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NON-LINEAR STATIC ANALYSIS AND SEISMIC PERFORMANCE OF MODERN ARCHITECTURAL HERITAGE IN EGYPT

Sayed Hemeda

Associate Professor, Conservation Department, Faculty of Archaeology, Cairo University, Giza, Egypt (hemeda_200x@yahoo.com)

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ABSTRACT

The objective of this paper is the investigation of the behavior of historic masonry structures subjected to static and seismic forces and the formulation of numerical models for use in structural analysis. The case study presented her regards the seismic analysis of El-Sakakini palace (1892 AC) in Cairo, Egypt (damaged by the earthquakes occurred in 1992 and 1995).
Firstly, we collected all data regarding the site, the geometry of manufact, the characteristics of materials of construction, the structure and the soil medium, etc. The paper presents the next part which includes the static and seismic performance of these structures is investigated according to statistical data from recent earthquakes and finally an analytical model of describing their performance is developed. An analytical, plasticity-based Model is presented for the inelastic analysis of traditional masonry structure. The proposed Model is applied to the analysis of El-Sakakini palace in Cairo.
A non-linear model is developed, aiming at capturing the key in elastic mechanisms. The analytical model is implemented in the finite element code Robout Millineum and validated against experimental results. The results obtained suggest that for such structures non-linear static analysis provides a reasonable prediction of damage at the base of the palace, but is not however suitable for predicting the overall damage along the palace's entire height. The conclusive aim of the project is then to develop guidelines for the evaluation of the static and seismic vulnerability of historic masonry palaces.
Ultimately, the analysis presenting the optimal structural interventions to remedy the existing damage and to prevent the formation of the same mechanisms under the action of future earthquake.

KEYWORDS: Architectural Heritage, multiple leaf-masonry, monitoring, restoration, structural analysis, Non-linear static analysis, Seismic Performance.

1. INTRODUCTION

Egypt is rich in monumental sites that covers span several civilizations starting from the pharonic times to the recent times. Yet these monumental sites are vulnerable due to both natural and man-made effects. As an example of such effects is Tell Basta an earthquake (2200 BC) that is believed to be responsible for the total collapse of pharonic temple located in eastern Nile Delta in Sharqiya Governorate (Khalil, 2011).

The knowledge of historical structural behavior permits to focus our attention on these macro-elements and to evaluate if the carried out retrofitting interventions or the suffered transformations. over the years have mitigated or increased the seismic vulnerability (Lagomarsino and Podesta, 2004a and 2004b).

Recently there is a rising interest in understanding the behavior of traditional structures which has survived over the centuries often in seismic-prone areas, and need preservation (Kouris and Kappos, 2009). The seismic assessment and risk mitigation of culture heritage represents one of the principal features of the protection of monumental buildings (Cardoso et al, 2005). The observation of damage mechanisms which have occurred during past earthquakes in addition to international literature highlights that the seismic behavior of architectural heritage can be correctly reproduced by subdividing the building into elementary parts characterized by independent structural response (macro-elements) (Podesta and Brignola, 2009). It is very important to prevent historic building against future large earthquakes (Gergova, 1995). the severe damage suffered by the palace for the Dahshour 1992 and Aqaba 1995 earthquakes (that probably determined the decay of the palace) is partially confirmed from the damage survey, still now detectable (Podesta and Curti, 2009).

Necessary constitutive models and analysis techniques have become available for unreinforced masonry structures. The analysis of these structures is usually performed with elastic methods. Elastic analysis is a useful tool for identifying regions with high stress; however, it fails sometimes to capture the final failure mechanism (Kouris and Kappos, 2009).

The intervention methods must be not only reliable and durable, but also if required easy to monitor and remove, the latter aspect corresponding to the widely shared policy of safeguarding existing buildings from inappropriate restoration interventions, with particular reference to historical and monumental constructions (Mazzolani and Mandara, 2009).

In earthquake conditions, structural components pass in a state of post- elastic, hysteretic cyclic state. This type of behavior is of high importance for the structural analysis in seismic conditions regarding the accumulation of dilation with the number of cycles (Petraskovic and Sedmak, 2011).

The observation of earthquake induced damage has highlighted that the out of plane turnover of the masonry wall is the main way of collapse in old masonry buildings especially because of ineffective connection with floors. in this case the collapse can be either global if the connection is completely ineffective or local when the lack of link is localized in some parts of structure only. Likewise, there are two different types of in-plane collapse of masonry walls: the first one involves the combination of bending and compressive resistance, the second shear failure (Alberto et al 2011).

2. TECHNICAL ASSESSMENT OF HISTORIC BUILDINGS

The Content of the Technical Assessment, according to the international legislation, consists of the following 16 chapters:

1) Reason and goal of the technical assessment;
2) Methods of investigations;
3) Comments regarding to condition of the historical building and on its location in a historical protection area;
4) General data on the building;
5) Architectural and Structural description of the building;
6) Geotechnical information on the foundation medium;
7) Description of the in-time modification of the building;
8) Detailed qualitative assessment;
9) Ambient vibration instrumental investigation;
10) Material non-destructive testing;
11) Advanced methods of investigations in order to assess the structural vulnerabilities of the building to seismic actions;
12) Correlations of the obtained results and conclusions;
13) Establishing the seismic risk class of the building;
14) Proposal of intervention and remedial measures;
15) Substantiating the decision for the necessity of structural intervention;
16) Final conclusions and cost estimate for the proposed works.

The Technical Assessment contains also: 1) A Historical study; 2) A Geotechnical study 3) Mapping of the existing cracks and damage; 4) Architectural and structural plans and details.

3. STRUCTURAL DESCRIPTION OF SAKAKINI PALACE

The overall structure system of the building consists of, superstructure, substructure, structure of foundation and foundation medium.

The superstructure comprises the storeys situated above the ground floor; first floor to the fifth floor, attics and the towers.

The vertical component of the structural system of the superstructure consists of structural multiple leaf masonry walls, disposed along the four axes : on the longitudinal direction (axes "1", "2", "3", and "5"); on the transversal direction (axes "A", "B", "C" and "E").

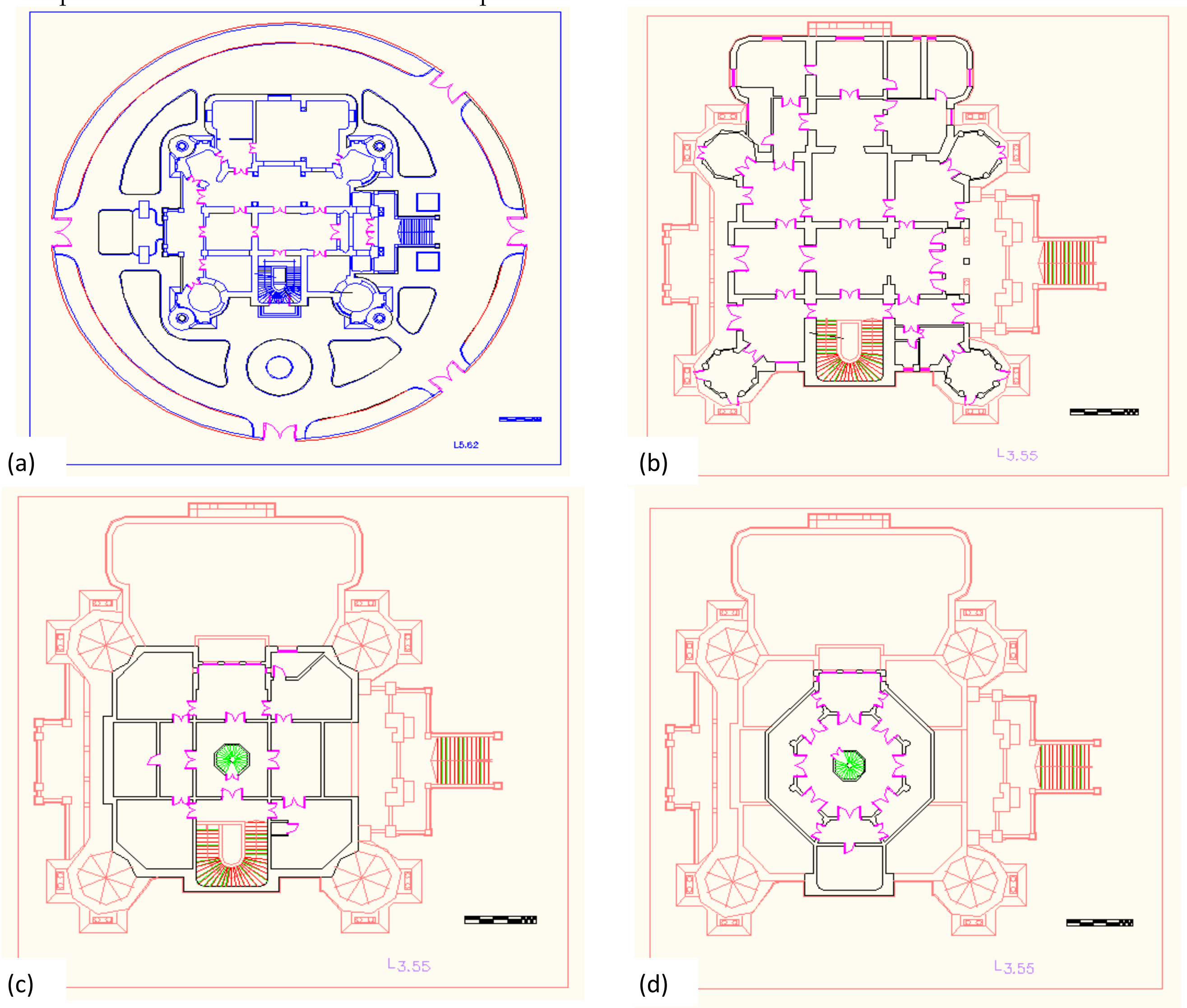

Figure. 1. Plans view of the first (a), second (b), third (c) and fourth (d) floor of Sakakini palace. Cairo, Egypt.

Figure.2. exterior and interior overview of Sakakini palace, Cairo, Egypt.

4. THE MAIN STRUCTURAL DEFICIENCIES OF SAKAKINI PALACE

The main structural deficiencies of the vertical component of the structural system are:

The superstructure and substructure of the palace has suffered considerable damage due to extensive cracking associated with soil settlement and foundations movements undergoing a shear failure accompanying with large settlement both towards to the western and northern directions as well as the centre of the palace;

Cracks widths were 10-20mm and exhibiting periodical widening. The cracks had a preferential orientation in two principal directions namely E-W, which includes typical views of major cracking in the masonry walls and foundations. Such effects were attributed to earthquake damages, permanent deformations and large settlement of the bearing soil 'due to the secondary Consolidation', natural wear of construction materials and the construction history of the complex;

Abnormalities in arranging entryway and window openings, together with the changeability of the measurements of these openings (both on "L" and "T");

The way that the auxiliary divider even segment regions vary on the two primary bearings of the building;

There are likewise inconsistencies of the auxiliary dividers level segments, at every story, on the vertical bearing;

The substructure of the building is 2.3 m stature and comprises of block and stone brick work dividers, constituting the general cellar.

The structure of the establishment comprised of proceeds with stone cyclopean dividers kind of roughly 3 m tallness, underneath all the substructure dividers. At the beginning, there were only information from the time press which clamed a bad foundation medium.

Figure.3a. present state of in plane and out of plane structural deficiency of Sakaini palace.

Figure.3b. present state of in plane and out of plane structural deficiency of Sakaini palace.

5. GEOTECHNICAL AND GEOPHYSICAL INVESTIGATIONS OF SAKAKINI PALACE

In view of the surrounding clamor battle the basic recurrence of the ground is of the request of 3.0 to 3.5sec near the crucial recurrence of the castle. Reverberation wonders ought not out of the ordinary and thought about genuinely in the nitty gritty investigation of the structure. There are strong affirmations that the upper two stories with wooden floors, which are showing high upgrade factors, are subjected to a couple of damages and debasement of their bearing point of confinement, (Table 1 and 2) (Hemeda, 2013).

Table. 1. P-wave refraction geophysical campaign conducted at El-Sakakini palace area.

	Layer A	Layer B		Layer C	
Profile N°	Velocity in m/s	Velocity in m/s	Depth (m)	Velocity in m/s	Depth (m)
1		300	10	1300	25
2		600	16	900	32
3		400	9	700	20
4		400	14	1200	30
5	< 300	300	5	500	10

Table. 2. Fundamental frequencies and amplification factors at five locations.

Floor	Fundamental resonance frequency (HZ)	Amplification Factor
Basement	3	1.3-2
1st Floor	3-3.5	1.4-2.8
2nd Floor	3-4	3-4.5
3rd Floor	3-4.3	5.5-8
4th wood Floor	3-4.5	7.5-8
5th wood Floor	2.5	12-20

6. THREE DIMENSIONAL STATIC AND SIESMIC STRUCTURAL ANALYSIS

The present work was done to evaluate the stability of El Sakakini Palace for risk assessment of static and earthquake loads as shown below in design criteria.

In order to meet the required structural evaluation the following procedure was followed:-

1 - Collecting the required data from the different available reports like:

- Seismic risk assessment - Draft report – March 2012, (Hemeda, 2013).
- The geometrical and geodetically survey in the present state of El – Sakakini Palace.
- The geotechnical investigation report – June 2012, (Hemeda, 2013).
- Characterization and durability aspects of stones and another construction materials of El - Sakakini Palace, (Hemeda, 2013b).
- Collaborative project " Performance based approach to risk assessment, seismic response analysis and protection of Arch. Heritage in Egypt" – March 2012.

2 - Using advanced structural analysis FEM program " Robot Millennium" in the solid analysis.

1. DESIGN CRITERIA

1.1 Codes of Practice and Standards

Egyptian Code Practice 2008

2. LOADING CRITERIA

2.1 Dead Loads

Dead loads have be taken as the following:

Self weight for floor & walls Actual weight

Floor Finish 200 Kg/m^2

2.2 Live Loads

Live loads have be based on the above listed codes, with the following specified live loads:

Typical Floors 500 Kg/m^2

Live load reduction has be implemented according to allowance made by the code.

2.3 Wind Loading

For south zone:

Basic wind speed 70 mph (130 km/hr), Exposure C & Importance factor 1.0.

For north zone:

Basic wind speed 87 mph (140 km/hr), Exposure B & Importance factor 1.15.

2.4 Seismic Loading

Seismic Zone has be based on ECP 2008 recommendations at project location (probability zone 3). Soil profile estimated based on the geotechnical report.

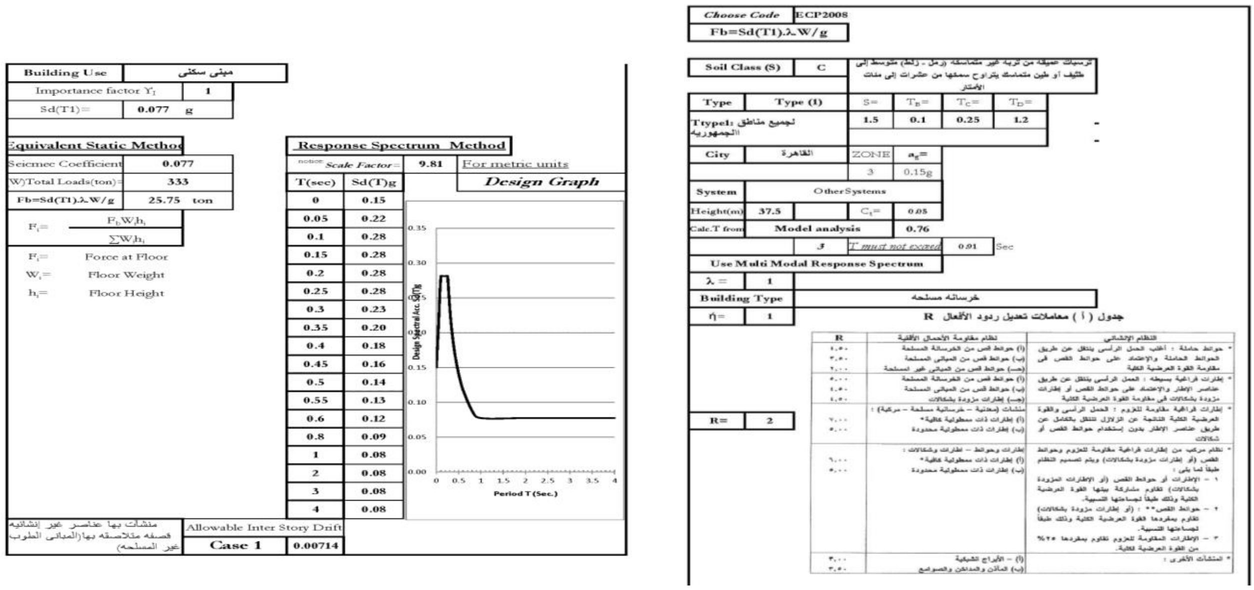

Figure.4. Egyptian Code of Practice (2008) recommendations at Sakakini palace location (seismic zone- probability zone 3).

2.5 Earth Pressure Loads and Differential Settlement.

The coefficient of earth pressure (ka, ko) has be taken based upon the angle of inertial friction of soil as determined by the geotechnical report.

The differential settlement for each structure has be determined on a case by case basis depending on the location of the building.

For rock, specific calculations will be done to take into account rock dip direction, dip angle. Such parameters will be done by geologist.

3. LOAD COMBINATIONS

Ultimate Load Combinations
Required strength U for all structural elements in this project should be at least equal to the effects of factored loads in the following equations:
U = 1.4 Dead + 1.6 Live
U = 1.12 Dead + 0.5 Live + Sx + 0.3 Sy
U = 1.12 Dead + 0.5 Live + Sy + 0.3 Sx

4. MATERIAL PROPERTIES

According to Characterization and durability aspects of stones and other construction materials of El - Sakakini Palace. The allowable stress is 20 MPa. The stone young's modulus is 20 GPa.

5. SOFTWARE

Autodesk® Robot™ Structural Analysis Professional

6. MODELING

Sakakini Palace was modeled using solid element module, the solid element properties taken as defined above in design criteria.

The upper roof of the palace was modeled using frame elements for columns and shell elements for slabs and domes with same properties in the mentioned reports above.

Rigid links are used to connect all walls together to act as one unit, rigid links defined with very high moment of inertial and weightless.

Elements Cross Sections

Solid elements thickness varies from 1600mm to 700 mm along the height.

Beams cross sections 250 mm * 800 mm.

Columns cross section at the top of roof is 300mm * 300 mm.

Slab thickness is 160 mm.

Dome thickness 160 mm.

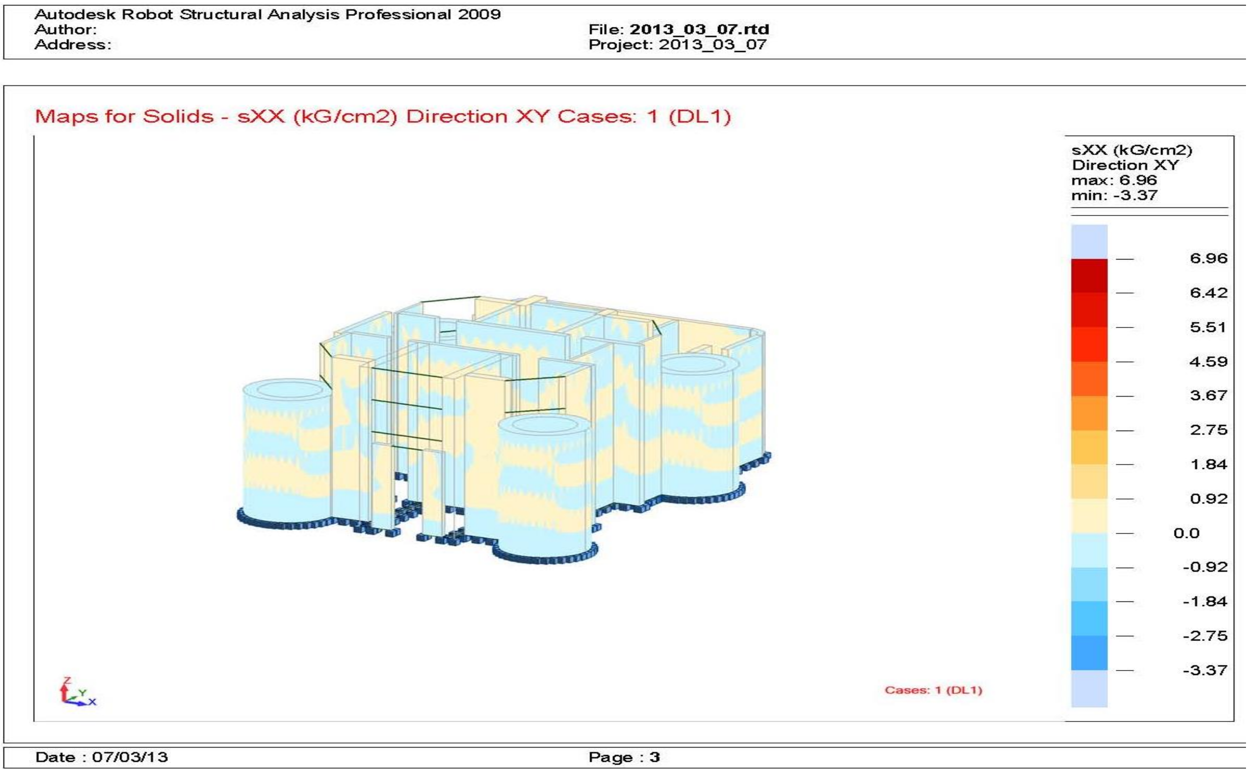

Figure.5. Maps of solids- Effective horizontal compressive stresses σ/xx (Kg/ cm2) Direction XY cases: (DL1).

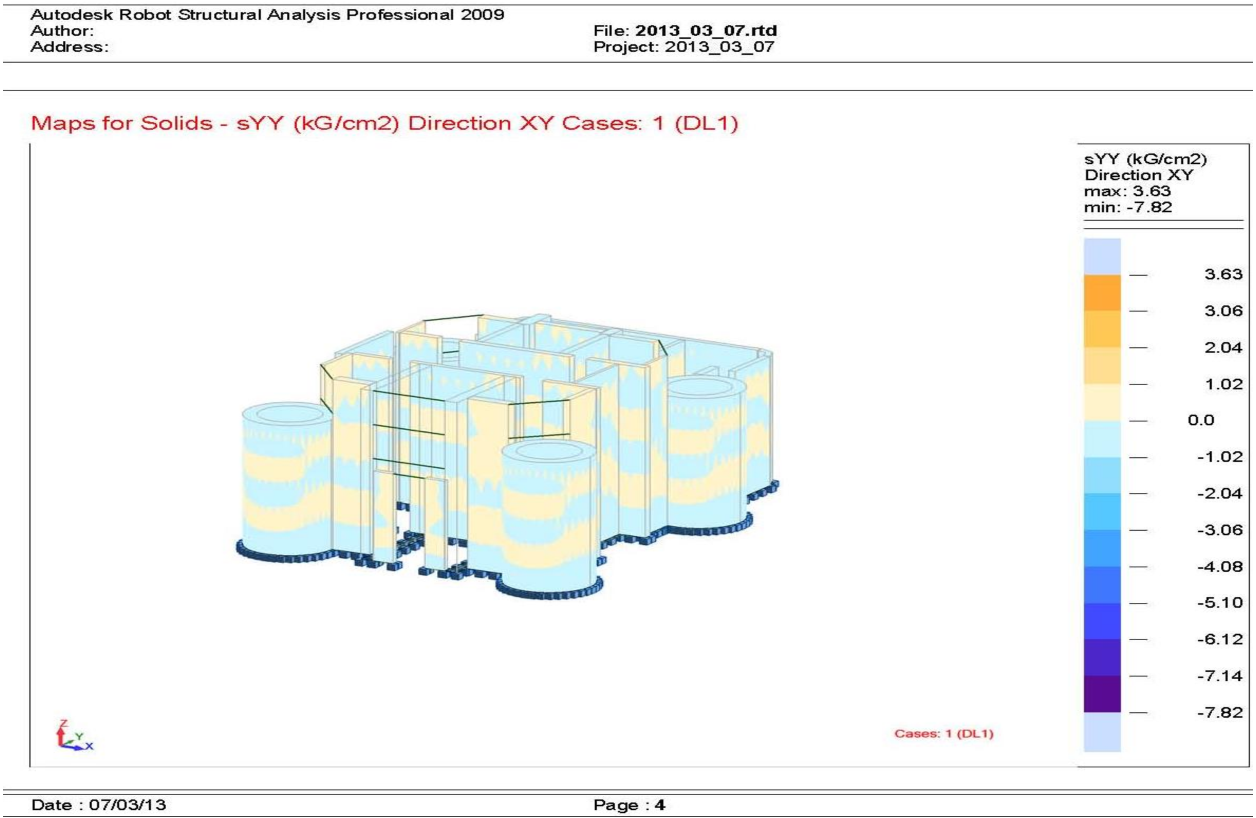

Figure.6 Maps of solids- Effective horizontal compressive stresses σ/yy (Kg/ cm2) Direction XY cases: (DL1).

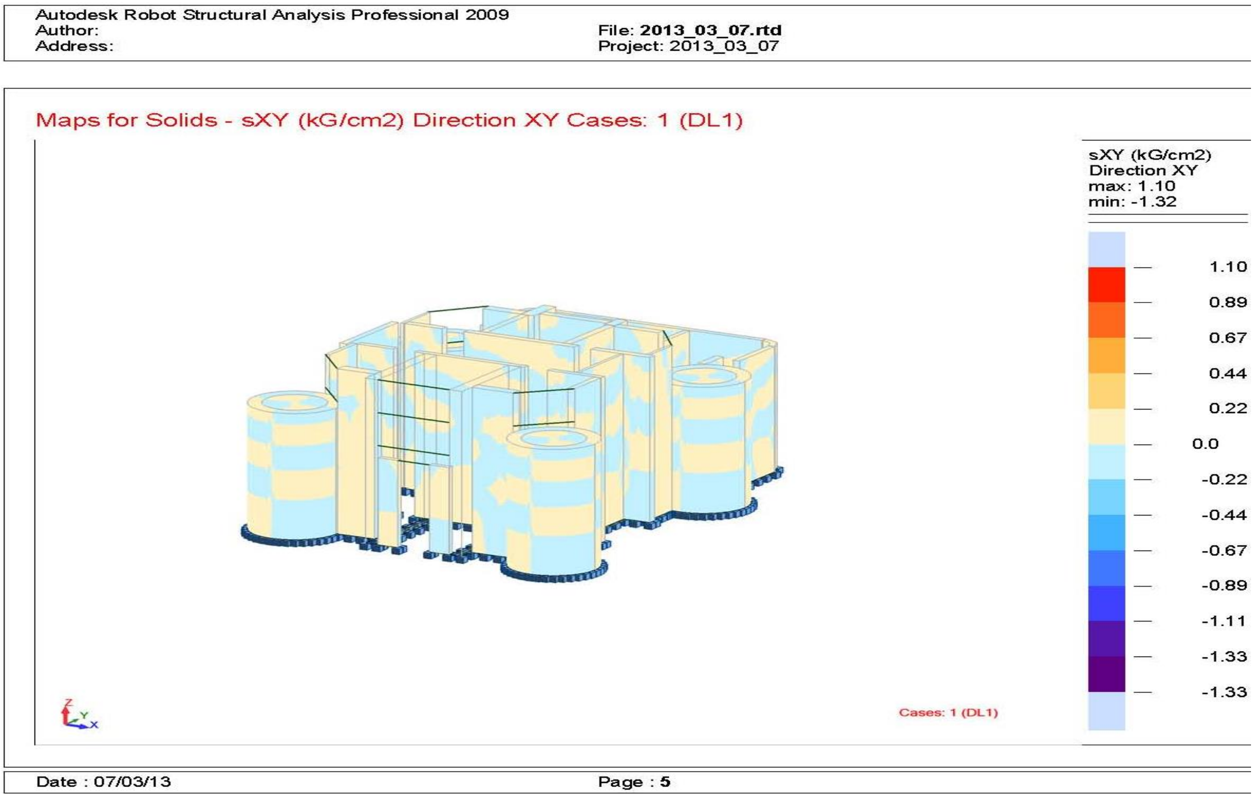

Figure.7. Maps of solids- Effective horizontal compressive stresses σ/xy (Kg/ cm2) Direction XY cases: (DL1).

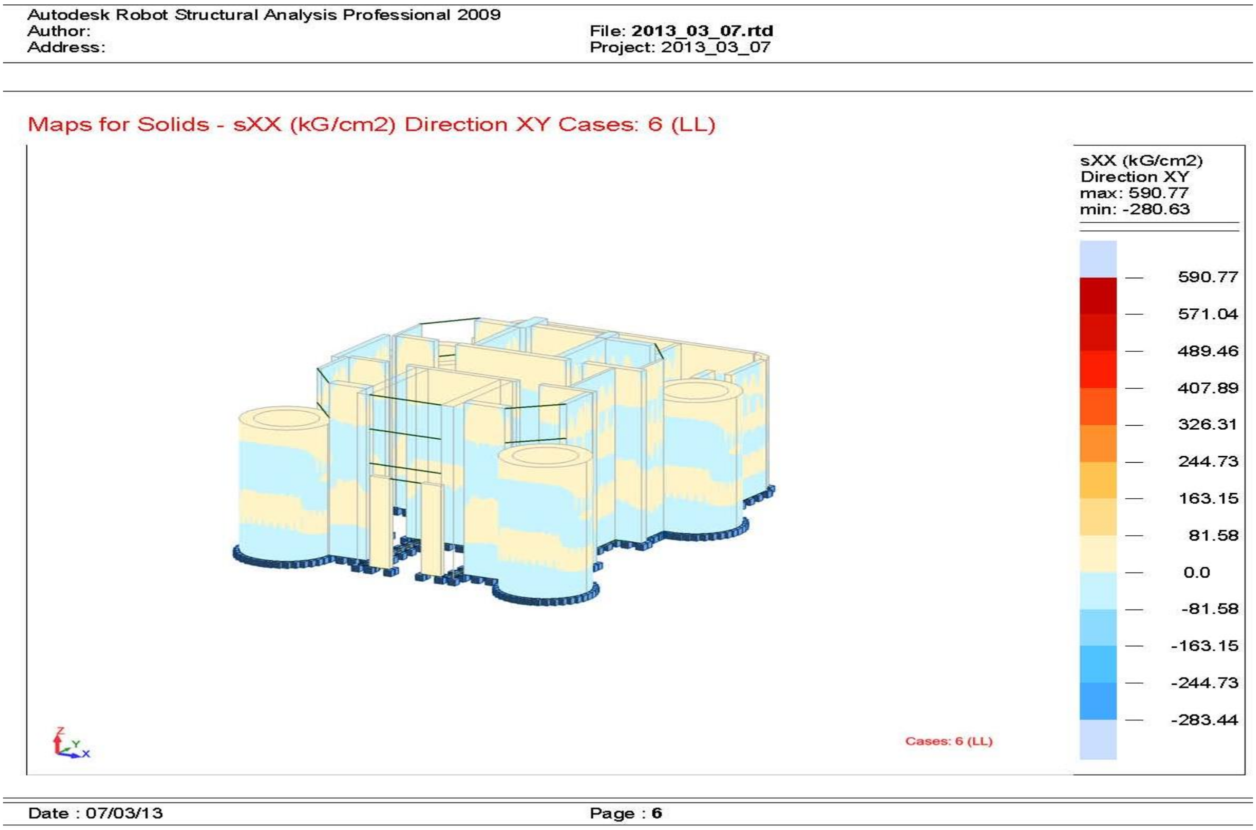

Figure.8. Maps of solids- Effective horizontal compressive stresses σ/xy (Kg/ cm2) Direction XY cases: (DL1).

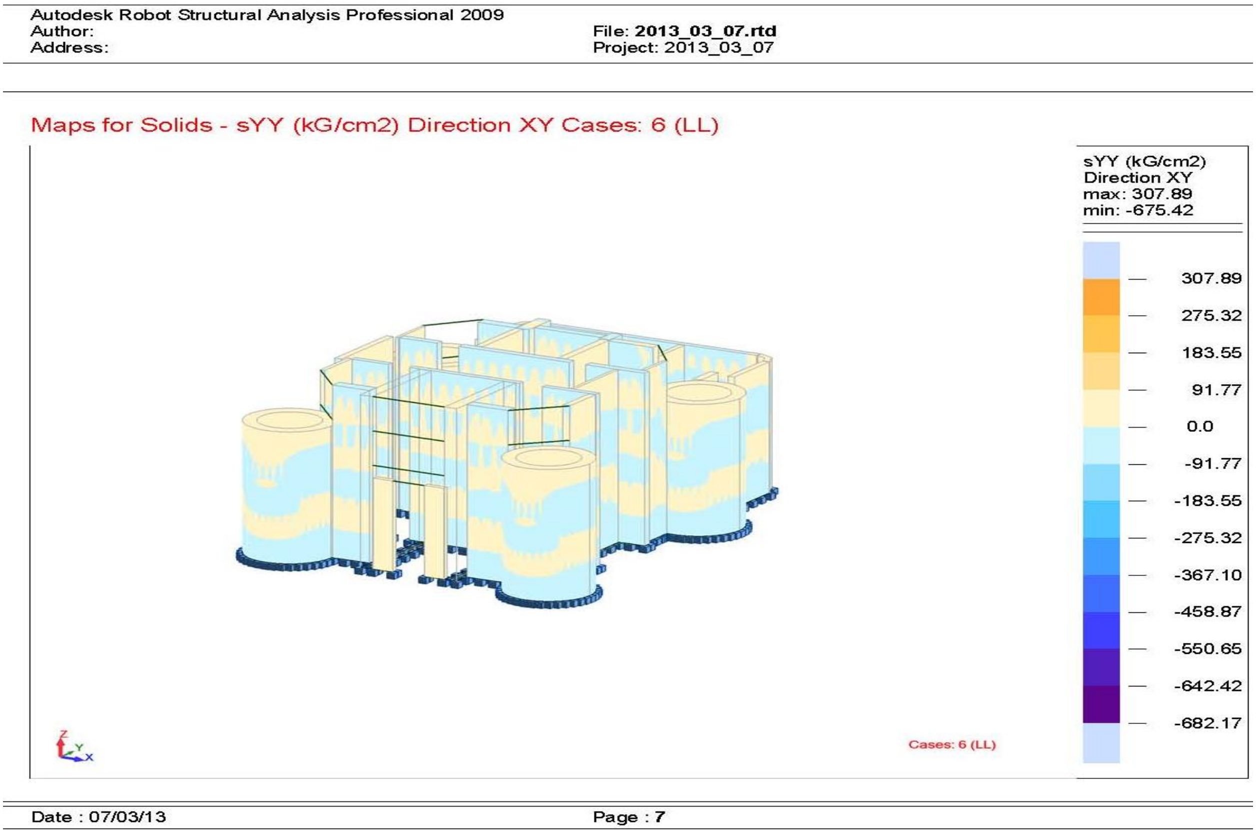

Figure.9. Maps of solids- Effective horizontal compressive stresses σ/yy (Kg/ cm2) Direction XY cases: (LL).

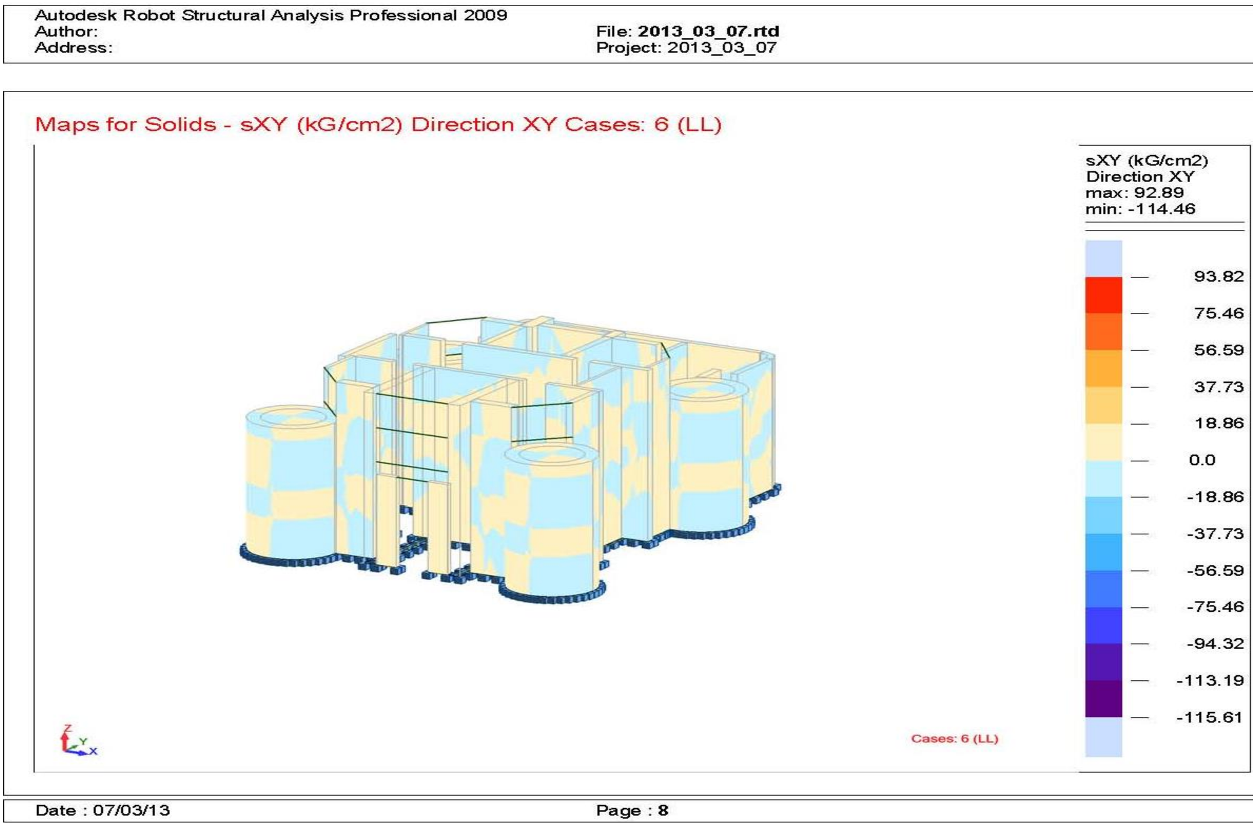

Figure.10. Maps of solids- Effective horizontal compressive stresses σ/xy (Kg/ cm2) Direction XY cases: (LL).

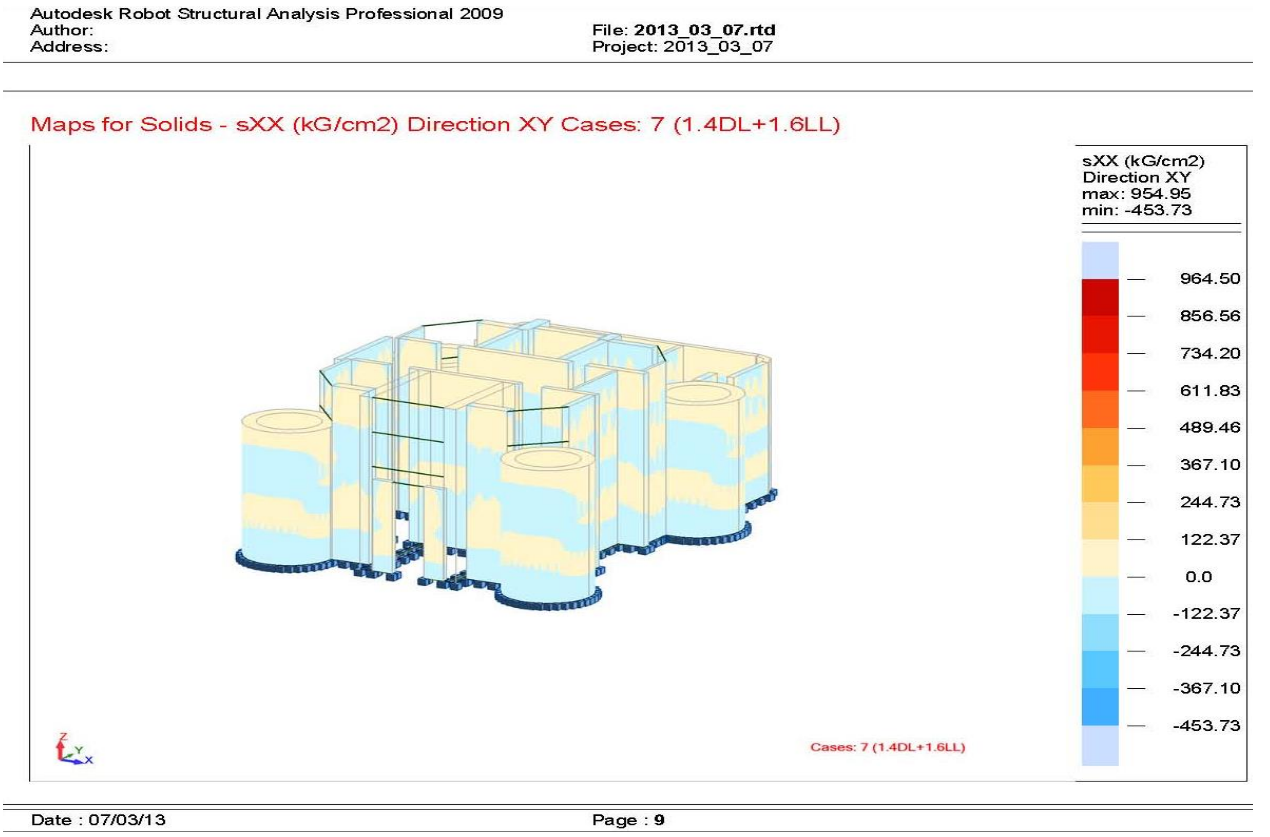

Figure.11. Maps of solids- Effective horizontal compressive stresses σ/xx (Kg/ cm2) Direction XY cases: (1.4 DL+1.6LL).

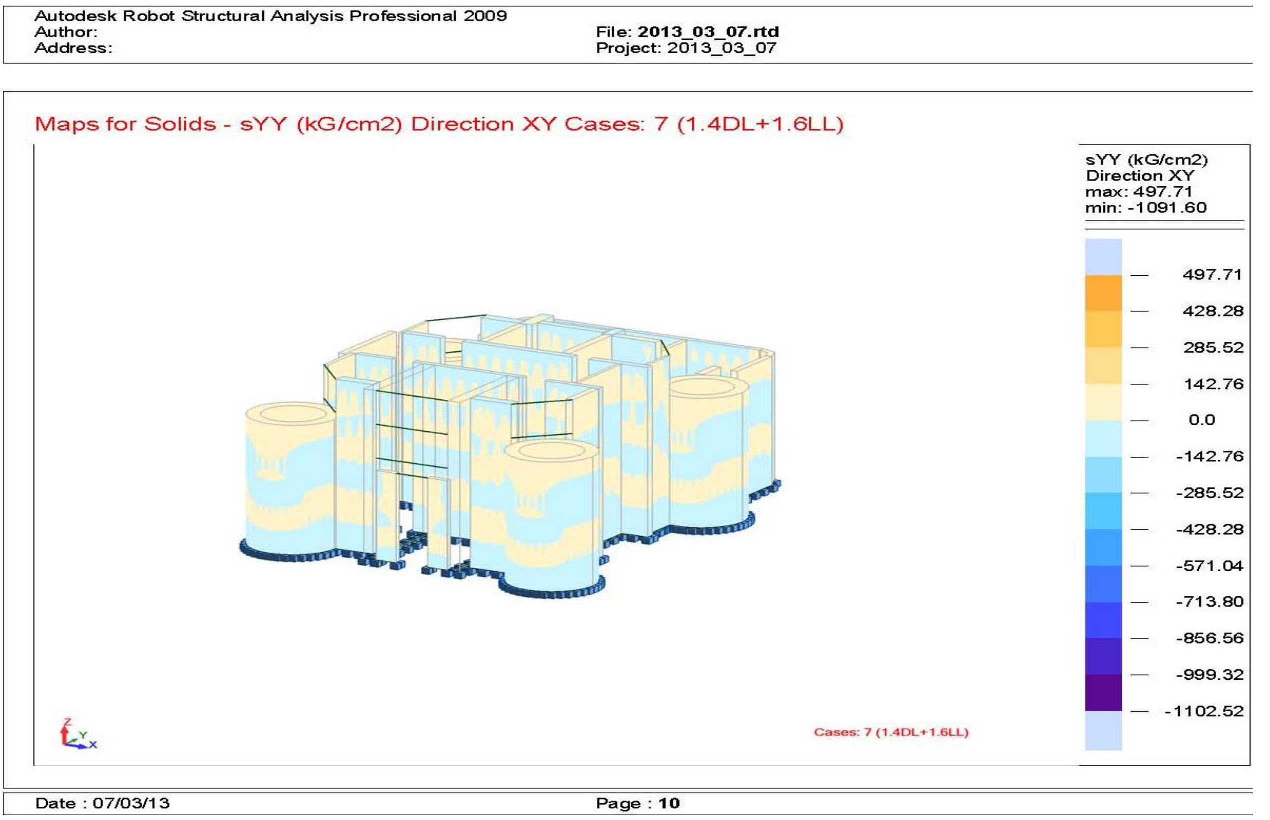

Figure.12. Maps of solids- Effective horizontal compressive stresses σ/yy (Kg/ cm2) Direction XY cases: (1.4DL+1.6LL).

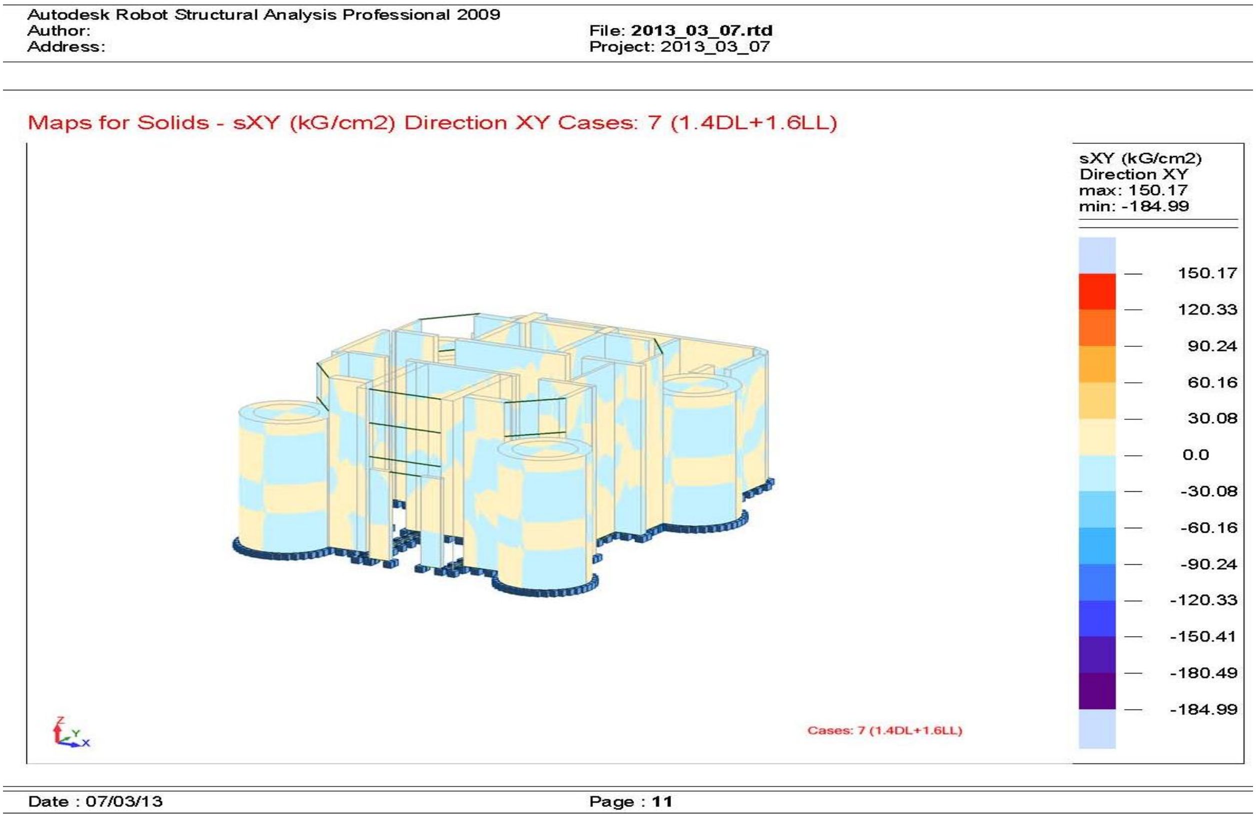

Figure.13. Maps of solids- Effective horizontal compressive stresses σ/yy (Kg/ cm2) Direction XY cases: (1.4DL+1.6LL).

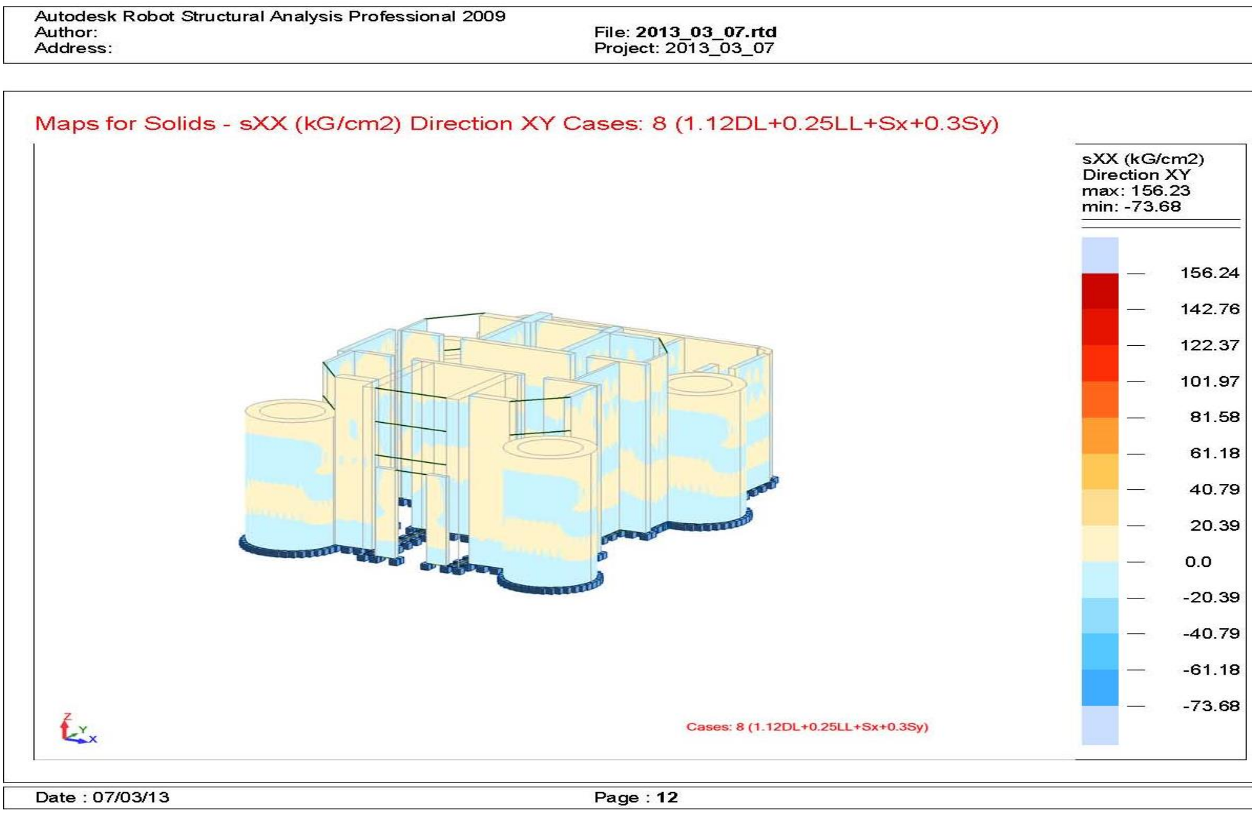

Figure.14. Maps of solids- Effective horizontal compressive stresses σ/xx (Kg/ cm2) Direction XY cases: (1.12DL+0.25LL+Sx+0.3Sy).

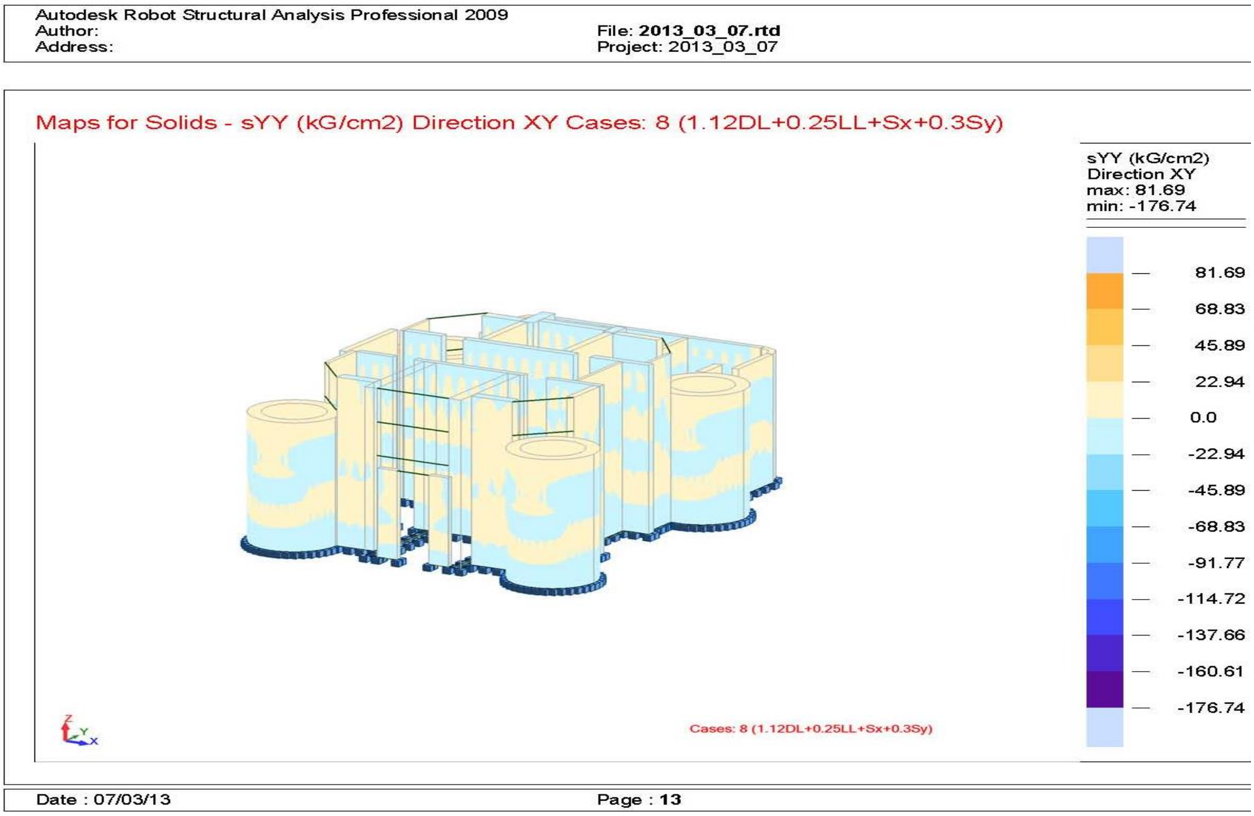

Figure.15. Maps of solids- Effective horizontal compressive stresses σ/yy(Kg/ cm2) Direction XY cases: (1.12DL+0.25LL+Sx+0.3Sy).

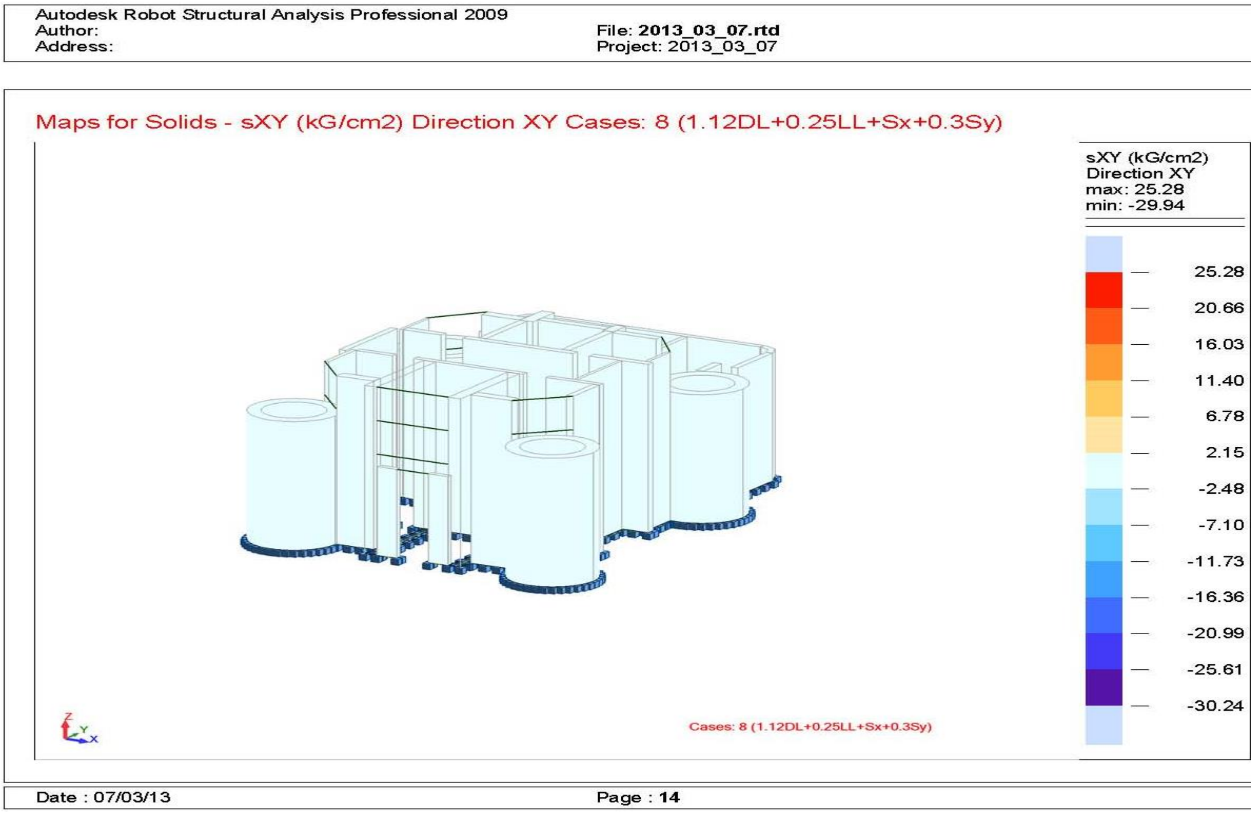

Figure.16. Maps of solids- Effective horizontal compressive stresses σ/xy(Kg/ cm2) Direction XY cases: (1.12DL+0.25LL+Sx+0.3Sy).

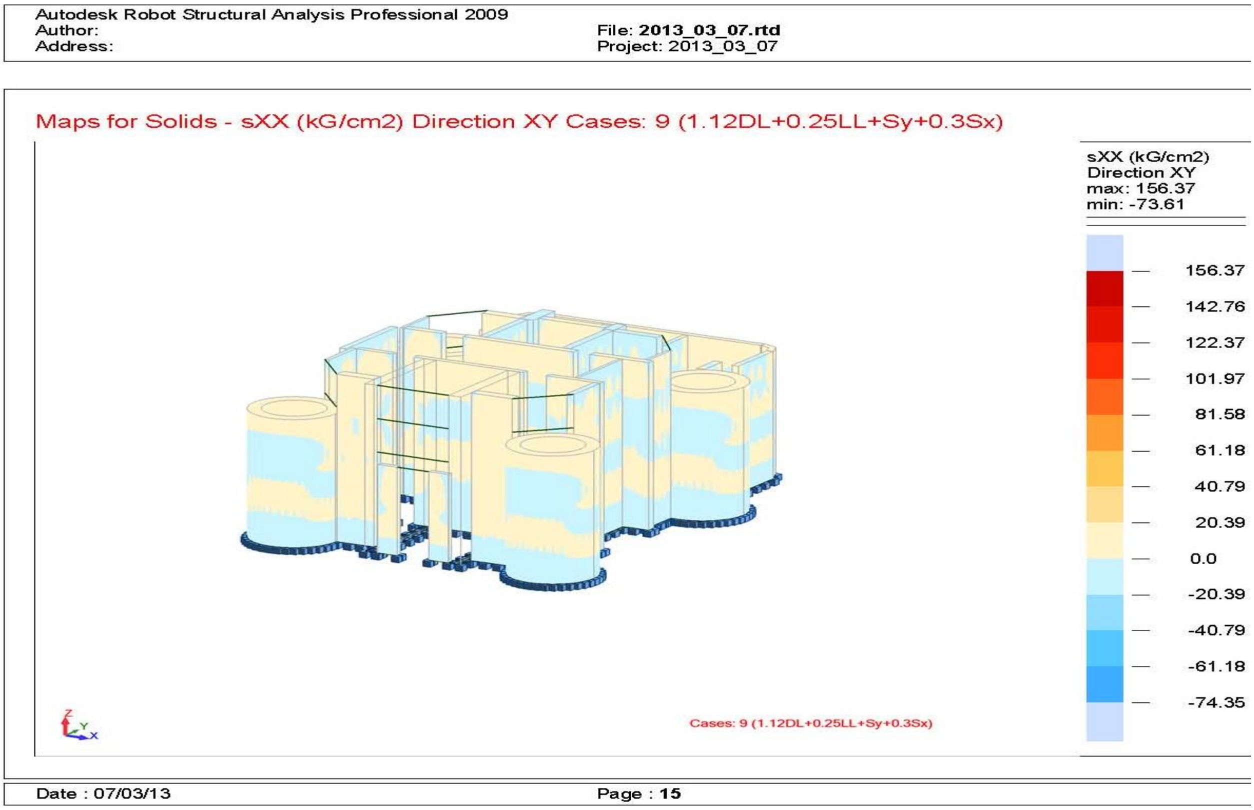

Figure.17. Maps of solids- Effective horizontal compressive stresses σ/xx(Kg/ cm2) Direction XY cases: (1.12DL+0.25LL+Sy+0.3Sx).

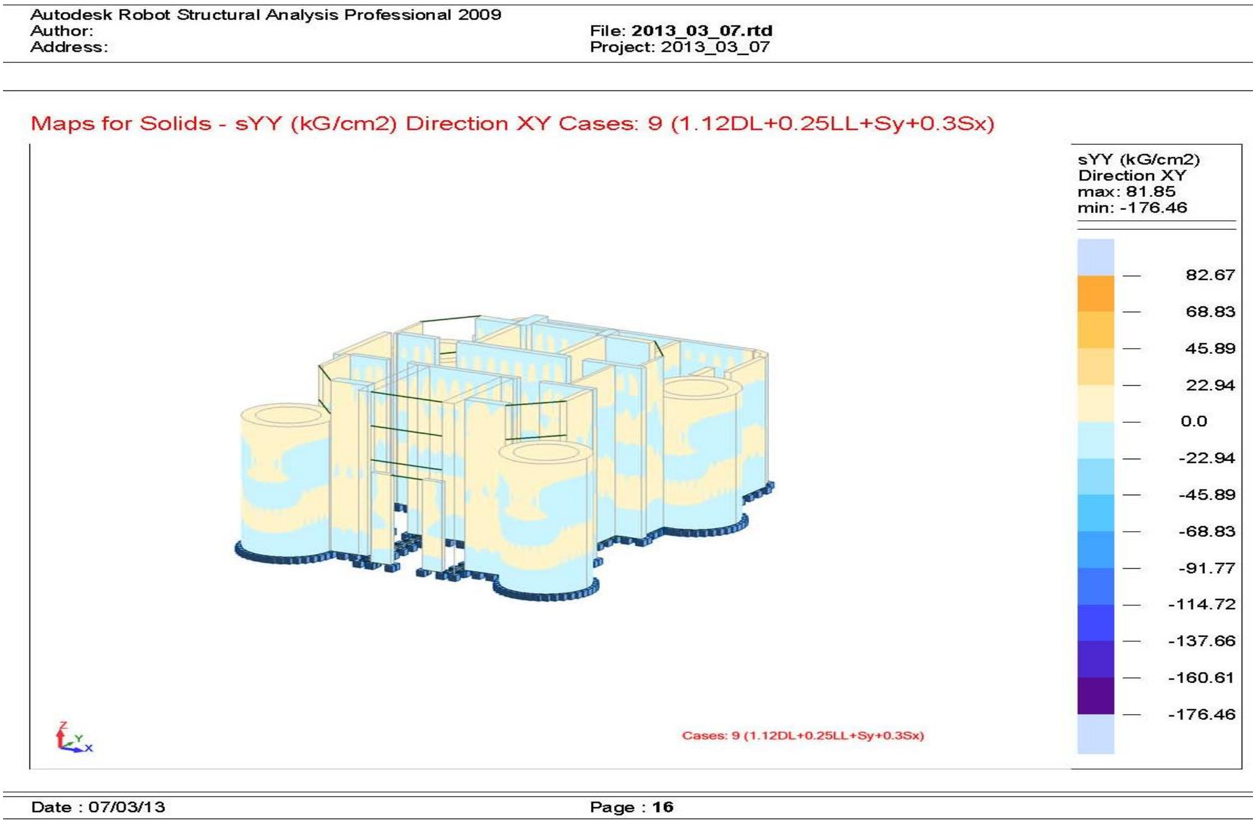

Figure.18. Maps of solids- Effective horizontal compressive stresses σ/yy(Kg/ cm2) Direction XY cases: (1.12DL+0.25LL+Sy+0.3Sx).

7. RESULTS OF NUMERICAL ANALYSIS

Based on the structural analysis, the maximum stresses due to static and lateral loads are less than the actual stone stress (200 Kg / cm^2), except the 2nd floor.

Max. Stress)$_{1.4 DL + 1.6LL}$ = 122 Kg / cm^2 < 200 Kg / cm^2 SAFE

Max Stress)$_{1.12DL + 0.5LL + Sx + 0.3Sy}$ = 23 Kg / cm^2 < 200 Kg / cm^2 SAFE

Max Stress)$_{1.12DL + 0.5LL + Sy + 0.3Sx}$ = 22 Kg / cm^2 < 200 Kg / cm^2 SAFE

For the second floor:

Max. Stress)$_{1.4 DL + 1.6LL}$ = 350 Kg / cm^2 > 200 Kg / cm^2 UNSAFE

Max Stress)$_{1.12DL + 0.5LL + Sx + 0.3Sy}$ = 23 Kg / cm^2 < 200 Kg / cm^2 SAFE

Max Stress)$_{1.12DL + 0.5LL + Sy + 0.3Sx}$ = 22 Kg / cm^2 <200 Kg / cm^2 SAFE

8. RESULTS OF THE TECHNICAL ASSESSMENT

The technical assessment revealed that almost all level masonry structural walls presented a brittle mode of failure and more than that, from the second to the fifth level were of "weak and soft stories" type.

Adding the main deficiencies of the vertical and horizontal components of the structural system, the building was classified in the first seismic risk class "Rsl" according to the Egyptian technical legislation (building with a high level risk of collapse in case of occurrence of an earthquake corresponding to the code of seismic intensity to Cairo city (a= 0.16g).

The ministry of antiquities, the owner of this historical and architectural monument decided to go ahead to the next step, which retrofitting of the building.

The structural system of the building placed in unfavorable spectral positions of the inelastic response spectra .

For a period of vibration $T_{n,1}$ = 0.4 s and for the two values of $C_{B,y}$= (0.2 and 0.13) large values of displacements can be observed.

These unfavorable "spectral positions" (on both directions) led to exaggerated value for the required ductility factors.

9. RECOMMENDATIONS

It is showing from result and calculations that, the second floor stresses exceed than the allowable, so it is highly recommended to repair it using a suitable materials determine by contractor and approve from the consultant.

10. ENHANCEMENT OF THE SEISMIC RESISTANCE

For the improvement of the seismic obstruction of the whole building, a few choices were considered:

1-Horizontal Prestressing: is compelling in reinforcing load-bearing dividers and repair wide breaks.

2-Vertical Prestressing of load-bearing dividers.

3-Reinforced solid section: which is more tough than the flexural wooden floor.

4-Horizontal fortified solid tie shafts in all dividers at floor and rooftop levels.

5-Reinforced solid tie segments at crossing points of load-bearing dividers.

6-Two-sided Shotcrete jacketing over the whole inside and outside load-bearing dividers, is a standout amongst the best yet costly retrofit conspire.

7-One-side Shotcrete jacketing, have preference more than two-side coats as they leave the outside façade of the building unaltered. They were viewed as not so much successful but rather more costly than different choices.

11. CONCLUSION

The technical assessment and the strengthening of an old monumental unreinforced masonry building are domains where decisions are taken based on risk analysis, in order to reach a compromise between the historical value and the cost of investigations.

It was discovered that the building has the inclination of limiting harm at the second and third level, with the advancement of a " delicate and frail levels" impact (circumstance which relates to a conceivable general dynamic crumple.

The present, the fortifying arrangement is required, and the works for retrofitting and restoration of the "Royal residence" exhibition hall is required, as the proprietor will designate the vital assets.

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