NONLINEAR ANALYSIS AND SEISMIC EVALUATION OF GRAND COVER STRUCTURE, FOIRE DE TRIPOLI

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Abstract

In this paper, we discuss the nonlinear analysis of a large one-story building. The Grand Cover Structure, located in Rachid Karami International Fair of Tripoli, North of Lebanon, was built in 1967. It was designed by the famous Brazilian Architect Oscar Niemeyer. The Fair is considered a rare masterpiece and one of the 14 most important exhibitions in the world. The Grand Cover building consists of a repeating pattern of 20 rectangular modules that measure roughly 33-meters by 70-meters, resulting in an overall building length of approximately 660-meters. The building curves gradually around the mid-length by about 54 degrees, which gives it a boomerang shape. The modules are seismically separated from each other using seismic shear key joints, and are typically supported by two parallel column bents, each consisting of two columns, a deep prestressed beam that spans 47-meters between columns and cantilevers 11-meters on each end. The roof slab consists of a 0.8-meter deep box section with dimensions of 17-meter by 45-meters. The structure is used for exhibition halls, fairs and conferences. It was subjected to several earthquake shakings and never underwent any maintenance activities. Linear Analysis was performed per ASCE 7-16 showed that most structural elements were failing, we used Nonlinear Pushover and Time History analysis to perform a more accurate assessment the seismic resistance of the structure and identify the most critical and controlling seismic deficiencies. A set of 11 ground motions that best match Tripoli's seismic design hazard were applied to the structure to obtain the inelastic deformations in the structural component hinges and compare to the acceptance criteria provided by ASCE/SEI 41-17. According to the nonlinear analyses results, the columns and roof are found to behave acceptably, but the deep inverted girders were found to be deficient. CFRP wrapping was proposed to mitigate shear deficiency, which leads to an improvement in the flexural ductility and rotational deformation capacity of the girders, allowing them to meet the drift demands. A re-analysis of the structure shows a significant decrease in the girder hinge rotations.

Keywords: Nonlinear Analysis; Pushover Analysis; Response History Analysis; Seismic Retrofit

1. Introduction

Lebanon straddles a well-known fault system in the eastern Mediterranean where it lies across the northern segment of the Dead Sea fault. Lebanon has experienced many devastating earthquakes that are part of the historical record. The recorded earthquake shakings were reported to have caused a wide-scale destruction and high death tolls [1]. While occurrence of earthquakes is unpredictable, engineers can adopt preventive measures to reduce loss to life and property during earthquakes. Modern performance-based design methods provide ways to determine the realistic behavior of structures under such conditions and can be used to design economical and effective retrofit solutions. [2]

2. Site Description

The "Grand Cover" structure is located in Rashid Karami International Fair, Tripoli, North Lebanon. It was designed by the famous Brazilian architect Oscar Niemeyer. The building construction started in 1964 (Fig. 1). The Fair is considered a rare masterpiece and one of the top 14 exhibitions in the world. At the time of construction, it was expected that the fair will receive an estimated two-million visitors annually; however, due to the civil war (1975-2000), construction was never completed, and the condition of the Fair deteriorated significantly. The Grand Cover Structure (Fig. 2) is a large one-story building that consists of 80 columns, 40 deep beams and a total roof area of 47,730 m^2. The Grand Cover building consists of a repeating pattern of 20 rectangular modules that each measure roughly 33-meters by 70-meters, resulting in an overall building length of approximately 660-meters. The building curves gradually around the mid-length by about 54 degrees, which gives it a boomerang shape. The modules are seismically separated from each other using seismic shear key joints, and are each typically supported by a pair of parallel two-column bents, each having a deep prestressed beam that spans 47-meters between the two end columns, and cantilevers 11-meters on each end. The reinforced concrete roof consists of a 0.8-meter deep box section with dimensions of 17-meter by 45-meters.

Fig. 1 - Grand Cover 1966, Ferdinand Dagher Collection

Fig. 2 - Grand Cover 2019

The structure is occasionally used for exhibition halls, fairs and conferences. Some of its elements are suffering from some deterioration and failures where cracks, concrete spalling, delamination and steel corrosion are clearly observed. Due to onset of the civil war and lack of funding and political initiative during and after the war, the structure did not undergo any serious maintenance activities.

3. Objective and Overview

The level of seismicity of Lebanon was not well understood in 1964, so the structure was likely designed to a low level of seismicity. Since the Grand Cover is the largest and most important building in the fair, it is useful for hosting large exhibits and may be occupied by a large number of people. For that purpose, it is important to study and assess the seismic safety and behavior of this landmark structure. The evaluation of the Grand Cover involved the following:

1. Data Collection: Performed non-destructive field testing and researched historical articles and photographs of the structure.
2. Linear Analysis: A 3D ETABS linear model was built using ETABS V17 where all structural elements and loadings were defined. Linear static and dynamic analyses were performed to obtain seismic demands and compare them to the available capacities. The Grand Cover performance was evaluated under seismic load using performance-based designs according to ASCE 41-17 [3].
3. Nonlinear Analysis: Used Nonlinear Static (Pushover) Analysis and Nonlinear Time-History Analysis to estimate the demand parameters and compare them to the applicable acceptance criteria, and identify seismic deficiencies.
4. A retrofit solution is proposed to reduce the lateral displacement, increase ductility and improve the structural capacity.

4. Data Collection and Field Testing

Since most of the required structural and architectural data were not available, non-destructive testing was conducted to obtain compressive strength using Schmidt Hammer and embedded longitudinal and transversal reinforcement using rebar scanner (Table 1).

Table 1 - Comparison between available data and field measurements

Structural Element	Results of Field Measurements	Compressive Strength (MPa)	Results of Rebar Scanning
Column	2 m × 0.8 m	50	3.3 % Stirrups: 10Ø10 / m
Girders @ Support	2.2 m × 0.8 m	45	Top Steel: 0.3 % Bottom Steel: 0.6 % 13Ø10 / m
Girders @ Midspan			Top Steel: 0.3 % Bottom Steel: 1% Stirrups: 10Ø10 / m 32½-in prestressing strands (constant ecc.)
Slab	0.74 m rib depth 1.85 m rib spacing Topping slab: 6 cm Top slab thickness: 4 cm Bottom slab thickness: 10 cm	45	2 ½-in prestressing strands Top Longitudinal Steel: 10Ø10 / m Bottom Longitudinal Steel: 10Ø10 / m Transverse Steel: 5Ø8 / m

5. Linear Model

CSI ETABS V17 was used to build and analyze a linear model of the building. The loads were calculated and load combinations were assigned according to ASCE/SEI 7-16. The section of columns, inverted girders, deep I-section section beams and chainages were defined in the model. Pinned supports were assigned to the columns at the foundation level (Fig. 3). The dead load (excluding self-weight of structural elements) assigned to beams was a 6-cm wire-mesh reinforced concrete slab, asphalt and 0.5 kN/m^2 roof-live load. The mass and weight of the suspended modules between seismic shear keys were added to the model as a line load applied through a dummy beam section at the edges of the model, which acted to the transfer the forces to the girders and columns through the equivalent slab and beams. The roof box section was modeled using an equivalent solid slab section with the same stiffness and area. Since it is not possible to match the area and moment of inertia of the section simultaneously, the thickness of the equivalent section was selected to match the area and mass of the original box section, and a stiffness modifier was then applied to match the moment of inertia. The Grand Cover structure is a combination of two seismic force-resisting systems in the two directions. In the X-direction, seismic force-resisting system is an Ordinary Moment Resisting Frame (OMRF) whereas in the Y-direction, there is an Intermediate Moment Resisting Frame (

Table **2**).

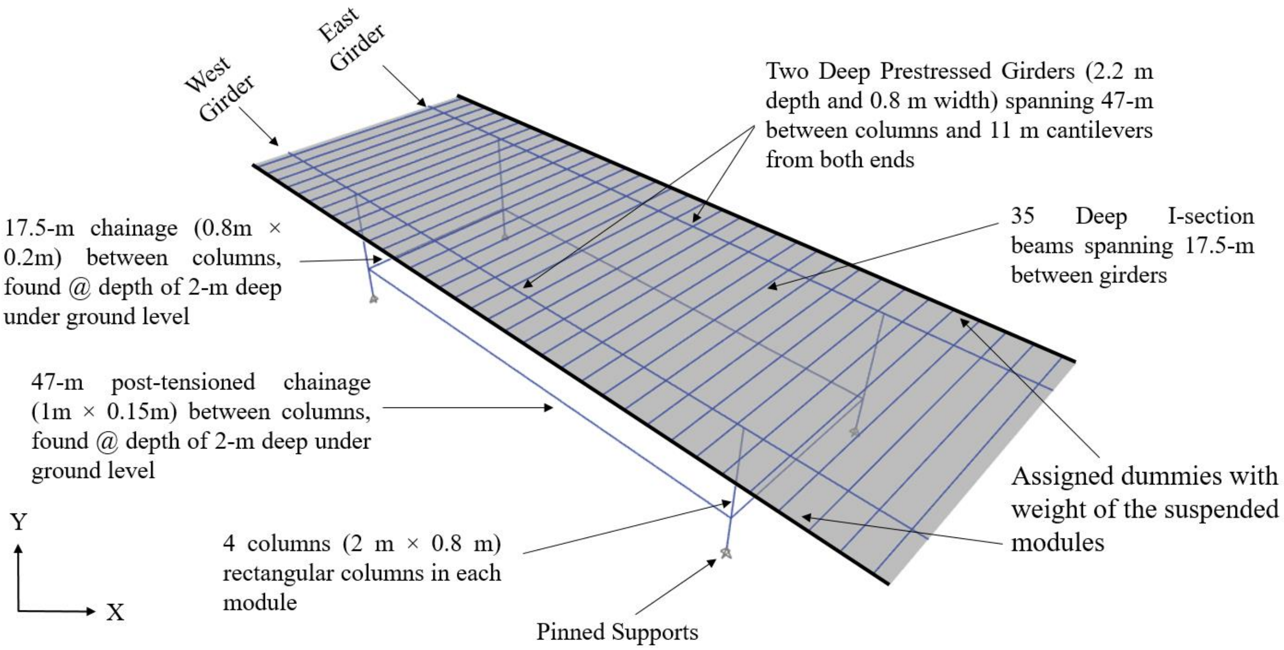

Fig. 3 - Grand Cover Structural Elements' Sections

Table 2 - Grand Cover Seismic Force-Resisting System in two directions

Direction	Seismic Force-Resisting System	R	Ω_0	C_d
X	Ordinary Moment Resisting Frame (OMRF)	3	3	2.5
Y	Intermediate Moment Resisting Frame (IMRF)	5	3	4.5

The fundamental periods of the structure, T, in each direction were established using the structural properties and deformational characteristics of the resisting elements (Table 3).

Table 3 - Structural Period

Mode	Period (s)	Direction	Modal Mass Participating Ratio
1	3.18	X	98%
2	2.92	Torsional	98%
3	1.48	Y	98%

In the linear static analysis, four load cases were defined in model. Static lateral loads in the X and Y directions with positive and negative loading, in addition to defining the structural system coefficients (R), overstrength factors and amplification factors in both directions. Since a seismic soil investigation for the site was not available, this study assumes the default Soil Class D, as allowed by ASCE/SEI 7-16, Section 11.4.3. In the dynamic analysis, the spectral response acceleration parameters (S_s and S_1) were defined in the model, and two response spectrum load cases in X and Y directions were defined in the model, depending on the structural system coefficient, with a scale factor equals to $(I \times g) \div R$. Using ETABS, structural demands under linear static and dynamic loading were obtained. The calculated DCR (Demand-to-Capacity ratio) was evaluated for each structural component and the failure mode was identified (Table 4).

Table 4 - Failure mode of structural elements

Structural Element	Type of Analysis	Failure Mode	DCR
Girder	Static	Flexure	1.96
	Dynamic	Flexure	1.90
	Static	Shear	1.13
	Dynamic	Shear	1.10
Columns	Static	Flexure	1.21
Beams	Static	Shear	2.09
	Dynamic	Shear	1.95

The design story drift (Δ) was computed as the difference of the deflections at the top and bottom of the story under consideration. According to Table 12.12-1 in ASCE/SEI 7-16, the allowable story drift Δ_a = 2%. According to the results, the story drift in the X-direction under static loading have exceeded the maximum allowed.

The maximum displacements, δ_1 and δ_2, with 0% and 5% diaphragm eccentricity respectively were computed to check torsional irregularity. According to the results, the ratio δ_2 / δ_1 does not exceed the limit (1.2) under dynamic loading for both directions, and hence no torsional irregularity was recorded.

The capacity and demand of each structural element of the Grand Cover were computed to determine if any of the elements are overloaded. The software Prokon V2.3 was used to determine the structural capacity of the deep prestressed girders and post-tensioned I-beams in the transverse direction. Section analysis software SpColumn was used to determine the P-M-M interaction diagram of the column with the available reinforcement.

6. Nonlinear Model

When the structural elements are subjected to forces moments higher than their yield strength, they no longer behave elastically and begin to yield; they start to experience inelastic deformations and eventually form a plastic hinge. Hinge components are used to define the location and nonlinear inelastic behavior of yielding components, and are used to model and simulate nonlinear behavior from the onset of yielding through strength and stiffness degradation. In the nonlinear model, inelastic hinges were placed at the ends of each structural element that is expected to yield. In the ETABS model (Fig. 4), Multilinear Plastic Link Elements

(Fig. 5) were assigned to the ends of each structural element. In ETABS, Links were used instead of Hinges because: they converge more quickly compared to Hinges, and more modeling features exist for links such as various hysteretic models. Links were assigned at (h/2) from the face of each structural element support. ASCE 41-17, "Seismic Evaluation and Retrofit of Existing Structures" describes deficiency-based and systematic procedures that use performance-based principles to evaluate and retrofit existing buildings to withstand the effects of earthquakes. ASCE 41-17 was used to check the structural compliance of the building, to assign and define the available plastic rotations, and set three different state limits (Intermediate Occupancy (IO), Life Safety (LS) and Collapse Prevention (CP)) to determine allowable deformations post-yielding of structural elements. The available moment capacity and allowable rotation of the structural element was assigned to each hinge based on the recommendations of ASCE 41-17 (Table 5).

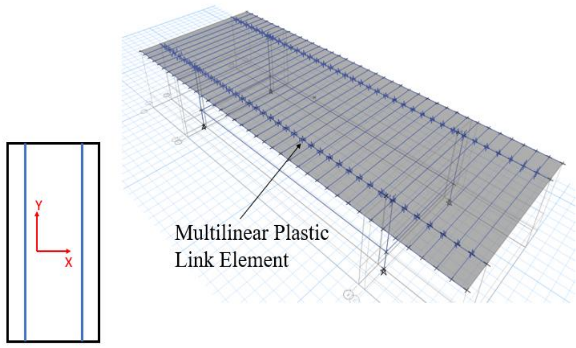

Fig. 4 - Multilinear Plastic Links

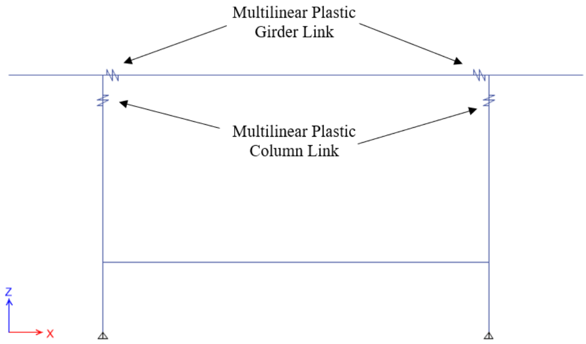

Fig. 5 - Nonlinear model

Table 5 - Characteristics of hinges based on ASCE 41-17

Structural element	$\frac{\rho-\rho'}{\rho_{bal}}$	Transverse reinforcement	$\frac{V}{b_w d\sqrt{f_{cE}}}$	a (radians)	b (radians)	c	IO	LS	CP
Girders	0	NC	0.313	0.017	0.026	0.2	0.004	0.017	0.026
Beams	0	NC	0.191	0.020	0.03	0.2	0.005	0.020	0.030
Columns	-	-	-	0.032	0.06	0.2	0.005	0.045	0.060

7. Pushover Analysis

Nonlinear static analysis approximates the dynamic response under earthquake ground motions through the application of a static lateral load. The Nonlinear Static Pushover analysis was used for early detection of structural deficiencies and served a valuable role as a companion to our nonlinear dynamic analysis. The building was subjected to monotonically increasing lateral loads until the target displacement in each direction was reached. Using Eq. 7-28 in ASCE/SEI 41-17, the target displacements in both directions were calculated:

$$\delta_t = C_0 C_1 C_2 S_a \frac{T_e^2}{4\pi^2} g \longrightarrow \begin{cases} \delta_x = 540 \text{ mm} \\ \delta_y = 450 \text{ mm} \end{cases}$$

The transverse beams' west hinges resulted in higher rotations compared to the east location hinges since the structure was pushed in the positive X-direction. The structure was also pushed in the negative direction to check convergence and symmetry, and the results confirmed that the structure is behaving symmetrically.

The base shear ratio (V / W) in both directions was computed and compared to the structural design value (Sa / R). According to results (Fig. 6), the structural design value (Sax / Rx) exceeded the pushover base shear ratio in the X-direction showing that the structure was under-designed in that direction.

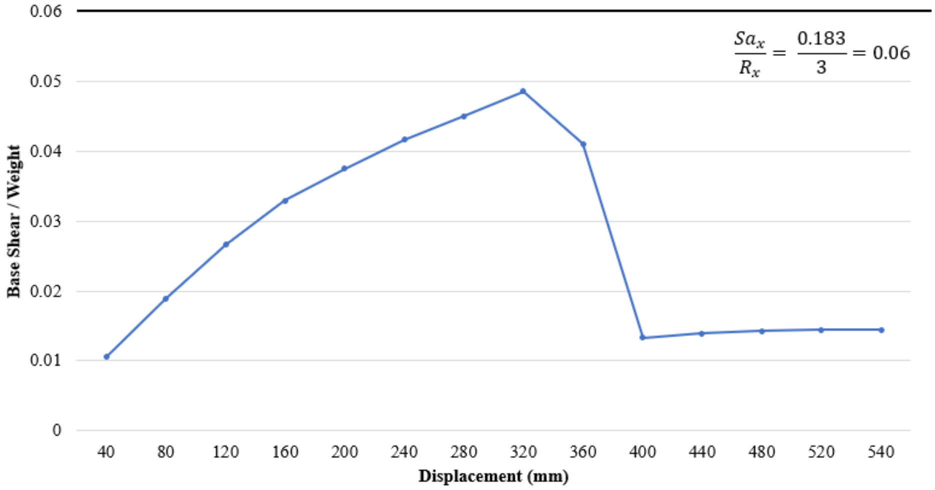

Fig. 6 - Base Shear-Weight Ratio in X-Dir

The most important predictors of structural performance in the inelastic ranges are the deformation demands in yielding components. Using the nonlinear static pushover analysis model, the columns, girders and beams link deformations were extracted to determine if the structure meets the performance objectives. The columns were considered to behave acceptably as the column link rotations were substantially below the Life Safety rotation limit of 0.045, and even met the Immediate Occupancy acceptance criteria. Concerning the girders and beams, Fig. 7 and Fig. 8 show that the link rotations exceeded the collapse prevention limit state, predicting that the girders and beams will fail under earthquake loads.

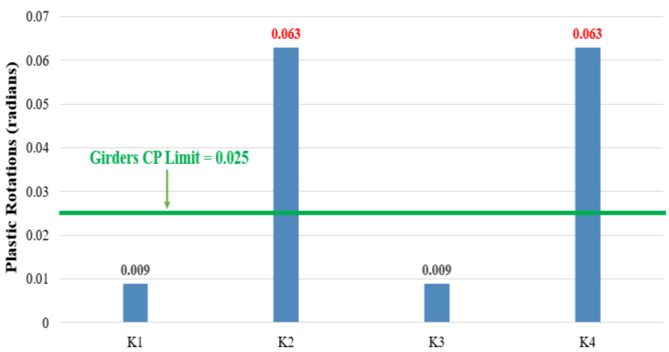

Fig. 7 - Girders rotations in Pushover analysis

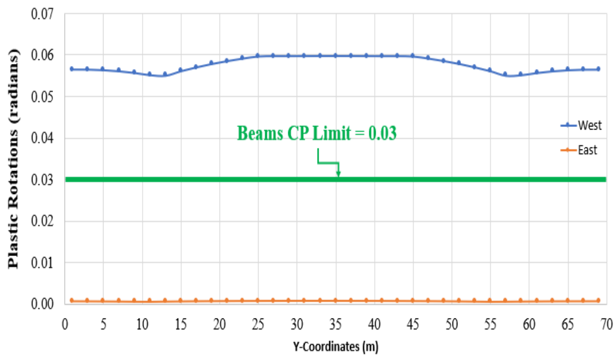

Fig. 8 - Beams rotations in Pushover analysis

From the Pushover analysis, the drifts (Table 6) were obtained and the results show that the peak drift values exceeded the maximum allowed by 120% in the X-direction and 70% in the Y-direction.

Table 6 - Drifts in Pushover analysis

Direction	Drift	Maximum Allowed
X-Drift	4.4 %	2 %
Y-Drift	3.4 %	2 %

8. Nonlinear Response-History Analysis

In the Pushover analysis, the plastic rotations in beams and girders exceeded by 100% and 142%, respectively, the maximum Collapse-Prevention rotations limits. Consequently, nonlinear Time-History analysis was performed in order to more accurately model the structural behavior, and validate the conclusions of the static pushover analysis. Nonlinear response-history analysis is a dynamic analysis where the structural model is subjected to a ground motion acceleration record. The response of the structure is calculated using step-by-step integration in the time domain over the full duration of the ground motion.

Using the QuakeManager software [4], a set of eleven bidirectional ground motions were selected and scaled to best match the MCE seismic design spectrum of Tripoli. The records were selected from NGA-West2 library collection and scaled by factors varying between minimum and maximum (Fig. 9) values of 0.2 and 4, considering the period range between 0.2 sec and 1.5 times the Grand Cover structural period (1.5x3.18=4.77sec). The error measure was computed using the square sum of the error between the suite average and the target spectrum for the maximum rotated spectral demand (RotD100). Note that for the seismic hazard analysis for Lebanon, the controlling earthquake magnitude has an approximate magnitude of 7.5. So, the generated records magnitude ranged from 6 to 7.5 at a maximum rupture distance (Rrup) equals to 40 km [5]. The spectral accelerations for site class D for MCE seismic hazard of Tripoli (S_a = 0.95 and S_1 = 0.53) were and used as the target spectrum [5]. Only one record from the same event was selected in the suite. The 11 ground motions were applied to the Grand Cover structure in both directions (X & Y) to obtain a more accurate estimation of the inelastic demands. 2% damping was used in the analysis.

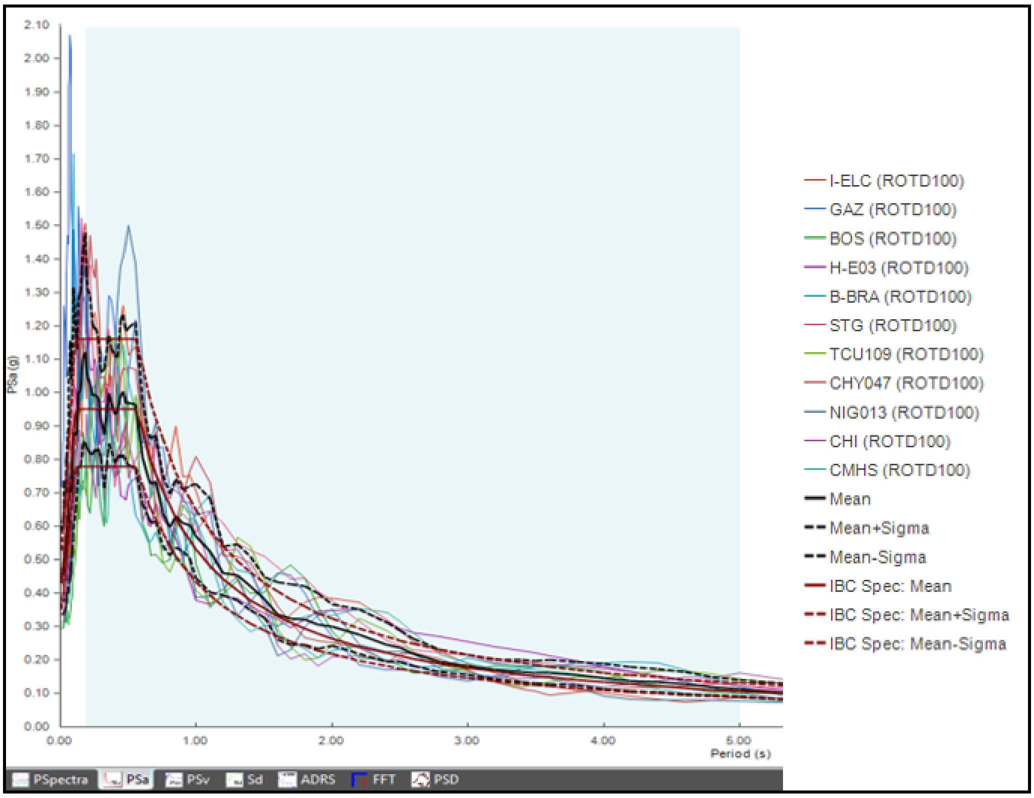

Fig. 9 - Generated ground motions

The obtained deformation demands (Fig. 10) were compared to the applicable limit states. The inelastic deformations at both ends of each girder were obtained under the applied 11 ground motions. In this case, the average (mean), taken over the 11 ground motions, of the of peak girder link deformations were computed and the average was found to exceed the acceptance criteria specified in Table 5. Fig. 10 summarizes the results of the girder links under the applied 11 ground motions.

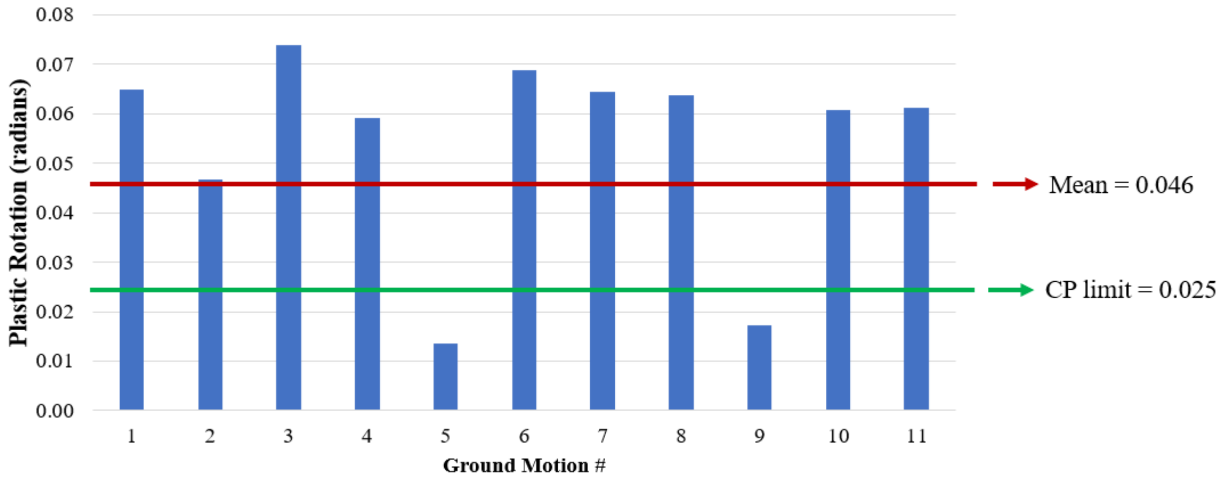

Fig. 10 - Results of girder rotations in Time-History analysis

Fig. 11 shows the variation of the inelastic deformations along the girder length in the east and west directions. The maximum rotations are observed at the intersection of the girders with the columns due to the additional restraint at the columns.

The beams were found to meet at least the Collapse-Prevention limit according to the acceptance criteria stated in Table 5.

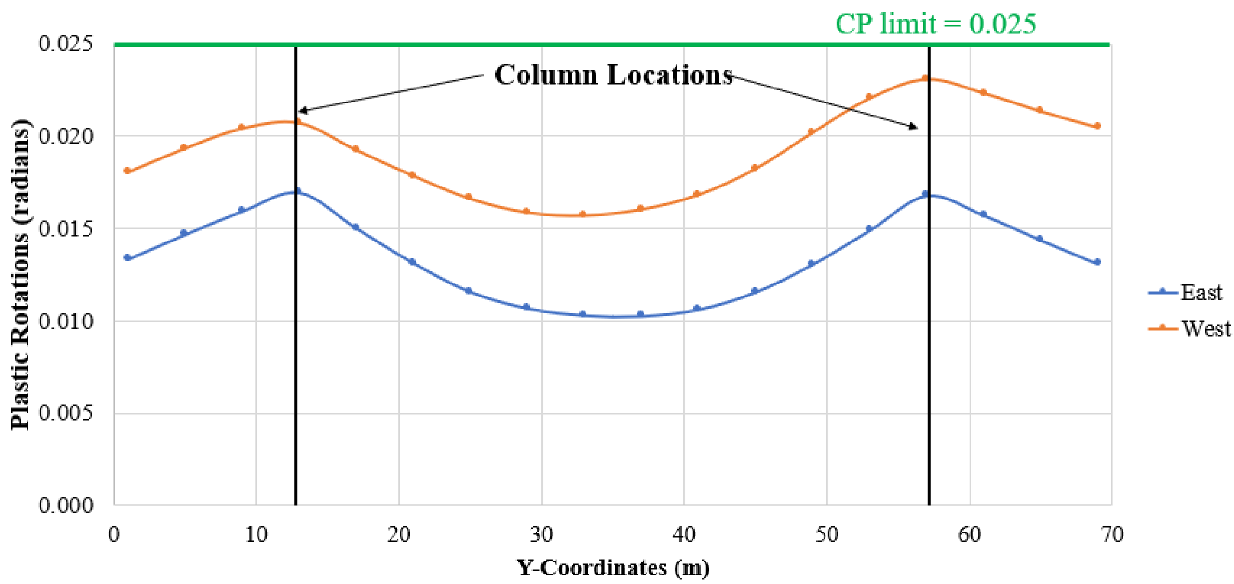

Fig. 11 - Mean rotations of beams

8.1 Drifts

The maximum drifts for the 11 ground motions were obtained in both directions (Table 7). Since ASCE 41-17 does not impose a limit on story drifts, we used ASCE 7-16 [6] drift limits. When evaluating the structure under MCE, ASCE/SEI 7-16, section 16.1.2 specifies that the allowable story drifts shall not exceed 150% of the drift limits of table 12.12-1.

⇨ Allowable drift = 2% × 1.5 = 3%

The mean drifts over the 11 applied ground motions were found to be safe in both directions where the mean drift didn't exceed 3%.

Table 7 - Drifts in Time-History analysis

	Ground Motion #												
Dir.	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**Mean**	**Allowable**
X (%)	2	3.2	3	2.8	3.9	3.3	1.2	2.2	4.2	3.6	4.1	3	3
Y (%)	3.8	1.9	4.4	3.2	0.6	3.5	3.8	3.5	0.8	3	3.3	2.9	3

8.2 Residual Drifts

Residual story drift is an important demand measure as it can have major implications on whether a building can be reoccupied and repaired after an earthquake. The residual drift of 3 corners of the building in addition to the center of mass (CM) were obtained (Fig. 12). Since ASCE 41-17 does not impose a maximum limit for the residual displacements, we referred to Tall Building Initiative (TBI) v2.03 [7] to obtain the allowable displacement limits. According to the results, the mean residual drifts in the existing conditions exceed the maximum allowable (1%) by 88% in the Y-direction and 53% in the X-direction.

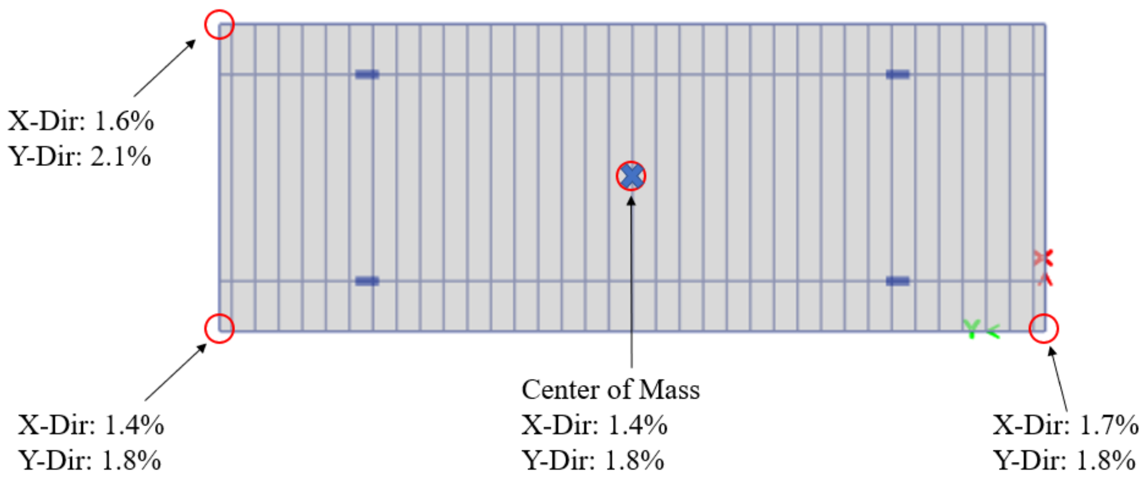

Fig. 12 - Residual Drifts in existing conditions

9. Retrofit

The shear demand of the girders was obtained from the 11 applied ground motions to check the girders conformance to ASCE 41-17. As shear in structural elements is considered a force-controlled action, ASCE/SEI 41-17, Table 7-8, specifies an amplification factor for force-controlled elements. For that purpose, the seismic shear force was multiplied by a factor of 1.5 to satisfy the Life Safety limit state. According to ASCE/SEI 41-17, Table 10-3, the steel shear capacity must be greater than ¾ of the design shear in order for the beam to be considered compliant. Based on Fig. 13, in the plastic hinges region, a shear deficiency was identified where three-quarters of the design shear demand exceeded the steel shear capacity of the girder. Hence, the shear capacity of the girder at the support must be increased to meet the maximum seismic shear demand (¾ Life Safety Demand Shear).

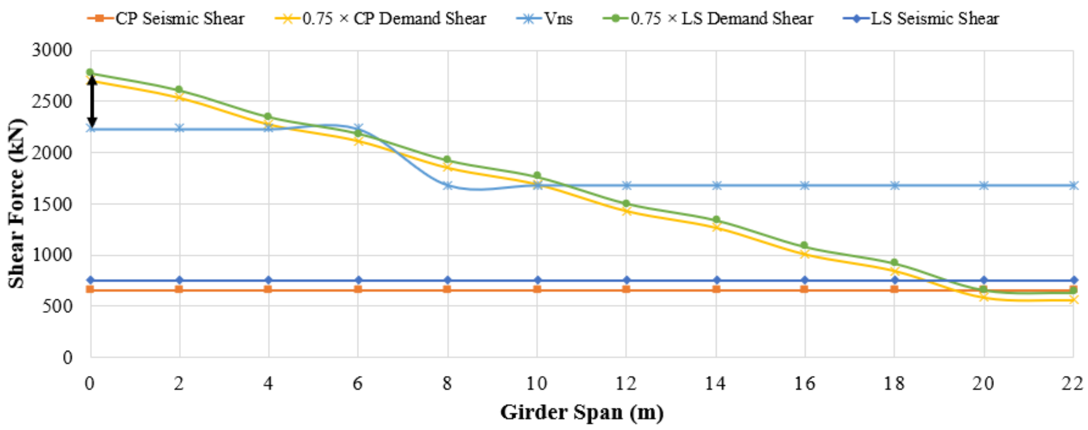

Fig. 13 - Variation of shear forces along girder span

Note that in Fig. 13, Vns is available shear capacity due to mild and prestressing steel in kN.

Carbon Fiber Reinforced Polymer (CFRP) was proposed to increase shear capacity for girders by wrapping the girder section to enhance resistance against seismic movement. FRPs have emerged as an alternative to traditional materials for repair and rehabilitation. Their well-defined material properties, high strength-to-weight and stiffness-to-weight ratios, and resistance to electrochemical corrosion as well as their easy handling make FRP material superior to other conventional materials in strengthening applications. The FRP design was performed on following ACI 440.2R-08 [8]. As a result, two FRP wraps were required to address the shear deficiency in the girders. With the retrofit, the combined shear capacity due to mild, prestressing and FRP reinforcement exceeded the ¾ demand shear showing that the retrofitted girders are conforming.

After applying the girder retrofit, the shear capacity and flexural ductility will increase. The building model was updated and re-analyzed to recheck the inelastic deformation demands and compare them to the acceptance criteria. According to Fig. 14, the hinge rotations in the 11 ground motions decreased by 84% on average. The mean girder hinge rotations passed the CP and LS limits.

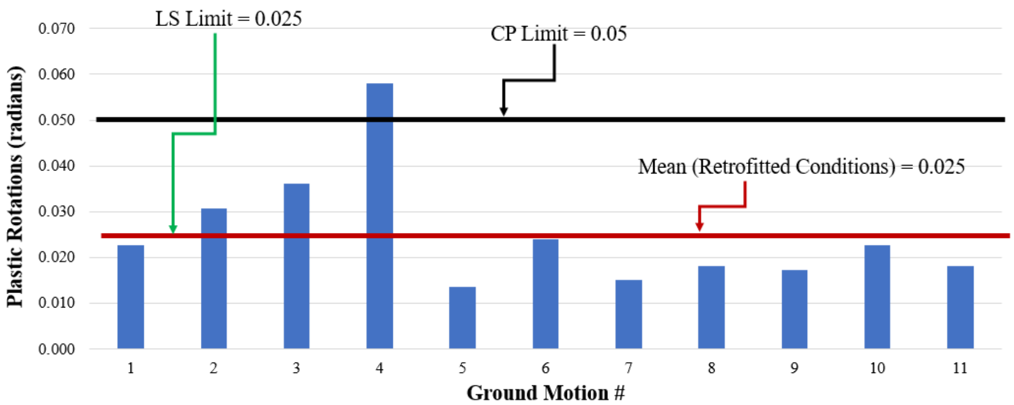

Fig. 14 - Girder rotations post-retrofit

Additionally, a significant decrease in the Y-direction drift was observed. The average drift decreased by approximately 100%. The drifts do not exceed the allowable drift limit (3%). The residual displacements were checked for the building retrofitted conditions at the corners and center of mass in both directions. The residual drifts were calculated and compared to maximum allowed. The residual drifts in the Y-Direction complied with the acceptance criteria where the drifts did not exceed 1%. No changes were recorded for the X-Direction.

10. Conclusion

In this paper, we discuss the seismic evaluation of the Grand Cover Structure. Grand Cover is located in Rashid Karami International Fair, Tripoli, North of Lebanon. The fair was accepted in the year 2018 on the UNESCO World Heritage tentative list, so it is important to assess the safety of this landmark structure. Since most of the structural and architectural data were missing, several site visits were performed to collect data and make field measurements, and non-destructive testing was conducted to determine concrete compressive strength using Schmidt Hammer, and embedded steel reinforcement using rebar scanner. The structure was analyzed under gravity and seismic loads using linear static and dynamic loads. The results obtained were very conservative and predicted that most of the structural elements will fail. ASCE 41-17 was used to build the nonlinear model and define the characteristics and parameters of hinge components. Link elements were utilized in the model due to their better capabilities compared to hinges. The links were assigned to the structural elements that were expected to yield and a nonlinear pushover analysis was conducted where an increasing lateral load was subjected to the structure. The results of the pushover analysis indicated that beams and girders were failing. So, we proceeded with Nonlinear Response-History Analysis, where a set of 11 ground motions were selected that best match Tripoli's design spectrum and applied to the structure. According to the Time-History Analysis results in the existing condition, only girders in the Y-direction needed retrofit to meet the Life Safety (LS) limit state. CFRP was chosen as the most appropriate retrofit solution because it allows the girders to meet the acceptance criteria of ASCE/SEI 41-17, and can be installed with minimal disruption and modification to the structure. The re-analysis and model of the retrofitted structure showed that columns, beams and girders were generally compliant with the Life-Safety damage state limit. Nonlinear Time-History analysis was very effective in estimating nonlinear demands and identifying the most critical components for retrofit, and in reducing the scope and cost of the retrofit. Furthermore, nonlinear time-history accurately estimated the story drift in both directions and the drifts results complied with the acceptance criteria. Using nonlinear analysis, we were also able to compute the residual drifts and check its compliance.

11. Acknowledgements

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