The Islamic University Journal (Series of Natural Studies and Engineering)
Vol.19, No.2, pp 37-56 , **2011**, ISSN 1726-6807, http://www.iugaza.edu.ps/ar/periodical/

ASSESSMENT AND STRENGTHENING OF A SIXTEEN STOREY RC BUILDING DAMAGED BY AIR ATTACK

Samir. M. Shihada[1] and M. Al-Jerjawy[2]
[1]Department of Civil Engineering, The Islamic University of Gaza, Palestine
E-Mail: sshihada@mail.iugaza.edu.ps
[2]Department of Civil Engineering, The Islamic University of Gaza, Palestine

Abstract: During the 2008-2009 Israeli aggression against Gaza Strip, about 4,100 residential units were completely destroyed and another 17,000 units suffered partial destruction. One of the damaged main buildings was a 16-story residential tower, which was attacked from air by a large number of missiles, causing a variety of structural damages. This work aims at assessing the structural damages incurred to the building. Visual inspection and structural modeling using ETABS are used to determine whether the building is to be demolished or repaired/ strengthened. Based on this, proposed rehabilitation schemes are set out. The results indicate that the building can be repaired by application of specified rehabilitation techniques, which are expected to ensure the structural safety of this building for current and future loading.

Keywords: Assessment; Strengthening; Jacketing; Model; Explosion

تقييم وتقوية مبنى ذو 16 طابقاً من الخرسانة المسلحة تعرض لقصف جوي

ملخص: خلال حرب عام 2008-2009 والتي شنها الإسرائيليون على قطاع غزة تعرضت 4100 وحدة سكنية للدمار الكلي و 17000 أخرى لأضرار جزئية. أحد المباني الرئيسية التي تضررت بسبب القصف الجوي بعدد كبير من الصواريخ كان برجا سكنيا مكونا من 16 طابقا من الخرسانة المسلحة مسببا أضرارا إنشائية مختلفة. ولهذا الغرض تم الإعتماد على الفحص البصري ونماذج التحليل الإنشائي لتحديد ماإذا كان المبنى يستدعي الإزالة أم بالإمكان إصلاحه وتقويته. هذا العمل يهدف إلى تقييم الأضرار الإنشائية الناتجة، وبناء على ذلك سيتم عرض أنظمة إعادة التأهيل المقترحة. النتائج أظهرت أن المبنى يمكن إصلاحه بإستخدام طرق تأهيلية مخصصة تضمن سلامة المبنى للأحمال الحالية والمستقبلية.

1- Introduction:

An explosion is defined as a large-scale, rapid and sudden release of energy. The detonation of a condensed high explosive generates hot gases under pressure up to 300 kilo bar and a temperature of about 3000-4000C°. The hot gas expands forcing out the volume it occupies. As a consequence, a blast wave forms in front of this gas volume containing most of the energy released by the explosion [1]. The pressure generated from explosive blast will weaken the structure and subsequently could cause structural failure. However, sometime damaged structures could be repaired or rehabilitated. Thus, an assessment method is required in order to identify and plan for the rehabilitation process. Such method is important to ensure all data is recorded in detail [2].

In general structural assessment is a process to determine, how reliable the existing structure is able to carry current and future loads. The first step of the assessment process must always be the clear specification of the assessment objective. There are two main objectives to conduct assessment of existing structures, the assurance of structural safety and serviceability and the minimization of costs. In general assessment procedures can be classified into three groups: measurement based assessment, model based assessment and non-formal assessment [3].

Restoration is the restitution of the strength the building had before the damage occurred. The main purpose of restoration is to carry out structural repairs to load bearing elements. It may involve cutting portions of the elements and rebuilding them or simply adding more structural material so that the original strength is more or less restored. The process may involve inserting temporary supports, underpinning, etc., [4].

Strengthening a RC element may be defined as an intervention to increase the original strength and stiffness of the RC element. In the case of a damaged and/or deteriorated RC element, strengthening must be associated with structural repair. The strengthening process must be preceded by the repairing operation [5]. The strengthening of reinforced concrete members is a task that should be carried out by a structural engineer according to calculations. The decision to rehabilitate must be made only after the inspection of the structure, its structural assessment and a cost/benefit study of the different solutions. Rodriguez and Park [6] published extensive bibliographic research on the repair and strengthening of RC structures in seismic areas. According to the authors, some buildings in Mexico City, repaired and strengthened after the 1985 earthquake, had a value between three and four times the operation cost. Basically the strengthening techniques for reinforced concrete structures can be divided into addition of new structural elements and strengthening of the existing structural elements [7]. Ramirez et al. [8] published an experimental study on repair of RC columns, where an interesting distinction between cosmetic repair and structural repair is introduced. The authors consider cosmetic repair if the strength loss is lower than 10% and structural repair if the strength decrease is above that value.

Aguilar et al. [9] performed a statistical study on the repair and strengthening methods of 114 RC buildings damaged after the 1985 earthquake in Mexico City. The authors stated that the most commonly used techniques are the addition of shear walls and the RC jacketing of columns. The RC jacketing method, unlike other techniques, leads to a uniformly distributed increase in strength and stiffness of columns and does not require specialized workmanship which makes RC jacketing an extremely valuable choice in structural rehabilitation [5]. Fiber reinforced plastic wraps are proved effective in restoring the flexural strength and ductility capacity of earthquake-damaged columns [10, 11]. Yalciner and Hedayat investigated the most effective strengthening method among column jacketing, adding steel braces and new shear walls to an existing four-story RC building. As a case study, a four- story RC existing building was selected and assessed using finite element method. Results showed that the column jacketing is the most effective and the most economic strengthening technique for the low-rise residential buildings in North Cyprus.

Repairing of reinforced concrete beams is one of the important works normally associated with the rehabilitation of concrete structures. It is important to ensure bonding between old concrete and the applied materials. For this purpose several bonding agents are used just before applying the repairing materials. Different types of repairing materials are available such as grout, mortar, concrete, sprayed concrete, resin based material and externally bonded steel plate and composite materials. The work is to be carried out by the experienced operators, working under experienced supervisors [4, 13].

Retrofitting of footings include enlargement of plan dimensions and the thickness, addition of top steel reinforcement, and placement of dowels to connect the existing and new concrete [4]. An experimental program is carried out on retrofitted isolated footings shows that both footing strength and deformation capacity increased substantially, and the damage to the retrofitted footing is minimal [14].

A report prepared by PCBS shows that the 2008-2009 Israeli aggression in Gaza Strip resulted in the total destruction of 4,100 houses, as well as the partial destruction of 17,000 houses, schools, government buildings and security forces headquarters [15]. The objectives of this work include assessment of the damages caused to the structural elements of a partially damaged 16-story RC residential building, based on non-formal assessment and model-based assessment. Depending on the degree of the damages incurred to the structural elements, proposed repair/strengthening techniques are to be outlined.

2- Methodology:

Given the special nature of this work, the following steps are followed in order to achieve the above-mentioned objectives.

2-1 Site visits and damage classification

Site visits are to be conducted for visual inspection and gathering of general information regarding the building condition. Furthermore, the original design drawings of the building are to be obtained from the related authorities.

The structural damages are to be assessed and classified, based on the extent of the damage. Testing of concrete is to be measured via core tests and NDT to assess the loss of strength, if any.

2-2 Structural Modeling

Structural analysis for the entire building is to be carried out using ETABS structural analysis and design software. The results are to be obtained, based on the damaged incurred to the structural elements.

2-3 Detailed analysis

Based on he preliminary analysis, critically damaged members are to be determined. Consequently, the building site is to be re-visited in order to confirm the analysis results and look for unnoticed cracking/deformation. Furthermore, the rehabilitation alternatives are set out.

2-4 Final design

Detailed structural design of the damaged elements is to be carried out, based on the proposed rehabilitation scheme.

3- Description of the damaged structure:

Al-Andalus residential tower consists of a basement floor, a ground floor, a mezzanine floor and 13 typical floors. It is constructed on a plan area of about 1046 m^2 as shown in Figure (1). The structural system of the building is "building frame"; where 49 columns, rectangular in cross section, are designed to resist the gravity loads. To withstand the lateral loads resulting from seismic/wind forces, 13 shear walls, in addition to elevator and staircase walls, are used. The floor and roof slabs are two-way hollow-block ribbed slabs, 26 cm in thickness, except for the basement slab which is 30 cm thick. The building foundation is 1.5 meter thick mat constructed on sandy soil having a bearing capacity of 2.0 kg/cm^2. Original structural design is based on ACI 318-95 Code [16] and UBC-97 [17] is used as the building code for determining the loads, especially seismic loads. Columns and shear walls are shown in Figure (2).

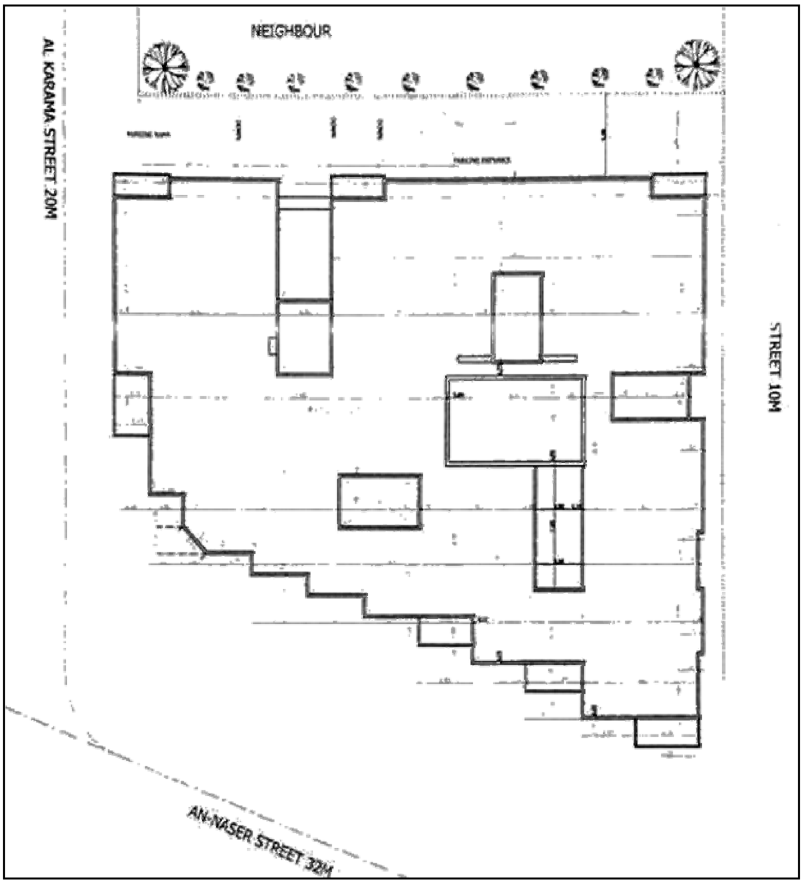


Figure (1): Building site plan

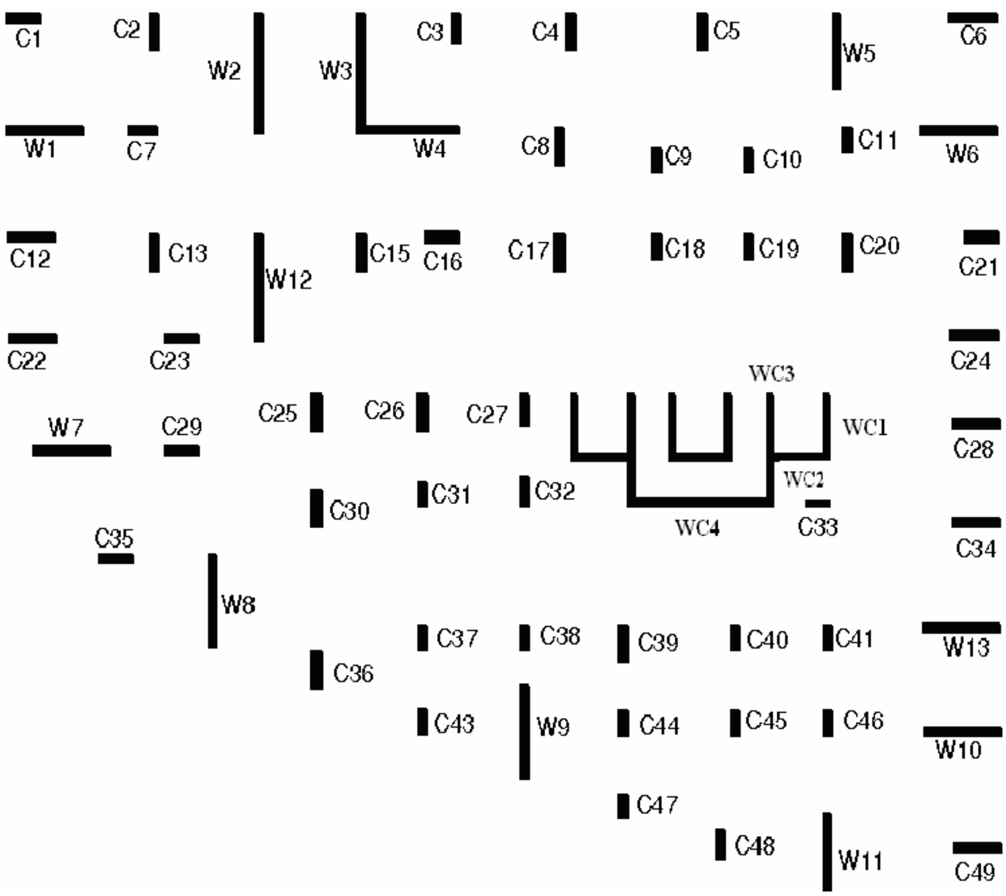


Figure (2): Columns and shear walls

4- Classification of the damages, based on visual inspection:

During the aggressive Israeli actions in Gaza Strip within the period between December 27th of 2007 and January 17th of 2008, the tower was exposed to jet fighter missiles causing extensive structural and nonstructural damages to the building. The structural damages included partially and totally-damaged columns, shear walls and slabs along the building height, as shown in Figure (3).

Figure (3-a): Damages in the western side

Figure (3-b): Damages in the eastern side

4-1 Columns

The damages are classified into the following categories.

4-1-1 Columns showing minor/partial damage

This category includes C17 and C21 in the basement floor. In the ground floor C20, C21 and C29 are included.

4-1-2 Columns showing severe damage

Column C9, C10, C11, C18, C19 and C20 in the basement floor. See Figure (4) for a sample of this category.

Figure (4): Column showing severe damage

4-1-3 Columns showing bucked shape/major cracking

This category includes C13, C23, C24 and C28 in the basement floor. Added to this are columns C10, C13 and C23 in the ground floor. In the mezzanine floor C9, C10, C16 and C18. A sample of this category is shown in Figure (5).

Figure (5): Buckled ground floor column

4-1-4 Columns, showing total destruction

Columns in this category need to be reconstructed due to their failure. These columns include C1, C2, C3, C4, C7 and C8 from the basement level up to the top of the building.

4-2 Shear Walls

4-2-1: Shear walls showing minor damage

This category includes W5 in the basement floor and W12 in the Mezzanine floor.

4-2-2 Shear walls showing severe damage

Shear walls of this category include the elevator and staircase cores (WC1, WC2, WC3 and WC4) in the basement floor only.

4-2-3 Shear walls showing total destruction

This category includes W1, W2, W3 and W4 for the entire height of the building.

4-3 Slabs and Beams

4-3-1 Slabs completely destroyed

The entire basement slab and all the slabs in the western part of the building (part 2), shown in Figure (6), are completely destroyed.

Figure (6): Part of the destroyed basement slab

4-3-2 Slabs severely damaged

Limited parts of the ground floor slab of the building are severely damaged, as shown in Figure (7).

Figure (7): Limited damages in ground floor slab

4-3-3 Slabs with limited damages

A small panel in mezzanine floor slab and another in first floor show excessive deflection and structural cracking. The excessive deflection and cracking are attributed to the damage caused to column C11 in the basement floor (see Figure 8). Also, one panel in the fourth floor slab is partially damaged. Moreover, one panel of the fourth floor slab of part (1) of the building shows limited damages.

Figure (8): Excessive deflection of a panel in second floor slab around column C11

4-4 Mat foundation

Building loads may be redistributed as a result of the damaged columns and shear walls. Due to the change in eccentricity of the loads, the soil pressure under the mat is to be evaluated.

5- Damage Simulation

The building before sustaining the damages was originally designed according to ACI 318-95 as the structural code and UBC-97 as the building code. Then, a three-dimensional model of the structure was created using ETABS-9 to carry out the static analysis. In addition, analysis of the mat foundation was carried out using SAFE-8.

The ETABS-9 model included defining a new model, defining materials and sections, adding the geometry, meshing, defining loads and load cases, assigning the supports, checking the model, running the analysis and obtaining the results. At the end of this work the model looks liked the one shown in Figure (9).

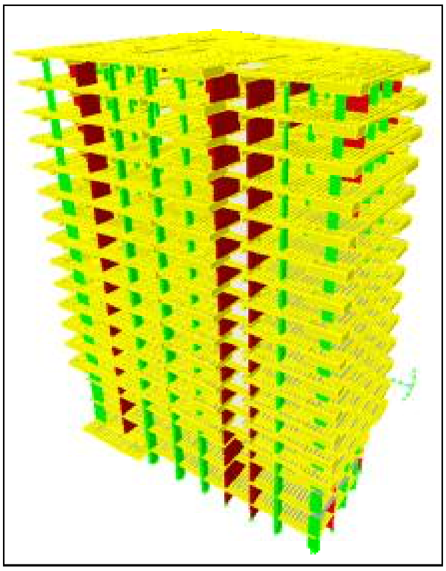


Figure (9): The 3-D ETABS-9 building model

The SAFE-8 model included importing from ETABS-9, assigning and importing data, running the analysis and obtaining the results.

6- Analysis of Results:

Based on the results obtained from ETABS-9 and SAFE-8 models, the structural design of the building was checked based on ACI 318-95 and UBC-97. Design checks included slab design, column design, shear wall design and mat foundation design. Moreover, soil pressures under the mat before and after the incurred damages were evaluated. These design checks showed the following:

- Slab design, column design and shear wall design are satisfactory.
- The soil pressure under the mat increased from 16.65 t/m^2 before the sustained damages to 20.85 t/m^2 after the damages. This is still within the allowable bearing capacity of the soil as stated in the geotechnical investigation of the building site (see Figure 10 for SAFE-8 results).

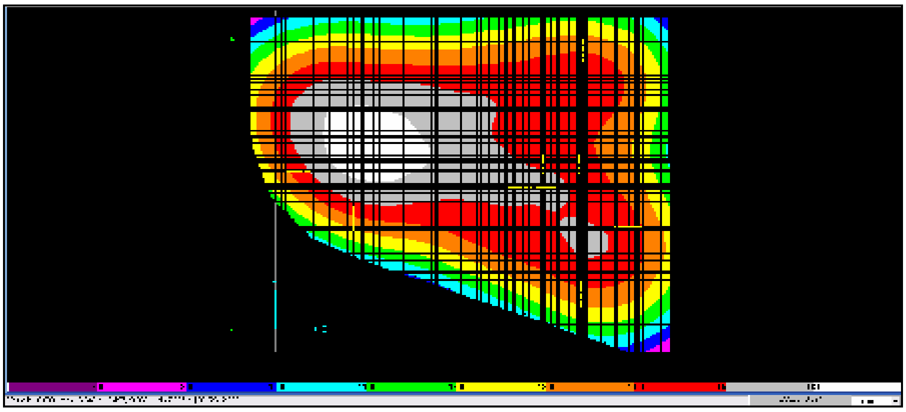
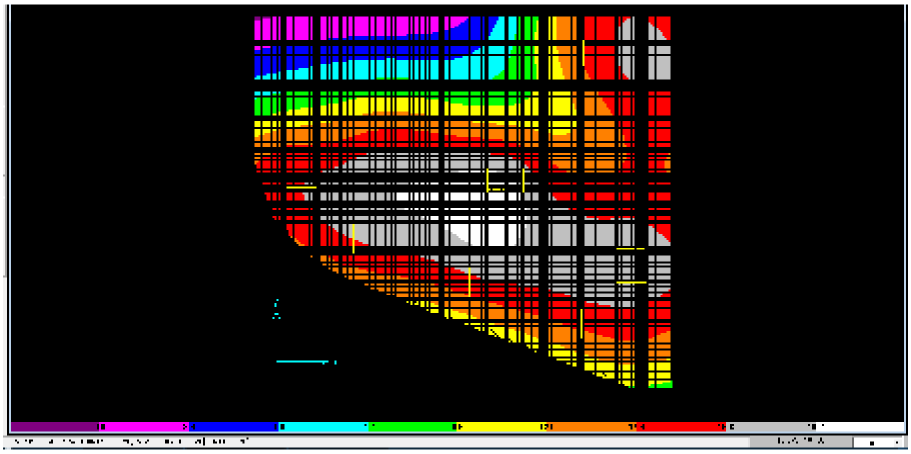


Figure (10-a): SAFE-8 results show that maximum soil reaction is 16.65 t/m^2 (before)



(10-b): SAFE-8 results show that maximum soil reaction is 20.85 t/m^2 (after)

• The mat thickness of 150 cm is sufficient for resisting the additional shear forces and bending moments caused by the increased the soil pressure under the mat, as shown in Table (1).

Table (1-a): Shear and flral strengths of a mat strip, 550 cm wide and 150 cm thick.

	Location	$V_{u,max}$ (ton)	$M_{u,\max}$ ton.m	Existing Reinft. cm^2	Flexural Strength	Shear Strength
Before Damage	Top	300	78	138.23	Adequate	Adequate
	Bottom	300	318	172.78	Adequate	Adequate
After Damage	Top	480	420	138.23	Adequate	Adequate
	Bottom	480	730	172.78	Adequate	Adequate
After Strengthening	Top	480	420	163.36	Adequate	Adequate
	Bottom	480	730	197.92	Adequate	Adequate

Table (1-b): Shear and flexural strengths of a mat strip, 400 cm wide and 150 cm thick

	Location	$V_{u,max}$ (ton)	$M_{u,max}$ ton.m	Existing Reinft. cm^2	Flexural Strength	Shear Strength
Before Damage	Top	172	43	100.53	Adequate	Adequate
	Bottom	172	200	125.66	Adequate	Adequate
After Damage	Top	370	175	100.53	Adequate	Adequate
	Bottom	370	725	125.66	Adequate	Adequate
After Strengthening	Top	370	175	116.24	Adequate	Adequate
	Bottom	370	725	141.37	Adequate	Adequate

- The severe damage incurred to column C11 is accounted for in the ETABS-9 model by including a 20-cm support settlement. This caused high moments in parts of the mezzanine and ground floor slab beams, around column C11, with decreasing effect towards the top of the building. The increase in bending moments for two affected beams is shown in Table (2).

Table (2): Moments in affected beams due to the inflicted damages (t.m)

Beam C9-C10-C11-W6		
Support	**Before damage**	**After damage**
C9	-3.50	5.15
C10	-3.50	-0.83
	-3.50	4.50
C11	-3.50	-4.10
	-3.50	-4.10
W6	-2.48	-26.6
Beam C19-C20-C21		
Support	**Before damage**	**After damage**
C19	0	0
C20	-8.6	-14.1
	-8.6	17.7
C21	-11.55	-16.2

7- Proposed Repair and Retrofit Schemes:

7-1 Columns

7-1-1 Columns showing minor/partial damage

These columns have partial damage that only requires patching, existing bars should be straighten or if they are cut, tied with new bars of the same diameter then new concrete can be cast on them. Epoxy material ought to be placed between old and fresh concrete.

7-1-2 Columns showing severe damage

These columns were severely damage. Thus, these column, which mainly exist in the basement floor, are to be reconstructed. Column C11 which is used for demonstration purposes is to be reconstructed based on the original design of the building, as shown in Figure (11).

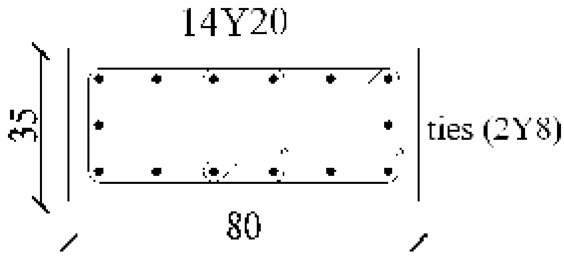


Figure (11): Column C11 after reconstruction

7-1-3 Columns showing bucked shape/major cracking

The columns of this category suffered from cracks and/or excessive deflection. These columns are to be jacketed assuming that the jacket carries the entire axial load. Column C13 is to be considered as a sample of this group.

From ETABS-9 analysis, the factored axial load on column C13, $P_u = 300\ tons$. Assuming a reinforcement ratio of $r_g = 1\%$ and knowing that compressive strength of column concrete $f_c' = 300\ kg/cm^2$, yield stress of reinforcement $f_y = 4200\ kg/cm^2$ and original cross sectional area of this column = 35 x 120 = 4200 cm^2.

Required cross sectional area of column's jacket is given by ACI Equation (10-2)

$$A_g = \frac{P_u}{0.56\left[0.85 f_c' + r_g\left(f_y - 0.85 f_c'\right)\right]}$$

= 1823.01 cm^2

Thus, the gross cross sectional area = 4200 + 1823.01 = 6023.01 cm^2

For a practical jacket thickness of 20 cm, the outside column dimensions are B = 160 cm and H = 75 cm.
The actual jacket cross section = 160 (75) – 4200 = 7800 cm^2 > 1823.01 cm^2 O.K
Based on article 10.8.4 of ACI 318-99, longitudinal reinforcement, $A_s = 0.005(7800) = 39\ cm^2$ and $20F\ 16\ mm$ are to be used. Based on ACI 7.10.5 transverse reinforcement of $F\ 10\ mm@25\ cm$ are used, as shown in Figure (12).

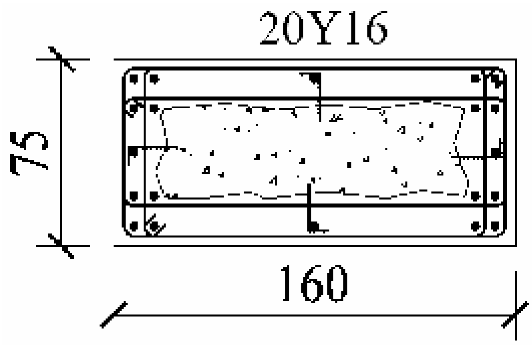


Figure (12): Column C13 after strengthening

7-1-4 Column showing total destruction

These columns need to be reconstructed based on the original design of the building.

7-1-5 Column, newly added

Nine columns CN1, CN2, CN3, CN4, CN5, CN6, CN7, CN8 and CN9 are to be constructed along the proposed construction joint. The joint is to be constructed since this part of the building was totally destroyed, which makes anchoring of the new floor slabs to the old ones in other parts of the building very difficult.
Column CN8 is to be considered as a sample of this group.
Required cross sectional area of column's jacket is given by ACI Equation (10-2)

$$A_g = \frac{P_u}{0.56\left[0.85 f_c' + r_g\left(f_y - 0.85 f_c'\right)\right]}$$

= 2061.96 cm^2

Use a cross section of 35 cm x 70 cm. Longitudinal reinforcement $A_s = 0.01(70)(35) = 24.5\ cm^2$ and 10Φ16mm+4Φ14mm are to be used. In addition, $F\ 10\ mm@25\ cm$ are to be used.

7-2 Shear Walls:

7-2-1: Shear walls showing minor damage

These walls are treated similar to columns of the corresponding group.

7-2-2 Shear walls showing severe damage

These walls are treated similar to columns of the corresponding group. The jacketing technique is used to repair and to strengthen the damaged shear walls.

7-2-3 Shear walls showing total destruction

These shear walls need to be reconstructed according to the original design of the building.

7-3 Slabs and Beams

7-3-1 Slabs completely destroyed

The basement slab is divided into two parts and a construction joint is proposed between the two parts, where columns CN1 through CN9 are to be added and anchored to the mat. Part (2) is to be designed as hollow-block ribbed slab, 30 cm in thickness. The main beams are to be designed as simply supported beams due to the presence of columns and shear walls (see Figure 13-a).

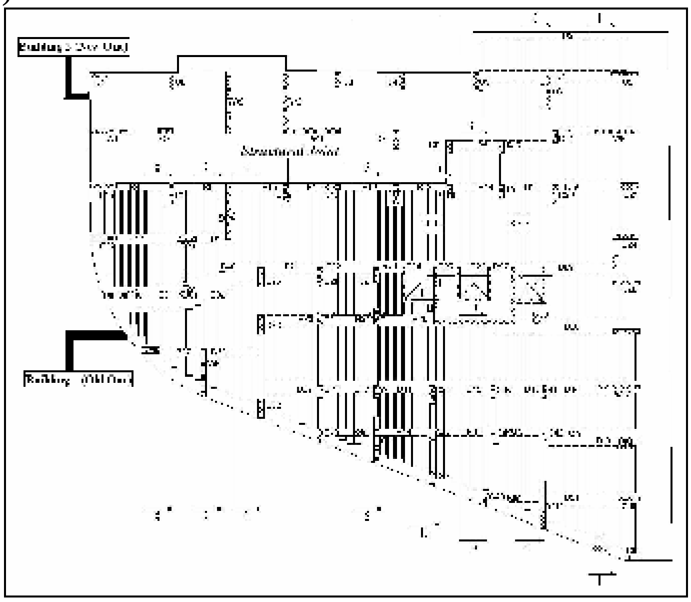


Figure (13-a): Newly constructed basement slab (part 1)

Basement slab in Part (2) is to be redesigned with the same thickness (30 cm), since additional columns are to be constructed, as shown in Figure (13-

b). Also the slabs of mezzanine, ground and typical floor slabs in part (2) are to be redesigned.

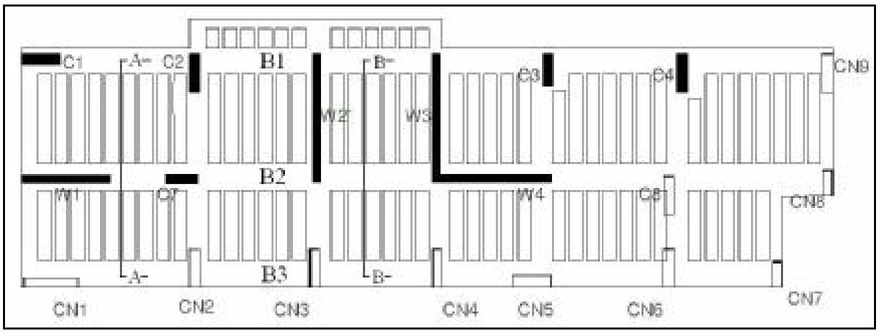


Figure (13-b): Newly constructed basement slab (part 2)

7-3-2 Slabs severely damaged

Damaged panels in the ground floor slab is to be reconstructed as shown in Figure (14).

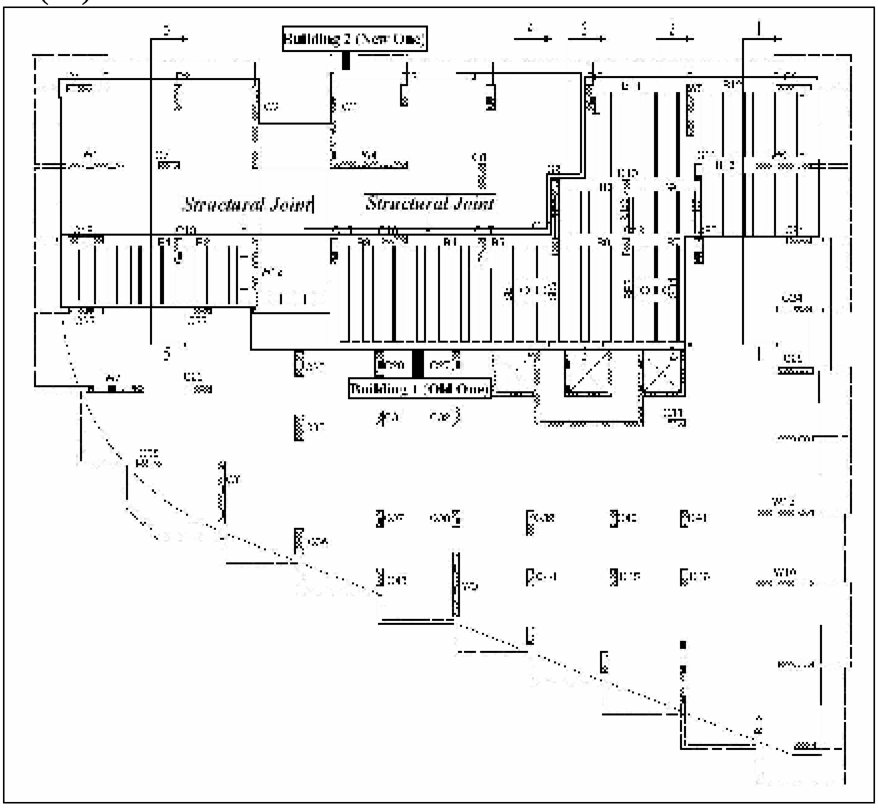


Figure (14): Reconstructed panels in ground floor slab (part 2)

7-3-3 Slabs with limited damages

Parts of the slab around column C11 are to be reconstructed in ground floor and mezzanine floor slabs, as shown in Figure (15-a). Reconstructed panel of the fourth floor slab is shown in Figure (15-b).

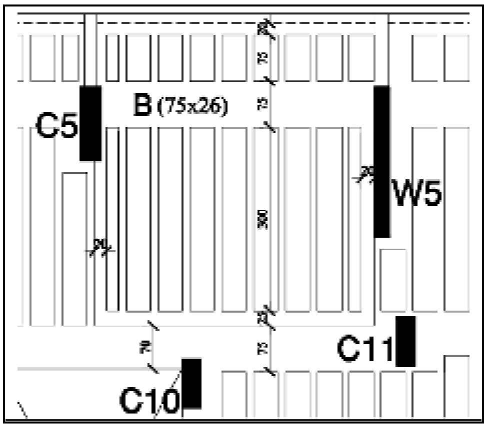


Figure (15-a): Reconstructed panels in mezzanine and first floor slab

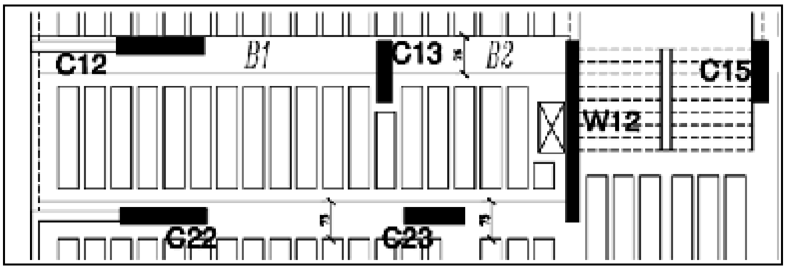


Figure (15-b): Reconstructed panel in fourth floor slab

7-4 Staircase

The main staircase is to be reconstructed according to the original design of the building.

8- Conclusions:

1. The feasibility study showed that it is more economical to rehabilitate the building than demolishing and reconstructing it.
2. The complete destruction in the western side of the building makes rehabilitation possibilities impractical. Therefore, a construction joint is proposed with nine more columns added to make this possible.

3. For the slab surrounded by (W5, W6 and C6), ETABS-9 analysis showed additional moment in some supporting beams, as well as in slabs caused by shortening of column C11. Visual inspection showed that this slab is deflected in the mezzanine and ground floor slabs. So, this panel should be removed and re-casted, based on its same design of slab reinforcements and shear connectors.

4. For all new concrete used for rehabilitation, expanding agents must be added to the mix to ensure gab fillings. Old surfaces must be cleaned, roughened and painted by epoxy agent to enhance bonding between old and fresh concrete. Shear connectors must be covered by bonding agents (concrete-steel) to guarantee fixation in old concrete, then driven into it.

9- Acknowledgements

The author would like to extend his gratitude to the Palestinian Ministry of Public Works and Housing for providing access to the damaged building and providing the interim inspection report.

List of Abbreviations:

PCBS:Palestinian Central Bureau of Statistics.

NDT: Non-Destructive Testing

ETABS-9: Extended 3D Analysis of Building Systems by Computers and Structures, Inc.

SAFE-8: Integrated Design of Flat Slabs, Foundation Mats & Spread Footings by Computers and Structures, Inc.

UBC: Uniform building Code

10- References:

[1] Ngo T., Mendis P., Gupta A. and Ramsay, J. 2007- *Blast Loading and Blast Effects on Structures- An Overview*, EJSE International Special Issue, Loading on Structures.

[2] Nor N., Zainuddin M., Ismail N., Yusof M., Husen H., 2010- *Damage Evaluation Procedure for Building Subjected to Blast Impact*, European Journal of Scientific Research, ISSN 1450-216X, Vol.39 No.3, p.326-335.

[3] Rücker W., Hille F., Rohrmann R., 2006- *F08a Guideline for the Assessment of Existing Structures*, SAMCO Final Report.

[4] National Information Center of Earthquake Engineering, 2004- *Guidelines for Earthquake-Resistant Non-Engineered Construction*, Indian Institute of Technology Kanpur, Kanpur, India.

[5] Júlio E., Branco F., Silva V., 2003- *Structural rehabilitation of Columns with Reinforced Concrete Jacketing*, Prog. Struct. Engng Mater., Vol., No. 1,(p: 29–37.

[6] Rodriguez M., Park R., 1991- *Repair and Strengthening of Reinforced Concrete Buildings for Earthquake Resistance*, Earthquake Spectra, Vol. 7, No., p: 439–459.

[7] Sugano S., 1981- *Seismic Strengthening of Existing Reinforced Buildings in Japan*, Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 14, No. 4, p: 209–222.

[8] Ramírez JL., Ba´rcena JM., Urreta JI., de Val B, Aurrekoetxea JR., 1993- *Repair of Concrete Columns with Localized Partial Loss of Corners or Cover*, Proceedings of the Conference on Structure Faults and Repair, Edinburgh, p: 195–203.

[9] Aguilar J., Juarez H., Ortega R., Iglesias J., 1989- *The Mexico Earthquake of September 19, 1985. Statistics of Damage and Retrofitting Techniques in Reinforced Concrete Buildings Affected by the 1985 Earthquake*, Earthquake Spectra, Vol. 5, No. 1, p: 145–151.

[10] Iacobucci R., Sheikh S., Bayrak O., 2003- *Retrofit of Square Columns with Carbon Fiber-Reinforced Polymer for Seismic Resistance*, ACI Structural Journal, November-December.

[11] Saadatmanesh H., Ehsani M., Jin L., 1997- *Repair of Earthquake-Damaged RC Columns with FRP Wraps*, ACI Structural Journal, March-April.

[12] Yalciner H., Hedayat A., 2010- *Repairing and Strengthening of an Existing Reinforced Concrete Building: A North Cyprus Perspective*, American Journal of Engineering and Applied Sciences, Vol. 3, No. 1, p: 109-116.

[13] Jumaat M., Kabir M., Obaydullah M.,2006- *A Review of the Repair of Reinforced Concrete Beams*, Journal of Applied Science Research, Vol. 2, No. 6, p: 317-326.

[14] Saiidi M., Sanders D., Acharya S., 2001- *Seismic Retrofit of Spread Footings Supporting Bridge Columns with Short Dowels*, Construction and building Materials, Vol. 15, No. 4, p: 177-185.

[15] PCBS, 2009- *Direct Losses in Infrastructure (in Arabic)*, http://www.pcbs.gov.ps/Portals/_pcbs/Pressrelease/gaza_losts.pdf, Ramallah, extracted on 14/12/2010.

[16] ACI Committee 318- 1995, *Building Code Requirements for Structural Concrete, (ACI 318-95*), American Concrete Institute, Farmington Hills, MI.

[17] International Conference of Building Officials, 1997- *Uniform Building Code*, Whittier, CA.