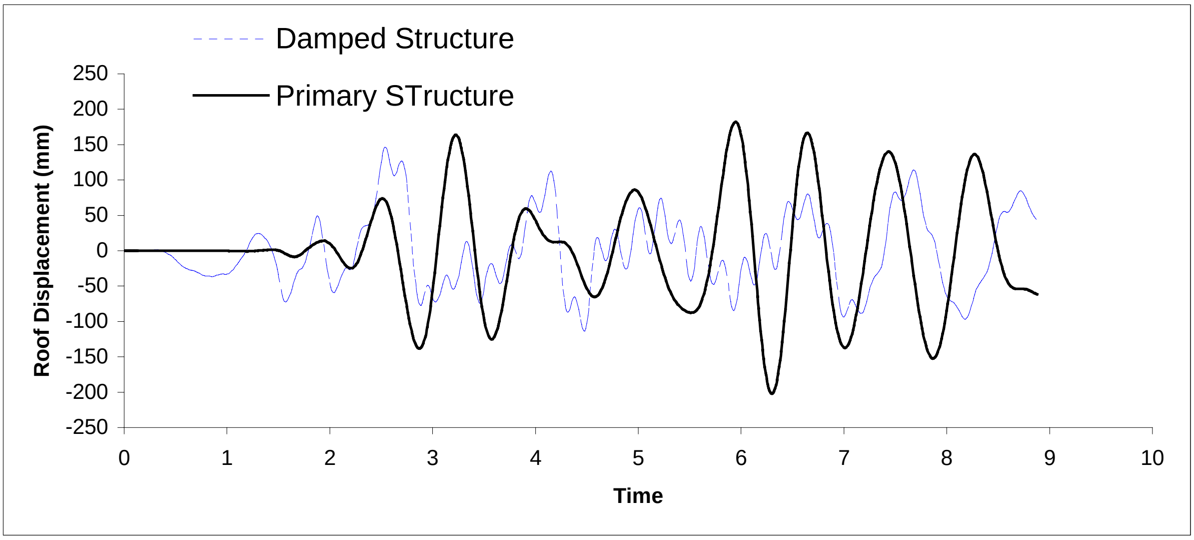

Figure8. Roof displacements of the primary and damped structure versus time in X direction

5. CONCLUSION

Pall friction dampers were proposed for the "Les Jardins Westmount" in order to upgrade the existing structure for seismic loading in light of standards set out in the National Building Code of Canada 2005. Several linear and non-linear time-history analyses were used in the design process in order to optimise the amount of supplemental stiffness and values of the dampers slip loads. After considering all options the adopted strategy was to find the best structural response in terms of story drifts and thus minimize elastic member response. Pall friction dampers were the most economical solution in terms of construction, since they could be easily installed and required no repair or replacement after an earthquake. Their ability to dissipate energy minimized the need to strengthen columns and foundation. The construction has since started and likely to be completed in 2013.

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