AN INNOVATIVE APPLICATION OF DAMPING DEVICES IN SEISMIC UPGRADE OF A WATER RESERVOIR

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SUMMARY

Seismic rehabilitation of a critical water storage facility by means of passive seismic energy dissipation devices - dampers has been presented in the paper. The reservoir considered is a concrete flat slab structure over twenty years old located in a park area of Burnaby, British Columbia, Canada. Due to inadequate strength of critical structural elements, the existing structure would possibly demonstrate excessively large lateral displacements and might experience a non-ductile failure at a significantly less than the design level earthquake. A feasible way of upgrading the deficient reservoir structure is by means of seismic dampers, which are used with an objective to enhance energy dissipation potential of the existing non-ductile structure. A possible layout and installation of damper units has been presented in the paper. Effectiveness of viscous and friction dampers has been evaluated by means of 2-D and 3-D dynamic time history analyses. The following results of the dynamic analysis have been discussed: acceleration and displacement response, the effectiveness of linear versus nonlinear viscous dampers, and the effect of recorded versus artificial earthquake time histories used in the analysis.

INTRODUCTION

The Central Park Water Reservoir was constructed in 1974-75 and it is considered to be a post-disaster facility in the Greater Vancouver area, Canada. The 36 million litre structure consists of a basin excavated into existing soil, lined with concrete over the flat central area and sloping sides. At the top of the slopes are short cantilever concrete walls that retain backfill and support the perimeter of the roof slab. The roof slab is primarily supported by the interior columns. The cast in place concrete structure consists of two separate “mirror-image” units each 50.3 m by 62.5 m in plan separated by a 50 mm wide expansion joint.

The two-way roof slab is 230 mm thick and it is supported by 560 mm square columns on a typical grid spacing of approximately 7.3 m in each direction, as illustrated in Figure 1. Majority of the columns (72 in number) are approximately 6.3 m high, whereas the shorter columns (40 in number) on the sloped portion of the slab on grade are approximately 4.3 m high. The flat slab is thickened in the region over the columns with 2.4 m square by 102 mm thick drop panels and 610 mm high tapered column capitals. The perimeter of the slab is structurally independent of the walls and it is supported atop the walls by a 25 mm thick and 76 mm wide continuous neoprene rubber pad. This joint acts as a sealant and allows for a freedom of lateral movement in all directions due to thermal expansion of the reservoir roof.

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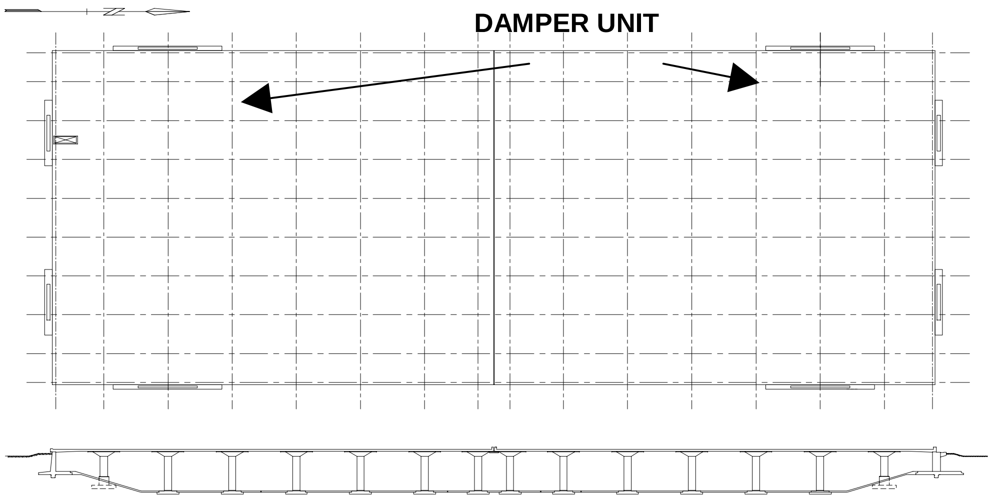


Figure 1: Roof plan and typical elevation of the existing reservoir structure

SEISMIC RETROFIT CRITERIA

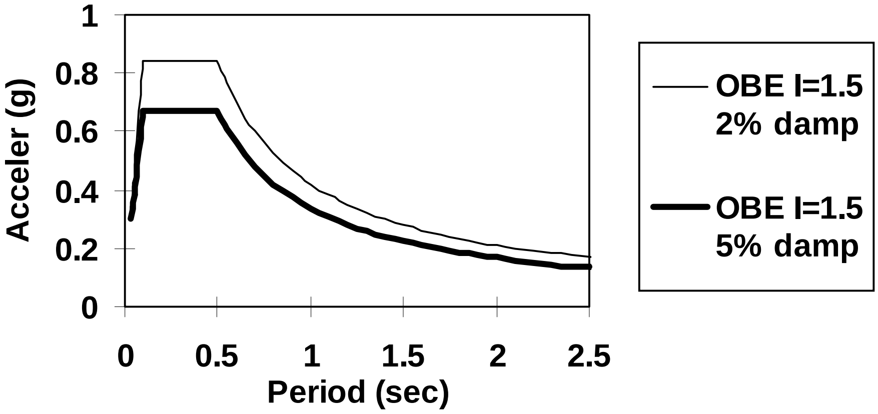
According to the National Building Code of Canada [NBC, 1995], the reservoir is located in the Seismic Zone 4 of Canada. Others previously conducted a deterministic seismic risk study for a location close to the reservoir site to estimate Peak Ground Acceleration (PGA) levels corresponding to design earthquakes. The design earthquake levels and the corresponding seismic performance criteria for this project are summarized in Table 1.

Table 1: Design Earthquake Levels and the Corresponding Seismic Retrofit Objectives

Design Earthquake	Earthquake Level (Return Period)	PGA	Seismic Retrofit Objectives
EQ-1	SLE (100 years)	0.07*g*	Reservoir exhibits elastic response with no damage.
EQ-2	OBE (475 years) I=1.5	0.30*g*	Reservoir remains operational but may experience cracking and moderate leakage that may be repaired, when convenient, within a year following the event.
EQ-3	MCE	0.50*g*	Reservoir may experience extensive damage, however, no sudden, catastrophic release of water occurs from the containment structure.

Note:
SLE= Service Level Earthquake
OBE= Operating Basis Earthquake
MCE= Maximum Credible Earthquake - an M 6.5 event occurring at a distance of approximately 10 km from the site, with an estimated firm ground PGA level of 0.5 g.
I= Importance factor (as per the NBC1995)

As a result of the seismic risk study, a set of response acceleration spectrum curves corresponding to the mean confidence level was developed. In addition, a set of the three design spectrum compatible artificial time histories were generated to serve as input for time history dynamic analysis; the 1940 El Centro earthquake (N270 and N180 components), the 1983 Coalinga earthquake (N45 component), and the 1985 Mexico City earthquake (N270 and N180 components) records were used as “seed” for this purpose. Both the amplitude and frequency content of the original records were modified in order to fit the target response spectrum curve using the computer software SYNTH [Naumoski, 1985]. The design acceleration response spectra for EQ-2 event (at 2% and 5% modal damping) are shown in Fig. 2a, whereas a couple of the acceleration response spectrum curves corresponding to the original and the artificial 1983 Coalinga earthquake record are depicted in Figure 2b.



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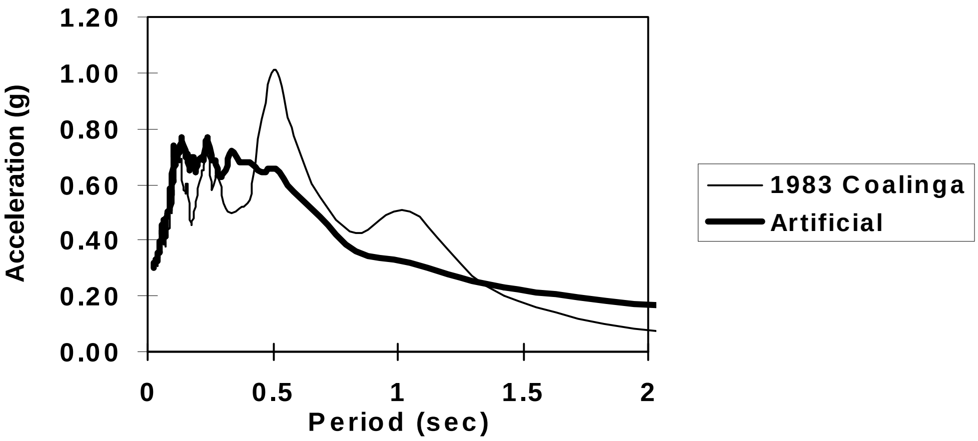

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Figure 2: Acceleration response spectrum curves: a) design acceleration response spectra (EQ-2 event) at 2% and 5% modal damping, and b) response spectra corresponding to the original 1983 Coalinga earthquake record and the corresponding artificial record (at 5% modal damping)

SEISMIC EVALUATION OF THE EXISTING STRUCTURE

The reservoir was constructed in 1974-75 and it is expected that the design was carried out in compliance with the 1970 National Building Code of Canada. At the time of the original reservoir construction, flat slab structures were designed to sustain mainly gravity load effects. Consequently, such structures are characterized with a rather low lateral deformation capacity. Some of the design and detailing deficiencies commonly found in the older flat slab structures are: i) discontinuous bottom slab reinforcement at the column locations, causing flexural slab failure at the roof slab-to-column joint; ii) lack of shear reinforcement at a critical slab perimeter around the columns, resulting in punching shear failure in the roof slab; iii) inadequate lateral confinement of the column reinforcement, thus resulting in the limited hinge rotational capacity, and iv) inadequate development length of the column longitudinal reinforcement at the column-to-roof slab connections, leading to anchorage failure in the connections. Current edition of the Canadian Concrete Code [CSA, 1994] referred to in the 1995 National Building Code of Canada implicitly advocates less than a nominal displacement ductility ratio (R) value of 1.5 for flat slab structures (as per Cl. 21.9.1 which addresses the two-way floor systems without beams).

Several 2-D and 3-D structural models were developed to evaluate lateral deformation capacity of the existing roof structure and the corresponding range of fundamental periods; both the equivalent static and dynamic analyses were carried out. Details of the structural models were discussed by [Nikolic-Brzev and Sherstobitoff, 1998]. A number of parameters affecting the seismic response of the roof structure were varied in the analysis e.g. column base support conditions (pinned or fixed), Young's modulus value (from 26,000 to 40,000 MPa), and moduli of inertia values for the slab and columns (cracked/uncracked); for the cracked structure, values of gross modulus of inertia for the columns and the slab were reduced by 30% and 60% respectively, as recommended by the CSA (1994).

Due to almost perfect symmetry of the roof plan with respect to the centre of gravity and very similar lateral stiffness values in the North-South (N-S) and East-West (E-W) directions, very similar values were obtained for the fundamental periods in the two directions. Modal frequency analysis has revealed that the effective mass ratio of the fundamental mode was in the order of 94% both for the N-S and E-W directions. The total weight of the roof structure considered in the seismic analysis was in the order of 4.2×10^6 kg. Modal damping ratio of 2% was used in the analysis of the existing and the retrofitted structure.

It should be noted that, as a result of the variation in column base support conditions and other parameters, fundamental period of the existing structure varies in a rather broad range from 0.2 sec (corresponding to the fixed-base uncracked structure) to 1.4 sec (corresponding to the pinned-base fully cracked structure). In the further text, the structural model characterized with the lower bound value of the fundamental period range (0.2 sec) will be referred to as the "stiff" model, whereas the model characterized with the upper bound value of the fundamental period range (1.4 sec) will be referred to as the "flexible" model.

Equivalent 2-D frame analysis (as per the CSA, 1994) was carried out to determine the lateral capacity versus demand (C/D) ratio for the critical elements of the existing roof structure. Typical frames were identified both in the N-S and E-W directions. The C/D ratios were determined for critical load-bearing structural elements: roof slab and columns, thereby revealing general inadequacy of these elements to sustain the effects of a design level earthquake. The analysis has shown that the negative flexural capacity of the slab in the vicinity of drop panels represents a major "weak link" in the system. In case of an EQ-2 event, the corresponding C/D ratio was found to be as low as 0.3.

Due to inadequate amount and poor detailing of the roof reinforcement, with high chances of a brittle structural failure at a design level earthquake, it was surmised that lateral displacement in the structure needs to be restrained to a level below the onset of yielding in the slab. Based on the analysis carried out, this requirement corresponds to a lateral drift level of 0.4% (i.e. lateral displacement of approximately 30 mm), corresponding to "elastic-cracked" response of a flat slab system; this is compatible with the seismic retrofit objectives for earthquake level EQ-2 as outlined in Table 1.

DAMPER RETROFIT SCHEME

The following seismic retrofit schemes were considered for upgrading the existing reservoir roof structure: i) new reinforced concrete shear walls within the reservoir basin, ii) upgrade of the flat slab structure to a moment frame, and iii) installation of seismic dampers. The first two schemes represent conventional seismic upgrade solutions, and they were used successfully in two other reservoir upgrade projects in the Vancouver area, as discussed by [Sherstobitoff and Nikolic-Brzev, 1998]. The third, less conventional option, entails the installation of seismic dampers to achieve a substantial increase in the modal damping ratio from the original level of 2 - 5% to over 20% and thereby reduce the lateral drift response and the overall seismic demand to this structure. One of the attractions of damper technology is that a major part of the earthquake input energy is being absorbed by damper devices and transformed into heat, whereas in the conventional retrofit schemes similar amount of energy is being absorbed through nonlinear response and concentrated damage to specially detailed "ductile" plastic hinge regions of the beams, columns and walls. The damper scheme was finally selected as the most feasible solution due to lower construction costs and considerably shorter construction time requirements as compared with the other two schemes. An additional advantage of the damper scheme is that a major part of the construction effort related to the structural upgrade can be carried out at the exterior of the reservoir, leaving the reservoir operational even during the installation of dampers and their attachments to the adjoining roof and wall members.

Two types of damping devices were evaluated in this project, namely friction dampers and viscous dampers. Friction dampers utilize the mechanism of solid friction that develops between two solid bodies sliding relative to one another to provide the desired energy dissipation. Viscous (fluid) dampers are hydraulic cylinders which operate on the principle of fluid flow through orifices; the means of energy dissipation in case of fluid dampers is that of heat transfer, i.e. the mechanical energy dissipated by the damper causes heating of the damper's fluid and mechanical parts. Each of these devices has its own benefits and drawbacks, and their features are discussed in detail by [Nikolic-Brzev and Sherstobitoff , 1998].

Both options entail damper installation at the perimeter of the reservoir, aligned parallel to the perimeter walls, as illustrated in Figure 1. All forces generated in the damping devices are transferred from the roof to the walls in their strong longitudinal direction. At one end, each damper is connected to the reservoir perimeter wall by means of a galvanized steel bracket attached to a new concrete transfer beam; the beam distributes damper force over a predefined wall length (depending on the wall in-plane shear strength). At the other end, a damper is connected to the roof structure by means of a steel bracket (similar to the damper-to-roof slab connection); length of the steel plate attached to the roof structure depends on the in-plane shear strength of the roof slab. Note that several additional retrofit operations were required at the interior of the reservoir as a part of the upgrade, i.e. the removal of the existing roof expansion joint and joining the roof segments together (in order to achieve a symmetrical layout of dampers at all four sides of the structure), and reinforcing of the perimeter wall.

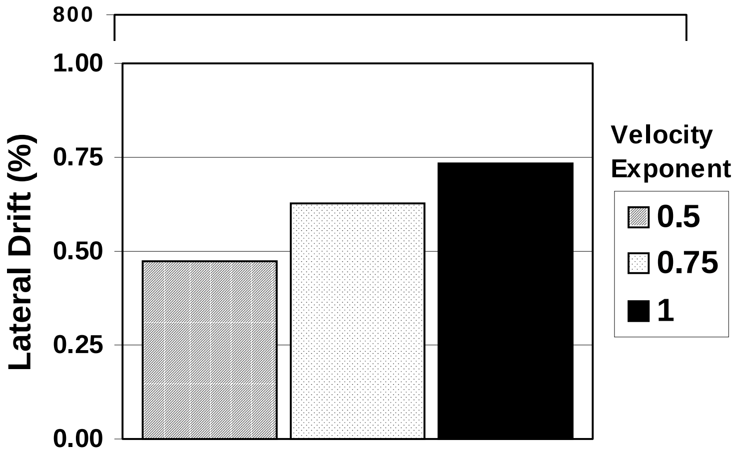
THE ANALYTICAL PROCEDURE

Two different mathematical models of the reservoir roof, including a simple 2-D frame model and a more complex 3-D FEM model, were developed to evaluate seismic response of the retrofitted structure, as discussed by [Nikolic-Brzev and Sherstobitoff, 1998]. The 2-D model was used in the preliminary design phase to determine the required level of supplemental damping provided by damper devices by means of a dynamic response spectrum analysis. Acceleration, velocity and displacement response spectrum curves corresponding to the design spectrum compatible artificial earthquake time histories were developed; modal damping ratio was varied in the range from 2% (corresponding to the original unstrengthened structure) to 40% (considered as the upper limit of supplemental damping provided by damper devices). The 3-D model was used to verify the results of the 2-D analysis, to confirm the number and capacity of damper devices required, and to determine the key response indicators for the retrofitted structure e.g. acceleration and displacement levels and member forces. Nonlinear time history dynamic analysis was performed using a step-by-step linear acceleration method using the SAP2000 software [CSI, 1998]. Damper devices were modeled using 16 NLINK elements (4 damper elements at each side of the reservoir roof); viscous dampers were modeled using DAMPER elements (spring-dampers with nonlinear damper force vs. velocity relation), whereas friction dampers were modeled using PLAST1 elements (elasto-plastic springs).

SEISMIC RESPONSE OF THE RETROFITTED STRUCTURE

Installation of external viscous dampers at the perimeter of the reservoir roof has confirmed the benefits of added damping without significant changes in the stiffness of the existing structure; the fundamental period of the roof structure remained virtually unchanged even though the damper elements were incorporated in the structural model. In total, 16 nonlinear viscous dampers were used in the model. The viscous damper output force can be expressed as CxV^{α}, where C is the damping coefficient, V is the relative response velocity developed in a damper during a seismic event, and α is velocity exponent (typically in the range from 0.4 to 1.0). In this analysis, C value of 1,500 kN-sec/m and α value in the range from 0.5 to 1.0 were considered. The maximum damper output force of 780 kN (175 kips) was obtained at the EQ-2 earthquake level; this force was used to specify the design capacity of damper devices. The effect of friction dampers and steel struts installed at the perimeter of the reservoir roof structure resulted in a significant stiffness increase in the existing structure and in a corresponding reduction in the fundamental period value from 1.4 sec to 0.35 sec. In total, 16 friction dampers were installed in the structure; each characterized with a 600 kN slip force. Friction dampers were installed at the same locations as viscous dampers.

Comparison of the peak seismic response for the original unstrengthened structure with the two models of the retrofitted structure equipped with friction and viscous dampers was discussed by [Nikolic-Brzev and Sherstobitoff, 1998]. The artificially modified Mexico City earthquake record (PGA of 0.3 g) was used in the analysis. The obtained values of peak response accelerations for the original structure were in the range from 0.3g ("flexible" model) to 1.02g ("stiff" model). Installation of viscous dampers resulted in a significant reduction in the acceleration response by 33% ("flexible" model) and 56% ("stiff" model) as compared to the original structure. It should be noted that the installation of friction dampers led to a multi-fold increase in the acceleration response for the retrofitted structure as compared to the original structure. Peak lateral drift values obtained for the unstrengthened structure model range from 0.4% ("stiff" model) to 1.8% ("flexible" model). Both viscous and friction dampers proved to be effective in reducing the displacement response in the structure. A considerable reduction in lateral drift levels by 60% and 97% as compared to the original structure was observed in the “stiff” model retrofitted with viscous and friction dampers respectively. A similar trend was noticed in the case of “flexible” model, with the corresponding reduction in the lateral drift ratio by 75% (viscous dampers) and 84% (friction dampers).



a)
b)

Figure 3: The effect of variation in viscous damper velocity exponent (α) value: a) damper output force, and b) lateral drift ratio

Velocity exponent (α) is an important parameter related to the design of structures equipped with viscous dampers. Most of the viscous damper applications to date were made using α value of 1.0, thereby resulting in the linear damper output force-velocity relation (so-called “linear” dampers). In some applications, however, it might be appropriate to use “nonlinear” viscous dampers, characterized with α value of less than 1.0 (and typically greater than 0.4). A potentially good “case” for the application of nonlinear viscous dampers would be if lateral deformation capacity of the original structure is rather limited, and there is a need to further reduce the lateral displacements beyond the level achieved using linear viscous dampers. It should be noted that, as a “trade-off”, the use of nonlinear dampers results in larger damper output force as compared with the linear dampers. The effect of variation in the velocity exponent (α) value on damper output force and lateral drift levels in the reservoir roof structure is illustrated in Figure 3 (“flexible” model, Mexico City artificial time history, damping coefficient of 1,500 kN-sec/m). It is noteworthy that a decrease in the α value from 1.0 (linear dampers) to 0.5 (for dampers with the same C value) results in a substantial decrease in the lateral displacement drift ratio by over 55% (see Figure 3b) and a corresponding increase in damper output force by 32% (see Figure 3a). It is important to note that the peak values of damper output force for linear viscous dampers could be either larger or smaller as compared to the corresponding forces for the nonlinear dampers (assuming the same value of damping coefficient C), depending on the response velocities developed in the structure equipped with viscous dampers. As an example, in the case: $\alpha = 0.5$, the maximum relative velocity (V) was equal to 230 mm/sec (corresponding to the damper output force of 706 kN), whereas for $\alpha = 1.0$, the peak response velocity was found to be approximately 330 mm/sec (corresponding to the damper output force of 482 kN). It is noteworthy that nonlinear viscous dampers characterized with α value of 0.5 were finally selected for this application.

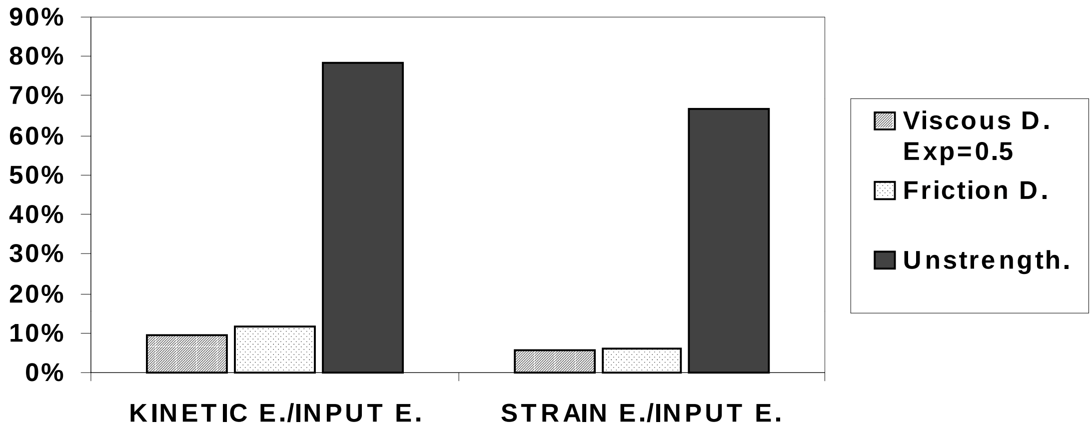


Figure 4: Ratio of the peak kinetic (or potential) energy vs. the cumulative input energy absorbed by the unstrengthened and the retrofitted structure (equipped with viscous/friction dampers)

Energy-based parameters were also used as seismic performance indicators for friction/viscous dampers installed in the reservoir roof structure. Cumulative input energy transferred to a structure subjected to an earthquake excitation (modified Mexico City record, duration 20 sec.) was compared with the peak values of the kinetic and strain (potential) energy. The chart shown on Figure 4 depicts the ratio of the peak kinetic/potential energy vs. the cumulative input energy for the original and the retrofitted structure. The chart indicates that both viscous and friction dampers are very effective in diminishing the damaging effects of both the kinetic and elastic strain energy in a structure during an earthquake. The ratio of peak kinetic versus input energy for the structure equipped with viscous/friction dampers is in the order of 10-12%, whereas the value of the same ratio for the original structure is as high as 78%. A similar trend has been observed for the ratio of peak strain energy vs. the cumulative input energy, as illustrated in Fig. 4. It is also noteworthy that, in case of a structure equipped with viscous dampers, over 85% of the total input energy is absorbed by the dampers.

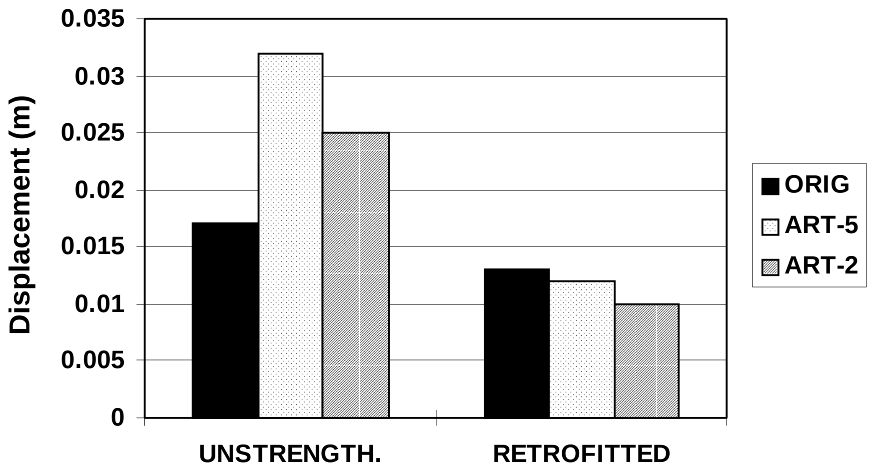


Figure 5: Peak displacement response for the unstrengthened and the retrofitted structure equipped with viscous dampers, subjected to the 1983 Coalinga earthquake record (ORIG) and the artificial records ART-2 and ART-5

An additional objective of this study was to compare seismic response of the structure subjected to the original recorded earthquake time histories and the corresponding spectrum-compatible artificial records. A comparison of the peak displacement response for the original unstrengthened structure ("stiff" model) and the structure equipped with the linear viscous dampers is illustrated in Figure 5; modal damping ratio of 2% was used in the analysis. The following three earthquake records were used: the original 1983 Coalinga record (ORIG), and the two artificial spectrum compatible records corresponding to the same target spectra shown on Fig. 2a at 2% and 5% modal damping respectively (denoted as ART-2 and ART-5 in the following text). The results indicate that that the difference in response displacements with respect to the use of recorded vs. the artificial time histories is more pronounced for the original (unstrengthened) structure. Displacement response appears to be higher by 88% in the model subjected to the ART-5 record as compared to the same model subjected to the ORIG record. Interestingly, in the retrofitted structure a decrease in the displacement response by 9% was observed in the model subjected to the ART-5 record as compared to the same model subjected to the ORIG record. It is also interesting to note that different displacement response values have been obtained in the otherwise identical model subjected to the two artificial records (ART-2 and ART-5). In the original structure, a 12% decrease in the displacement response was obtained for the model subjected to the ART-2 record as compared to the ART-5 record, whereas in the retrofitted structure this difference is on the order of 17%.

CONCLUSIONS

The paper outlines the world's first reported application of seismic dampers used in the seismic rehabilitation of a partially buried water reservoir structure. Installation of seismic dampers at the perimeter of the existing reservoir roof structure has emerged as an effective seismic retrofit scheme for the deficient water containment facility. Special attraction of the presented retrofit scheme lies in the minimized construction efforts and a possibility of undisrupted reservoir operation during the installation of dampers and the connecting attachments. The aforementioned attractions of the damper retrofit scheme and its cost-effectiveness as compared with the conventional retrofit solutions point out to the great potential of this innovative technology in retrofitting partially buried water reservoirs and other similar facilities.

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