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SEISMIC RETROFIT OF A TEN-STOREY CONDOMINIUM BUILDING WITH PALL FRICTION DAMPERS

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ABSTRACT

For the conversion of the Ontario Building, a 10-storey industrial building located in downtown Montreal, into a condominium, important modifications were required. In this regard, the building had to undergo a reevaluation of the structural system in conformity with Quebec Construction code and National Building Code of Canada. The skeleton of the existing structure is a non-ductile steel moment frame with concrete slabs, completed in 1929 and designed to carry gravity and wind loads only. The proposed solution for seismic upgrading was to incorporate Pall friction dampers with one-way diagonal braces connected to existing frame members, such that the overall stiffness and damping of the retrofitted system was largely improved. Several linear and non-linear time-history dynamic analyses were performed. It was found that staggering Pall friction-type dampers in single diagonal bracing at different stories was sufficient to meet code requirements in terms of roof displacement, storey drift and base shear. Due to high damping provided by the Pall friction dampers, strengthening of column and foundations was avoided. This innovative solution responds well in terms of cost efficiency, design flexibility and speedy construction.

Introduction

Over the past decade, the use of passive energy dissipation devices (PEDD) has become an attractive option for seismic design and retrofit of structures. Following the Northridge and Kobe earthquakes, PEDD has been accepted as a viable alternative for seismic retrofit in the U.S and Japan. Among different types of damping devices, Pall friction-type dampers have been used in several buildings in Canada and U.S. (Pall, 1991, 2004; Vezina, 1992; Pasquin, 2002). The aim of adding energy dissipation devices to an existing structural system is to concentrate the hysteretic behaviour in specially designed and detailed zones, and to insure that the forces acting on the primary gravity load resisting system are maintained in the elastic range. Many researchers have focused on experimental testing of Pall friction dampers and several simplified force-displacement models have been developed (Filiatrault and Cherry 1987), (Aiken and Kelly 1990), (Pall and Pall 1993). Furthermore, Aiken et al (1993) have conducted shake table experiments for retrofitting a 6-storey steel frame using moment resisting, concentrically-braced and eccentrically braced configurations.

Pall friction dampers were included in the model as part of a chevron bracing system. It was found that by adding braces, the stiffness of the structure increased, inelastic behaviour in primary structural members

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was avoided and energy was instead dissipated in the friction devices. The optimal slip load was found through a series of non-linear time-history analyses.

Energy dissipation mechanisms based on friction of steel plates are easy to manufacture and to install. In order to avoid the formation of a soft storey mechanism, dampers should be arranged in such a way that structural integrity and strength remain uninterrupted. Furthermore, the slip forces of the friction dampers and their layout on the building should insure the uniform distribution of storey drift over the height of the structure. After an earthquake, the building should return to its initial equilibrium position under the spring action of elastic forces.

However, it is noted that current NBCC does not include any comprehensive design procedures for designing structures with a damping system. The quasi-static design procedure given in NBCC is ductility based and does not explicitly apply to friction-damped buildings. However, structural Commentary J of the NBCC permits the use of friction dampers for seismic control of buildings. FEMA 356 (Prestandard for the Seismic Rehabilitation of Buildings) provides the most comprehensive specifications and has been adopted for the seismic retrofit of this building.

Earthquake Response of Existing Building

Building Description

The building, as shown in Fig. 1, is located at the intersection of Ontario Street and St-Laurent Boulevard in Montreal. The existing structure, fabricated and erected by Dominion Bridge Limited Company, has one basement level and 10 stories above ground. The skeleton of the existing building is a non-ductile steel moment frame with concrete slabs, completed in 1929 and designed to carry gravity and wind loads only. Originally, the structure served as an industrial building for garment manufacturing. Its floor plan is approximately square in shape and measures 18m by 18m (see Fig. 2). The typical story height is 3.4m, except for the ground floor which is 5.3m, while the typical storey bay is 6.0m in both directions. It is of interest to note that all existing structural drawings were available for this project. The beam and columns members are typical W sections. For fireproofing, beam and columns are covered in concrete. The building has beautiful brick facades, adorned with cast-iron prefabricated wall panels, details that are typical for buildings of this generation.

Figure 1. Building facades.

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The existing beam-column connections were riveted stiffened seat angles that were widely used as rigid joints in steel frame of this period (Chessman and Bruneau 2000). The floor structure consisted of poured in place reinforced concrete joists that were monolithic with the floor slab and also the fire protecting concrete cover over the existing steel frame. This configuration was considered to be a rigid diaphragm.

Because the steel beams supporting the floor joists were significantly stiffer than the beams in the opposite direction and those beams coincided with the orientation of the strong axis of the columns, the existing structure was stiffer in the East-West direction (the y-direction) than in the x-direction.

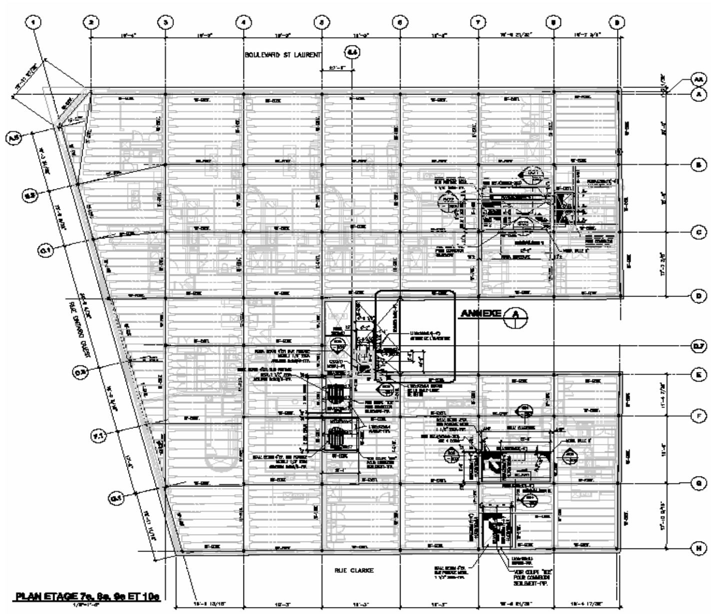


Figure 2. Typical plan view.

For the conversion of the original industrial building to a condominium, a major architectural intervention was required including: demolishing of interior brick and terra-cotta masonry partition walls (especially at ground floor level), providing new openings for windows located on grid line 9 (rear facade), and enlarging the existing elevator shaft. Furthermore, the developer wished to incorporate a recreation space on the roof of the existing building including a swimming pool, garden, and terrace.

Seismic Loads

The design was performed according to the upcoming 2005 National Building Code of Canada. The minimum lateral seismic force, V, is given by the formula: $V = S(T_a)M_vI_EW/(R_dR_o)$, where S is the design spectral response acceleration, M_v is the higher mode factor, I_E is the importance factor, W is the seismic weight of the structure, and R_d, R_o are force modifications factors that are related to the ductility and the overstrength of the structural system. The value of S is obtained from the 2 percent in 50 years Uniform Hazard Spectrum for Montreal area and is dependent on T_a, the fundamental period of the structure, and the soil class. In general, for a period T, where T < 0.2s, the ordinate of acceleration spectra is $F_aS_a(0.2)$, for T= 0.5s, the smaller of $F_vS_a(0.5)$ and $F_aS_a(0.2)$, for T= 1.0s, $F_vS_a(1.0)$, and for T= 2.0s, $F_vS_a(2.0)$. For T longer than 2.0s, a minimum seismic load determined with S at T = 2.0s must be considered. In these expressions, Fa and Fv are the acceleration-based and velocity-based site coefficients, respectively. For Montreal, S_a values at periods T=0.2s, 0.5s, 1.0s, and 2.0s are equal to 0.69g, 0.34g, 0.14g and 0.048g. Firm ground conditions (Site Class C) with F_a =1.0 and F_v=1.0 was considered. For design purposes, the period of the steel moment resisting frame is taken as $1.5T_a$, where $T_a=0.085(h_n)^{3/4}$ and h_n is the building height. By applying the aforementioned formula, the fundamental period of the building is $T=1.5T_a$ =1.85s. However, the fundamental period of the building, computed from free vibration analysis and considering no moment resistant connections is quite longer than that for similar newly design structures, and largely exceeds the code value (T=1.85s). The seismic weight included the 25% of the roof snow load is 120,360KN. For non-ductile steel structure, the values of R_d and R_o are: R_d =1.0; R_0=1.0. Therefore, based on the static equivalent method, the lateral seismic force would be V=8060KN, which is slightly higher that the required $V_{min} = S(2.0)M_vI_EW/(R_dR_o)$.

Linear and non-linear dynamic analyses were performed to evaluate the building response, and the structure was subjected to eight simulated ground motion time histories typical for east-cost source zones matching the two dominant magnitudes -hypocentral distance scenarios for the Montreal area: M_w6 at 30Km and M_w7 at 70Km (Tremblay and Atkinson 2001). All ground motions were scaled to match the design spectra – 2% in 50 year probability, for Montreal. In this context, the proposed scale factor applied for simulated ground motions R1 to R4 is 0.85 and for R5 to R8 is 0.9, as is shown in Table 1.

Table 1. Characteristics of two sets of scaled ground motions for Montreal.

No.	Time history	Scale factor	PHGA	Scale factor Sa(T=2.3s)	Scale factor Sa(T=3.2s)	Scale factor Sa(T=3.6s)
R1	Simulated Trial #1 (Mw 6-30Km)	0.85	0.43g	1.28	2.40	2.90
R2	Simulated Trial #2 (Mw 6-30Km)	0.85	0.52g	1.25	1.51	1.78
R3	Simulated Trial #3 (Mw 6-30Km)	0.85	0.47g	1.20	1.53	1.82
R4	Simulated Trial #4 (Mw 6-30Km)	0.85	0.44g	1.70	1.18	2.80
R5	Simulated Trial #1 (Mw 7-70Km)	0.90	0.30g	1.04	1.23	1.45
R6	Simulated Trial #2 (Mw 7-70Km)	0.90	0.29g	1.53	1.33	1.41
R7	Simulated Trial #3 (Mw 7-70Km)	0.90	0.34g	1.00	1.70	1.45
R8	Simulated Trial #4 (Mw 7-70Km)	0.90	0.29g	1.00	1.80	1.40

Records R1 to R4 have duration of 6.3s, while records R5 to R8 have duration of 20.1s. Spectra for two sets of uniform hazard spectrum (UHS) – compatible records for Montreal: R1 to R4; R5 to R8; and the design spectrum are shown in Fig. 3. Table 1 also gives the peak horizontal ground acceleration (PHGA) for all ground motions and the adjusted scale factor to match the design spectrum over the following period ranges: T= 2.3s; T= 3.2s and T= 3.6s.

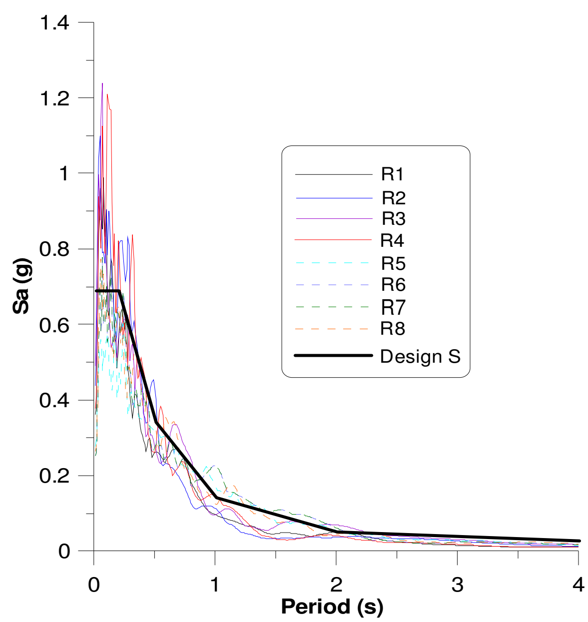


Figure 3. Absolute acceleration response spectra of two sets of scaled ground motions for Montreal.

Structural Response

A three-dimensional dynamic analysis was used to determine the distribution of seismic forces within the structure. Consequently, the envelope of the lateral force distribution over the height of the structure is often significantly different than that obtained from an equivalent static load method. To evaluate the structural response of the existing building, ETABS (Nonlinear version) computer program (Computer and Structures Inc.) was employed and three-dimensional dynamic analyses using response spectrum and linear time-history seismic loading were performed. Rigid diaphragm behaviour was assumed at every floor. For this project, a 2 percent modal damping was considered for linear time-history analysis. For each principal direction of primary structure, the period of the first and the second mode is: T_{1x}=3.6s; T_{2x}=1.7s; T_{1Y}=2.9s, and T_{2Y}=1.4s. Therefore, especially in the x direction, the building was generally found to be flexible and prone to large storey-drift. In spite of its flexibility, the seismic base shear response is reduced by 30% from the NBCC requirements for regular structure (Vd = 0.8V) and by 10% from the Quebec Building Code requirements for existing structures. It is mentioned that this structure could match FEMA 356 previsions if, 35% damping would be added to the building. To estimate the value of building lateral deflection, the 2005 code allows the results obtained from a dynamic analysis to be normalized, so that the maximum base shear may be scaled to match the code requirement design base shear (Vd = 0.8V). In this respect, the largest inter-storey drift deflection, 3.3%hs, occurred in x direction at the 8^{th} floor when building was subjected to the ground motions R6 and R7. In the y direction, the largest inter-storey drift occurred at the 8^{th} floor under the ground motion R7. As shown in Fig. 4, different ground motions records, at equal intensity, have different profiles over the building height. The response of the building under the ground motions R2, R5, R6, R7 and R8 were retained for further analysis. In general the dominant contributor to long-period ground motion hazard comes from larger earthquakes at greater distance from the source.

Based on largest storey-drift responses that resulted from the analysis, the seismic upgrading of the building was indeed justified.

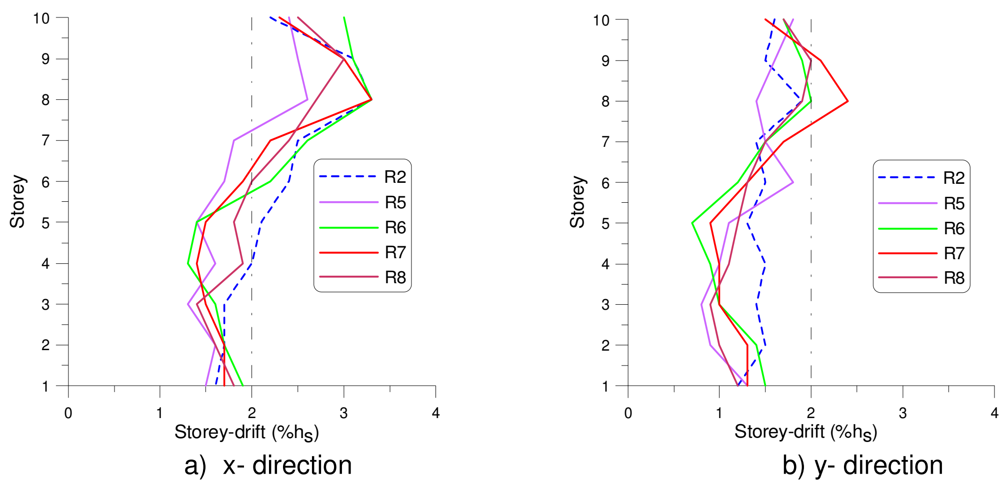


Figure 4. Inter-storey deflection of the primary building under different ground motions

Seismic Upgrade

It is well known that adding stiffness to a building will reduce storey-drift and structural damage. Conventional methods of stiffening consist of adding rigid steel braces, steel plate shear walls or even concrete shear walls. These elements are typically vertically continuous over the building height and as a consequence, reinforcing of columns and foundations located in the braced bays is often necessary. The limited budget and compressed construction schedule made these conventional options unfeasible. In contrast with classical braced system, the friction-damped bracing need not be vertically continuous. The damped bracings do not need to go down through the basement to the foundation. At the ground floor level, the lateral shear from the bracing is transferred through the rigid floor diaphragm to the perimeter retaining walls of the basement. These aspects were particularly appealing to the project architects as they offered great flexibility in space planning. The principle of adding friction dampers to the building frame is to reduce lateral displacements and to move energy dissipation out of gravity frame.

Energy Equation in a Damped System

The seismic input energy is a constant quantity and is mainly dependent on the total mass of the building and the fundamental natural period. Design parameters such as strength, mass, stiffness and the slip load of friction dampers influence the input energy distribution over the height of the structure. A general shape of the energy input (energy spectrum) shows that a damped system is not highly dependent on the period, which is not the case in a purely elastic system (Akiyama, 2001). Pall friction dampers are not dependent on excitation frequency and hence the phenomenon of resonance is avoided. The equation of energy balance in a damped system is:

$$E_I = E_K + E_S + E_\xi + E_H \tag{1}$$

Where E_I is the earthquake input energy, E_K is the kinetic energy, E_S is the strain energy; E_ξ is the viscous damping energy, and E_H is the hysteretic damping energy.

Pall friction dampers consist of series of steel plates, which are specially treated to develop controlled friction. They possess large rectangular hysteresis loops with negligible fade. In a typical un-damped structure, the inherent damping is merely 2-5% of critical, while adding dampers the hysteretic damping energy could increase the inherent damping to as much as 70% of critical. For non-linear time-history analysis, a 2 percent modal damping applied in the first two modes of vibration.

Therefore, the role of a passive energy dissipater is to increase the hysteretic damping in the structure so that for a given energy input $E_{I,}$ the elastic strain energy is minimized and also the forces acting on the gravity structure are considerably reduced. This means that the structure will undergo reduced deformations for a given level of energy input.

Damped System Design

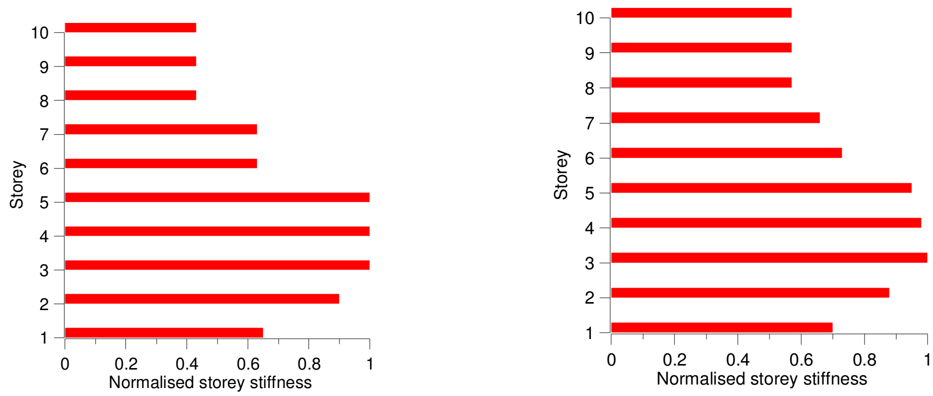
By adding Pall friction-damped single diagonal braces to the structure, the building stiffness and damping increase simultaneously and the imposed target control parameters would be achieved.

The following target control parameters were used in agreement with code requirements: i) storey-drift displacement less than 0.75%hs and ii) factored forces developed in structure members to be smaller then member resistance. In order to optimize the amount of supplemental stiffness added to the structure, the distribution of the existing stiffness over the structure height was evaluated. For best performance, the stiffness of the lowest storey should exceed that of the storey immediately above. Even if the structure is stable, but the stiffness in one storey is much lower than the storey above, as happens at ground floor level, then at this location, the formation of a soft storey mechanism is inevitable. However, the storey stiffness is not well defined. Paulay and Priestley (1992) proposed the calculation of the storey stiffness based on the following procedure. At the centre of the mass belonged to the roof diaphragm, an arbitrary horizontal force is applied and then, the storey stiffness is calculated by dividing the resulted storey shear force to the storey drift. Fig. 5 shows the normalized value of storey stiffness in both principal directions "x" and "y", for the building prior to retrofit. It can be seen that, the 3rd, 4th, and 5th floors are the stiff floors, while ground floor and the 2nd floors are prone to soft storey mechanism.

The number and location of diagonal braces added to the structure (un-damped stiffened system) should meet the following three criteria:

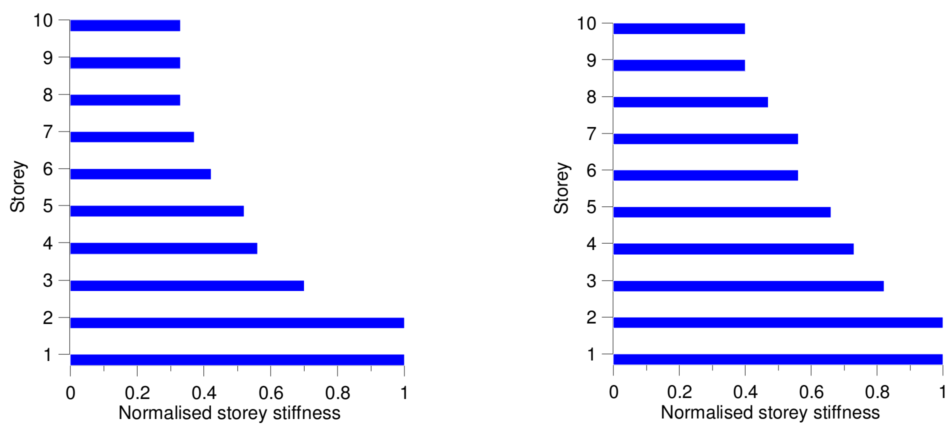
i) the stiffness of the each lowest storey should exceed that of the storey immediately above;
ii) the base shear for the stiffened structure (considering braces behave elastically) should equal the require code base shear, Vd (Vd =0.8V), for the building prior to retrofit;
iii) maximum storey drift displacement should be less than 0.02hs.

In this respect, the following pattern was proposed: 6 braces at the 10th and 9th floor, 8 braces at the 8th to the 3rd floor, 10 braces at the second floor and 12 braces at ground floor. The new distribution of storey stiffness (gravity frame + braces stiffness), in both directions, as is shown in Fig. 6, was able to avoid the soft storey mechanism formation and to reduce building lateral displacement. In general, the distribution of story drift demands over the height of the structure is a function of the roof drift. This relationship is strongly dependant on the ground motion type and structure characteristics. However, flexible structures and those with significant higher mode effects exhibit larger storey-drift displacements for the upper stories (Tirca et all 2003). Adding stiffness to the existing building, the fundamental period is reduced to T_{1X}=2.3s and T_{1y}=1.9s, the maximum storey drift shift less than $0.02h_s$, while the forces in the frame members increase.



a) x-direction b) y-direction

Figure 5. Normalised storey stiffness over the structure height for the building prior to retrofit.

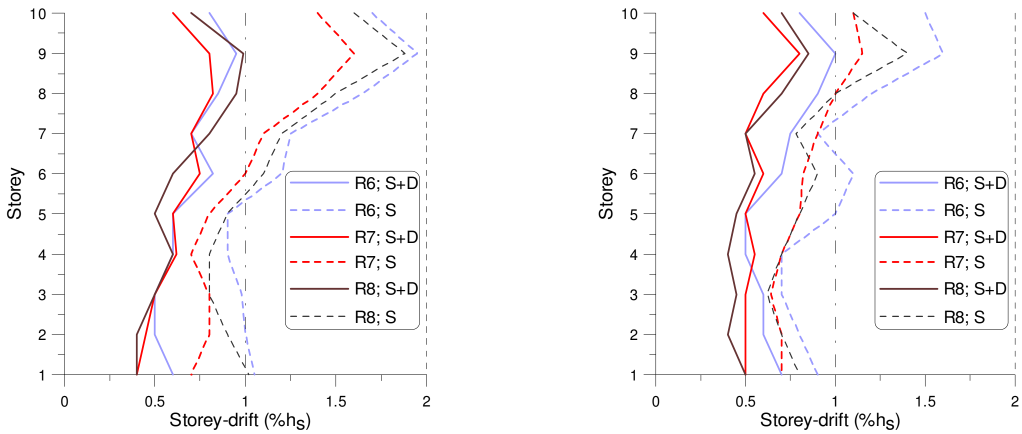


a) x- direction b) y- direction

Figure 6. Normalised storey stiffness over the structure height for retrofitted building.

From the linear time-history analyses of the un-damped stiffened structure, it was found that the maximum value of interstorey drift is less than 2%h_s and occurs at the 9th floor under R6, R7 and R8. The interstorey drift envelope is shown by a dashed line in Fig.7. The elastic axial force in the braces of the un-damped structure exceeded approximately two times their un-damped capacity. Following Rao et all (1996), the preliminary horizontal component of the design slip force at the base of the building was estimated by dividing the base shear of the un-damped stiffened structure resulting from a spectral analyses by a ductility factor of R_dR_o =2 (similar to FEMA 356 damping modification factor B1=1.8), then subtracting the shear of the pure primary structure (frame actions) and divided the obtained value by the number of braces, n, in one direction (for instance n = 6) and projecting the force by the angle Θ (where Θ is the angle between the brace and the horizontal.) The slip force value was thusly estimated as being 500KN at ground floor level and then gradually decreased to 360KN at the 10th floor. It is recommended that the damper slip forces vary gradually up the height of the building in the same manner as the storey shear. Once the slip force was found, braces were verified to carry an axial tension compression force equal to 130% design slip load, in agreement with FEMA 356, issued in 2000. Connections and adjacent gravity frame members (columns and floor diaphragms) are verified to 130% design slip force, too.

FEMA 356 requires the designer to consider a minimum of three accelerograms and to design for the maximum response. If the analysis is carried out for more than 7 accelerograms, than an average value in term of deflection and forces may be used. For a brief presentation, in this paper, the maximum response is shown for all considered ground motions.



a) x- direction   b) y- direction

Figure 7. Inter-storey drift deflection under ground motions: R6, R7,R8 for un-damped (stiffened) and damped (stiffened + damped) building.

The single diagonal tension/compression brace with friction damper was modeled as a damped brace using ETABS non-linear link element. It is represented by a bilinear perfectly rectangular hysteretic loop, similar to perfectly elasto-plastic material, and having the yield force equal to slip load. During seismic event, friction damper devices follow the complete hysteretic cycles and dissipate an amount of energy equal to the area of each cycle multiplied by the number of cycles. Following equation (1), the term represented the hysteretic damping energy increases, while strain energy diminishes. Otherwise, the axial force in the brace increases until the slip force is reached and at this point the inelastic behaviour begins. From analysis it was found that the period of the damped structure shift dawn on the acceleration spectrum curve to T_{1X}=3.2s and T_{1Y}=2.6s in comparison with the un-damped structure.

Several time-history “non-linear” analyses were performed to confirm that the gravity frame of retrofitted structure behaves elastically and that drift limits are met. In this respect, the retrofitted building shows a nearly uniform storey drift distribution over the height of the structure, with a peak storey drift value less than 0.01h_s at the 9th floor and 0.0065h_s at ground floor. Interstorey drift envelope for the damped structure is shown by a solid line in Fig.7. Because there were more than 7 accelerograms in this analysis, an average value of storey drift may be used. For instance, for the 9th floor the average storey drift is 0.0071$h_{s.}$ This value of storey-drift was chosen as the drift performance target in order to reduce the damage to the building facades. Researcher suggests that masonry strength deteriorates at approximately 0.01h_s. A comparison between the lateral roof displacement of the damped and primary structure is depicted in Fig.8 a). It is clearly shown that adding damped staggered braces to existing structure do not change the lateral oscillation pattern of the primary structure. However, the roof displacement history depends on the frequency content of the ground motions. Fig.8 c) shows the perfectly rectangular hysteretic loop of a 500KN slip load damped single diagonal brace.

Furthermore, since the building period did not shift much (from 3.6s to 3.2s in x direction), while the maximum storey drift reduced from 3.3%h_s to an average drift of 0.71%h_s, it resulted in reduced floor accelerations and thus enhanced damage control of non structural components.

As required in Chapter 9 of FEMA 356, two prototypes and production control testing on each damper was performed by Pall Dynamics Ltd., to ensure their performances. A quality control program concerning dampers installation was implemented. The damper installation was completed in December of 2004. A typical installed friction damper is shown in Fig.9.

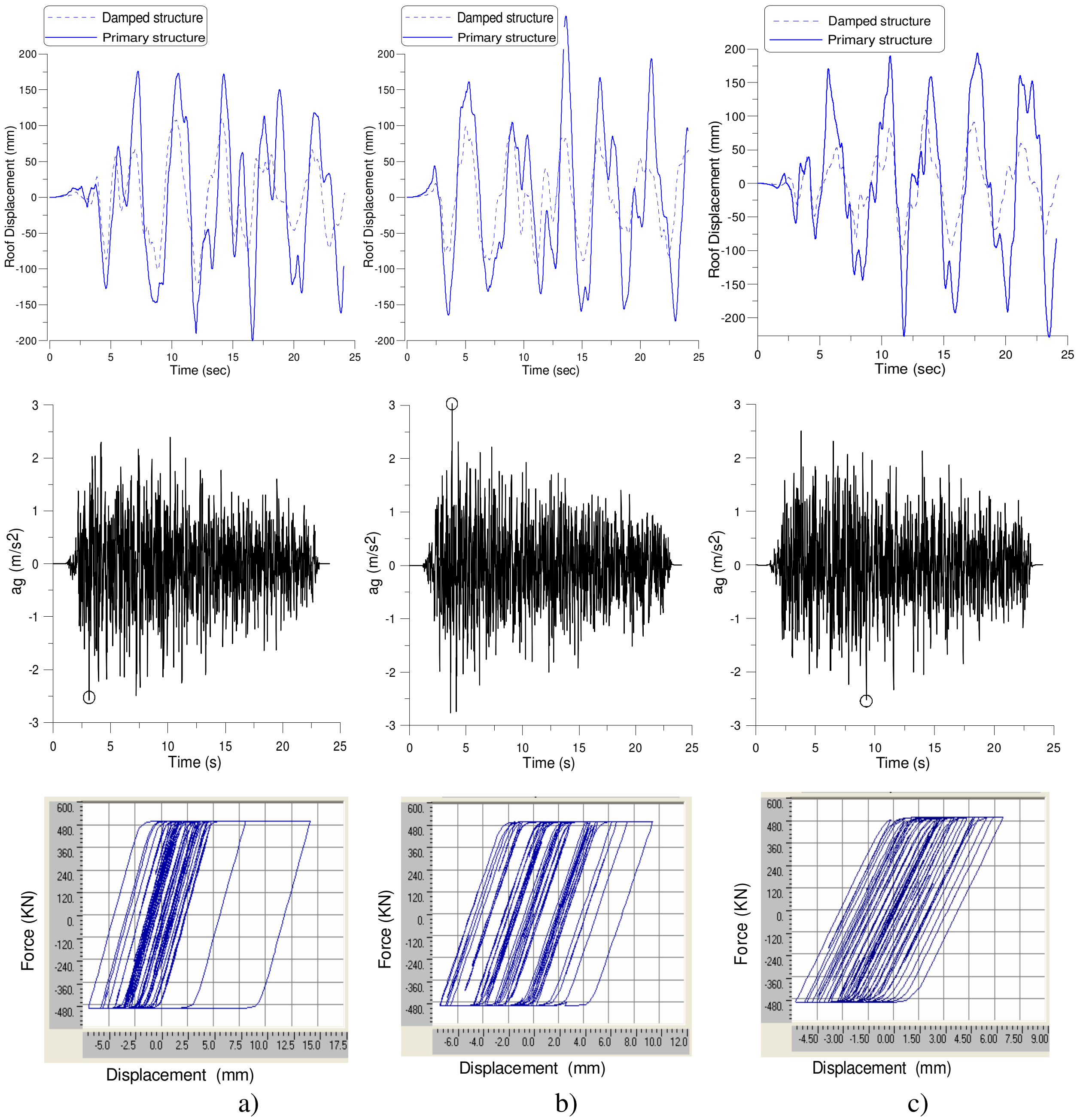


Figure 8. Roof displacements for retrofitted versus primary structure under ground motions: a) R6; b) R7; c) R8; and the hysteresis loop of a ground floor damped brace.

Conclusions

Friction dampers were found to be an economic solution for upgrading the seismic performance of a gravity steel moment frame, even for flexible structures. The design approach utilizes several linear and "non-linear" time-history analyses to set the design values of damper slip loads, so that, the retrofitted system to best fit the imposed target control parameters. Several scenarios for dampers location were analyzed to minimize the elastic member responses when the target storey drift displacement is achieved. Due to high damping provided by the Pall friction dampers, the strengthening of column and foundations was avoided.

Figure 9 Retrofitting of existing structure with Pall friction dampers

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