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DYNAMIC CHARACTERIZATION AND SEISMIC ASSESSMENT OF HISTORIC MASONRY STRUCTURE OF RUMI DARWAZA

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ABSTRACT

Historical monuments portray the architectural heritage and cultural wealth of a civilization. It is of paramount importance that these monuments be preserved for future generations. Foremost symbol of Lucknow Awadhi architecture, *Rumi Darwaza* is an 18th century gateway structure characterized by a half-spherical dome resting on half-octagonal plan and further supported by an arch. The masonry structure, built using thin burnt clay bricks (Lakhauri) and lime-crushed brick aggregate (surkhi) mortar, has developed major cracks in the arch due to natural aging and other environmental factors. Past earthquakes in India have highlighted the poor seismic performance of our monuments. Seismic assessment of historic monuments needs to be carried out to evaluate their structural response and draw out retrofitting plans to ensure their longevity. To assess the seismic performance, dynamic characteristics, such as natural frequencies, mode shapes and damping ratios, are required for realistic numerical simulation which have been obtained from field vibration tests. Laboratory experiments have been conducted to evaluate the mechanical properties of Lakhauri bricks, lime-surkhi mortar, and masonry assemblages also. A detailed Finite Element (FE) model, closely resembling the original structure, has been developed to understand the structural behaviour under seismic loads. Dynamic characterization results are seen to match closely with analytical predictions validating the FE model. Response Spectrum analysis showed the stiffening arch at the open face of half-dome to be the critical structural element with high tensile stresses at same locations where damages have been observed in the structure. Based on the developed understanding, two strengthening techniques using latticed structure of concrete filled hollow steel tubes to support the dome-arch from the inside have been proposed to reduce its vulnerability for seismic forces.

KEYWORDS: *dynamic characterization, finite element modelling, historic masonry, seismic assessment, strengthening*

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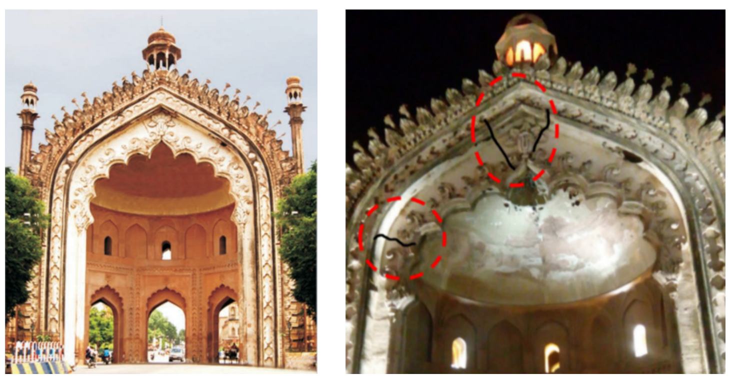
INTRODUCTION

Historic preservation is a worldwide endeavor aimed at protecting and conserving buildings of historical significance. Historical monuments being symbols of pride of our civilizations help us appreciate our past. Therefore, it is a society's duty to preserve and protect these monuments. The city of *Lucknow* derives its charm from the glorious Awadhi architecture and monumental heritage. *Rumi Darwaza*, a magnificent and unparalleled creation of Hindu-Muslim architecture, is a gateway structure characterized by half spherical dome resting on half octagonal plan and the half dome further supported by an arch. Due to natural aging, vehicular movement through the gate and other deteriorating natural influences, major cracks have developed in dome and arch region (Figure 1). Repair work carried out by Archaeological Survey of India (ASI) has worn out within a very short span of time. Furthermore, cracks that have been observed are due to structural problems, but no structural remedial measures have been undertaken. In the current study, seismic assessment of the historic masonry structure of *Rumi Darwaza* has been carried out and strengthening techniques to mitigate its vulnerability to seismic forces have been proposed.



(a) (b)

Figure 1: (a) Rumi Darwaza, Lucknow and (b) Cracks at dome crown and arch

FIELD VIBRATION TEST

Forced vibration testing of *Rumi Darwaza* used a small APS 113 ELECTRO-SEIS linear shaker which is a long stroke, electrodynamic force generator that can input sine waves, swept sine waves or random force waveforms. It can generate a modest peak force of 133 N within the 2-20 Hz frequency range which can be transferred directly through friction between its base and structure, and hence does not require any special anchoring to the structure. Four SS-1 Ranger Seismometers (Kinematics, USA), were used to measure the small vibrations induced in the structure. The experimental testing was performed in two phases. First phase involved ambient and broad sine sweep forced vibration tests for identification of natural frequencies of the structure. The second phase involved fixed frequency forced vibration tests at each identified frequency for estimating the mode shape. Figure 2 shows the different locations on the structure at which sensors and shaker were placed. Measurements were made in both orthogonal directions: x-direction along the in-plane direction of the monument, and y-direction along the out-of-plane direction of the monument along the road. The sampling rate for data acquisition was kept at 2000 samples/s.

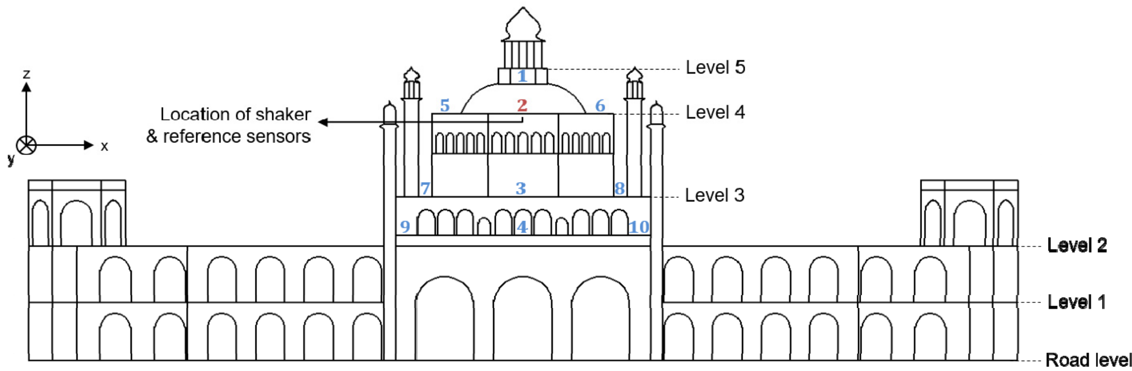
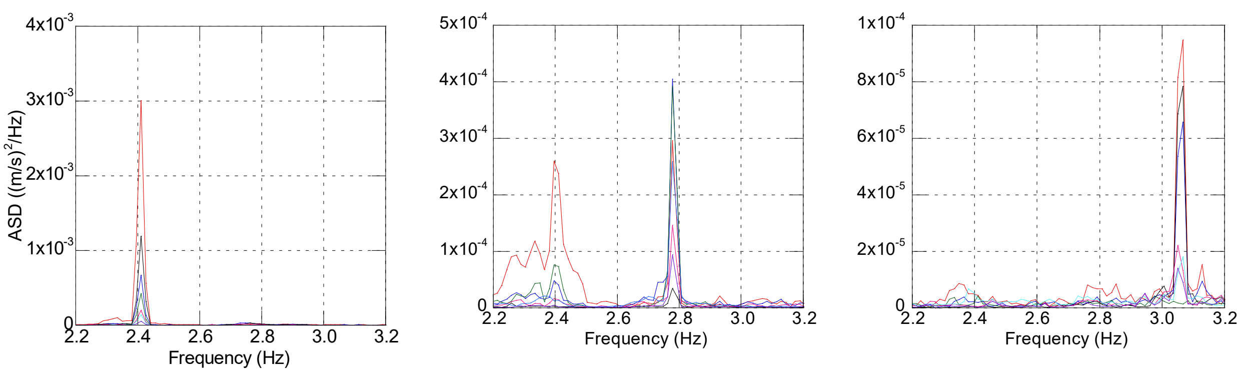


Figure 2: Location of sensors and linear shaker on monument

First, the natural frequencies of the structure were identified by ambient and forced vibrations exciting the structure over a broad range of frequencies in the two directions. In the broad range frequency sweep, multiple runs of sine sweep waveforms with frequencies ranging from 0.1-9 Hz in a 150 s total duration were used to get a rough estimate of the natural frequencies. This broad sweep was followed by a narrow range of sine sweep around the rough estimate to get a better estimate of natural frequency. A simple and fast frequency domain technique called Peak Picking (PP) was used for approximate estimates at the site for the modal parameters. The natural frequencies were identified from peaks in ASD (Auto Spectral Density) and CSD (Cross Spectral Density) plots where the cross spectral phases were either zero or π radians. This data was later processed for damping ratio by taking the piece of SDOF density function around natural frequency back into time domain and estimating damping from logarithmic decrement of the corresponding SDOF auto correlation function.

The second phase of testing involved measuring the structural response at strategically chosen locations on the structure to find the mode shape coordinates. Since the number of locations to be measured were greater than the number of sensors available, multiple test setups had to be performed to cover the entire structure. Before the beginning of second phase, two sensors were fixed as the reference sensors at Location 2 in the two orthogonal directions, close to the linear shaker as shown in Figure 2. The other two sensors were employed as the rover sensors which were moved to a different location in each test setup. During each test setup, the rover sensors were first oriented in y-direction while the structure was excited at fixed frequencies, i.e., natural frequencies identified in the first phase of testing. The input force waveform was a sine wave at the determined frequency for a duration of 180 s. The procedure was repeated for orientation of the rover sensors changed to x-direction. Another frequency domain approach known as Frequency Domain Decomposition (FDD) [1] was used for processing data measured in this phase of testing. FDD procedure also begins with estimation of spectral density matrix (ASD and CSD). Both procedures, PP and FDD, use modified periodogram method [2] for ASD and CSD estimation which involves dividing the time history record into periodograms with certain overlap between them and then averaging individual FFTs together with windowing applied. The velocity time history records were sectioned into periodograms of 2^{17} points with 50% overlapping for spectral averaging. A Hanning window was applied to prevent spectral leakage. Frequency resolution of 0.0153 Hz was achieved for sampling rate of 2000 samples/s. The spectral matrix is then decomposed by taking the Singular Value Decomposition (SVD) at each frequency. The curves representing the singular values can be inspected to yield the natural frequency and corresponding mode shape. Since the measurements were made in multiple test setups, mode shape estimations from different test setups had to be rescaled to the same level first before combining them together. This rescaling to the same level was carried out through the Post Global Estimation Rescaling (PoGER) approach [3].

From the first phase of testing, two modes in y-direction (out-of-plane) having natural frequencies of 2.41 Hz and 2.78 Hz, and one mode in x-direction (in-plane) having a natural frequency of 3.06 Hz were identified. It was observed that the structure lies in the acceleration sensitive region of the seismic design response spectrum, and therefore could be expected to attract large forces and experience high base shear in event of an earthquake. Figure 3 shows the ASD plots of data measured from select locations from different setups which show dominant response at identified frequencies of different modes, demonstrating the ability of the small shaker to excite the structure. A summary of results of the field vibration testing is provided in Table 1. Damping ratio is observed to decrease for higher modes. Mode shapes for the three identified modes are shown in Figure 4.

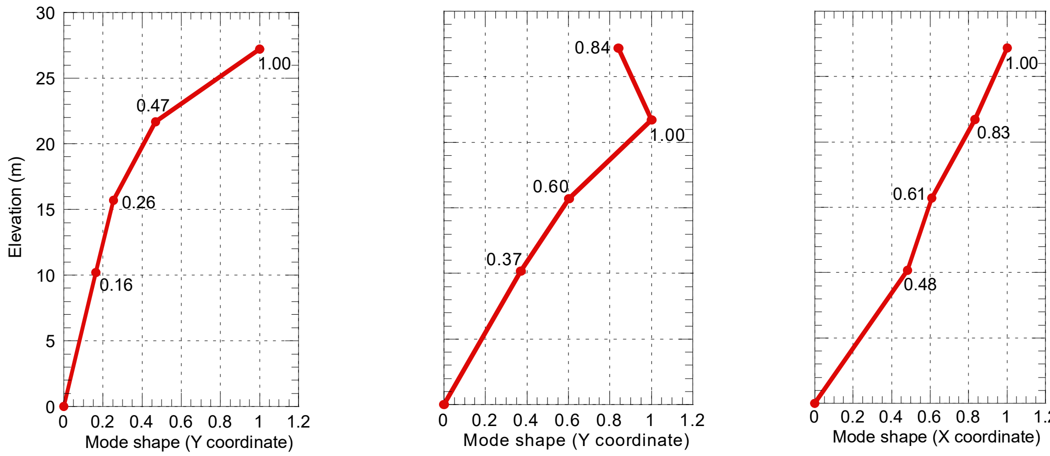


(a) (b) (c)

Figure 3: ASD plots from fixed frequency tests for (a) Mode A, (b) Mode B and (c) Mode C

Table 1: Summary of results of field vibration testing

Mode	**Natural Frequency (Hz)**		**Damping Ratio**	**Nature of mode**
	Ambient	Forced		
Mode A	2.41	2.41	2.7%	Out-of-plane (y-direction)
Mode B	2.82	2.78	2.3%	Out-of-plane (y-direction)
Mode C	3.14	3.06	1.8%	In -plane (x-direction)



(a) (b) (c)

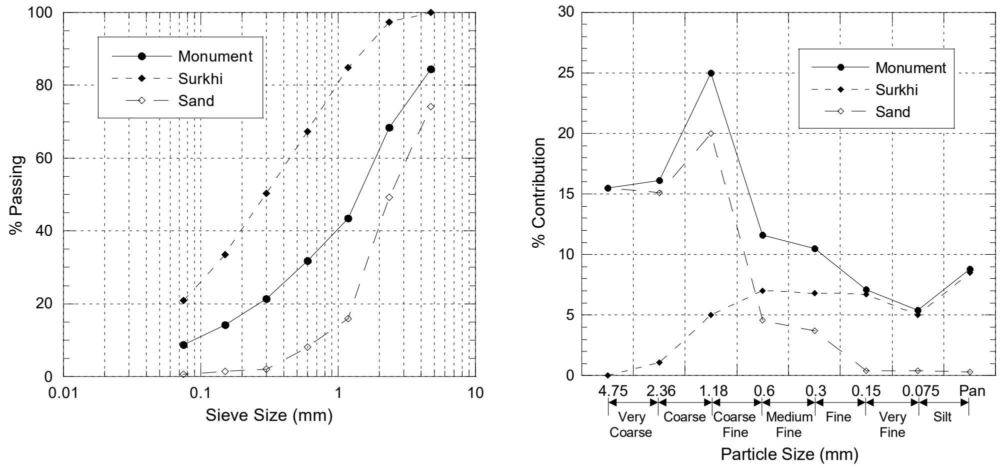
Figure 4: Normalized mode shapes for (a) Mode A, (b) Mode B and (c) Mode C

MECHANICAL CHARACTERIZATION

Any structural intervention for a monument requires detailed material characterization of the existing materials of the monument. Interventions for heritage structures and historic monuments are generally guided by the concept of retreatability/repairability, which implies that the new materials should not have any negative consequences on the existing material in the short or long term. However, the material used in the intervention work by ASI was found to differ in several aspects from the existing mortar such as binder aggregate ratio, particle size distribution and hydraulicity based on laboratory tests on reclaimed Lakhauri bricks and lime-*surkhi* mortar [4]. Experiments conducted on masonry materials used in the construction of the structure for their mechanical properties have been described here.

Lime-*surkhi* mortar, composed of lime as binder with crushed brick (*surkhi*) and sand as aggregate, was formulated based on mortar proportion and aggregate fraction suggested in an earlier study [4]. A mortar proportion of 1:1:1.5 (lime:*surkhi*:sand) by volume was used with aggregate size distribution of *surkhi* and sand shown in Figure 5. The contribution at each grain size from *surkhi* and sand is such that the combination is close to the original size distribution of mortar used in the monument. Locally available lime was used in the formulated mortar. Eight mortar cubes of 50 mm size were tested for their compressive strength at 28 days age as per ASTM C109 [5]. Compressive strength for mortar was found to be 1.06 MPa (COV = 13%). Mortar showed a very ductile behaviour, reaching 12-15% strain before losing 50% of its strength. The low strength of mortar is attributed to slow rate of carbonation which takes several years to complete.

Six fully baked (red) and partially baked (pale yellow) *Lakhauri* bricks were tested for their compressive strength. Because of the irregular shape, the bricks were cut with a tile cutter to an approximate size of 120 mm x 80 mm x 20 mm. The surfaces were levelled and capped as per ASTM C67-14 [6] with plaster of paris. Compressive strength for fully and partially baked bricks was found to be 36.2 MPa (COV = 15%) and 19.0 MPa (COV = 13%) respectively.



(a) (b)

Figure 5: (a) Aggregate size distribution and (b) Contribution of *surkhi* and sand in formulated mortar

Masonry prisms prepared using fully baked *Lakhauri* bricks and formulated lime-*surkhi* mortar were tested for their uniaxial compressive strength and modulus of elasticity. The masonry prisms were prepared with a mortar course thickness almost 3/4th of brick thickness, typical of what has been observed for Lucknow monuments. Standard five brick prisms, 120 mm × 80 mm × 180 mm in size, were tested according to ASTM C1314-16 [7] at 28 days. The compressive strength of stack bond prisms was corrected for their aspect ratio by applying the correction as described in ASTM C1314-16. Modulus of elasticity was calculated as secant modulus between ordinates corresponding to 5% and 33% of the compressive strength of specimens [8]. Compressive strength and modulus of elasticity was found to be 2.72 MPa (COV = 5%) and 964.3 MPa (COV = 5%) respectively. Since stack bond prisms may not actually simulate the conditions of masonry material present in the structure, as the actual masonry has many more interactions between brick and mortar along the bed joints, head joints and collar joints, multi-wythe masonry specimens were prepared to simulate the actual field interaction. Multi-wythe masonry prisms, 250 mm x 250 mm x 410 mm in size, were tested for compressive strength and modulus of elasticity. Compressive strength and modulus of elasticity was found to be 2.04 MPa (COV = 9%) and 65.1 MPa (COV = 15%), respectively. The average compressive strength of multi-wythe prisms was about 75% of compressive strength of stack bond prisms with the increased mortar bonds along the head joints and collar joints causing the reduction in compressive strength. Stress-strain curves from stack bond prism and multi-wythe prism compressive strength tests have been shown in Figure 6.

Tensile strength of masonry was evaluated by the diagonal tension test on specimens prepared as per RILEM standard LUM B6 [9]. Since *Lakhauri* bricks are very thin, a specimen prepared as per ASTM standards would be too slender. RILEM standard LUM B6 lays the guidelines for diagonal tension testing of smaller wall specimens with at least four brick units long length.

Masonry specimens, with a size 520 mm x 520 mm x 85 mm were tested at 28 days age. The tensile strength of masonry was evaluated to be 4.7% of the compressive strength of masonry.

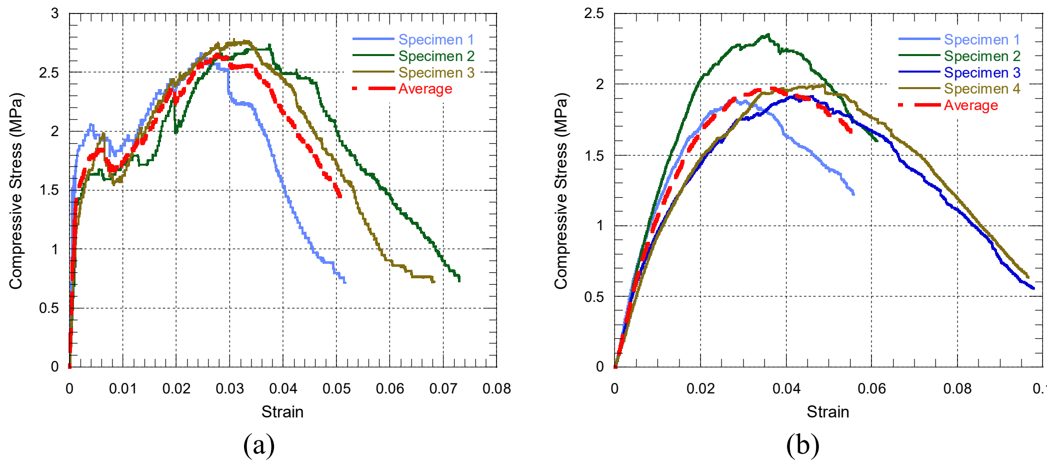


Figure 6: Stress strain curves for (a) Stack bond prism and (b) Multi-wythe prism compressive strength test

SEISMIC ASSESSMENT USING FINITE ELEMENT ANALYSES

Seismic response of a structure can be predicted through Finite Element (FE) modelling of the structure. Depending on the level of accuracy and simplicity desired, different modelling strategies can be used to model the composite masonry material. For large structures, such as monuments, macro modelling yields sufficiently accurate results about the global behaviour. In the current study, macro modelling approach has been adopted. Material properties defined in Abaqus for masonry included density as 1790 kg/m^3, modulus of elasticity as 964.3 MPa and Poisson's ratio of 0.2.

The structure of *Rumi Darwaza* presents a very complex geometry with several features that mandate the use of a sophisticated software which has the capabilities to define the complex geometry as close as possible to the actual shape. A 3-D model of the monument was prepared in AutoCAD software. Some simplifying assumptions such as modelling the curved arches as semi circles were made. The ornamentation around the main arch gate and *Guldastas* (flower petals) on parapet walls were not considered to be important owing to their insignificant mass. Due to the complexity presented by the geometry, HyperMesh was used as a pre-processor to mesh the geometry. Volume tetra meshing with 4-node tetrahedral element C3D4 was used for meshing the geometry. Element size and minimum element size for the elements were defined based on the fineness level of meshing required. The final mesh had a total of 211,087 nodes and 843,359 elements. The total mass of the monument is estimated at 120 × 10^3 kN. The 3-D geometry mesh shown in Figure 7 was imported into Abaqus for carrying out subsequent FE analyses.

Modal analysis of FE model was performed to obtain modal parameters such as natural frequencies, mode shapes and associated parameters of modal and mass participation factors. These parameters can be compared with those obtained from dynamic characterization for calibration of FE model. Based on the fundamental frequency obtained from modal analysis and dynamic characterization, the modulus of elasticity was updated to get a better estimate of global stiffness of structure, as shown in Equation 1. Results of modal analysis using the updated modulus of elasticity have been compared with the results obtained from dynamic characterization in Table 2, with Modal Assurance Criteria (MAC) as a measure of closeness between experimental mode shapes and analytical mode shapes. A comparison of experimental and analytical mode shapes has been shown in Figure 8.

$$E_{updated} = E_{old} \times \left(\frac{f_{updated}}{f_{old}} \right)^2 = 964.3 \times \left(\frac{2.41}{2.74} \right)^2 = 744.7 MPa \tag{1}$$

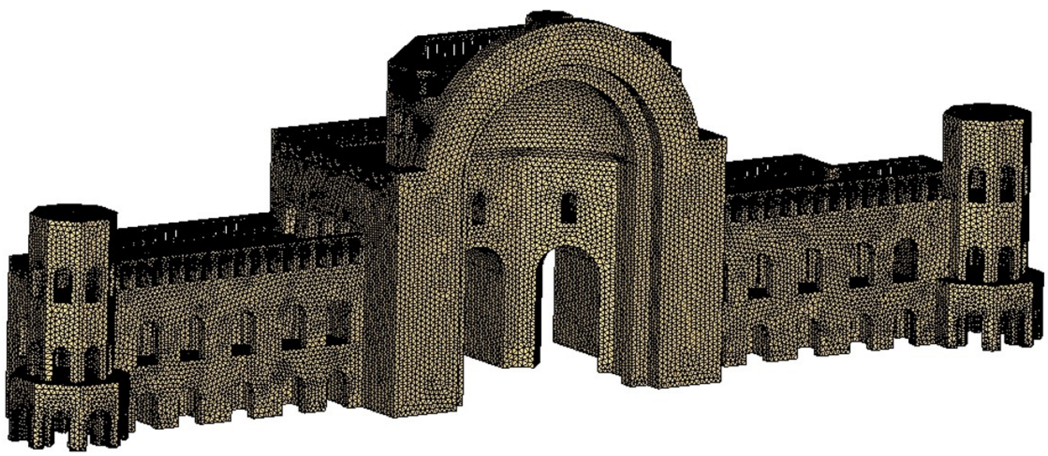
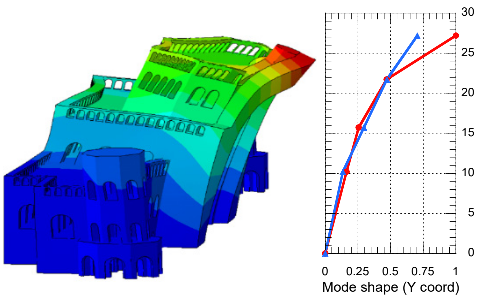


Figure 7: Meshed 3-D geometry of Rumi Darwaza

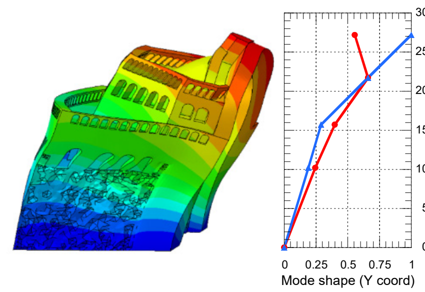
Table 2: Comparison between dynamic characterization results and FE modal analysis

Mode	Mass participation (% of total mass)		Natural Frequency (Hz)		MAC
	In-plane	Out-of-plane	Experimental	Analytical	
Mode A	0.0	31.2	2.41	2.41	0.94
Mode B	0.0	7.4	2.78	2.95	0.83
Mode C	33.2	0.0	3.06	2.56	0.98

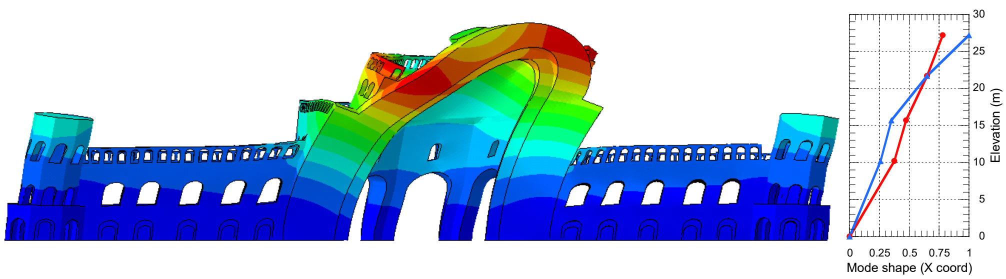
The seismic demand on the structure is evaluated using response spectrum analysis, considering all modes with natural frequency below 25 Hz such that the mass participation in both orthogonal directions is at least 90%. Since *Rumi Darwaza* is situated near *Gomati* river basin, the response spectrum given in IS:1893[10] for soft soil 5% damping, is scaled for Zone III for *Lucknow* city and Importance Factor of 1.5 for heritage structures. The structural response was evaluated for earthquake loads in two orthogonal directions. The responses were summed as per SRSS summation rule. Maximum stress and displacement and their locations were monitored during the analysis. Displacement and tensile stress (S_{11}) contours are shown in Figure 9.



(a) Mode A: Analytical (2.41 Hz) and Experimental (2.41 Hz); MAC = 0.94

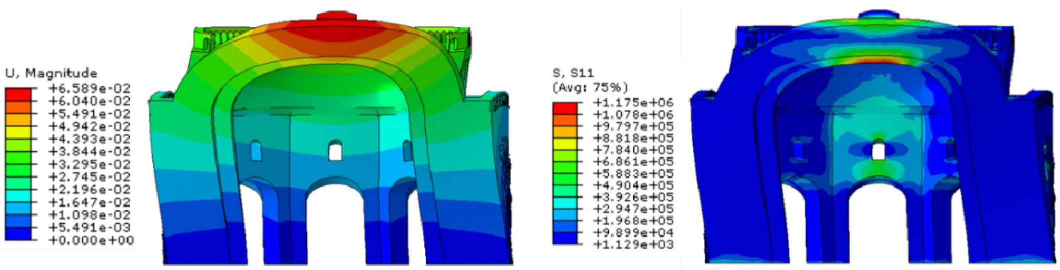


(b) Mode B: Analytical (2.95 Hz) and Experimental (2.78 Hz); MAC = 0.83

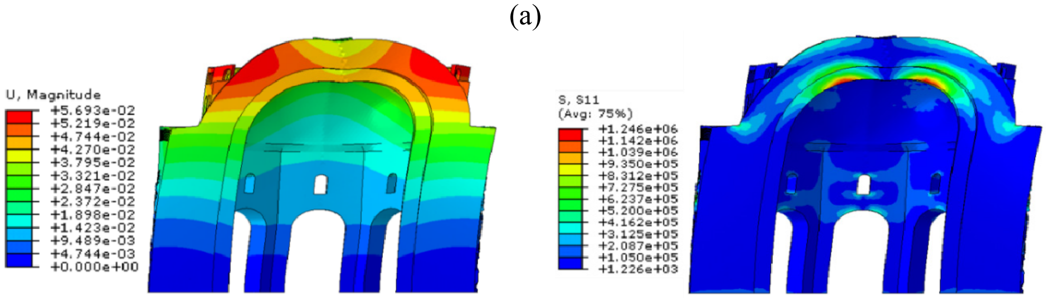


(c) Mode C: Analytical (2.56 Hz) and Experimental (3.06 Hz); MAC = 0.98

Figure 8: Comparison of experimental and analytical mode shapes



(a)

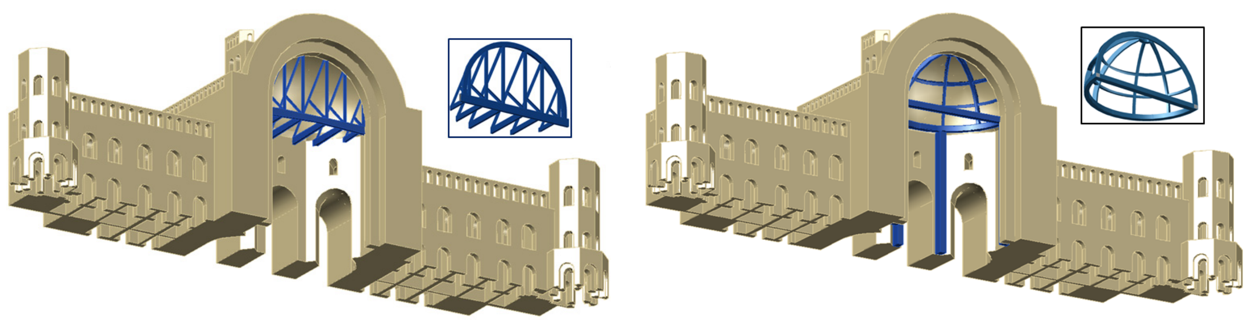


(b)

Figure 9: Displacement and principle tensile stress (S_{11}) contour for (a) Out-of-plane and (b) In-plane direction earthquake loading

Base shear demand (as % of seismic weight) for both the orthogonal directions was found to be 30%. Predicted tensile stresses were seen to exceed the tensile capacity of masonry, while compressive stresses remained below the compressive strength. Displacement and stress contours indicated that half dome and arch at the open face edge are the vulnerable elements of the structure. Due to its instability, the half-dome is seen to deform excessively in the out-of-plane direction. Additionally, the arch-pier junction is seen to be pushed laterally outward due to local bending of half-dome as there is no lateral restraint available. Damages can be observed in the same locations of high tensile stress in the actual structure also, as shown in Figure 1b.

Based on the developed understanding of structural response, two strengthening techniques have been proposed to support and strengthen the dome/arch portion of the monument. The aim of these techniques is to restrict the movement of dome crown, to relieve the stress in the structure under its own load and to diffuse the high tensile stress in critical locations by adding structural elements increasing the structures' stiffness. First strengthening technique employs concrete filled hollow steel tubes (CFT) of 25 mm thickness. It is composed of a relieving arch supported by cantilever beams through truss like members. The weight of the frame is supported by the thick masonry walls by anchoring using threaded rods grouted into the walls of structure. The aim of this strengthening technique is to prop the arch/dome portion and reduce the high tensile stress concentration. The second technique provides strengthening using a relieving half-dome, which employs concrete filled steel tubes of 50 mm thickness. It is composed of arch beams along the dome intrados. These arches rest on a curved beam which is tied together by a horizontal member. They are connected to each other through cross-ribs which curve along the curvature of dome. The frame is supported on columns of composite section of 50 mm thick concrete filled steel tubes which are located such that traffic flow is not hindered. The aim of the strengthening technique is to provide support to half-dome by transferring some portion of dead load through the columns. The frame is supported by columns and hence no additional dead load is added to the structure. Both the strengthening techniques, Relieving Arch (RA) and Relieving Half-Dome (RH) are shown in Figure 10. Material properties used for FE modelling are listed in Table 3. Effect of the two proposed techniques on structural response is summarized in Table 4.



(a) (b)

Figure 10: Strengthening using (a) Relieving Arch (RA) and (b) Relieving Half-Dome (RH)

Table 3: Material properties in FE modelling

Material	Density	Modulus of Elasticity	Poisson's Ratio
Steel	7850 kg/m^3	200 GPa	0.3
Concrete	2400 kg/m^3	35 GPa	0.2

Table 4: Effect of strengthening techniques on structural response

Structural Response	Strengthening Technique	Effect
Fundamental Frequency	RA	Increased from 2.41 Hz to 2.51 Hz
	RH	Increased from 2.41 Hz to 2.78 Hz, considerable stiffness added to structure in out-of-plane direction
Movement of dome crown and arch	RA	Reduced by 11% in out-of-plane direction
	RH	Reduced by 40% in out-of-plane direction
Lateral outward movement of arch-pier junctions	RA	Reduced by 44% (under self weight)
	RH	Effectively tied against laterally outward thrust with 95% reduction (under self weight)
Tensile stress concentration at arch intrados & dome crown	RA	High tensile stress concentration reduced by 50%.
	RH	High tensile stress concentration reduced by 67%

SUMMARY AND CONCLUSIONS

The historic masonry structure of a monument, *Rumi Darwaza* of *Lucknow*, a hallmark of 18^{th} century Awadhi architecture, has been assessed for its seismic performance. Seismic analysis of the structure revealed the dome crown and stiffening arch at the front face to be the critical structural elements. Tensile stress concentration on the inside of the arch was seen to exceed masonry tensile strength, however compressive stresses remained below masonry compressive strength. Based on understanding of structural response, two strengthening techniques have been proposed using concrete filled hollow steel tubes. Relieving Arch (RA) composed of arch supported by cantilever beams through truss like members aimed to prop the arch/dome. The high tensile stresses were reduced by ~50%. Strengthening technique using Relieving Half-Dome (RH) composed of arch beams on the dome intrados restrict its movements under seismic loads. The high tensile stress concentration was effectively reduced by 67% and the movement of dome crown and arch was reduced by 40%. Strengthening technique RH offered several advantages over technique RA in terms of lower tensile stress, reduced displacement of dome, bearing part of structure's weight and adding considerable stiffness in out-of-plane direction.

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