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Seismic Retrofitting Study on an Industrial Building in Aqaba - Jordan

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ABSTRACT

This paper presents a seismic retrofitting study conducted on one of the structures at the Industrial Complex in Aqaba. The study aimed at checking the current capacity of the structure in resisting seismic forces due to some considered earthquake events according to recognized codes of practice, as well as the findings of a specially commissioned study concerned with the seismic hazard at the site. A 3-D finite element model of the structure was developed and "Time History" dynamic structural analysis was carried out. The 1995 Aqaba earthquake was utilized in the analysis. The model was verified by comparing the analysis results with the actual damage sustained during the earthquake.

Moreover, the study discusses different options for retrofitting, particularly the preferred option of adding shear walls. The study involved carrying out dynamic analysis using "SAP2000 non-linear" computer software on a three-dimensional model of the original as well as the retrofitted structure.

KEYWORDS: Aqaba earthquake, Peak ground acceleration, Seismic retrofitting, Structural analysis, Shear walls, Time history.

INTRODUCTION

During the past decade, Jordan experienced several earthquakes. They ranged from moderate to strong ones, such as the 6.2 degrees on Richter scale- November, 1995- Aqaba quake. The effect of these earthquakes on different structures varies according to structure type, system, age and ability to resist excessive earthquake forces (Abdel-Halim and Al-Tarazi, 2004).

Many reinforced concrete buildings have either collapsed or experienced different levels of damage during past earthquakes. Many investigations have been carried out on buildings that were damaged or ruined by earthquakes (Moehle, 2000).

Low-quality concrete, poor confinement of the end regions, weak column-strong beam behaviour, short column behaviour, inadequate splice lengths and improper hooks of the stirrups were some of the important structural deficiencies. Most of those buildings were constructed before the introduction of modern building codes. They usually cannot provide the required ductility, lateral stiffness and strength, which are definitely lower than the limits imposed by the modern building codes. Due to low lateral stiffness and strength, vulnerable structures are subjected to large displacement demands, which cannot be adequately met, as they have low ductility (Moehle, 2000; Armouti, 2003).

Nowadays, most of the strengthening strategies are based on global strengthening schemes as per which the structure is usually strengthened for limiting lateral displacements in order to compensate low ductility (Jirsa and Kreger, 1989). In these schemes, global behaviour of the system is transformed. Another

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approach is modification of deficient elements to increase ductility, so that the deficient elements will not reach their limit state conditions when subjected to design loads (Al-Dwaik and Armouti, 2013). However, the latter strategy is more expensive and harder to implement in cases of many deficient elements which is the reason for that global strengthening methods have been more popular than element strengthening.

Among the global strengthening methods, addition of RC shear walls is the most popular one. Many researchers have focused on this subject and found that installation of RC shear walls greatly improves lateral load capacity and stiffness of the structure (Albanesi et al., 2006). Even in cases of application to damaged buildings, this method yields satisfactory results (Canbay et al., 2003). In some other researches, the use of wing walls, attached to two sides of columns was investigated. The systems strengthened with wing walls exhibited ductile behaviour (Higashi et al., 1982; Bush et al., 1991).

Steel bracing for RC frames has also been used to reduce drift demands. Bracing can either be implemented inside the frame (Masri and Goel, 1996) or applied from outside the system (Bush et al., 1991). Post-tensioned steel bracing is also an efficient alternative for vulnerable framed buildings (Gilmore et al., 1996) and it compensates structural irregularities. Experimental results for another alternative, knee bracing with shear links replaced with masonry infills, lead to improvement in energy absorption capacity (Perera et al., 2004).

Although each of these methods satisfactorily increased strength and stiffness, all of them with the exception of external steel bracing require construction work inside the building, which means disturbance of users and results in the buildings being out of service. Consequently, research efforts in this field have shifted their focus to new methods that could overcome this difficulty (Frosch et al., 1996; Baran, 2005).

Some other researchers perpendicularly installed RC shear walls outside the building (Kaltakci et al., 2008). This kind of shear walls was also applied to strengthen structures with an external diaphragm at the roof level (Kaplan et al., 2009). This method has increased lateral load capacity and strength of the structure as well.

The literature review presents numerous strengthening techniques. However, most of them require long-term construction works inside the building, rendering the building out of service for that period of time. On the other hand, external strengthening techniques offer advantages with respect to cost and ease of construction (Sucuoglu et al., 2006).

In recent years, non-linear static analyses (ATC, 1996; FEMA, 1997) have received a great deal of research attention within the earthquake engineering community. Their main goal is to describe the non-linear capacity of a structure when subjected to horizontal loading with a reduced computational effort with respect to non-linear dynamic analysis. Pushover methods are particularly indicated for assessing existing structures (Ferracuti et al., 2009).

Sonia et al. (2012) checked common software SAP2000 in non-linear analysis of retrofitting flat slab buildings. To analyze the retrofitting building methods, it is necessary to have software where the analysis of these structures can be made.

Pushover analysis was performed on a nineteen storey concrete building with shear wall lateral system and certain unique design features. Utilizing the results from this analysis, some modifications were made to the original code-based design, so that the design objective of life safety performance is expected to be achieved under design earthquake (Rahul et al., 2004).

Marco et al. (2010) compared the results obtained from the test program and finite element analyses using two programs: SAP2000 and SeismoStruct. The results of comparison showed that it is possible to get good accuracy of the highest load that an RC frame can reach through pushover analysis in SAP2000 or in SeismoStruct.

The primary objective of this study is to evaluate the seismic capacity of the structure under consideration, which has been designed before 1980 (earlier codes before the new seismic regulations), and, if necessary, to propose an appropriate repair and seismic retrofitting scheme to resist future seismic events. The magnitude of expected future events would be based on historical events in the Aqaba Gulf region and on statistical methods, as well as on soil conditions and foundation type pertaining to the structure.

This was an owner - option - seismic retrofitting of the industrial structure. Therefore, maintaining the architectural appearance of the structure, the earthquake response performance of the structure and cost were primary considerations in establishing the retrofit design.

In this study, analyses have been performed using SAP2000 which is a general purpose structural analysis program for static analysis and dynamic analysis of structures.

The study includes inspection, documentation, evaluation of the structure, as well as devising a strengthening technique to withstand future events.

OBJECTIVES

The technical study conducted aimed at:

a. Evaluating the seismic capacity of the structure and defining the main deficiencies by adopting static and dynamic analysis.
b. Evaluating different up-grading/strengthening techniques.
c. Devising the most adequate up-grading/strengthening technique and furnishing the structural details and tender's documents.

SCOPE OF WORK

The work involved inspecting the columns, beams and slabs which are the main structural elements, taking necessary measurements, observations by photographs and sketches. The below mentioned procedure was followed during the course of this investigation:

a. Reviewing the available engineering "As Built" drawings, design calculations and the geotechnical report.
b. Conducting a preliminary survey of the structure to check compliance between "As Built" drawings and construction on site, as well as checking the dimensions of the structure and its components, in addition to the openings in the slab, location of the fixed machinery such as the filter, crane, pipes,… etc.
c. Inspecting the present structural condition of the building. The inspection included crack mapping of the columns taking into account crack size, location, continuity and penetration.
d. Performing dynamic structural analysis on the structure after modeling by using SAP2000 non-linear computer program. The following steps were included in the analysis process.
    - Conducting time-history dynamic analysis utilizing the acceleration record of the November1995 Aqaba quake (east/ west component), as recorded at the Aqaba hotel strong motion recording station (NRA, 1995). The analysis was carried out with peak ground acceleration (PGA) on the record scaled to 0.11g, 0.156g and 0.3g.
    - Reanalyzing the structure utilizing time-history of the Nov. 1995 Aqaba earthquake (east/west component) with PGA of 0.175g after modifying the model of the structure to up-grade its resistance to future expected events.
e. Verifying the accuracy of the main results obtained from the computer analysis. This was achieved by comparing the results obtained from SAP2000 program with those calculated manually.
f. Checking column capacity, column ductility, confinement, reinforcement details and lap splices in accordance with the provisions of Chapter 21 of the ACI318M (ACI, 1999) and the Uniform Building Code.
g. Checking beam capacity and reinforcement details in accordance with the above mentioned standards,

and the British Standards (BS, 1984).

h. Checking joint capacity and reinforcement details in accordance with the above mentioned standards.

PROJECT DESCRIPTION

General Site Description

The structure is one of the industrial complex structures that are located on the southern shore of Aqaba city. The complex comprised a number of structures with different dimensions, materials and functions. Reinforced concrete and steel are the main materials used in the construction of these structures. Some of the structures come under the effect of regular vibrations due to the nature of the production process. In addition, accumulation of dust on surfaces and spillage of liquids (water or chemicals) appear to be a daily exposure condition in the structure under consideration.

Site Investigation

The "As Built" drawings, the original calculation sheets plus the soil test report were checked after a preliminary reconnaissance and detailed inspection of the structure at site. The inspection included checking all the structural elements for signs of distress and damage, if any. Moreover, columns, beams and slabs were checked for cracking, spalling… etc. Generally, no serious signs of damage or distress were observed, except that the short columns exhibited some signs of distress "cracking" that have been reported.

Building Description

- The structure is a 2-storey cast *in situ* reinforced concrete skeleton of 12 m height - superimposed by steel super structure of 14 m height. So, the total height of the structure is 26 m, as illustrated in Figure (1).

Figure (1): The industrial structure

- The total area in plan for each storey is (34m x 28m). The structure was founded on sand with some fine gravel deposit of bearing capacity in the range of (1.75 - 3.0) kg/cm^2.

- Foundations, 2 m below the ground level, consist of separate (isolated) column footings. There are no infill walls in the first and second stories; whereas the upper storey is covered with steel sheets (cladding).
- In the long building side (X-direction), the lateral load resisting system consists of 5 frames; whereas it consists of 7 frames in the other directions.

The salient aspects of the structure that were taken into consideration during analysis and evaluation were as follows:

- Isolated footing at level -2.0 from the ground.
- Combined footings along G-axis, as shown in Figure (2).

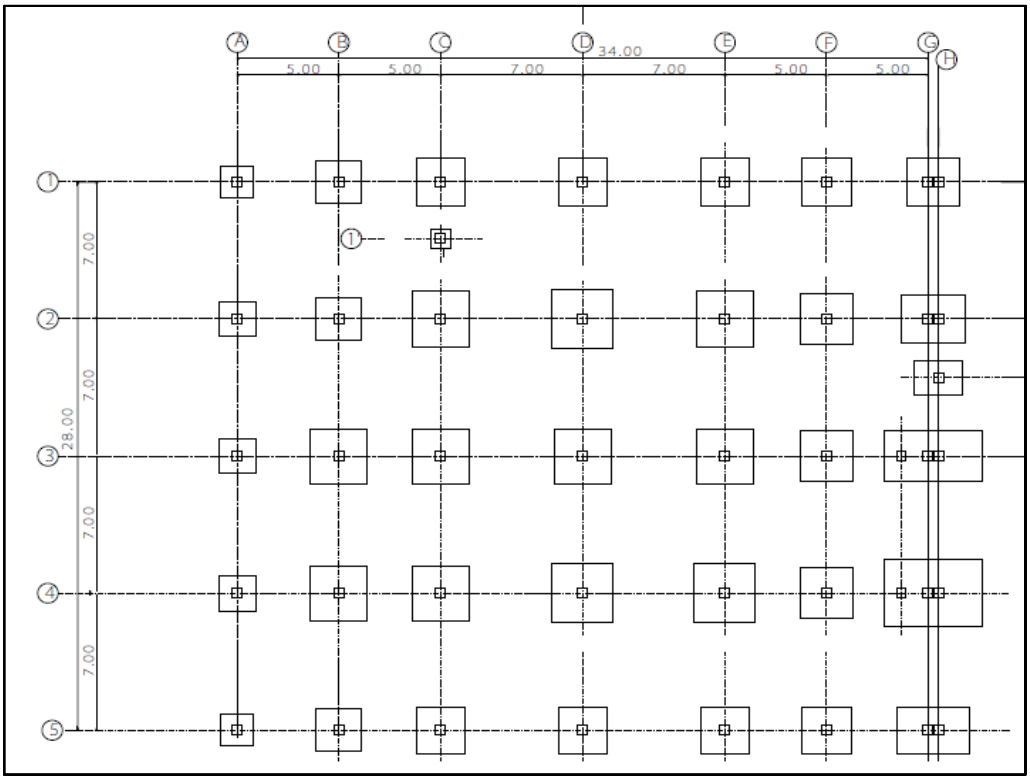

Figure (2): Footings at level - 2.00

- Expansion joint of 20 mm separating the structure and another building, as illustrated in Figure (2).
- No basement is available.
- No tie beams are available between column necks.
- Ground slab at level + 0.30, based on fill materials.
- Solid slab with openings at level + 6.80 with corners thickened (thickness = 700 mm) forming a base for 14 implanted concrete columns, as shown in Figure (3).
- Concrete ring beam at level + 10.58, setting on the implanted columns, as shown in Figure (4).
- Solid slab at level + 11.80, with openings as shown in Figure (5).
- The special provisions for resisting earthquake loading were not considered in the original design.
- Generally, no serious signs of damage or distress were observed. Nonetheless, site inspection revealed that the short columns, as illustrated in Figure (6), exhibited some signs of distress "cracking".

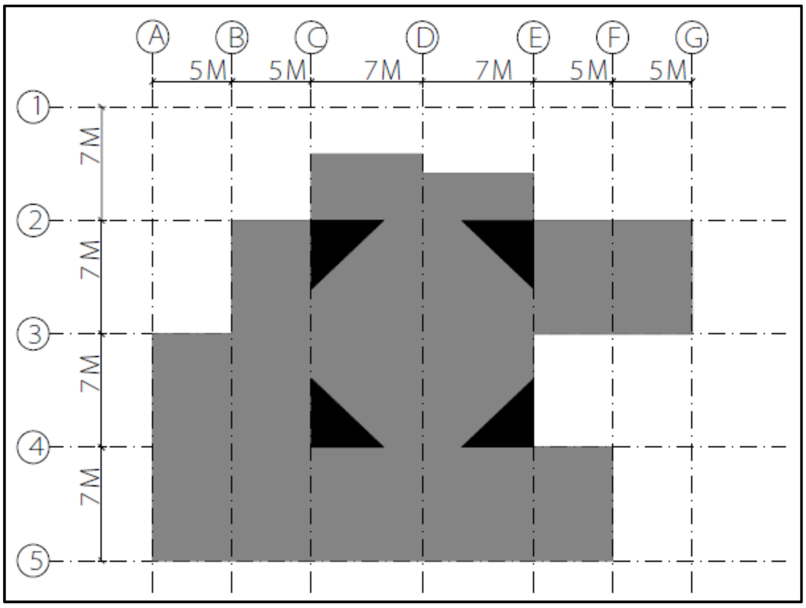

Figure (3): Solid slab at level + 6.80

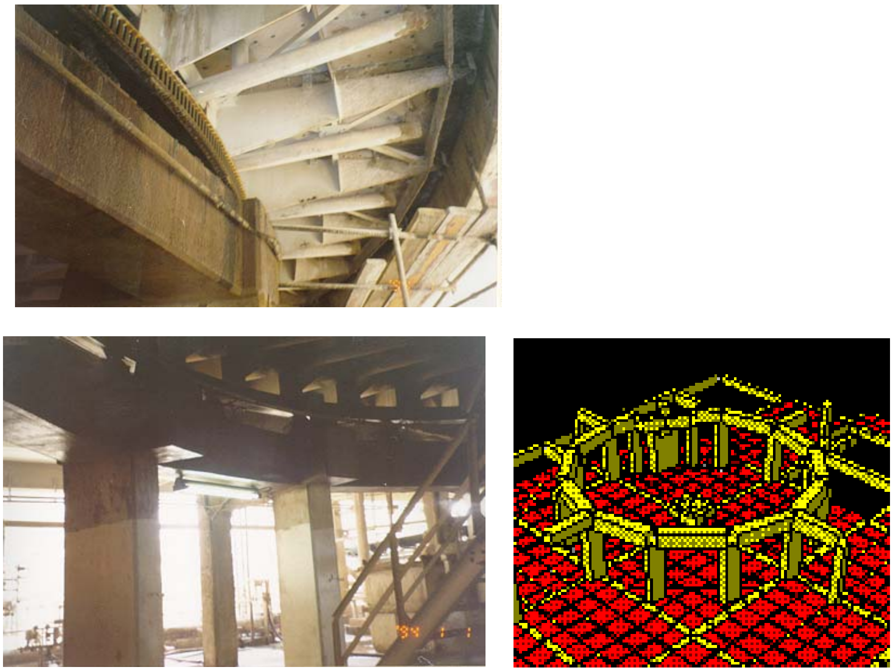

Figure (4): Ring beam at level + 10.58

Structural Analysis

To study the behavior of the structure under the effect of seismic loading, it was necessary to carry out dynamic structural analysis (Derecho, 1989; Syal and Ummat; Tanvir Wasti). This was accomplished by creating a finite element model, as shown in Figure (7) and thereafter carrying dynamic structural analysis using SAP2000 computer software. The model consisted of beams and columns, which had been represented by frame elements, and slabs that were meshed into a series of shell elements. The model reflected the geometric characteristics of the building, as well as its boundary conditions, material properties, relative stiffness, mass distribution, energy dissipation "damping", dead loads and live loads,… etc**.**

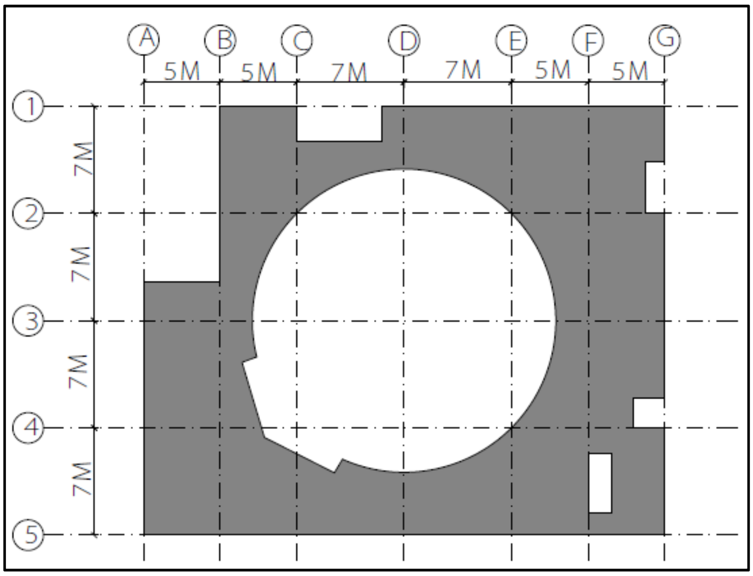

Figure (5): Solid slab at level + 11.80

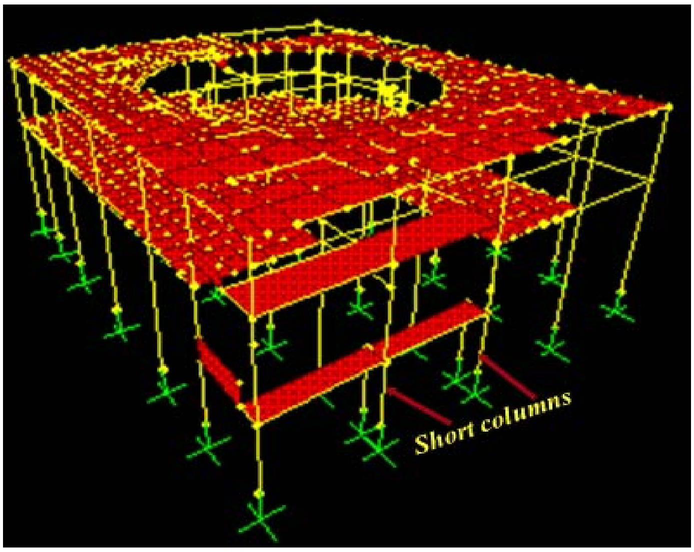

Figure (6): Short columns

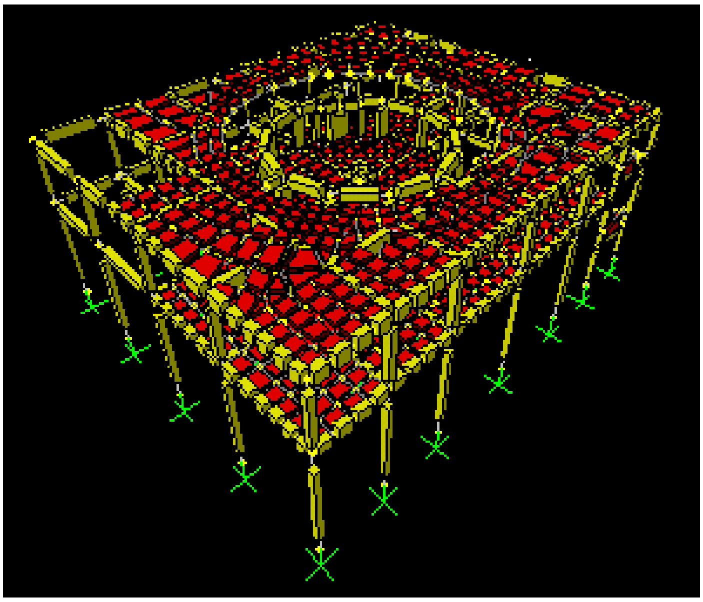

Figure (7): Finite element model of the original structure

Ground Motion

A seismic hazard study was conducted (Malkawi and Fahmi, 1996) for the area of concern, using the time-history (TH) acceleration of the Nov. 1995 Aqaba earthquake as recorded by the strong motion recording station at the Aqaba hotel, on the north beach of Aqaba city. The same TH was used together with borehole logs at the Aqaba hotel site to calculate the TH at bedrock; hence an attenuation equation was used to calculate the PGA at bedrock in the Industrial Complex south of Aqaba city. Then, the TH at ground level was calculated utilizing the borehole "Soil" information at the building location and the TH at bedrock at the Aqaba hotel modified accordingly. The SHAKE 91- computer software was utilized in the course of the study.

According to this study, the computed peak horizontal acceleration at the site during the Nov. 1995 Aqaba earthquake was equal to (0.11) g, while that at the site of the Aqaba hotel station was equal to (0.156) g. For the design life of the building and based on the probabilistic hazard assessment study conducted for the site considering (90) % probability of non-exceedance in (100) years lifespan, the maximum PGA at the ground level could reach (0.3) g.

Seismic analysis of the structure to check its adequacy and its need for structural up-grading was based on the TH of the east/west acceleration of the 1995 Gulf of Aqaba earthquake as recorded by Aqaba hotel station scaled to PGA of (0.11g, 0.156g and 0.3g), as illustrated in Figure (8). Three angles of excitation (0º, 45º and 90º) were used in the analysis to account for the directional uncertainty of earthquake motions.

Verification of Analysis Results

Verifying the accuracy of the main results obtained from the computer analysis was achieved by comparing some key results obtained from SAP2000 program output with those calculated manually. Moreover, the correlation between the resultant forces from the analysis and severity of damage documented was used to verify the adequacy of the finite element model and analysis approach. The verification of the fundamental results is presented below.

Comparing No. of Equations of Equilibrium with No. of Degrees of Freedom

To verify the equilibrium of the model, no. of equations solved by the program must equal no. of degrees of freedom.

No. of joints = 1057.

No. of restrained joints = 38.

No. of master joints = 1.

No. of body constraints = 6.

$\therefore$ No. of degrees of freedom = 6(1057-38-6+1)=6084.

Given from SAP2000 output

Total no. of equilibrium equations is = 6084.

$\therefore$ Structure is stable.

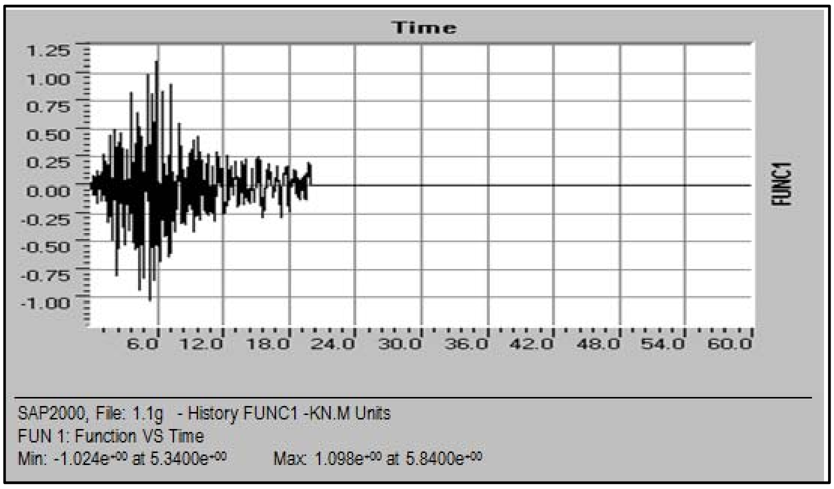

Figure (8): TH of the east/ west acceleration of the 1995 Gulf of Aqaba earthquake with PGA of 0.11g

Total Self-Weight of the Filtration Unit

To verify the analysis results, the self -weight of the structure was manually calculated taking into account its components' dimensions. The dead loads for all columns, beams and slabs were calculated. In addition, the weight of the permanent fixed components, such as the filter, super steel structure, fixed tanks,… etc. was added to the calculated weight. The weight of the unit volume in all calculations was taken as 24.0 kN/m^3.

The grand total of the dead load of the above mentioned elements was found to be 25729.639 kN, as calculated by hand. This result was compared with its equivalent one obtained from computer analysis of 25242.425 kN. The comparison indicated that the two values have good correlation with an acceptable percentage of deviation of 1.89%.

Natural Period of Vibration

The natural period of vibration was calculated manually by (a) Rayleigh's equation and (b) Simplified structural dynamics principles and compared to the result of SAP 2000 output.

The comparison indicated that the two values have good correlation with an acceptable percentage of deviation of 1.436% for case (a) and 0.645% for case (b).

Comparison between Actual and Expected Damage

The results of the TH analysis and the capacities of columns and beams showed that the structure should have been able to sustain axial and shear forces and moments from the Nov. 1995 Aqaba quake without showing any sign of distress, except the 4 short columns which have been deficient in shear and moments. This was also confirmed by the results of the structural inspection mentioned earlier, where these columns had been cracked.

Hence, the accuracy of the model and the analysis carried out is implicitly verified.

As a general conclusion, such verification of the results confirms that the computer analysis produced fairly accurate outcome, which can be used for the objectives of this investigation.

RESULTS AND DISCUSSION

The detailed inspection of all the structural elements showed that these elements in general did withstand the Nov.1995 Aqaba earthquake (6.2 on the Richter scale) with PGA of about (0.11) g at the site. Only minor damage occurred in the form of cracking of some short columns.

The ultimate resistance capacities of the framing elements; namely, the columns, beams, slabs and foundations were checked against both vertical and lateral loading using the envelope values of the combined dead, live and seismic forces. Different levels of PGA (0.11g, 0.156g and 0.3g) at three different angles of excitation (0º, 45º and 90º) were considered. All the framing sections were checked for compression, tension, bending moments and shear resistance.

The check demonstrated that the beams and slabs have enough capacities to resist all the loads to which they would probably be exposed during a future earthquake event, but the beams are much stronger than the columns, thus violating the "strong columns-weak beams" design requirement for earthquake resistance (Moehle and Nicoletti, 1994). The columns themselves were deficient in resisting shear forces and bending moments generated by the TH of PGA of (0.11) g with angles of excitation different from (0º), as illustrated in Figure (9). For PGA of (0.156) g and (0.3) g, most of the columns were deficient for all angles of excitation. Furthermore, the footings were deficient regarding to stability and reinforcement considerations.

The second issue to be addressed was drift; i.e. the lateral displacement of the building. It was found that the structure under consideration would sway by a maximum value of (21) mm when subjected to Aqaba earthquake TH scaled to PGA of (0.11) g with an angle of excitation of 0º, notwithstanding the sway of the adjacent building, and by a value of about (58) mm if Aqaba earthquake TH was scaled to (0.3) g with an angle of excitation of (0º). As a result, pounding between the building and the adjacent structure separated by (20) mm is very likely to occurr.

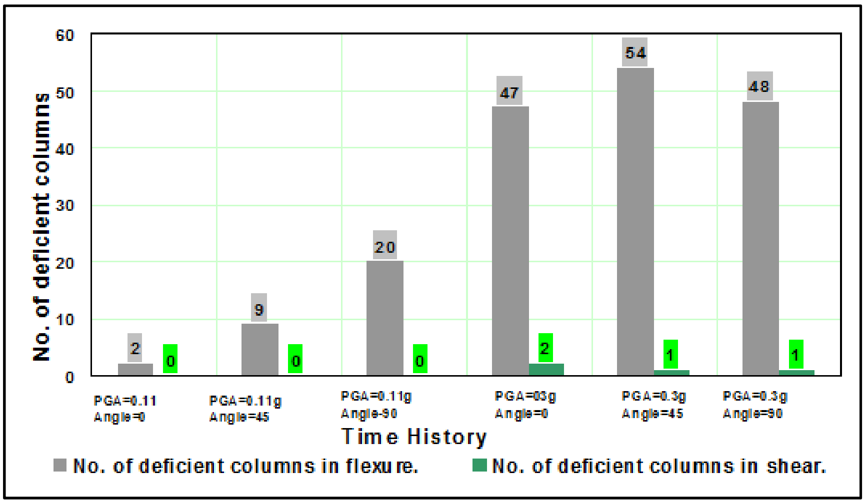

Figure (9): Columns deficient in shear and flexure

The third issue to be addressed was the detailing of reinforcement. The longitudinal steel for the short columns was less than the code requirement. Transverse reinforcement of the columns was found unsatisfactory. Spacing and detailing of the transverse reinforcement were not adequate to confine the concrete, and the locations of lap splices were at the ends of columns (critical locations of possible inelastic action) and not in the middle third as required by the code.

As a conclusion, the building which has been designed as a gravity-load bearing structure might not perform well under earthquake loading, especially if the peak horizontal ground acceleration exceeded (0.11) g. Therefore, the building would be better up-graded to be able to resist future earthquake events.

Retrofitting

Retrofitting and strengthening of the building was necessary to:

a. Reduce the internal forces and effects generated from earthquakes on the deficient columns. The most efficient scheme to perform such a task is adding shear walls.
b. Reduce lateral displacement (sway) of the building, especially in the long direction (X-direction), to avoid pounding with the adjacent structure. Again, shear walls may prove to be the most efficient method to achieve this goal.
c. Remedy the deficiencies in reinforcement details (violations of code provisions), such as lack of confinement.
d. Mitigate the absence of tie beams between the isolated footings, since one of the most important pre-requisites of adequate performance of the building during an earthquake is the provision of a foundation system which would enable the base of the building to move as a unit and thus forbid the differential movement of the column bases relative to each other.

Proposed Strengthening Technique

To strengthen the building to resist possible future earthquake loading, it was necessary to provide a solution for the whole structure; i.e., provide a new lateral-load resisting system. Therefore, it was decided to construct shear walls of reinforced concrete at chosen locations at the periphery of the building for maximum efficiency and to avoid the difficulties of working on the inside area. After several computer analyses of different shear wall configurations, the shear wall arrangement shown in Figure (10) was deemed to be the most efficient one. New computer analyses were carried out on a modified model of the structure as shown in Figure (11) including the suggested shear walls utilizing the acceleration record of the Nov.1995 Aqaba earthquake scaled to PGA of (0.175) g (design earthquake) based on (50)% probability of non- exceedance in (50) years lifetime of the building.

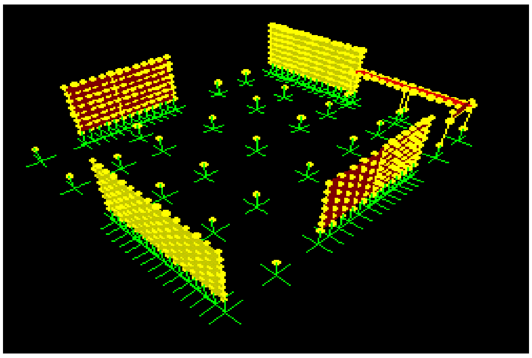

Figure (10): Locations of the proposed shear walls

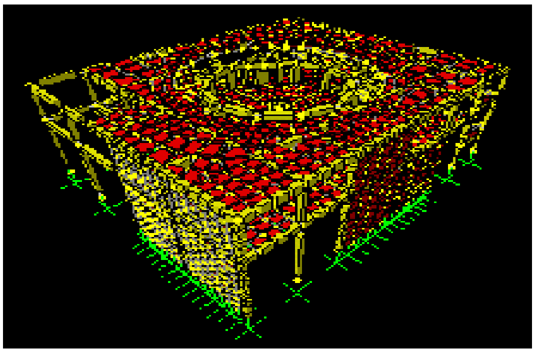

Figure (11): Finite element model of the retrofitted structure

The choice of PGA 0f (0.175)g was based on NEHRP Guidelines for the Seismic Rehabilitation of Existing Buildings (FEMA, 1997) and the seismic hazard study at the site of the industrial complex. Since the structure under consideration was an existing building constructed in 1980, the PGA at the ground level can be based on 50% probability of non-exceedance in 50 years lifetime exposure. According to the hazard study, the aforementioned design earthquake had a PGA of 0.175g. Therefore, the Nov. 1995 Aqaba earthquake acceleration TH was scaled to 0.175g.

Moreover, the maximum base shear force that corresponds to an earthquake having PGA of 0.3g was found to be (8327) kN by dynamic analysis, exceeding that calculated by the equivalent static method (5225 kN) being the absolute minimum required by UBC Code, which further supports the reduction of the design (retrofitting) earthquake to 0.175 g. The structure should be able to resist similar earthquakes with PGA of 0.175g elastically, whilst it should be capable of resisting earthquakes with higher magnitudes plastically.

Dynamic analysis of the structure was carried out after modifying the model to reflect the effect of adding the shear walls, hence running the analysis again and checking the response of the whole structure and its components. The existing slabs, beams, columns and foundations were evaluated to ascertain that they have sufficient strength to withstand the design earthquake elastically (PGA of 0.175g).

The results indicated that the columns became capable of resisting the maximum factored forces, especially bending moment forces, which were reduced to very small values as the structure became braced. The structure as a whole became stiffer with a natural period of vibration of (0.21) sec instead of (1.124) sec (original structure) as shown in Figures (12 and 13), and the sway was reduced to (3.5) mm.

The resistance capacities of the beams around the openings in the slab were also checked, and the results indicated that they were capable of transmitting lateral loads and thereby effectively acting as rigid links.

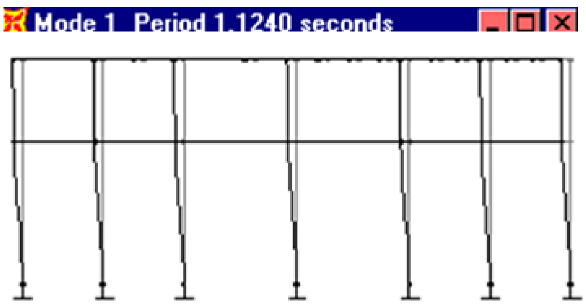

Figure (12): Natural period of vibration for the original structure

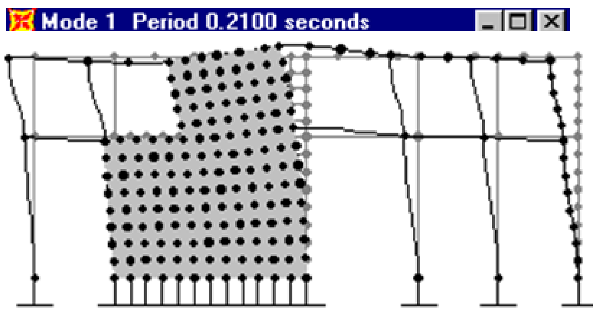

Figure (13): Natural period of vibration for the retrofitted structure

CONCLUSIONS

In light of the investigation results, the following conclusions can be drawn.

a. Detailed dynamic structural analysis was performed to evaluate the structural condition of the structure under consideration. Computer software SAP2000 non-linear program was used for the dynamic analysis using time history representation of the recorded 1995 Aqaba earthquake with different PGA values (0.11g, 0.156g and 0.3g) and with different angles of excitation for checking the capacity of the original structure.

b. Based on the analysis results, a capacity check was performed for all columns and beams. This check showed that the columns which are the main items in the lateral load resisting frame structure, with the exception of the short columns, were capable of resisting the force induced by the Aqaba quake of PGA 0.11 g with an angle of excitation = 0°.

c. Strength capacity of the columns was found deficient in resisting forces induced by Nov. Aqaba quake of 0.11g with angles of excitation different from 0°, such as 45° and 90°.
d. Strength capacity of the columns was found deficient in resisting forces induced by Nov. Aqaba quake scaled to higher values of PGA such as 0.156g and 0.3g for all angles of excitation.
e. The beams are much stronger than the columns in resisting the moments and shears, which leads to “strong beam - weak column” formation, in violation of code requirements for earthquake resistant design. This condition generates plastic hinges in the columns instead of the beams, which may threaten the stability of the whole structure.
f. Reinforcement detailing of the columns according to UBC Code provisions is inadequate, which contributes to the deficiency of their ductile behaviour.
g. Dynamic analysis was performed reflecting the proposed strengthening work necessary for the industrial building. The results proved that adding structural walls could overcome the problem of inadequacy in resisting maximum moments in the long slender columns, since the structure became a braced one with negligible sway effects. The structure became stiffer with a natural period of vibration of 0.21 sec instead of 1.124 sec (original structure) and the sway was reduced to 3.5mm.

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