Investigation on Performance of Heritage Masonry Structures

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Abstract

A structural assessment study on the "Humayun Palace" in Tamil Nadu, a cultural heritage structure, constructed in the early 1700's was done. Based on the original design documentation collected through records, a static analysis model of the building was generated. This structure is a 3-storey building and details of all the provisions from the slab system to the roof tiles were made. The results developed by these models, aimed at fully understanding the original design concept of the various members, as well as at evaluating their current static and seismic safety conditions, are reported in this paper. This analysis was examined using E-TABS software, whose geometric limitations lead to minor encumbrances. Some complex geometric façade features were approximated to convenience and overcome. The results of the analyses highlight safe conditions and good performance objectives in general, but for some important exceptions. The overall satisfactory performance may be attributed to the overdesigning with excessive safety factors carried out by builders of that century, enabling the structure to withstand conditions way past its intended life. However, the question as to how well the structure will hold good in the future is hypothetical. It is for the same reason that the Public Works Department has allocated nearly 96 crore rupees for the rehabilitation of the same. This investigation is synchronous with the objectives of the said rehabilitation project and has established an exhaustive analysis of the structural health of the structure.

Keywords: Un-reinforced masonry, Rehabilitation, Response reduction factor, Importance factor, Retrofitting

1. Introduction

1.1 General

World-wide experience from past earthquakes reveals the particular vulnerability of un-reinforced masonry structures to seismic action. Hence, these structures represent a severe threat to human and other resources in the case of similar future seismic events. This requires an urgent assessment of existing buildings in terms ofthe strength, expected performance and safety of existing buildings during earthquake as well as for carrying out the necessary rehabilitation. The Chepauk Palace comprises two blocks—the northern block is known as Kalas Mahal while the southern block is known as Humayun Mahal. Located within a 117 acre campus, the Humayun Mahal is spread over 66,000 square feet and has ventilation on the terrace and a connecting corridor to the Kalas Mahal. Fig 1. exhibits the structure's original physical appearance during time of construction. This investigation was focused on Humayun Mahal specifically.

Fig 1. Original Construction of the Chepauk Palace.

Fig 2. Renovation of the Structure during fire accident in 2016.

1.2 Need for Study

Humayun Mahal has suffered decades of neglect: languishing in dilapidated condition for years and later ravaged by fire. Due to such improper maintenance and the recent fire accident deteriorating the structure even further, the current strength, stability and durability of the structure is questionable. For the same reason the Public Works Department of India has decided to allocate Rupees 95.98 crores for the renovation of the same. Figure 2 shows the renovation that took place for the structure after the devastating fire accident. The significance of architectural heritage and the desirability of conserving it are now more readily recognized, asnoticed in the largely increasing public concern in favor of preserving and enhancing the familiar local scene.

1.3 Objective

The main agenda in this project is to model and analyze the said structure. The intention for experimentation is to assess the performance of the structure. The structure was built 252 years ago and hence, there is a pertinent need to model the structure using the E-tabs software and adopt the static analysis model for the same. Relevant loads such as dead load, live load, wind load and seismic load are applied on the model in accordance to currently prevalent Indian Standard Codes. In doing so, an assessment on to what extent the structure will fare in today's context can be obtained. Through this investigation, an exhaustive report on the health of the structure and possible retrofitting or rehabilitation for the sections of the structure which do not show promising results is done.

2. Literature Study

2.1 General

Ancient monumental structures are complex structures but have not been engineered to undergo the many transformations they encounter during their lifeand often present lack of connections among the structural elements. For this purpose, extensive research and investigation on this agenda has garnered popularity over the years. The following research papers discussed in the below section are the basis and inspiration of this study.

2.2 Literature Review

Ahmad et al (2014) [1] have investigated the structural inadequacy of a century old unreinforced masonry heritage building, to withstand seismic loads and have extended that such similar structures require an urgent assessment in terms of the strength, expected performance and safety. In this study they have investigated and experimented to evaluate the seismic performance of a 137-year-old unreinforced masonry heritage school building located at Aligarh Muslim University, Aligarh (seismic zone IV). This given structure is a load bearing structure. The seismic analysis of the building is done using equivalent static method as described in the code IS 1893(Part I): 2002. Natural frequencies and mode shapes of the building have been obtained through modal analysis approach using STAAD Pro. The building is modelled using finite element technique and its seismic evaluation is carried out assuming a homogeneous and nonlinear behavior of the material. The building is subjected to different peak ground acceleration (PGA) levels (0.1g, 0.2g, 0.3g, 0.4g) as input ground motion. The portions around the openings were found to be highly vulnerable in all cases.

Miceikaitė and Parasonis [2] have investigated the representative palace of Biržai castle. The palace was rebuilt and tailored to become a museum and library. All these rebuilding works were done without sufficient geotechnical data. The purpose is to find out the explanation for the exterior walls splitting and present solutions to stabilize the current situation. The model was simulated using STAAD Pro and Finite Element Analysis was performed for the same. Two methods adopted were (a). Modelling the wall without taking splits into account in the current wall, (b). Modelling the wall with its natural splits. It is inferred that splits are influencing both stresses values and their distribution in the wall. Authors conclude that an improper remains analysis of masonry structures was done, leading to the difficulty in ascertaining the distinction between original and new wall places. For that reason, it is hard to decide on masonry loadbearing values. Computing analysis results assert that a hypothesis for compressive strengths of 1.6-2.6 MPa leads to over-limit principal stresses without taking the splits into account. The results highlight that wall stresses and deflection results are influenced by thrust loads and splits places. This information is useful to decide on suitable walls consolidation techniques.

Saloustros et al (2020) [3] have presented the systematic use of numerical analysis as a tool for addressing some of the most common challenges encountered in the structural analysis of complex historical masonry structures. Conservation of built cultural heritage is a challenging task that relies on the implementation of a multidisciplinary scientific approach based on inspection, structural analysis, and monitoring. This study deals with the application of FE macro-modeling analysis for the structural assessment of complex historical masonry structures against long-term deformation, past historical alterations, and seismic actions. The first part is devoted to the presentation of a FEM model based on continuum mechanics theory. This model is able to reflect mechanical phenomena typically encountered in masonry structures. The second part presents two applications of the FEM on two important monuments in Spain: the cathedral of Mallorca and the church of the Poblet monastery, a UNESCO World Heritage Site. The third part includes a probabilistic analysis for assessing the effect of uncertainties related with material and numerical properties on the seismic response of the church of the Poblet monastery. The proposed mechanical model for masonry is able to simulate the time-dependent strain accumulation in the material because of long-term exposure to constant stress, including the description of tensile and compressive damage. It also elucidated on the use of numerical tools for structural analysis and their combination with historical investigation, in-situ inspection, and monitoring activities for two important large masonry historical structures in Spain. A numerical strategy based on the use of a macro-modeling approach was chosen as a way to reduce the computational cost and to permit the efficient analysis of the large-scale structures. The constitutive model plays an important role in this strategy; it was chosen to properly represent the distinct nonlinear tensile and compressive behavior of masonry, as well as its time-dependent response. The application of this methodology to case studies permitted the interpretation of their current pathology and allows to take optimum decisions of future studies, such as the definition of monitoring activities.

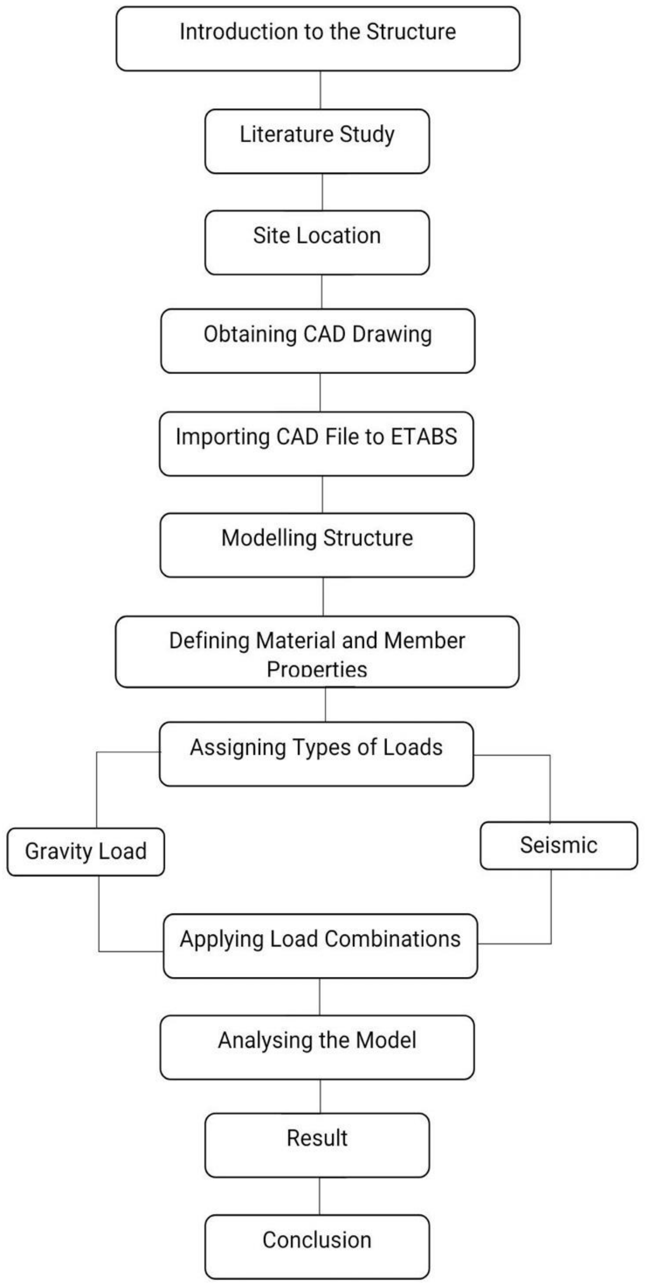
Dizhur et.al. [4] have observed the earthquake damage to masonry buildings and churches as a result of the 22nd of February 2011 Christchurch earthquake. The study focused on investigating commonly encountered failure patterns and collapse mechanisms. A detailed study was done based on the observations that were made on the performance and the deficiencies that contributed to the damage formed in unreinforced clay brick masonry (URM) buildings, unreinforced stone masonry buildings, reinforced concrete masonry (RCM) buildings. This was achieved by investigating the failure patternsand collapse mechanisms that were commonly encountered. The shapes and modelling of the structure is done by suitable software. The modelling of the building was carriedout by finite element analysis. From the above observation, details are provided of retrofit techniques that were implemented within relevant Christchurch URMbuildings prior to the 22nd of February earthquake. A very brief outline on the possible retrofit strategies and repair techniques for stone masonry buildings was also presented.

2.3 Summary of Literature

From these references, it was construed as to how various investigations on various heritage structures were conducted. The usefulness of the results of a model simulation in aiding the structural health of a structure was assessed. The various retrofitting and rehabilitation strategies given help us understand that such results from simulations can be implemented in real time work, which is extremely efficient and accurate. To dwell more on the same lines,a similar investigation is undertaken on the Humayun Palace.

3. Methodology

Work methodology is given in a flowchart as follows:



3.1. Layout of the Structure

The plan layout (various views and sections)of the Humayun palace is shown in Fig.3 a-f.

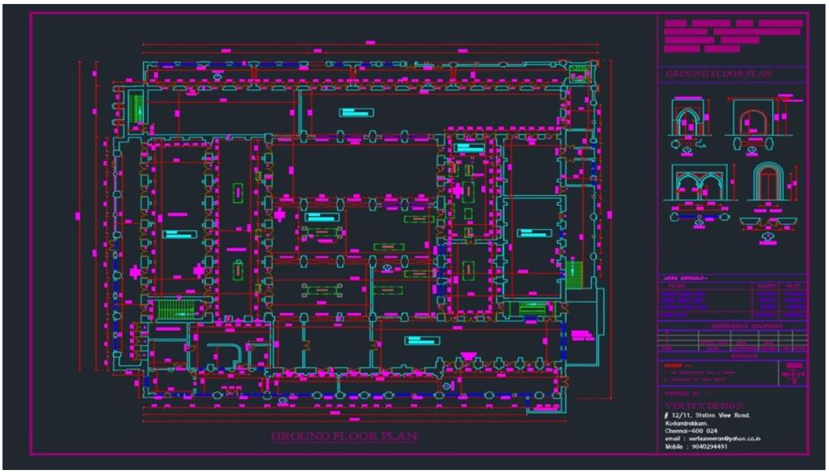


Fig.3.a Shows the Plan layout of the Humayun Palace in the format of a CAD drawing.

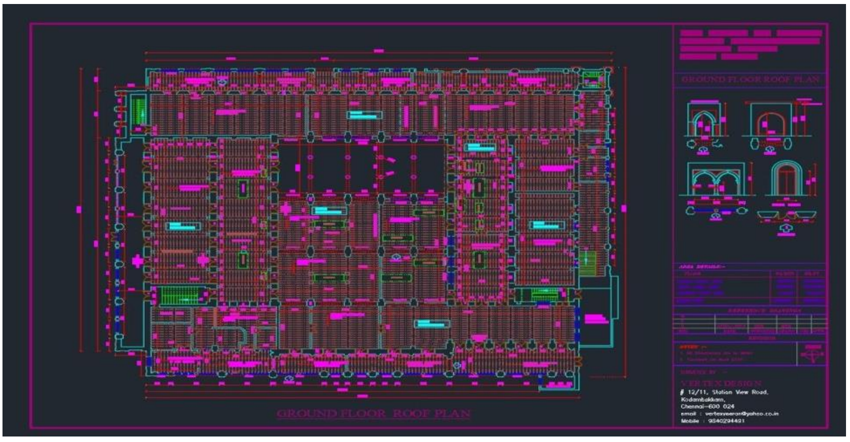


Fig.3.b. Shows the Plan layout of the Humayun Palace in the format of a CAD drawing.

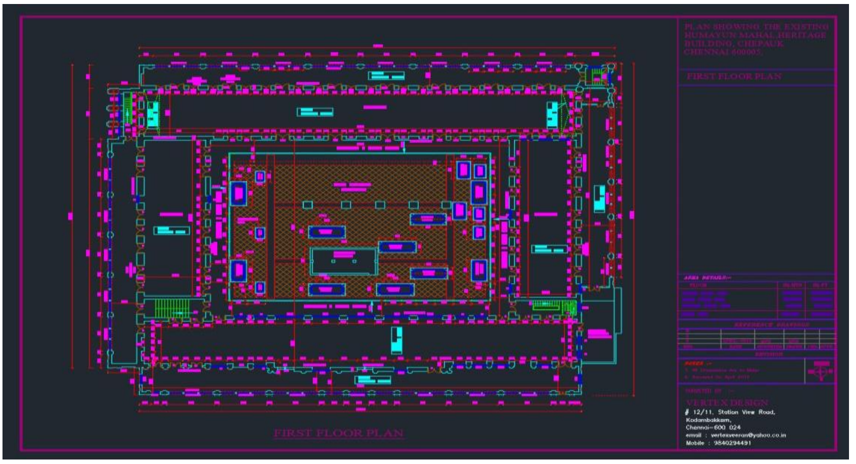


Fig.3.c. Shows the Plan layout of the Humayun Palace in the format of a CAD drawing.

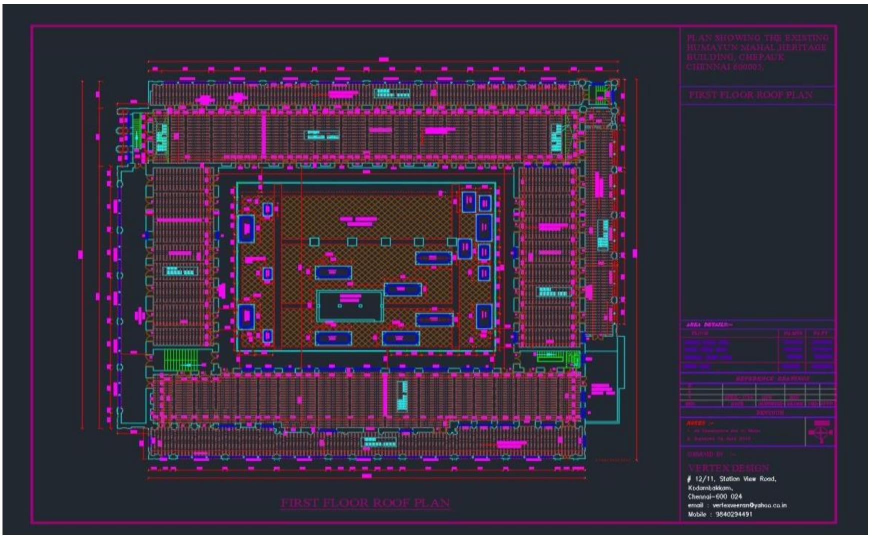


Fig.3.d. Shows the Plan layout of the Humayun Palace in the format of a CAD drawing.

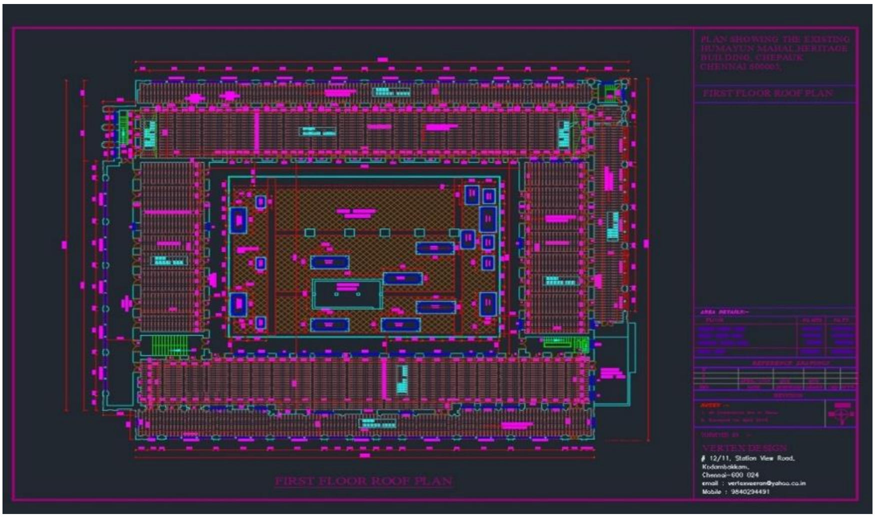


Fig.3.e. Shows the Plan layout of the Humayun Palace in the format of a CAD drawing.

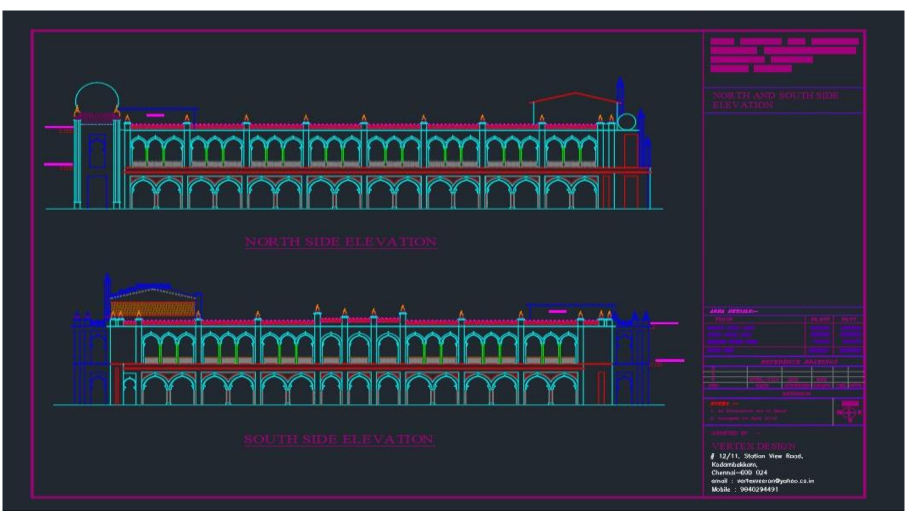


Fig.3.f. Shows the Plan layout of the Humayun Palace in the format of a CAD drawing.

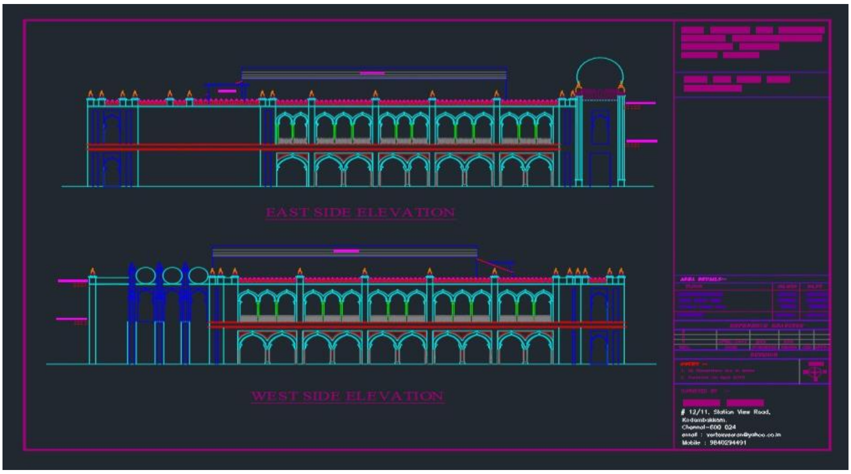


Fig.3.g. Shows the Plan layout of the Humayun Palace in the format of a CAD drawing.

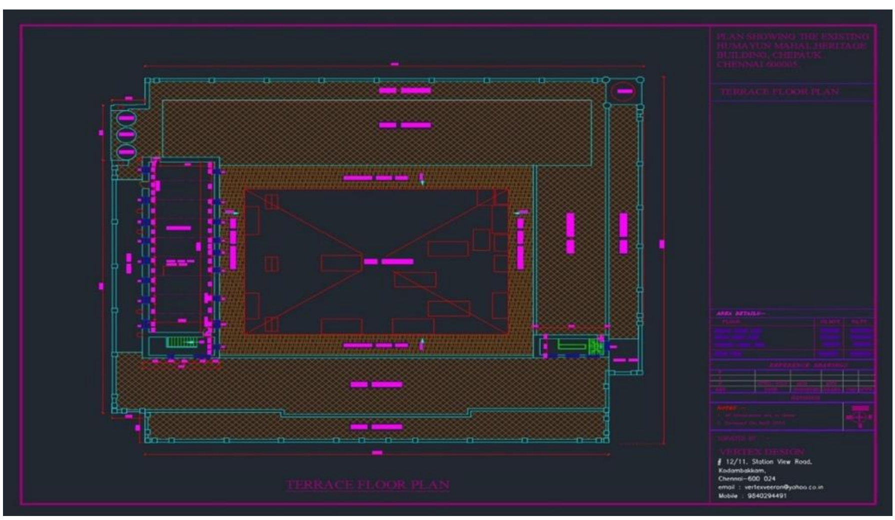


Fig.3.f. Shows the Plan layout of the Humayun Palace in the format of a CAD drawing.

3.2. Working Procedure

1. The plan layout of the Humayun Palace (Chepauk Palace) was obtainedas a CAD file.
2. A centerline drawing of the ground floor layout was drawn and this CAD file was exported to the ETABS software.
3. The material properties for masonry walls, wooden beams, woodenrafters, tile slab and brick slab were inputted.
4. The section properties were defined for the masonry walls, woodenbeams, wooden rafters, tile slab and brick slab.
5. Walls are designed with provisions for necessary arch and door openings.
6. Openings for arches and doors were made.
7. Main wooden beams of 400x800mm cross-section were drawn.
8. Small wooden rafters of size 76 x 127mm were used as secondary beams.
9. 300 mm thick slabs were used. They were assumed to be made ofbrick masonry.
10. All the catch basins in the structure were filled with walls.
11. This model was replicated for the first floor, with necessary modificationsmade with respect to the original plan layout.
12. Relevant loads were applied. The load combinations are as follows –

1.	Dead Load	Live load	
2.	Dead Load	Live Load	Seismic Load X direction
3.	Dead Load	Live Load	Seismic Load -Y direction
4.	Dead Load	Live Load	Seismic Load-

5 .	Dead Load	Live Load	Seismic Load-
6 .	Dead Load	Seismic Load X direction	
7 .	Dead Load	Seismic Load -X direction	
8 .	Dead Load	Seismic Load +Y direction	
9 .	Dead Load	Seismic Load -Y direction	

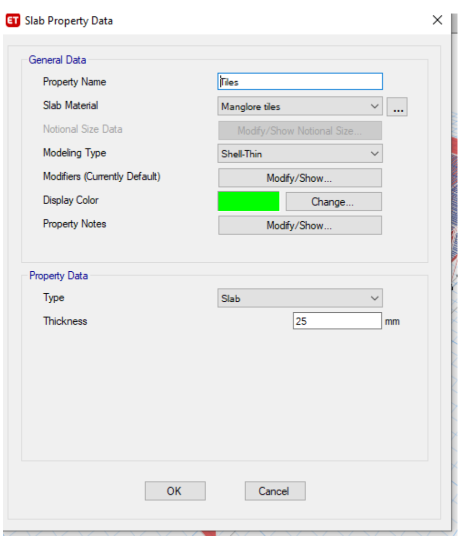


Fig.4.a Material and slab properties adopted during

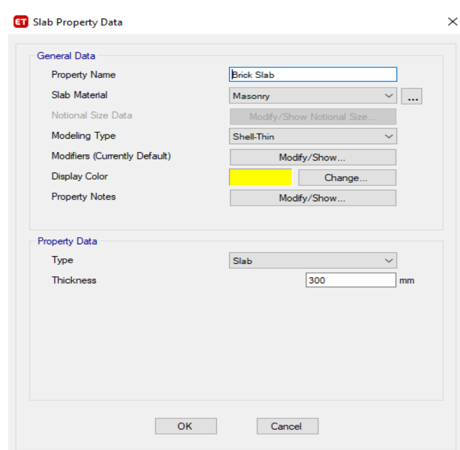


Fig.4.b. Material and slab properties adopted during

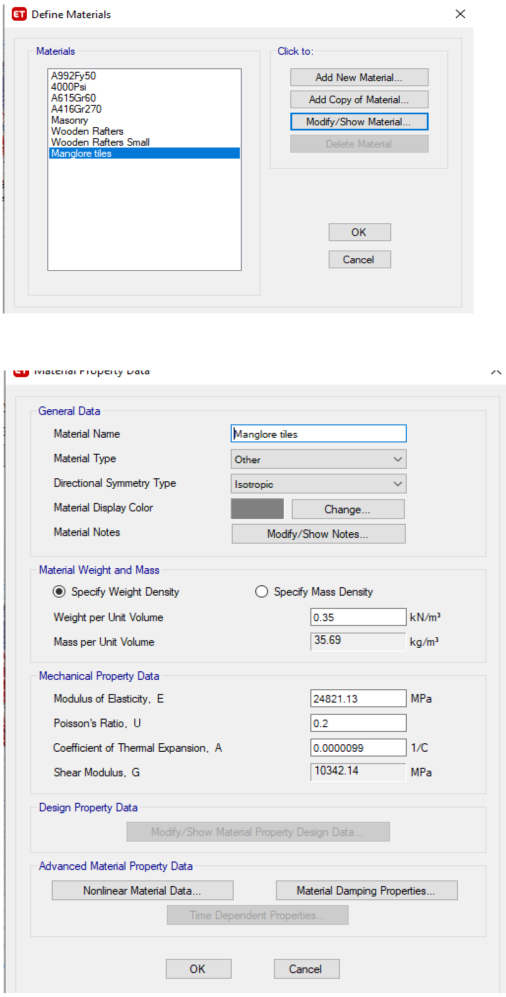


Fig.4.c. Material and slab properties adopted during analysis.

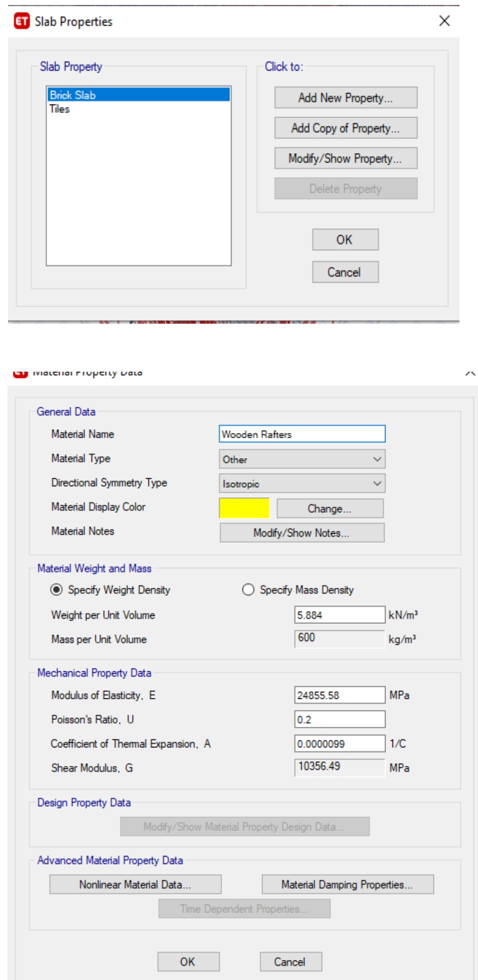


Fig.4.d. Material and slab properties adopted during analysis.

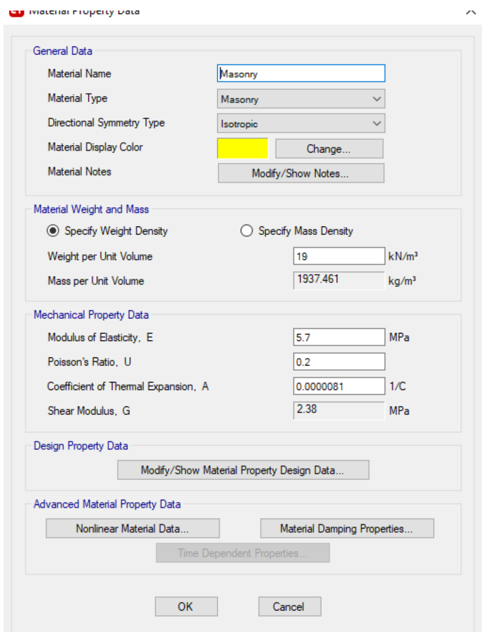


Fig.4.e. Material and slab properties adopted during analysis.

3.3. Seismic load considerations

- Seismic Loading was taken as per IS 1893:2016 Part 1. As, Chennai comes under Zone III as per the code, the following considerations were taken:
- Response Reduction Factor=1.5 for unreinforced masonry.
- Seismic Zone Factor = 0.1 for Zone III, shown in Fig.5.
- Importance Factor for important buildings such as monuments and government buildings
- Time period was calculated by the software.
- The seismic forces in the x and y directions are considered. Vertical seismic forces are ignored as the building is only two stories tall.

Table 3 Seismic Zone Factor *Z*
(*Clause* 6.4.2)

Seismic Zone Factor (1)	II (2)	III (3)	IV (4)	V (5)
Z	0.10	0.16	0.24	0.36

Fig.5 Seismic zone factor as per IS 1893

3.4. Combinations for Design of horizontal earthquake loadsAs Per IS Codes

1) 1.2 [DL+IL± (EL, ±0.3 EL,)] and 1.2 [DL+IL± (EL, ±0.3 EL¸)].

2) 1.5 [DL± (EL, +0.3 EL,)] and 1.5 [DL (EL 0.3 EL¸)]; and

3) 0.9 DL 1.5 (EL 0.3 EL) and 0.9 DL ± 1.5 (EL, +0.3 EL).

3.4.1.Combinations to be accounted to design of a three-dimensional structure according to IS 1983 2016

The seismic combination obtained the response due earthquake force (EL)is the maximum of the following three cases:

1) ± ELx±O.3 ELy±0.3ELz

2) ±ELy±O.3 ELx±0.3 ELz

3) ± ELz± 0.3 ELx±0.3 ELy

1) 1.2 [DL+IL+(EL, ±0.3 EL, ±0.3 EL,)] and
1.2 [DL + IL ± (EL, ±0.3 EL, ±0.3 EL)].
2) 1.5 [DL (EL, ±0.3 EL, 0.3 EL,)] and
1.5 [DL± (EL, ±0.3 EL, +0.3 EL,)]; and
3) 0.9 DL 1.5 (EL, ±0.3 EL, ±0.3 EL,) and
0.9 DL+ 1.5 (EL, +0.3 EL ±0.3 EL).

where x and y are two orthogonal directions and z is vertical direction.

Where,

$EL= \sqrt{} (ELx)^2 + (ELy)^2 + (ELz)^2$

3.5. Assumptions

13. Minor masonry projections were neglected.
14. Arches were taken as rectangular shapes. This was done as ETABS cannot factor in arch shapes and replacing it with a rectangular shapewouldn't cause much of a difference. Irregular shapes were taken as definitive shapes, such as squares or rectangles etc. This is illustratedin Fig.3. Fig 6(b) is the actual plan layout of the ground floor and Fig 6(a) is the center line diagram for the ground floor, which we took foranalysis purposes.
15. We considered the domes as concentrated loads.
16. We took the live load to be 3 kN/m.
17. We have considered the dome as a concentrated load and thecalculations are given below:

Diameter of the dome = 4.7m Thickness of the dome =100mm
Unit weight of brick is taken as 19Kn/m3 (as the dome is constructedusing brick masonry)

As we know, Surface area of hemisphere is

$$\frac{2}{3} \times \pi \times (r)^2 \; x \; (thickness \; of \; dome)$$

Load = unit weight of brick x is

$$\frac{2}{3} \times \pi \times (\frac{4.7}{2})^{2} \; x \; 0.1$$

By the equation, we got the concentrated load as 21.9Kn ~ $22kN$

Which we applied it as a gravity concentrated load to the structure.

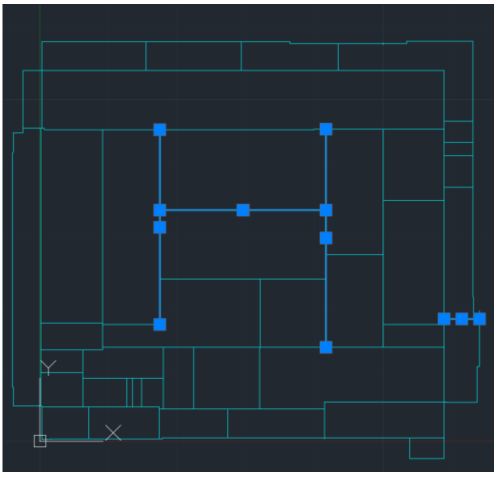


Fig.6.a. Centre line diagram of the ground floor used for analysis.

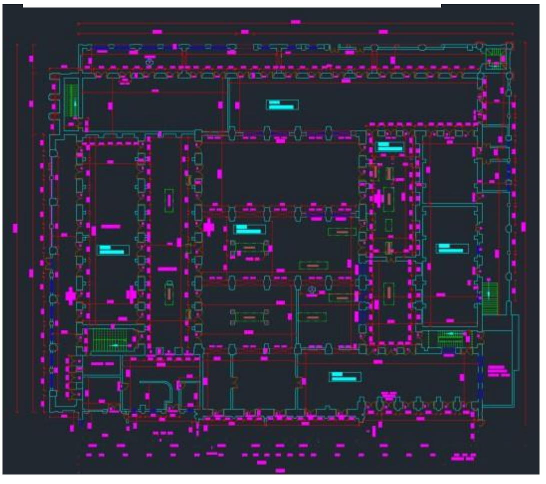


Fig.6.b. Actual plan layout of ground floor.

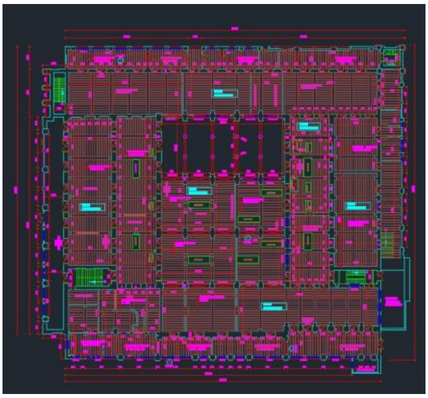


Fig.6.c. Ground floor top view CAD layout

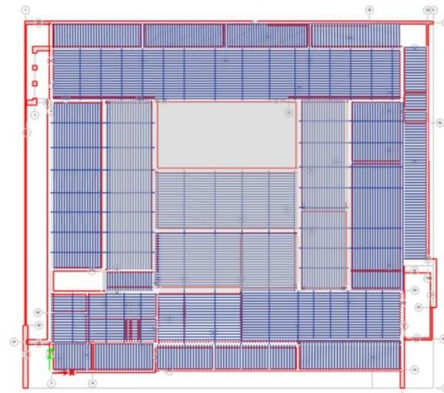


Fig.6.d. Modelled ground floor top view in ETABS

Fig. 6. Relative study of both the ground floor roof drawing.

3.6. Analysis

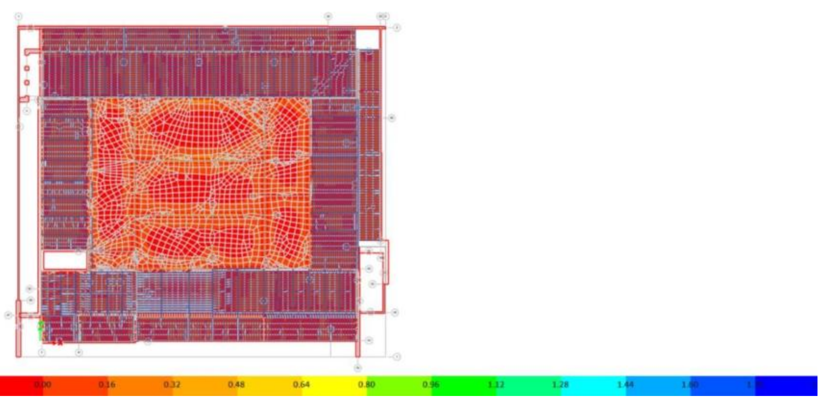


Fig.7.a. represents the maximum shell stress (kN/m2) distribution diagram in the plan view.

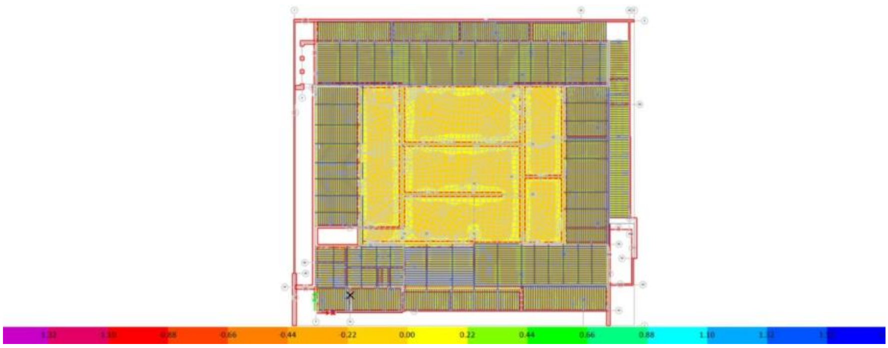


Fig .7.b. represents the average stress (kN/m^2) distribution diagram inthe plan view.

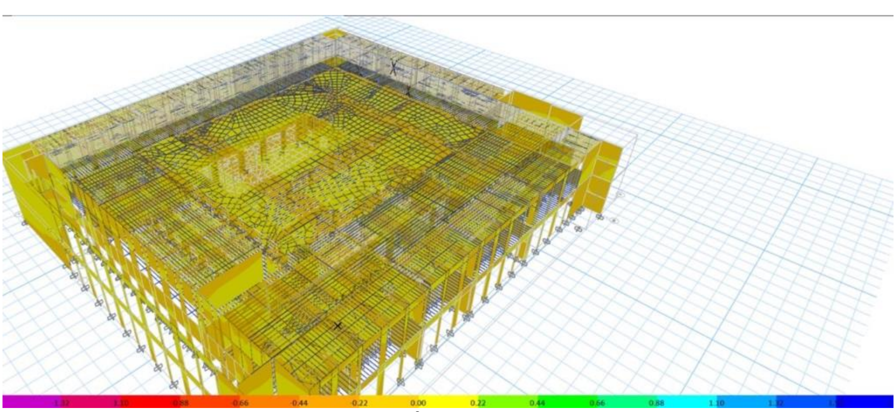


Fig.7.c. shows the average stress (kN/m^2) distribution in a 3-dimensional format.

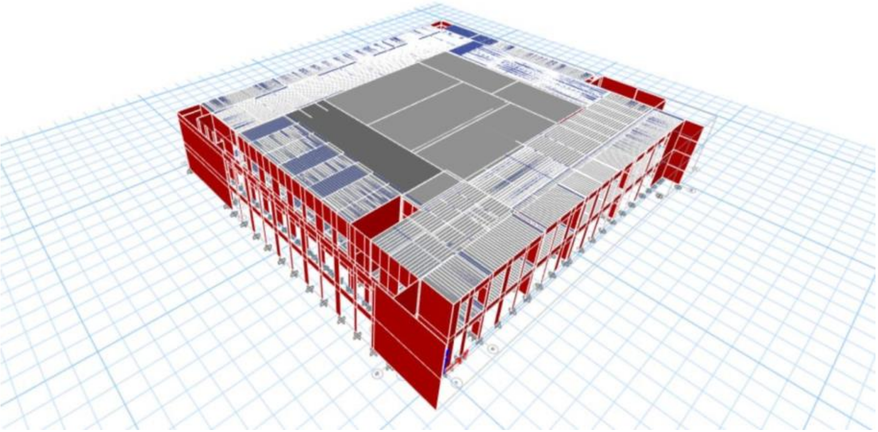


Fig. 7. d. shows the extruded 3-dimensional model and the 3-dimensional view if the model respectively.

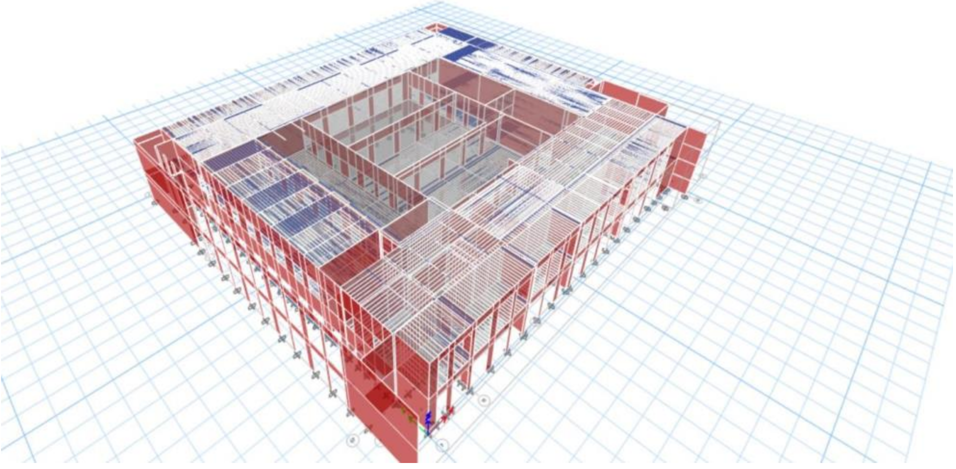


Fig. 7. e. show the extruded 3-dimensional model and the 3-dimensional view of the model respectively.

3.6.1. Important discussions from analysis

As from the analysis we could understand that the above shell stress/ forces contours obtained from its deformed shape under extreme envelope seismic design combinations. All the stresses are within the permissible limit which is ranging between 0.00 – 0.22 kN/m^2. So, the structure does not require any retrofits or seismic control systems for the building. The said structure was found to be structurally over designed and can withstand the present relevant seismic IS codes.

As mentioned already, arch openings are considered regular rectangular openings, which is why there aren’t any failure zones observed there. If so, any failure zones were visible on the arch openings, and necessary seismic control systems can be provided depending on Indian needs,

The generally used seismic control systems are namely:

1. Viscous Dampers
2. FRP (Fibre Reinforced Polymer)
3. Steel Dampers
4. Bracing systems
5. Shape memory Alloys
6. Hybrid control Systems

Out of which hybrid control systems are the most commonly used seismic control methods, they achieve the performance of the building by control thebase displacement of the building.

4. Results and Discussions

The given heritage structure was modelled with the ETABS software using staticanalysis modelling. It shows that the shells have undergone minimal stress and strain. This may be attributed to the fact that proper foresighted planning was done. However, a few possible reasons may be deduced for the said results, thembeing: -

- Most modern commercial structures are constructed with 110 to 115 mm thickness slabs, but in this structure, the predominant thickness adopted was roughly 3 times more, i.e., 300mm.
- Usual walls constructed are 230 mm thick, but in this structure most wallsare of 400-1000 mm thickness.
- This structure also follows a different madras terrace slab system. Usually, rafters are placed in a certain centre-to-centre distance over which tiles are placed. But in this case, a main beam of comparatively bigger size is placed,and secondary beams of relatively smaller sizes are placed perpendicular to the same.
- Most importantly we have considered the arch openings are regular rectangular openings as there were some limitations working on E-Tabs if arch openings are incorporated, we might have identified some failure zones for the same and suitable retrofitting for the buildings can beprovided.

Following all these reasons, it is evident that the structure has been overdesigned, which capacitates it to have added stability and shows promising results. The Public Works Department has allocated nearly 96 crores for the aesthetic restoration of the structure, which was damaged during the fire of 2014.

5. Conclusions

Through this investigation, the value of the structure was brought out. It is evident that the E-TABS software and the static analysis method, though having a few restrictions, are brilliant means of simulations that can be used for further studies in monitoring or checking the health of the structure. The structure, through the ravages of time and deterioration, has stayed strong and shows promising results through the analysis. This can be owed to the overdesigning and sustainable provisions incorporated by the constructors during the time of construction. However, because of the restrictions the software laid, such as not allowing the actual shapes of the arches and other irregular shapes, the necessity to replace them with definitive shapes was inevitable. This may constitute the reason forthe structure not showing failure. In retrospect, if these provisions were to be incorporated, the structure may be prone to failure. The solutions to solve the issueof failure have also been furnished. Such cultural heritage sites must be preservedwith utmost care. The recent fire accident compounded the need to rehabilitate this structure, which has drawn the attention of the Public Works Department to take this project up for the same cause.

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