



Old masonry tower analysis visual inspection, NDT, and macro-modeling

Ambareesh Kumar¹ · Kumar Pallav² 

Received: 8 June 2023 / Accepted: 10 June 2023 / Published online: 27 June 2023
© The Author(s) 2023

Abstract

A comprehensive analysis was conducted on a 106-year-old masonry tower to assess its response to gravitational forces and wind effects. Various techniques, including visual inspection, non-destructive testing, and finite element analysis (FEA), were employed in this study. Visual inspection played a vital role in evaluating the Tower's exterior and interior components, aiming to detect signs of damage or wear at any level and comparing them to the analyzed model. Dimensioned and assembled drawings were utilized to create a detailed 3D finite element model, employing the Ansys Workbench's macro and homogeneous modeling techniques. Non-destructive testing was carried out on multiple structural parts of the tower, using techniques, such as the rebound hammer and ultrasonic pulse velocity tests, to gather the mechanical properties of the stone and brick masonry. These properties were incorporated into the finite element model to evaluate the Tower's structural responses during analysis. The tower's structural response under gravitational forces was determined using standard code regulations and guidelines, and stress and strain responses were compared to the actual structural morphology observed during the inspection. The highest stress was found in the stone elements between the connection of the dome and the drum on the second floor. Furthermore, the tower's response to wind, including stress and deformation, was thoroughly examined at the same location, revealing the maximum response under gravity loading. This study pinpointed critical and weak areas that require retrofitting and strengthening using modern techniques to safeguard these monumental historical structures for future generations. The combined use of visual inspection, non-destructive testing, and finite element analysis proved to be an effective approach in assessing the response of the 106-year-old masonry tower to gravity loading and wind effects.

Keywords Old structures · Visual inspection · Unreinforced masonry · Cracks and damages · Finite element modeling · Static analysis

Introduction

The durability and the permanence of masonry materials like brick and stone have played a significant role in preserving the unwritten record of human history in the form of temples, fortresses, sanctuaries, and cities. Early efforts to build permanent shelters were limited to the materials available, and masonry provided a way to create long-lasting

structures that could withstand war and natural disasters. The history of civilization is closely tied to its architecture, and the history of architecture is closely tied to the history of masonry. Brick is the oldest manufactured building material, with a rich history that dates back centuries (Mosoarca & Gioncu, 2013). Despite experiencing harsh conditions like damage, material deterioration, and collapse, many ancient monuments and structures have survived. Researchers have conducted various studies to evaluate the response of old structures, including visual inspections to assess their geometrical drawings, construction techniques, materials, and damage (Dubey et al., 1996; Kaveh & Maniat, 2015). These surveys offer valuable information about the actual condition of these structures and help preserve their architectural legacy for future generations.

Studying the behavior of old monuments accurately can be challenging as it requires knowledge of the geometrical

✉ Kumar Pallav
kumarp@cput.ac.za

¹ Civil Engineering Department, Motilal Nehru National Institute of Technology Allahabad, Prayagraj, Uttar Pradesh 211004, India

² Department of Civil Engineering and Geomatics, Cape Peninsula University of Technology, Bellville, Cape Town 7530, South Africa

model and mechanical properties of the materials used in construction. Without accurate data, studies have relied on assumed mechanical properties obtained from old reports, codes, and structures constructed in the same era. Static and dynamic analyses are performed using approximated mechanical properties to understand old structures' stresses and deformation responses over time. Some studies have evaluated mechanical properties through non-destructive and minor destructive tests like Rebound Hammer and Ultrasonic Pulse Velocity tests (Hatir et al., 2019; Martínez-Soto et al., 2021; Tuğla et al., 2018). Often, old structures have been analyzed through self-weight analysis, while others have focused on the dynamic modal or seismic analysis to assess their responses and performance (Cundari & Milani, 2013; Pintucchi & Zani, 2016). Developing a finite element model of old structures is challenging without any background knowledge of the geometrical drawings, construction techniques, materials, etc. Macro- and Micro-modeling techniques have been employed to prepare 3D finite element models following a visual inspection of the structures.

Old structures were built using large construction materials to assemble structural elements, resulting in spacious and bulky designs visible in geometrical and architectural views. Macro-modeling techniques have been used to accurately simulate these structures and design the load-bearing and decorative components. Properly designing these components is crucial to maintaining the structural integrity of these historical structures and preserving their esthetic value. Most of the structures are cyclic or regular. Kaveh (2013) has shown an efficient way to analyze such structures.

The study focused on the tower element, the third-highest tower of the Senate Hall building in Allahabad University,

India, built in 1915. The study conducted a visual inspection survey to gather authentic information about the tower's design, construction materials, construction techniques, cracks and damages in load-bearing and decorative structural elements, connections, deterioration of construction materials, renovation, and strengthening of the tower. Stone played a vital role in the tower's facade, columns, arches, vaults, and railings, while brick masonry construction was used for load-bearing and partition portions at different levels (Kumar & Pallav, 2020). The tower is an important structural element for offices, stairs, and architectural views. The masonry construction used different structural components, such as plinths, walls, towers, arches, domes, etc., combining brick masonry and stone construction used for walls, arches, roofs, and dome elements. The study focused on ensuring the adequate structural performance of the tower, as it has survived for more than a century and is still in use.

The tower, a significant structural element of the Senate Hall building at Allahabad University, has been constructed using a combination of brick masonry and stone materials (Fig. 1). The tower comprises a plinth base, stone vaults, and wide openings connected by masonry walls and stone facades in the east and west verandahs. A 3D finite element model of the tower has been prepared on the AutoCAD and ANSYS workbench with the help of visual inspection and drawings. Non-destructive tests, such as Rebound Hammer and Ultrasonic Pulse Velocity tests, have been performed on the tower elements to evaluate their mechanical properties. These properties have been used for the finite element model of the tower to gather the stresses and deformation capacities and check the structural response during the gravity and wind load as per the Indian standard code. Finally,

Fig. 1 Structural view of the tower



the structural responses (stresses and deformation) have been compared between visual inspection surveys of the tower.

Visual inspection

Visual testing is a critical non-destructive test for understanding a structure's condition. A thorough visual inspection can provide extensive information and give a preliminary indication of the structure's condition. It can also help formulate subsequent testing programs. However, the visual inspection should not be confined only to the structure under investigation. It should include neighboring structures, the surrounding environment, and climatic conditions. This can be challenging as what may appear obvious to one engineer may not be apparent to another. Nevertheless, a comprehensive visual inspection is crucial to thoroughly understand the structure's condition (Saisi et al., 2016a, 2016b; Stepinac et al., 2013). In the present work, only a preliminary inspection has been presented, emphasizing the importance of further testing.

A complete set of relevant drawings is essential for recording observations during the construction of a tower. This tower is the third highest of four Towers constructed in different locations in the senate hall building, using similar geometrical dimensions, architectural style, and construction practices. The geometrical cross-sectional drawing of the tower is prepared after visual inspection and is shown in Fig. 2. The tower has been constructed from three partitions, each with unique designs and materials. Four stone arches

are constructed symmetrically at the ground floor level for North/East and South/West directional views. The tower is a testament to careful planning and attention to detail.

The front face of the North and East sides of the tower features two arches with symmetrical cross-sectional dimensions of height 3.54 m, width 2.44 m, and radius of arch 1.08 m. The front verandah arch is designed with a height of 5.49 m, width of 3.66 m, and a radius of arch 1.83 m, while the side verandah arch is 5.18 m high, 2.44 m wide, and has a radius of arch 1.22 m. The Tower's load-bearing masonry walls are constructed from 0.61 m thick brick. The Towers second floor features a masonry platform with eight stone arches of the same dimensions, each 2.44 m high, holding up the stone drum that the dome rests. The dome is circular and 2.44 m high with a radius of 2.1 m (Fig. 2).

The systematic survey has been conducted to cover various aspects, such as present defects, past and current use of the structure, the condition of adjacent structures, and environmental conditions. It is crucial to identify all defects, classify their degree, and determine their causes if possible. The distribution and the extent of defects must be recognized accurately. A visual inspection is an essential tool for gathering information about the actual condition of a structure without causing any damage. Human eyes can visualize the condition of a structure accurately, including its architectural style, geometrical plan, elevation, construction practice, and workmanship. Visual inspection is beneficial for collecting evidence about heritage and modern structures constructed in different periods.

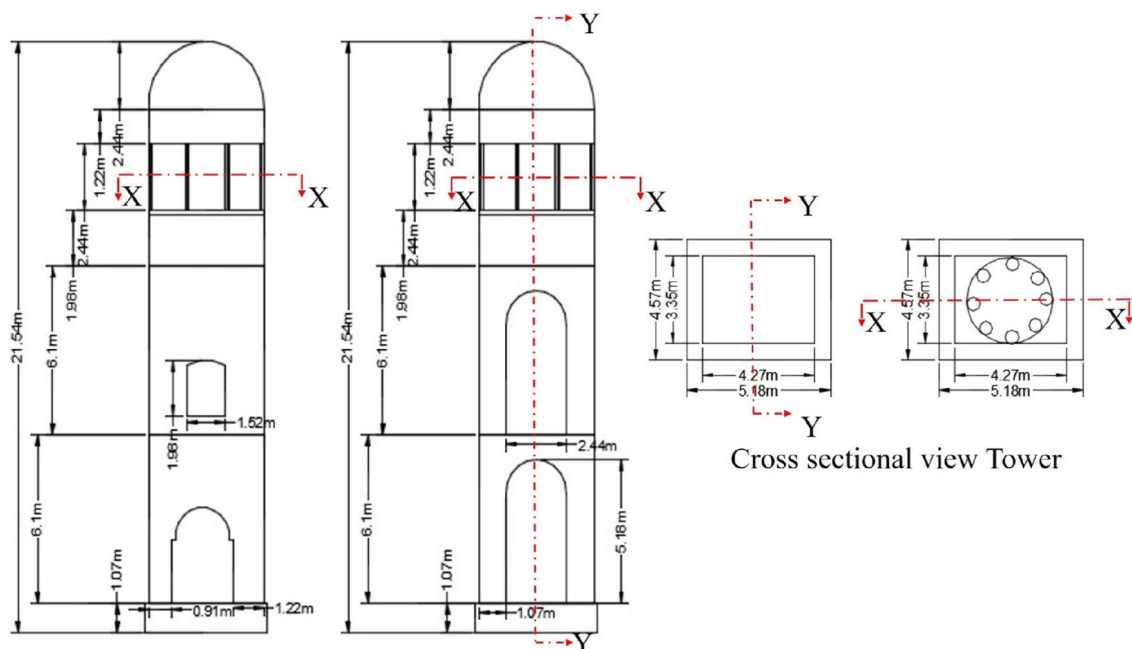


Fig. 2 Cross-sectional view of the tower

The tower is an example of Indo-Saracenic style architecture, constructed using unreinforced masonry techniques using brick and stone and decorated with stone chips. British architects and engineers built the tower in the nineteenth century combining Hindu, Mughal, and British architecture, serving several purposes, such as a clock and stairs. The tall tower is connected to the senate hall building at two levels. A systematic survey and visual inspection of a structure are essential to accurately determine its condition, defects, and history. The information collected can help identify potential problems, develop solutions, and preserve the structure for future generations. Figure 3 shows that the front corner tower is a two-story structure combining stone arches and brick masonry. The tower has a 1.07 m thick plinth base constructed using brick and stone masonry materials and decorated with stone chips.

Figure 3a shows that the entire tower is symmetrically constructed on the ground floor using a combination of four stone arches and brick masonry walls with and without plaster on the first floor. Figure 3b shows eight octagonal stone columns connecting the peripheral dome and resting on an octagonal stone drum. The towers serve specific purposes, such as providing access to upper floors, office spaces, and clocks, which are important architectural features of the building.

The deteriorating condition of a building, specifically the masonry walls, load-bearing elements, and decorative elements, has been observed during the survey. Major losses and material deterioration are visible on both the plastered and without plastered walls at the first-floor level. The site engineer confirms that the internal masonry walls have been plastered with 25 mm thick surkhi materials, but the external walls have only been painted red. The corner at the first-floor

level is coated with stone chips to protect the masonry without plaster.

The external masonry walls have huge cracks and damage in the rectangular peripheral wall and decorative elements of windows, railing, and stone shades. Construction materials deteriorate due to increased moisture and a lack of maintenance work. The color of the building is fading, and the materials are deteriorating. Systematic crack mapping is recommended, and further studies are required to understand the cause and progression of deterioration. Examination of crack patterns may provide a preliminary indication of the cause. Observation of masonry walls' surface texture and color shows variations due to the atmosphere effect on the tower.

The condition of the tower shows that proper maintenance work needs to be done to avoid further damage.

Non-destructive testing

The Non-Destructive testing on Tower's masonry walls, arches, and roof elements has been conducted, such as Rebound Hammer and Ultrasonic Pulse Velocity, at similar locations on the tower. After testing, the compressive strength of the Rebound Hammer index was calibrated based on the experimental results obtained in the laboratory (Kumar & Pallav, 2020).

Figure 4 displays the Schmidt rebound hammer and the testing locations. The versatile hammer weighs approximately 1.8 kg, making it appropriate for use in laboratory and field settings. The hammer comprises four primary components: the outer body, the plunger, the hammer mass, and the main spring. Pressing the hammer firmly against the

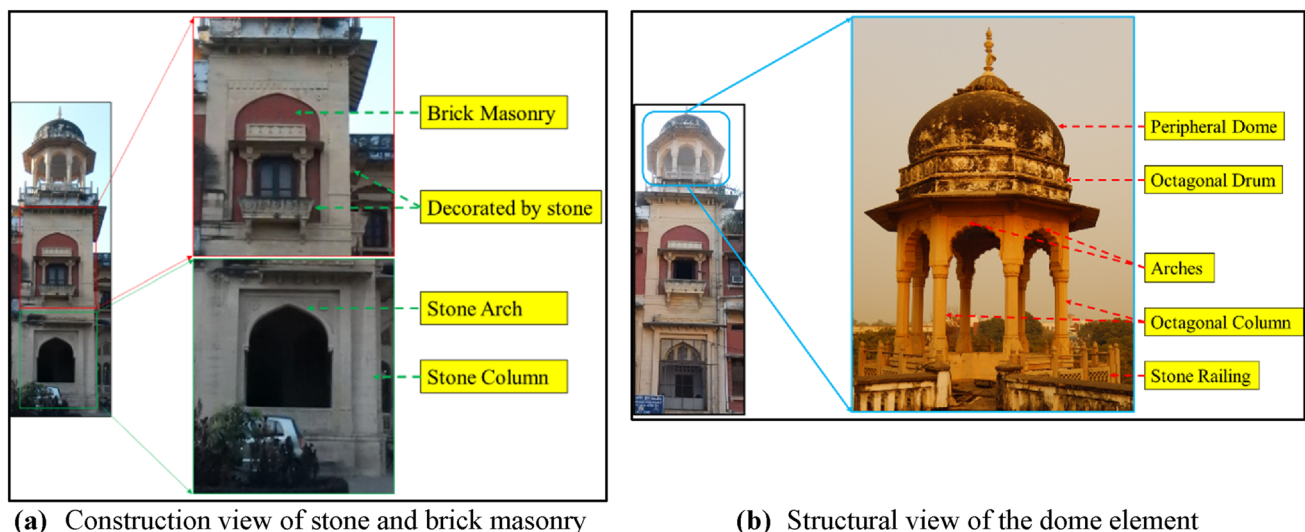


Fig. 3 Structural elements and materials of the tower

surface allows the body to move away from it until the latch connects the hammer mass to the plunger. This movement extends the spring that holds the mass to the body. The hammer can be used in various positions, including horizontally, vertically overhead, vertically downward, or any angle in between, as long as it is perpendicular to the tested surface. However, the mass's position relative to the vertical impacts the rebound number due to the influence of gravity on the hammer's mass.

The testing has been conducted in carefully selected locations of the Tower's elements following the guidelines and procedures of the RH. The tower is an old monumental structure, and the testing locations were chosen after a visual inspection revealed no signs of damage, cracks, or moisture. Six locations were marked for testing in the Tower's walls, arches, and roofs, with load-bearing walls

and stone elements prioritized for testing. Rebound hammer testing was performed sixteen times in each location for the masonry walls and stone arch elements and nine times for the roof elements. The rebound hammer values for each location are presented in Tables 1, 2, 3. The masonry walls' rebound hammer index ranged from 29.31 to a minimum of 14.53, while the stone arches' rebound hammer index ranged from a maximum of 44.88 to a minimum of 35.75. The roof elements' testing was challenging due to the material deterioration, severe cracks, damage, and moisture on the surface. Nonetheless, the testing yielded valuable results, with a maximum rebound hammer index of 21.06 and a minimum of 18.28.

The rebound hammer testing has been conducted well on the tower. Figure 5 shows the compressive strength determined in the Tower's walls, arches, and roof elements.



Fig. 4 Schematic view of the Schmidt Rebound Hammer

Table 1 Rebound Hammer value of wall

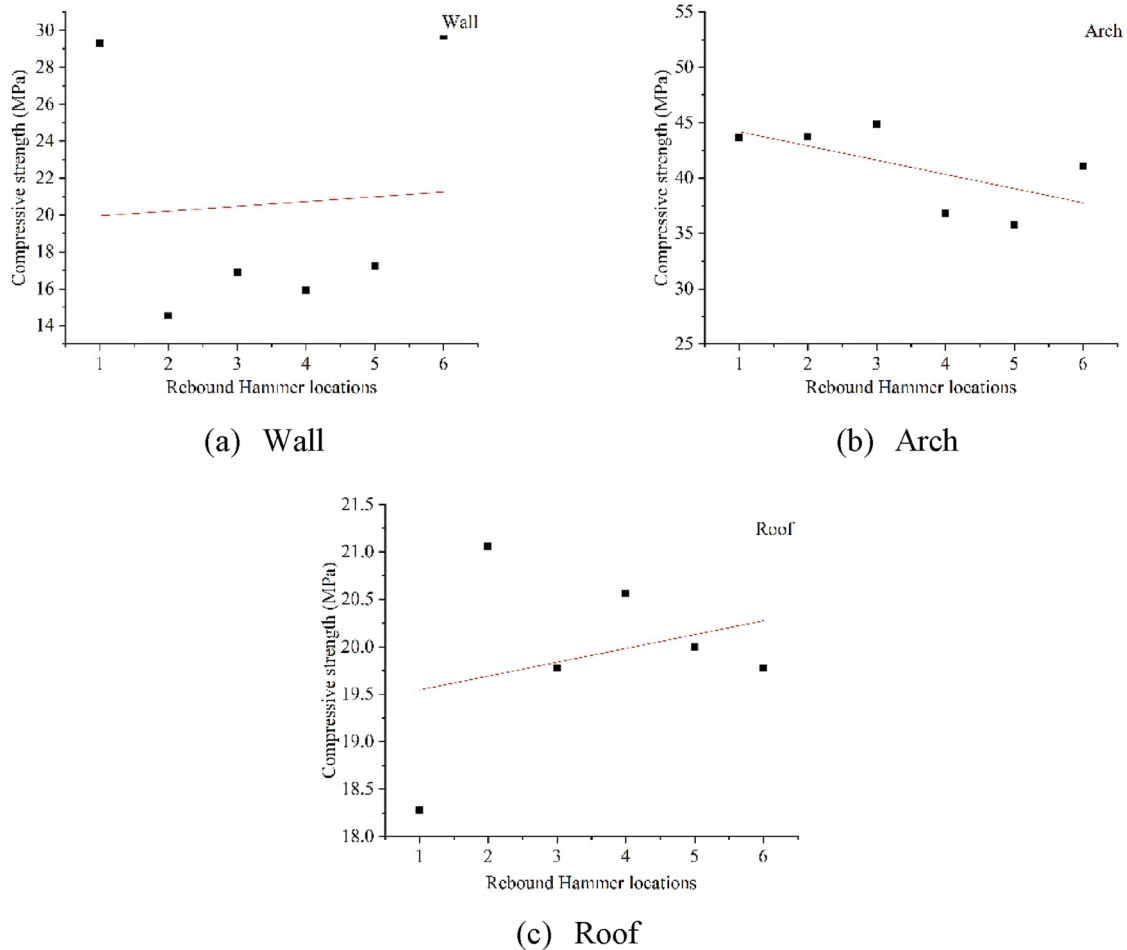
Location	Rebound Hammer Index																Average
GF-M 1	33.5	39.0	36.5	28.0	41.0	38.0	29.0	32.5	28.5	29.0	27.0	17.5	17.5	23.5	25.5	23.0	29.31
GF-M 2	17.0	18.0	16.5	9.5	14.5	20.5	16.0	14.0	14.5	16.0	14.5	13.0	11.0	11.0	14.5	12.0	14.53
GF-M 3	15.5	14.5	16.5	14.0	14.0	20.0	15.5	17.0	15.5	16.0	19.5	22.0	20.5	18.5	15.5	15.5	16.88
GF-M 4	14.0	21.0	17.0	18.5	15.5	17.5	10.5	11.0	13.5	15.0	14.0	15.0	18.5	22.5	19.0	12.5	15.94
GF-M 5	20.5	16.0	24.5	21.0	18.0	17.5	19.0	18.0	16.5	14.5	13.5	18.5	16.0	16.0	11.0	15.0	17.22
GF-M 6	28.0	26.0	25.5	35.5	24.5	37.0	29.5	25.5	33.5	30.0	27.0	34.5	35.0	28.0	26.0	29.5	29.69

Table 2 Rebound Hammer value of arch

Location	Rebound Hammer Index																Average
GL-FV1	49.5	40.5	41.5	41.0	35.5	39.5	39.5	33.0	51.5	43.5	45.0	41.5	49.0	49.0	53.0	45.5	43.63
GL-FV2	44.0	40.5	40.0	53.0	43.0	41.5	50.5	49.5	36.0	37.5	53.0	39.0	41.0	43.5	53.0	35.0	43.75
GL-FV3	47.5	44.0	36.5	25.0	42.5	50.0	41.0	53.5	48.5	44.5	45.0	58.5	43.0	46.0	52.0	40.5	44.88
GL-FV4	37.5	35.0	34.5	50.5	40.0	32.0	42.0	32.0	39.5	37.5	31.0	30.5	32.5	38.5	35.0	40.5	36.78
GL-FV5	29.5	36.0	31.5	26.0	25.5	36.5	34.5	41.5	39.5	48.0	42.0	38.5	40.0	39.0	30.0	34.0	35.75
GL-FV6	37.0	26.5	36.5	42.5	41.5	44.5	41.5	45.5	52.0	37.5	42.0	49.0	45.0	41.0	31.0	44.0	41.06

Table 3 Rebound Hammer value of roof

Location	Rebound hammer index									Average
BS-1	15.5	15.0	24.0	16.5	20.5	17.5	16.5	19.5	19.5	18.28
BS-2	18.5	28.0	22.5	23.0	17.0	23.0	18.5	16.0	23.0	21.06
BS-3	16.0	23.5	15.5	15.0	24.0	23.5	19.0	18.5	23.0	19.78
BS-4	20.0	15.0	15.5	21.5	20.0	21.0	23.0	26.0	23.0	20.56
BS-5	15.0	23.0	18.5	26.5	19.5	18.5	16.0	21.0	22.0	20.00
BS-6	16.0	22.5	20.0	19.5	18.5	18.0	23.0	21.0	19.5	19.78

**Fig. 5** Compressive strength of tower element from Rebound Hammer

The average compressive strength of the load-bearing masonry wall and roof element is found to be 20.6 MPa and 20.0 MPa, respectively, as shown in Fig. 5a. The average compressive strength of the stone element has been determined to be 41.0 MPa, as shown in Fig. 5c. These results provide the current state of mechanical properties of the Tower's elements, which can be used in the model for the analysis of the tower.

Ultrasonic pulse velocity testing

The ultrasonic pulse velocity test was performed on the Tower's structural elements, with testing locations selected during the Rebound Hammer. Before testing, locations were cleaned, and grease was applied as a lubricant to access the signal between the transducer and receiver. Three testing methods were used: direct and indirect testing for the stone assembly for the wall and roof elements. Direct and indirect

testing has been performed on the same 0.5 m path length between the transducer and receiver. The testing locations were marked and shown in Fig. 3, with results in Tables 4, 5, 6.

During the ultrasonic pulse velocity testing on the Tower's structural elements, six locations were selected for indirect testing on the wall elements, represented as UPV-W-1 to UPV-W-6 in Table 4. The maximum and minimum travel time gathering for walls was found to be 499.6 μ Sec and 233.7 μ Sec, respectively, while the pulse velocity was observed to be a maximum of 2.140 km/sec and a minimum of 1.001 km/sec for locations UPV-W-2 and UPV-W-6, respectively. In the arch element, direct testing was performed using the path length and travel time described from ME-1 to ME-6 in Table 5. The pulse velocity was observed to be a maximum of 2.447 km/sec and a minimum of 1.288 km/sec for the locations ME-4 and ME-2, respectively. The path length used for testing the roof element was 0.5 m for each location, with UPV testing performed on the underside of shell elements. The maximum and minimum pulse velocities for the roof element were 2.067 km/sec and 1.449 km/sec, respectively, at locations UPV-6 and UPV-1, as shown in Table 6.

The six locations have been selected for indirect testing on the wall elements showing as UPV-W-

The UPV testing has allowed for the evaluation of both dynamic and static Young's modulus for the Tower's wall, arch, and roof elements. The results have been plotted in Fig. 6, with the ultrasonic pulse velocity on the x -axis and Young's modulus on the y -axis. The wall element has been evaluated to have an average velocity of 1270.5 m/s, with dynamic and static Young's modulus of 3030.39 MPa and 1451.33 MPa, respectively. The arch element has an average velocity of 1812.67 m/s, with dynamic and static Young's modulus of 5933.5 MPa and 2844.81 MPa, respectively. The roof element has an average velocity of 1762.67 m/s, with dynamic and static Young's modulus of 5381.4 MPa and 2579.81 MPa, respectively.

Interestingly, the stone response has been observed to be more significant than that of the wall and roof elements. The mechanical properties evaluated from the UPV testing

Table 5 UPV value of arch

UPV location	Path length [m]	Travel time [μ sec]	Pulse velocity [km/sec]
ME-1	0.5	291.9	1.713
ME-2	0.5	388.3	1.288
ME-3	0.5	384.4	1.301
ME-4	0.5	201.9	2.477
ME-5	0.5	238.2	2.099
ME-6	0.5	250.3	1.998

have been used for the analysis, with the results shown in Table 7. Finally, these properties have been used for the static analysis of the tower element to evaluate the stresses and deformation response.

Finite element modeling

The finite element modeling of heritage structures is challenging due to unknown history, construction practice, geometrical drawing, etc. All the basics and applied knowledge of physical problems, finite elements, and solution algorithm contributes to modeling expertise (Burman et al., 2014). Kaveh (2014). The physical verification of the structural drawing is challenging for knowing the accurate behavior during the analysis. The documentation of a huge general-purpose program usually contains some modeling advice on the elements and connections. In the modeling, the connections generated major problems during the meshing and faced difficulties in analysis. In meshing, automatic mesh generators make it easy to use too much fine detail (Tzmatzis & Asteris, 2003; Valente & Milani, 2016). In computational cost and dimensionality, occasionally, one may select to analyze a one- or two-dimensional model rather than a two- or three-dimensional model, particularly when doing an initial simplified analysis. A rough guide to relative costs can be obtained by comparing the equation-solving costs of one-, two-, and three-dimensional meshes of comparable elements (Aprile et al., 2001; Ceroni et al., 2007; Shabani et al., 2022). The 3D finite element model of the tower has

Table 4 UPV value of wall

UPV location	Path length [m]	Travel time [μ sec]	Pulse velocity [km/sec]
UPV-W-1	0.5	490.6	1.019
UPV-W-2	0.5	233.7	2.140
UPV-W-3	0.5	412.1	1.213
UPV-W-4	0.5	417.6	1.197
UPV-W-5	0.5	474.8	1.053
UPV-W-6	0.5	499.6	1.001

Table 6 UPV value of roof

UPV location	Path length [m]	Travel time [μ sec]	Pulse velocity [km/sec]
UPV-1	0.5	345.1	1.449
UPV-2	0.5	316.4	1.580
UPV-3	0.5	274.2	1.823
UPV-4	0.5	279.8	1.787
UPV-5	0.5	267.4	1.870
UPV-6	0.5	242	2.067

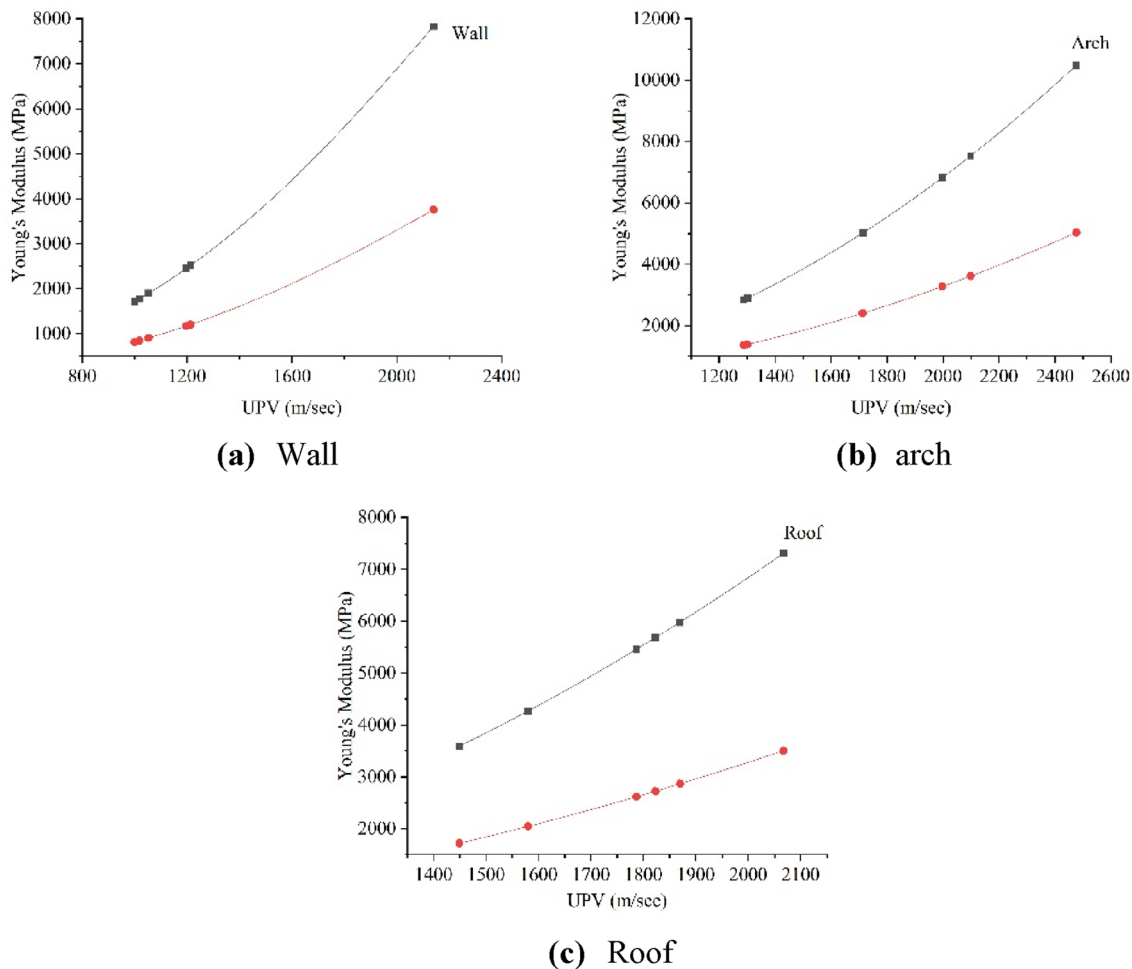


Fig. 6 Dynamic and static Young's modulus vs UPV of the tower element

Table 7 Mechanical properties of the tower element

Material	Density (kg/m ³)	Young's modulus (MPa)	Poisson's ratio
Masonry wall	1900	1451	0.2
Stone (arch)	2800	2844	0.2
Roof	1900	2579	0.2

been simulated on the Ansys workbench with the help of visual inspection and previous geometrical drawings shown in Fig. 7. The macro modeling technique has opted for the numerous structural components, such as masonry walls, arches, domes, etc., of the tower. Every material of the tower has been specifically modeled due to assigning the mechanical properties. The bonded condition has connected all the elements of the tower. The meshing has been used discretely for each structural element of the tower shown in Fig. 7. Fine meshing has been opted during the meshing used with

active assembly. In the present modeling, only load-bearing structural elements have been considered for the model. The decorating structural elements and materials are negligible due to increased computation cost and analysis time. Tower is modeled using Solid 186 and Solid 187 elements (20-Noded and 10-Noded 3D solid elements) in the load-bearing masonry walls, arches, and Surf-154 used in the dome. TARGE 170, and CONTA 174 used contact and connections of tower components. Solid 186 and Solid 187 elements are 20-Noded and 10-Noded tetrahedral solid elements, having three degrees of freedom at each node (translations in the x , y , and z directions). It has various capabilities like plasticity, hyperelasticity, creep, stress stiffening, large deflection, and large strain. Moreover, it is capable of mixed formulation simulation of deformations of nearly incompressible elastoplastic materials and fully incompressible hyper-elastic materials that agree well with the material of tower. For comprehensive computation analysis and finite element methods, one can see more details (Kaveh, 2014).

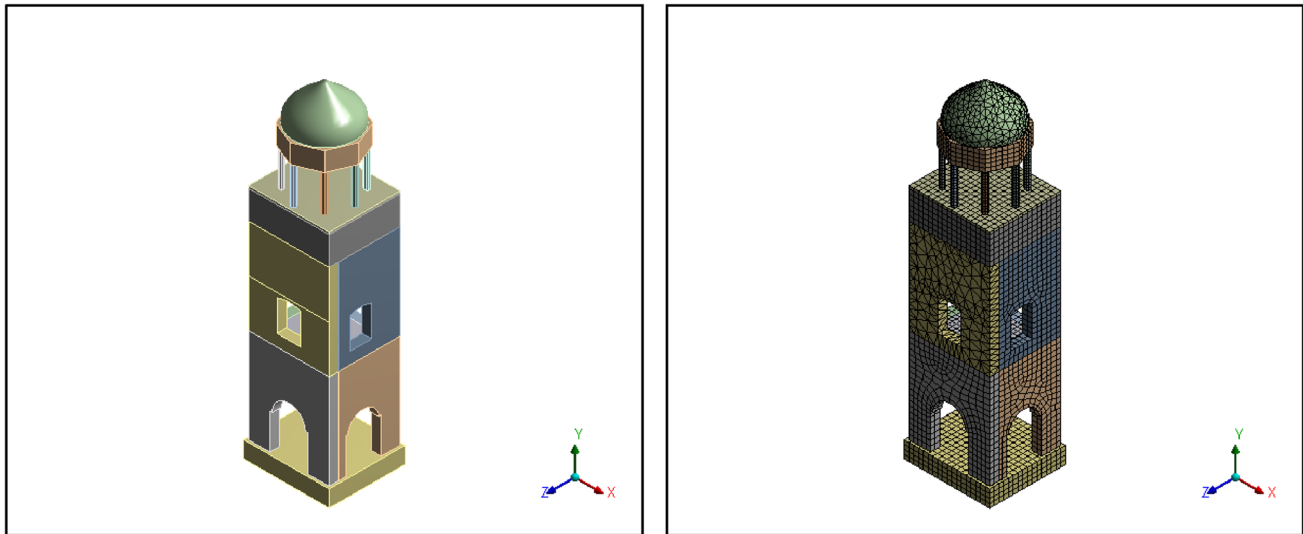


Fig. 7 Finite element meshed model of tower

Analysis

Analyzing old structures can be challenging, as accurate response evaluation requires background information on the structures. Researchers typically conduct studies on tower elements to gather structural responses while assessing previous information and actual conditions. Simulation software, such as Abaqus, Ansys, and Tri-muri, is used with approximate mechanical properties to conduct static and dynamic analysis on old structures. 3D finite element models were created after visual inspection surveys to simulate behavior, and Indian codal provisions and guidelines were followed to apply loads in the static analysis. Dead load (self-weight) IS: 875-1987 (Part 1), live load IS: 875-1987 (Part 2), and wind load IS: 875-2015 (Part 3) were used to evaluate stress and deformation responses, respectively (IS 875: 1987, 1987; IS 875 and Part 1, 1987; Suthar & Goyal, 2021). Mode shapes and frequencies have been evaluated through modal analysis by IS: 1893–2016 (Part 1) (IS 1893, 2016). The 3D model, built on a real scale, used brick and stone materials in load-bearing structural elements. Bonded and restrained boundary conditions are applied to the tower, and mechanical properties are simulated through non-destructive testing. These properties are separately assigned to the tower and its structural elements. The global responses were captured in contour form, showing load-bearing structural elements' stresses and deformation capacities. The study showed the structural response of the tower, demonstrating how the analysis of old structures can be a challenging task requiring extensive research and computational tools.

The 3D model has been built on the real scale used for different structural mechanisms of combining brick and

stone materials in load-bearing structural elements. The structural base has been fixed (bonded) and has been used in all directions. Boundary conditions of the tower are used bonded (plinth base) and restrained (connections of structural elements) on stone and brick masonry elements concerning each floor level. The restrained conditions have applied for the ground and first-floor levels due to the tower connecting two sides between the verandah of the senate hall building. But the second-floor level of the tower is free from all directions, only some elements, such as columns, arches, drum, and dome, are connected by bonded conditions. The mechanical properties have been simulated from non-destructive testing performed on the tower elements, as shown in Table 7, used during the static analysis to evaluate the response. In the present study, mechanical properties have been separately assigned to the tower and structural elements, such as brick masonry, stone (arch), and roof materials. Finally, the structural responses have been evaluated on the entire tower. Stresses and deformation responses have been captured globally in the tower, and the response has shown in contour form. The actual responses have gathered in the form of stresses and deformation capacities of load-bearing structural elements.

Results and discussion

The tower is a crucial structural component of the north and south corners of the senate hall building. It has been constructed with a combination of brick masonry and stone materials. The geometrical model was created based on a visual inspection survey that gathered detailed information about the structure, including construction techniques,

materials, cracks, and damages. A 3D finite element model was prepared on ANSYS with a homogeneous geometrical proportion of the masonry and stone materials separately from the actual geometrical proportion. Static and modal analyses are also performed to evaluate the actual behavior of the global and local coordination of the tower. The results were discussed based on stress and deformation capacities, with major stresses and deformation responses recorded on the whole geometrical model of the tower and checked for important structural components. The visual inspection revealed huge cracks and damage on the connections, partition walls, and facades at each floor level. The gathered responses were compared with the actual condition of the tower, with maximum responses observed on the crowns and openings of stone arches and facades on the ground- and second-floor levels.

The dead load analysis of the tower has been conducted according to Indian codal IS:875 (Part 1) to evaluate its structural capacities and responses during self-weight. The Tower's load-bearing components have responded better due to their large geometrical dimensions and material properties. The stress and deformation capacities of the tower during self-weight have been assessed, and the maximum stress has been observed on the stone drum and dome at the second-floor level. The stress response has been negligible on the masonry construction at the first-floor level, but some stress response has been visible on the stone elements, such as arches, facades, and columns. The tower has performed well for the masonry construction, but the stone components have performed differently for load-bearing structural components and decorative stones. Major cracks and damages have been observed on the masonry platform at the second-floor level, and a deformation capacity of 2.56 mm has been observed on the roof element at the same level (Fig. 8).

In the given context, the tower has been evaluated for its response to live loads on the first and second floor levels. The live load considered was 0.75 kN/m^2 , which was applied to non-assessable portions of the tower. However, the stairs portion had been closed due to major cracks and damages during numerous atmospheric behaviors. The stresses and deformation responses have been evaluated, and the results have been presented in Fig. 9. The maximum stress of 2.16 MPa has been observed on the connection between the stone drum and the arch. This location formed the maximum stress response during dead load analysis as well. The stress contour visible in Fig. 9a shows the formation of stresses on the opening and connections between elements like stone arches, vaults, columns, roofs, and connections on the respective floor levels. A deformation capacity of 2.56 mm has been observed on the second-floor roof level, and negligible deformation response has been observed on

the thick masonry walls and stone arches constructed on the ground floor level.

However, on the second-floor level, the deformation has been observed huge on the roof due to visible major cracks and deterioration on the masonry platform. Major deterioration has been generated on the masonry construction and plaster materials on the roof at the bottom of the ground floor and second-floor roof. The deformation vectors have been generated on the arches and dome assembly of stone mechanisms at the second-floor level, and the deformation vectors have been visible minimum on the ground floor and maximum on the top of the dome, which increased concerning the height of the tower. The stress and deformation responses have been observed in similar locations during dead and live load analysis. In the live load, the masonry walls and arches have been performing better, but the roof element has formed a serious condition on the top surface after placing 2.5-inch concrete plaster. The bottom side roof has been renovated with new material.

Ensuring the structural stability of a building requires a thorough evaluation of stresses and deformation responses caused by wind pressure acting on tower elements. Stress and deformation responses of the tower are depicted in Fig. 10, showcasing maximum stress and deformation observed at various levels. The wind pressure was applied 10 m above the ground level following codal guidelines for this analysis.

At the second-floor level, the connection between the stone columns and the roof experiences the highest stress, measuring 2.448 MPa. This stress primarily affects the stone elements on the second floor, with the maximum stress contour visibly concentrated on the openings within the stone arches on the ground floor. Furthermore, significant crack patterns have been observed in the roof and arch openings.

Regarding deformation, the roof at the second-floor level exhibits the maximum response, with a deformation of 2.723 mm. This deformation occurs at the connection points between the stone drum and columns and the masonry wall that connects the roof platform resting on octagonal columns at the second-floor level.

The analysis has revealed significant losses in the structural construction and constructional materials on the masonry and stone elements at the second-floor level. Minor and negligible stresses and deformation responses have been observed on the brick masonry walls at the first-floor level due to the sturdy construction, with the walls simulated from thick mortar and plastered from the internal side. However, minor cracks in load-bearing brick masonry wall elements have been visible from the longitudinal and transverse directions. Severe cracks and material deterioration have been visible from the connection between the internal side roof and walls at each respective floor level.

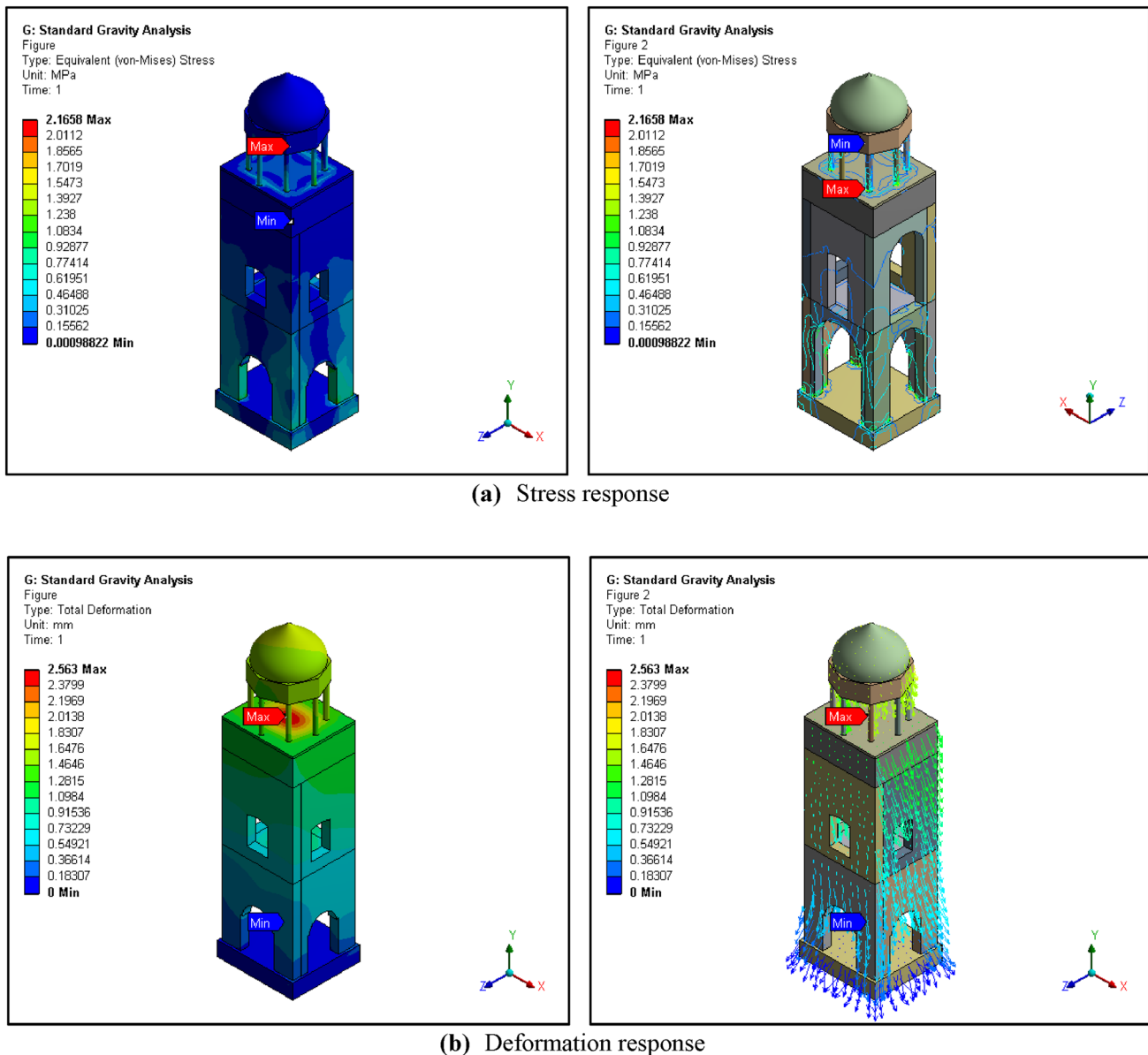


Fig. 8 Stress and deformation response due to gravity load

Due to stress variations, major losses have been detected in the decorating structural elements and materials of windows, balconies, arches and domes. Overall, this analysis provides valuable insights into the Tower's structural integrity and the potential vulnerabilities that must be addressed to ensure its long-term stability.

The modal analysis conducted on the tower provided insights into its dynamic behavior under different loading conditions. The first six modal shapes and frequencies were evaluated, and their responses are shown in Fig. 11. The

modal frequencies were obtained using the block Lanczos method and the same boundary conditions as the dead, live, and wind load analysis. The Tower's frequencies for the first six mode shapes are 2.294, 2.589, 3.943, 4.495, 4.518, and 6.326 Hz, respectively. The modal response analysis shows longitudinal and transverse directions for the first and second modes, while rotation deformation is observed for the third mode. These results are essential in understanding the Tower's structural behavior and designing appropriate interventions to ensure its safety and longevity.

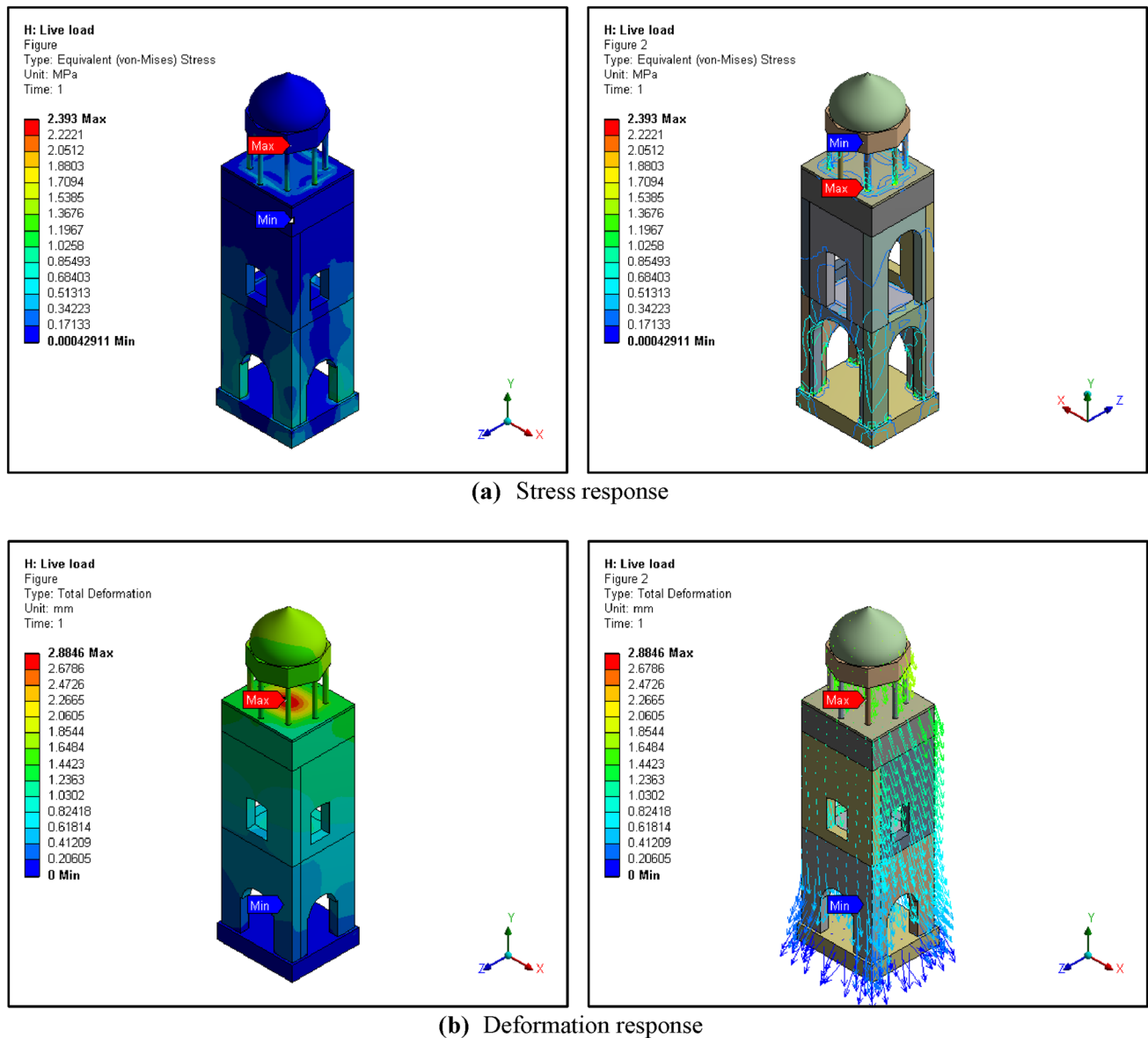


Fig. 9 Stress and deformation response due to living load

Conclusion

The study has provided a comprehensive analysis after visual inspection and NDT of the structural condition and response of the Indo-Sarasanic style tower. The visual inspection has revealed severe cracks and damages on the external side of the tower, primarily at the second-floor level. The tower is closed on the second floor due to significant damages observed on the masonry walls and roof elements. The 3D finite element model analysis has shown the maximum stress and deformation response on the stone column and roof at the second-floor level, mainly due to wind load.

The findings of this study can be used to develop appropriate strategies for restoring and maintaining the tower. The analysis results can also be used to enhance the future design and construction of similar structures, considering the response to different loads and the importance of proper material selection and interconnectivity of different building components. The monitoring of the structural elements and construction materials should be extended to the second-floor level to prevent further damage.

This study highlights the importance of regularly monitoring and maintaining historic structures to ensure their longevity and preservation for future generations.

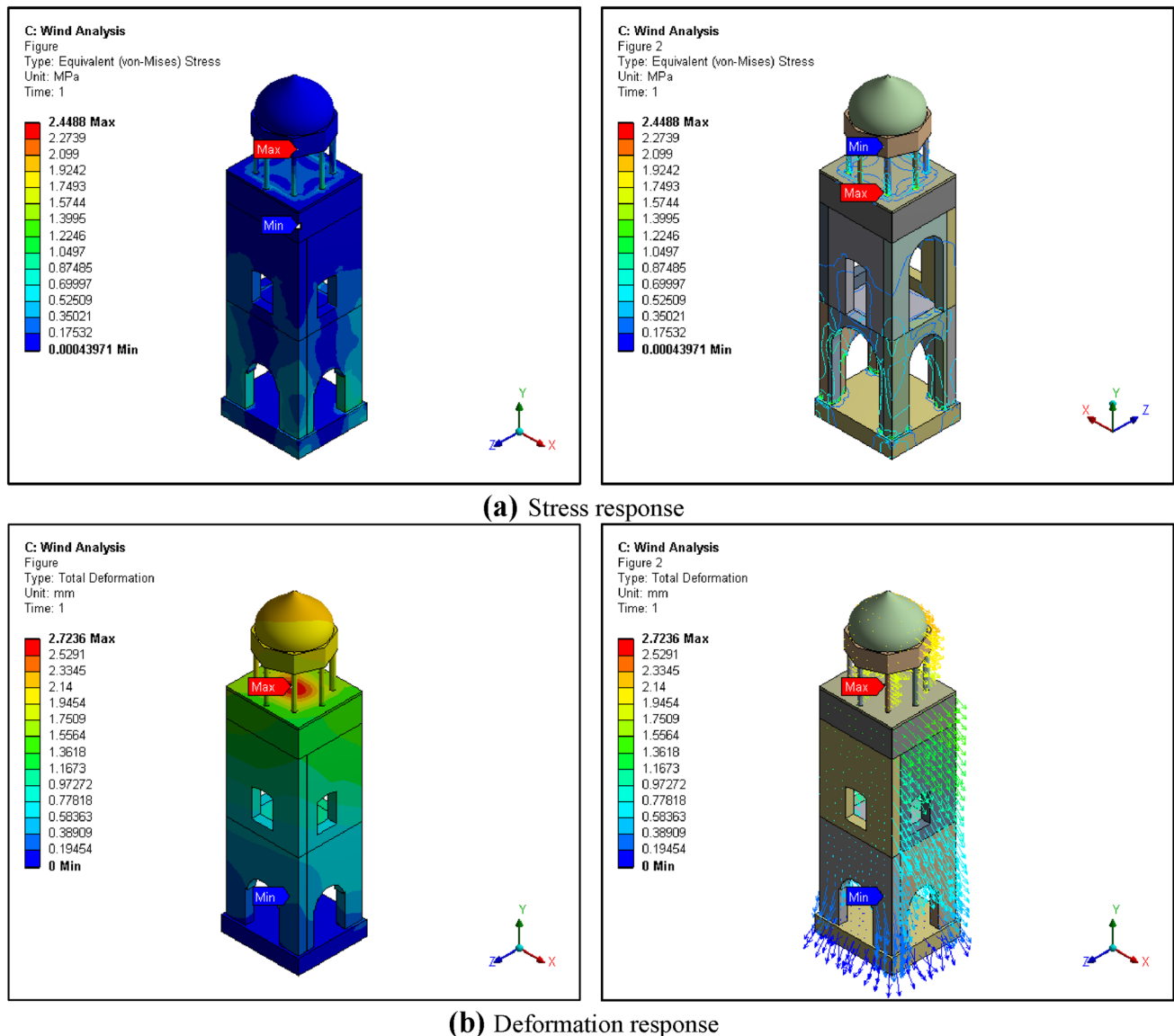


Fig. 10 Stress and deformation response due to wind load

Acknowledgements The authors thank Mr. Naveen Kumar, Civil Engineer and other supporting staff for graciously providing access to the Masonry tower. Also, the authors gratefully acknowledge the valuable information about the geometric survey, renovation and retrofitting of the Senate Hall building.

Author contributions The first author conducted the experimental work and prepared the initial draft of the article, while the second author conceptualized the work, determined the experimental order, and finalized the manuscript.

Funding Open access funding provided by Cape Peninsula University of Technology. The authors declare that no funds, grants, or other support was received during the preparation of this manuscript.

Data availability The NDT test data are used in this paper are shown in Tables. Research can cite while using them in there research.

Declarations

Conflict of interest The authors have no relevant financial or non-financial interests to disclose.

Open Access This article is licensed under a Creative Commons Attribution 4.0 International License, which permits use, sharing, adaptation, distribution and reproduction in any medium or format, as long as you give appropriate credit to the original author(s) and the source, provide a link to the Creative Commons licence, and indicate if changes were made. The images or other third party material in this article are included in the article's Creative Commons licence, unless indicated otherwise in a credit line to the material. If material is not included in

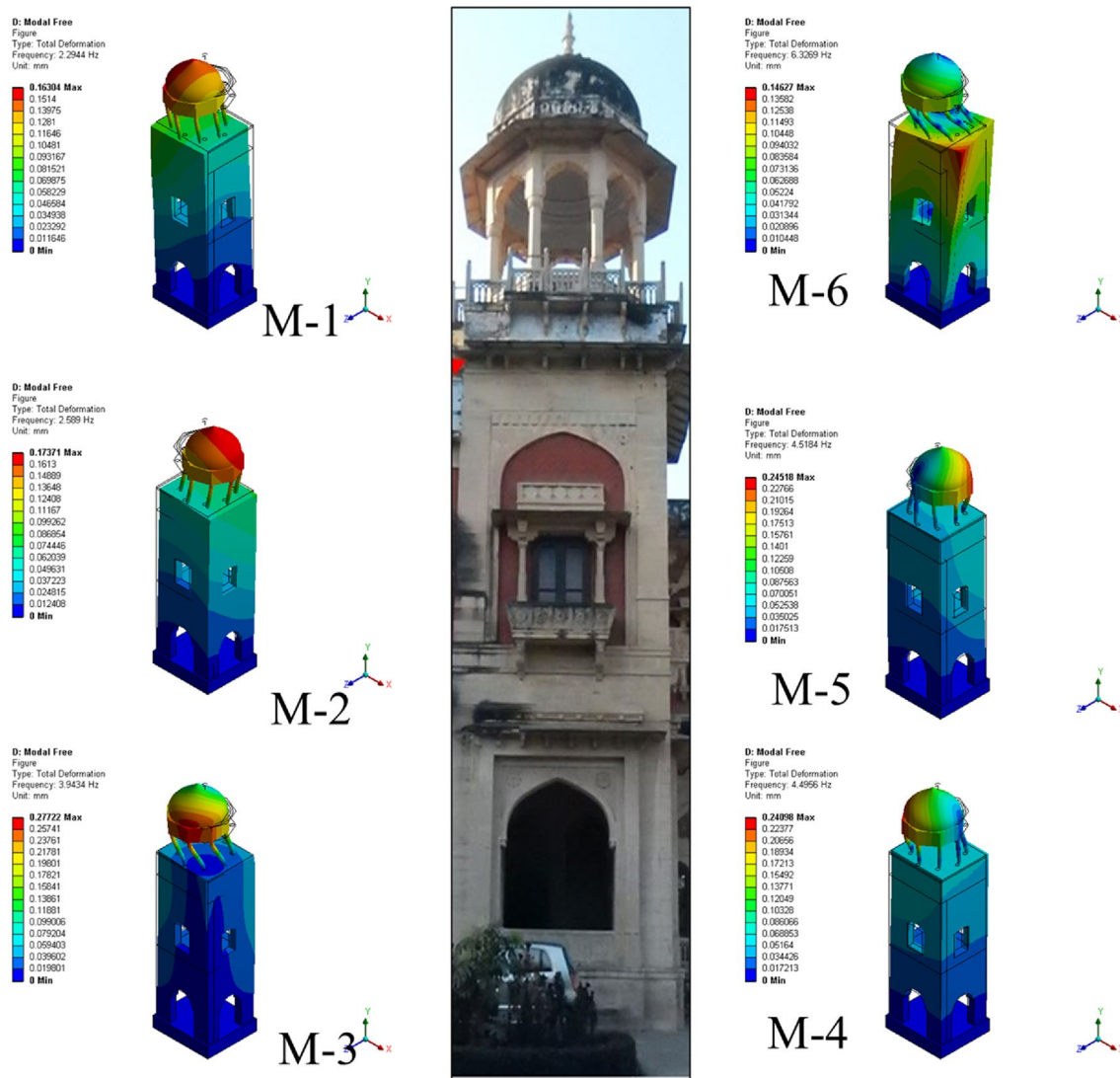


Fig. 11 Mode shape of the tower

the article's Creative Commons licence and your intended use is not permitted by statutory regulation or exceeds the permitted use, you will need to obtain permission directly from the copyright holder. To view a copy of this licence, visit <http://creativecommons.org/licenses/by/4.0/>.

References

- Aprile, A., Benedetti, A., & Grassucci, F. (2001). Assessment of cracking and collapse for old brick masonry columns. *Journal of Structural*. [https://doi.org/10.1061/\(ASCE\)0733-9445\(2001\)127:12\(1427\)](https://doi.org/10.1061/(ASCE)0733-9445(2001)127:12(1427))
- Burman, S., Pallav, K., & Singh, K. D. (2014). Finite element modelling of 'Rang Ghar' monument, Assam. *International Journal of Innovative Research in Science, Engineering and Technology*, 03, 144–151.
- Ceroni, F., Pecce, M., Palmaccio, F., & Manfredi, G. (2007). Structural investigations and modelling of the Bell Tower of Santa Maria del Carmine. *WIT Transactions on the Built Environment*. <https://doi.org/10.2495/STR070611>
- Cundari, G. A., & Milani, G. (2013). Homogenized and heterogeneous limit analysis model for pushover analysis of ancient masonry walls with irregular texture. *International Journal of Architectural Heritage*. <https://doi.org/10.1080/15583058.2011.640737>
- Dubey, R.N., Shashi, K.T., & Gupta, A. (1996). Seismic analysis for safety evaluation of Taj Mahal Monuments. In: *Eleventh World Conference on Earthquake Engineering*. Pergamon, ©1996., Aca-pulco, Mexico.
- Hatır, M. E., Korkanç, M., & Başar, M. E. (2019). Evaluating the deterioration effects of building stones using NDT: the Küçük-köy Church, Cappadocia Region, central Turkey. *Bulletin of*

- Engineering Geology and the Environment*. <https://doi.org/10.1007/s10064-018-1339-x>
- IS 1893. (2016). Criteria for Earthquake resistant design of structures, Part 1: General Provisions and buildings. Bureau of Indian Standards, New Delhi.
- IS 875:1987. (1987). Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures, Part 2: Imposed Loads. Bureau of Indian Standards, New Delhi.
- IS 875, Part 1. (1987). Indian Standard Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures (Dead Load). Bureau of Indian Standards, New Delhi.
- Kaveh, A. (2013). Optimal analysis of structures by concepts of symmetry and regularity. *Optimal Analysis of Structures by Concepts of Symmetry and Regularity*. <https://doi.org/10.1007/978-3-7091-1565-7>
- Kaveh, A. (2014). Computational structural analysis and finite element methods. *Computational Structural Analysis and Finite Element Methods*. <https://doi.org/10.1007/978-3-319-02964-1>
- Kaveh, A., & Maniat, M. (2015). Damage detection based on MCSS and PSO using modal data. *Smart Structures and Systems*. <https://doi.org/10.12989/sss.2015.15.5.1253>
- Kumar, A., & Pallav, K. (2020). Experimental and numerical investigation of old masonry wall using a macro-modeling approach. *The Open Civil Engineering Journal*. <https://doi.org/10.2174/1874149502014010334>
- Martínez-Soto, F., Ávila, F., Puertas, E., & Gallego, R. (2021). Spectral analysis of surface waves for non-destructive evaluation of historic masonry buildings. *Journal of Cultural Heritage*. <https://doi.org/10.1016/j.culher.2021.09.002>
- Mosoarca, M., & Gioncu, V. (2013). Failure mechanisms for historical religious buildings in Romanian seismic areas. *Journal of Cultural Heritage*, 14, e65–e72. <https://doi.org/10.1016/j.culher.2012.11.018>
- Pintucchi, B., & Zani, N. (2016). A simple model for performing nonlinear static and dynamic analyses of unreinforced and FRP-strengthened masonry arches. *European Journal of Mechanics, A/ Solids*. <https://doi.org/10.1016/j.euromechsol.2016.03.013>
- Saisi, A., Gentile, C., & Ruccolo, A. (2016a). Pre-diagnostic prompt investigation and static monitoring of a historic bell-tower. *Construction and Building Materials*. <https://doi.org/10.1016/j.conbuildmat.2016.04.016>
- Saisi, A., Guidobaldi, M., & Gentile, C. (2016b). On site investigation and health monitoring of a historic tower in Mantua Italy. *Applied Sciences (switzerland)*. <https://doi.org/10.3390/app6060173>
- Shabani, A., Ademi, A., & Kioumars, M. (2022). Structural model updating of a historical stone masonry tower in Tønsberg, Norway. *Lecture Notes in Civil Engineering*. Springer. https://doi.org/10.1007/978-3-030-90788-4_45
- Stepinac, M., Rajčić, V., & Čizmar, D. (2013). Learning from case studies: Timber tower of Gospić cathedral and traditional timber church. *Advanced Materials Research*. <https://doi.org/10.4028/www.scientific.net/AMR.778.927>
- Suthar, N., & Goyal, P. K. (2021). Comparison of response of building against wind load as per wind codes [IS 875—(Part 3)—1987] and [IS 875—(Part 3)—2015]. *IOP Conference Series: Earth and Environmental Science*. <https://doi.org/10.1088/1755-1315/796/1/012007>
- Tuğla, R., Demircan, R. K., & Kaplan, G. (2018). A study on the use of advanced nondestructive testing methods on historic structures. *Lecture Notes in Civil Engineering*. Springer. https://doi.org/10.1007/978-3-319-64349-6_43
- Tzmatzis, A., & Asteris, P. (2003). Finite element analysis of masonry structures: part I-Review of previous work. In: 9th North American masonry conference.
- Valente, M., & Milani, G. (2016). Non-linear dynamic and static analyses on eight historical masonry towers in the North-East of Italy. *Engineering Structures*. <https://doi.org/10.1016/j.engstruct.2016.02.004>

Publisher's Note Springer Nature remains neutral with regard to jurisdictional claims in published maps and institutional affiliations.