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Seismic Vulnerability Assessment of Historic Masonry Structure: A Case Study of Bindhyabasini Temple

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Abstract

The seismic vulnerability assessment of Bindhyabasini Temple, a 250-year-old stone unreinforced masonry structure in Nepal, is presented in this paper. The study encompasses both qualitative and quantitative evaluations. Qualitative assessments involved rapid visual screening and empirical vulnerability index methods, supported by field visits and interviews with locals and the temple committee. Quantitative assessments entailed computer software-based building design and seismic behaviour evaluation through time history analysis. Following both evaluations, a vulnerability curve was generated and a checklist was filled. The vulnerability curve showed the expected damage at different intensities which may be slight, moderate, and extensive damage grade. The structure is vulnerable to both tension and shear in the openings and dome-wall connection. This research serves as a valuable resource for assessing seismic vulnerability in similar building typologies.

Keywords

Non-destructive Test, Rapid Visual Screening, Stone Masonry, Time History Analysis, Vulnerability Curve, Vulnerability Index

1. Introduction

1.1 Background of Study

Nepal, situated between the Indian plate and the Eurasian plate, is highly vulnerable to earthquakes due to its tectonic setting and the presence of major fault lines. The collision of these two plates has led to the formation of the Himalayan range, making the Himalayan belt seismically active. The tectonic activities result in frequent earthquakes including major events of 1990 B.S. and 2072 B.S. According to the Bureau of Crises Prevention and Recovery, UNDP, Nepal has been ranked as the eleventh most vulnerable area in terms of earthquake risk [1]. Figure 1 delineates the various seismic zones within the country, each color-coded to indicate different levels of seismic risk. As per this zonation map, Kaski lies in Zone III. Recent studies showed that masonry structures in the region can suffer moderate to heavy damage under earthquakes comparable to the Jajarkot Earthquake [2].

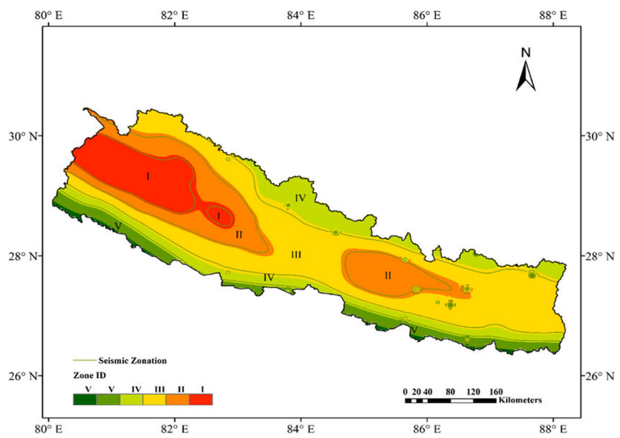
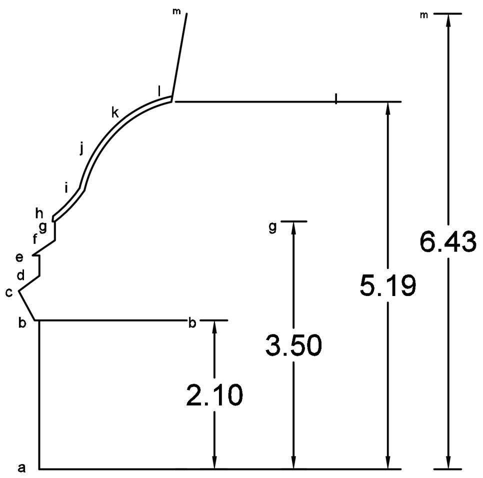


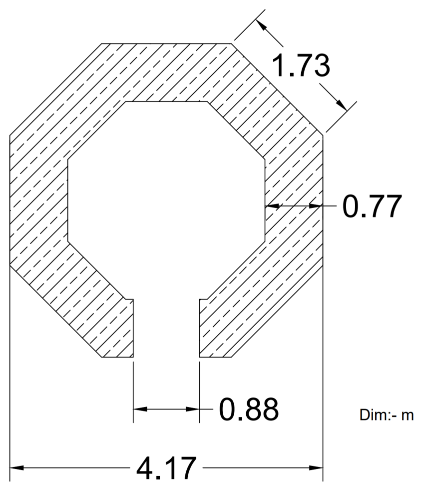
Figure 1: Seismic Zoning Map of Nepal. [3]

The main goal of seismic vulnerability assessment is to generate a vulnerability curve which quantify the damage that might occur in a structure due to seismic activities. The vulnerability curve is one of the important parameters for post-earthquake building assessments [4]. It can in turn help to develop risk reduction and mitigating measures and strengthening techniques of the structure. Seismic vulnerability assessments also help to establish building codes, standard design to enhance structural integrity and safety, and to analyze existing structures. Seismic vulnerability assessment can be categorized into two phases, qualitative and quantitative assessments.

The Bindhyabasini Temple stands as a significant Hindu place of worship in the city of Pokhara, Nepal. The Bindhyabasini Temple is a remarkable demonstration of the nation's abundant cultural heritage and deep-rooted religious traditions. It is situated on a top hill, offering a commanding view of the beautiful Pokhara valley, with majestic mountain ranges. Historians trace the establishment of the Bindhyabasini Temple back approximately 250 years to the legendary King Siddhi Narayan Shah of Kaski. The legend of the Bindhyabasini Temple in Pokhara, Nepal, traces back to a dream experienced by King Siddhi Narayan Malla of Kaski. The architecture showcases a harmonious blend of traditional Nepali *Shikhara* style. *Shikhara* translates to mountain peak or summit, and it refers to the rising tower or pinnacle that crowns many Hindu temples. *Shikhara* style temples are commonly constructed using stone or brick. *Shikhara* style architecture often exhibits a high degree of symmetry and proportion. The dome consists of stone only from basement steps which are supported by walls constructed using many individual and regularly heavy stones [5]. The Bindhyabasini temple is said to be constructed using a harmonious combination of stone masonry, mud, and surkhi. The temple is painted with limestone-based paint, presents a captivating sight, and seamlessly combines beauty and durability. Figure 2 shows the height, plan, and the section of the temple structure (all dimensions are measured in meters).



(a)



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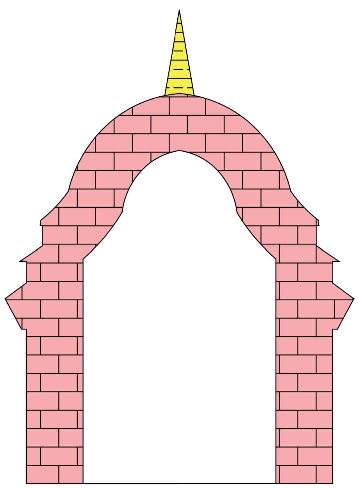


(c)

Figure 2: Bindhyabasini Temple; (a)Height, (b)Plan, and (c)Section

1.2 Statement of Problem

The problem lies in understanding the specific vulnerabilities of Bindhyabasini temple to seismic forces. The seismic vulnerability of historic masonry structures presents a significant challenge due to their architectural significance, cultural heritage value, and susceptibility to earthquake-induced damage. The Bindhyabasini Temple, an iconic historic masonry structure located on a hilltop, represents a quintessential example of such heritage sites. However, the potential seismic risk to the temple raises concerns regarding its structural integrity and preservation. Inadequate lateral load resistance, frail mortar, absence of reinforcement, and deteriorated materials all contribute to the vulnerability of the temple to earthquake damage.

1.3 Objectives of Study

The general objective of this study is to investigate the seismic vulnerability of Bindhyabasini temple. Moreover, the specific objectives are summarized as follows:

- To generate a vulnerability curve from the qualitative assessment.
- To evaluate stresses developed in the temple structure from the quantitative assessment.

2. Methodology

The seismic vulnerability assessment of the Bindhyabasini Temple follows a structured methodology outlined in a three-phase process namely; Data collection, Qualitative assessment and Quantitative assessment as shown in Figure 3. These steps culminate in the completion of a vulnerability checklist, from which the final results regarding the temple's seismic vulnerability are extracted.

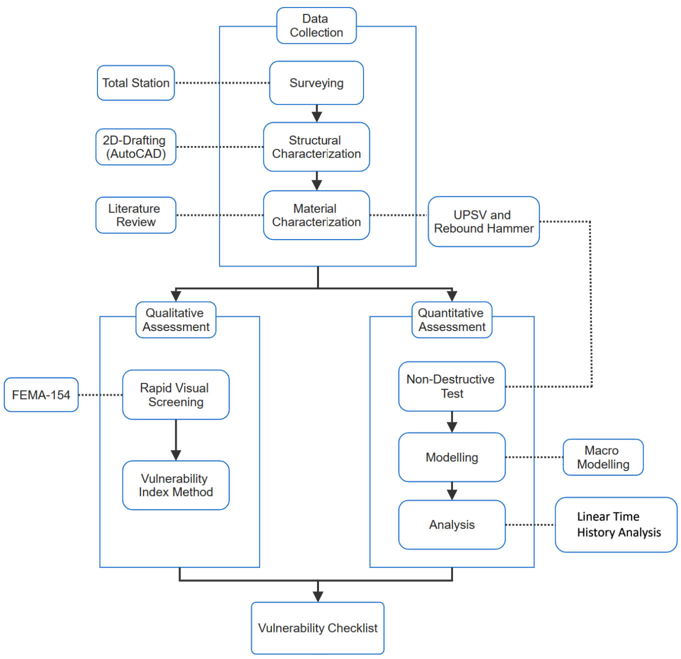


Figure 3: Methodology for Vulnerability Assessment

2.1 Qualitative Assessment

Tier 1 evaluation, also known as qualitative assessment, primarily aims to determine whether the building, in its current state, possesses the desired seismic performance capability. This method involves reviewing available drawings and visually inspecting the building for signs of damage [1]. It serves as a qualitative measure to identify seismic weaknesses in a building before conducting a detailed evaluation [1, 6].

2.1.1 Rapid Visual Screening

The RVS process is a qualitative assessment method that employs a methodology centred on conducting a sidewalk survey of a building and completing a Data Collection Form. This form is filled out by the surveyor based on visual observations of the building's exterior and, if feasible, its interior [7].

2.1.2 Vulnerability Index Method

The vulnerability of a structure to earthquake-induced damage is often assessed through the Vulnerability Index method, which is based on the GNDT (Gruppo Nazionale per la Difesa dai Terremoti or National Group for Earthquake Defence) II approach [8, 9] outlined as in GNDT-SSN (II). This approach is widely used for identifying and characterizing potential seismic risk in buildings [2]. It involves assigning points to significant structural components, enabling the calculation of a Seismic Vulnerability Index (I_v). The vulnerability index that has been calculated can subsequently be utilized to predict structural damage, correlating it with a specified seismic event intensity. Both qualitative and quantitative parameters are considered in this process, encompassing factors such as building materials, construction quality, and numerical measurements related to structural elements. The indirect method is used for determining the vulnerability index which determines the vulnerability index of the structure at first and then relates the damage grade based on the European Macroseismic scale (EMS-98) and seismic intensity based on the Modified Mercalli Intensity Scale (MMI).

1. **Wall Slenderness (P1)**: Wall slenderness is a critical factor affecting the out-of-plane behaviour of walls [10]. In masonry buildings, walls are notably thick, generally not less than 0.45–0.5 meters. The slenderness of these walls, influenced by their height and thickness, typically varies between a ratio of 4 and 22.5 [11].
2. **Maximum Wall span (P2)**: The maximum wall span is an additional geometric parameter that impacts the out-of-plane response of walls [11]. Vulnerability index formulations that include this factor categorize it by the ratio of span to thickness [8]. However, since wall thickness has been addressed in an earlier parameter, this factor specifically examines the variation in maximum wall span (s), which is measured in meters.
3. **Type of Material (P3)**: Masonry constructions typically employ materials such as rammed earth, stone, adobe, and fired clay brick, each contributing to diverse structural typologies and wall morphologies [11]. The variations in masonry include:
   a) Different types, sizes, and shapes of masonry units, such as fired clay brick masonry, ashlar stone masonry, and irregular rubble stone masonry.
   b) Variations in the masonry layout, including irregular or regular horizontal courses, the presence of multiple leaves, and the absence of connections between leaves.
   c) The use of different types of mortar, when used. These aspects collectively determine the quality of the masonry, which in turn influences the building's seismic resilience.
4. **Wall-to-Wall connection (P4)**: Longer walls are more prone to overturning as they often do not have horizontal supports, such as cross walls, roofs, or floor systems along their lengths [12]. The quality of connections between walls, particularly at building corners and at junctions between internal and external walls, is critical for the building's seismic performance. In stone masonry structures, this requires ensuring adequate interlocking at intersections of orthogonal walls, which can typically be identified by the presence of vertical joints [11]. A lack of proper interlocking at these junctions can undermine the building's structural integrity and increase its vulnerability to seismic forces.
5. **Horizontal Diaphragms (P5)**: Horizontal diaphragms play a crucial role in transferring lateral earthquake loads to the vertical resisting elements of a structure. The flexibility of traditional timber floors in unreinforced masonry structures can result in significant bending and shear deformations when subjected to horizontal loads [13]. This excessive flexibility or inadequate connections with load-bearing walls can cause the walls to function independently, leading to local out-of-plane failure during earthquake loading events [12].
6. **Roof Thrust (P6)**: Certain roof structures can exert lateral thrust, which may lead to the out-of-plane failure of load-bearing walls that support them [11, 12]. Whether a roof structure generates lateral thrust depends on its geometry or specific structural enhancements. The degree of thrust exerted by these roofs is influenced by several factors [11]:
   a) The roof span: Wider spans tend to produce greater lateral thrust.
   b) The weight of the roof: Heavier roofs intensify the lateral thrust exerted.
   c) The roof's slope: The angle or steepness of the roof influences the distribution and strength of the lateral thrust impacting the walls.

   These elements collectively affect the potential for lateral thrust and, consequently, the structural stability of load-bearing walls.
7. **Wall Openings (P7)**: Openings in walls specifically designed for seismic resistance primarily affect their in-plane behaviour, reducing their capacity to handle in-plane forces [14]. This issue is especially critical in buildings that are more likely to experience in-plane damage. Additionally, the inclusion of strong,

well-connected diaphragms is crucial in preventing premature out-of-plane collapses.

8. **Number of floors (P8)**: Higher number of floors causes centre of gravity of the structure to rise [11]. The rise of centre of gravity causes an increase in the overturning moment of the wall making the building susceptible to collapse.
9. **State of conservation (P9)**: The state of conservation plays a significant role in preserving the stiffness and strength of masonry structures. Unmanaged and unrepaired deterioration can significantly increase the vulnerability of such structures [15].
10. **In-plane index (P10)**: The in-plane index parameter helps to check the shear strength of the structure in a perpendicular direction. Plane irregularity and in-plane stiffness assessed using this parameter can offer insight into its seismic performance potential [16].

Table 1: Vulnerability index formulation [11]

Symbol	Parameter	Class(C_{vi})				Weight (P_i)
		A	B	C	D	
P1	Wall slenderness	0	5	20	50	1
P2	Maximum wall span	0	5	20	50	0.5
P3	Type of material	0	5	20	50	1.5
P4	Wall-to-wall connections	0	5	20	50	0.75
P5	Horizontal diaphragms	0	5	20	50	1.5
P6	Roof thrust	0	5	20	50	0.5
P7	Wall openings	0	5	20	50	1.5
P8	Number of floors	0	5	20	50	1.5
P9	State of conservation	0	5	20	50	0.75
P10	In-plane index	0	5	20	50	0.5

Vulnerability Index (I_v) can be calculated using the weighted value of each parameter whose weightage is assigned based on it's importance [9, 11] using Table 1 and equations below:

$$\text{Vulnerability Index}(I_v) = \sum_{n=1}^{10} C_{vi} P_i \tag{1}$$

$$\text{Normalized Index}:\ 0 \le I_v \le 100 \tag{2}$$

Further, vulnerability curve is generated using equations 3 and 4 [11]:

$$V = 0.56 + 0.0064 I_v \tag{3}$$

$$\mu_D = 2.5 * \left(1 + \tanh\left(\frac{I + 6.25 * V - 13.1}{Q}\right)\right) \tag{4}$$

2.2 Quantitative Assessment

Unlike Qualitative assessment, which relies on visual observations and subjective judgment, Quantitative assessment employs models of the structure created digitally, using expected ground motion to analyze its behaviour, capacity, and performance. The second phase of the assessment involves a comprehensive seismic evaluation that includes a detailed analysis of the building to determine seismic strengthening measures. These modifications aim to address or mitigate seismic deficiencies identified during the initial evaluation phase.

2.2.1 Non-Destructive Test

The UPV machine generates ultrasonic pulses, and the time taken for these pulses to traverse the temple materials is recorded. Different sets of data i.e. indirect velocity are collected from the seven walls and domes of the temple structure. These data are used to estimate direct velocity using equation developed by Turgut and Kucuk [17] in their study:

$$V_d = 0.6867 V_h + 1561.3 \tag{5}$$

where, V_d= Direct velocity and V_h= Indirect velocity. The estimated direct velocity is then compared to the standard values as shown in Table 2 as per IS 516 (Part 5/Sec 1). The velocity data obtained from various sections contributes to a thorough assessment of the material integrity, helping identify potential defects or anomalies within the temple structure even though Equation 5 and Table 2 are studies made on concrete.

Table 2: Velocity Criterion for Concrete Quality Grading [18]

S.N.	Average velocity of Pulse velocity(m/s)	Result
1	Above 4400	Excellent
2	3750 to 4400	Good
3	3000 to 3750	Doubtful
4	Below 3000	Poor

Another methodology for assessing the condition of the temple structure involves employing the Rebound Hammer test with an abrasive stone. Nine sets of data are systematically collected from seven walls of the temple, with each wall considered separately. The test is not conducted in one wall due to the presence of door openings. A comprehensive chart is then utilized to establish the correlation between rebound numbers and corresponding compressive strength, facilitating a thorough evaluation of the structural integrity and overall condition of the temple for informed maintenance and preservation efforts. Table 3 is the correlation between the average rebound number and quality based on past work [19].

Table 3: Quality of concrete according to rebound number

Average Rebound number	Quality
>40	Very good hard layer
30-40	Good layer
20-30	Fair
<20	Poor
0	Delaminated

2.2.2 Numerical Modelling

The numerical modelling methodology employed in this study utilizes ETABS v18.1.1 software, adopting a macro modelling approach and implementing the finite element method (FEM). Focusing on the seismic performance of the Bindhyabasini

Temple, a multi-tiered structure, the foundation is characterized by a wide plinth platform, essentially functioning as a mat foundation [20]. Given this configuration, it is anticipated that stepped footings will be incorporated for the main wall. The temple's high plinth design, incorporating such a substantial plinth base, aims to establish robust foundations that mitigate earthquake risks associated with soft soils [21]. Key input parameters includes material properties and ground motion data. The dome and wall materials are assumed to share identical properties and are modelled with thick shell elements, treating the masonry as isotropic and homogeneous. Despite masonry being among the oldest structural materials, the investigation of its behaviour under seismic loads began relatively late compared to other materials such as concrete and steel [22]. NBC 102:1994 [23] suggests to follow Indian Standard IS 875 (Part I) -1987 due to the similarity of materials and their uses in Nepal and India. As per IS 875:1987, the unit weight of stone masonry varies from 20.40 KN/m3 to 26.5 KN/m3. Adhikari and Chaulagain [2023] conducted the compressive test on stone mud masonry using a UTM (Universal Testing Machine) and found the compressive strength to be 2.95 MPa and modulus of elasticity to be 71.78 MPa. However, for modelling purposes mechanical properties presented by Build Change [24] through laboratory test is used and is presented in Table 4 and Poisson's ratio is taken as 0.213 as used by Adhikari and Chaulagain [2023] in their model.

Table 4: Mechanical properties of Stone mud masonry through Laboratory test [24]

S.N.	Properties	Values
1	Compressive Strength	2.40 MPa
2	Young's modulus	65.10 MPa
3	Density	2200 kg/m^3

Ground motion data from the Gorkha earthquake as obtained from Kritipur station [25], are specified for Linear Time History Analysis. Figure 4 displays plot of PGA vs Time.

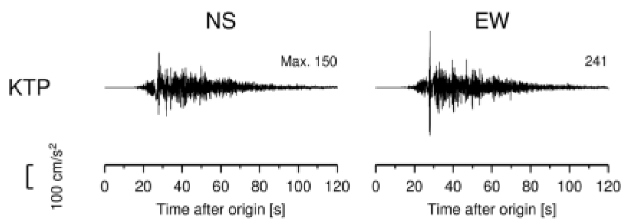


Figure 4: Peak Ground Acceleration vs Time plot of Gorkha Earthquake ground motion [25]

Further, response reduction factor R = 1.5 and the zone factor Z = 0.36 (for the most severe zone) are utilized with reference to IS 1893(Part-I):2016. Soil type II (for N=10 to 20 [26]) is used in the model based on core cutting test performed. The dry density was obtained to be 1.447 g/cc and the corresponding value of N for Standard penetration test was calculated to be 10.28 using equation below [27]:
$\rho_d = 1.267N^{0.057}$, where:- ρ_d = dry density, N= SPT number
Moreover, damping ratio is assumed to be 5% in the model [28]. The study employs linear time history analysis with a particular emphasis on deriving critical structural responses such as shell stresses, displacements, shear forces, and drift. Linear Time History method can offer simplicity and computational efficiency but the non-linearity of the structure is compromised [29]. Understanding the non linear behaviour of masonry can be a quite challenging, and time consuming so linear methodology provide a simpler way by underestimating the non-linear behavior of masonry under seismic loading. Neglecting non-linearity can produce inaccurate and conservative results [30].
Hence it is recommended for further investigation of structure using non-linear method.

3. Result and Discussion

3.1 Qualitative Evaluation

While filling out the FEMA-154 RVS form, the seismicity of the temple region (Pokhara) was taken from the seismic hazard map provided in the national building code (NBC 105:2020), and the form was selected as moderately high seismicity form accordingly. The soil type was selected as dense soil (Type C) by core cutting and visual inspection. No geological hazards were found at the site. There is no diaphragm instead there is a dome in the structure. Sloping Site vertical irregularity is found at the site. Moreover, non-parallel system plan irregularity is identified. As a result, the final level 1 score is calculated as 0.1 i.e. the minimum score which indicates higher damage risk and necessity of detailed investigation.

Expert opinions are utilized to assign class values (A, B, C, and D) to each parameter considered in the vulnerability index method. The corresponding class values (C_{vi}) are provided, and the product of the weight (P_i) and the class value (C_{vi}) for each parameter is calculated. The sum of these products for all 10 parameters yields the required Vulnerability Index (I_V) which is tabulated in Table 5.

Table 5: Vulnerability Index I_v

Symbol	Parameter	Assigned Class	Class Value (C_{vi})	Weight p_i	$C_{vi}*P_i$
P1	Wall Slenderness	A	0	1	0
P2	Maximum wall span	A	0	0.5	0
P3	Type of material	B	5	1.5	7.5
P4	Wall-to-wall connections	B	5	0.75	3.75
P5	Horizontal diaphragms	C	20	1.5	30
P6	Roof thrust	C	20	0.5	10
P7	Wall openings	A	0	1.5	0
P8	Number of floors	A	0	1.5	0
P9	State of conservation	B	5	0.75	3.75
P10	In-plane index	D	50	0.5	25
					$I_v = 80$

The mean damage grades are calculated for each level of intensity and the corresponding points are plotted on graph as shown in Figure 5.

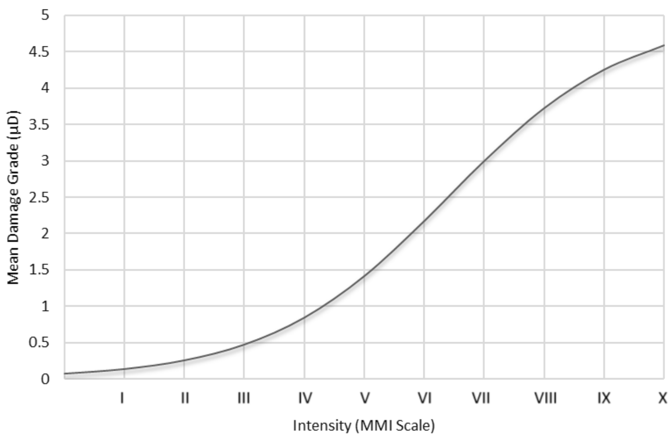


Figure 5: Vulnerability Curve

Correlating this index with the damaged grade across various MMI scales reveals a progression of damage severity, ranging from grade 2.16 at MMI intensity level VI to grade 4.58 at MMI intensity level X. Moreover, EMS- 98 defines Damage Grade 3 as Moderate structural damage and Heavy non-structural damage which corresponds to Intensity VII (MMI Scale) as per Figure 5. Conclusion can be made that as earthquake intensity increases, the expected damage grade also escalates, highlighting the critical need for preparedness and resilience measures in regions vulnerable to high-intensity seismic activity.

3.2 Quantitative Evaluation

The mean velocity for the wall section obtained from the Indirect method of Ultrasonic Pulse velocity method is 3259.78 m/s with a standard deviation of 972.61 m/s. Similarly, the mean velocity for dome section was obtained to be 352.33 m/s with a standard deviation of 256.62 m/s. Table 6 shows the estimation of direct velocity using Equation 5 and corresponding result as per Table 2.

Table 6: Result of UPV test

	Velocity (m/s)	Result
Indirect Velocity in wall	3259.78	Doubtful
Estimated Direct velocity in wall	3802.72	Good
Indirect Velocity in Dome	352.33	Poor
Estimated Direct in Dome	1802	Poor

The average rebound number was obtained from the rebound number obtained as presented in Table 7 and the quality check is done as per Table 3.

Table 7: Average rebound number and corresponding Quality of different walls

Wall	Average Rebound Number	Quality
1	36	Good Layer
2	39.44	Good Layer
3	39.9	Good Layer
4	41.5	Very Good Hard Layer
5	38.14	Good Layer
6	33.9	Good Layer
7	33.9	Good Layer

Time history analysis was done in both directions to find displacements, drift, and shear. Table 8 displays maximum values of displacement (Top), drift, and shear (Base) observed in both directions.

Table 8: Maximum values of Displacement, Drift and Shear

	Displacement (mm)	Drift (mm/mm)	Shear (KN)
X-Direction	10.451	0.002363	140.36
Y-Direction	10.820	0.002011	180

The allowable drift ratio is 0.004 [26]. This shows that the structure has not exceeded allowable drift in both directions. Shear stress is then calculated from base shear and the Demand Capacity Ratio is then determined taking capacity to be 0.1 MPa as per IS 15988. Calculations are shown in Table 9.

Table 9: Calculation of DCR

	Shear Stress(MPa)	DCR
X-Direction	0.043	0.43
Y-Direction	0.032	0.32

Here, DCR in both directions is obtained below 1 which indicates that the structural capacity of a building is adequate to withstand the expected seismic demands, suggesting a lower risk of damage during an earthquake.

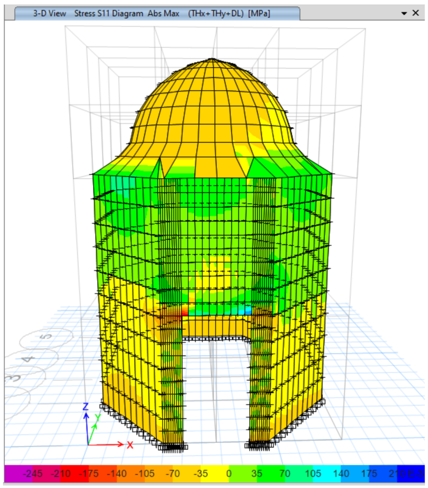
The stresses in the shell element are calculated and the contour is plotted. The most vulnerable part of structure is opening as the maximum stress is seen in the opening. The acceptable stress limits for compression, tension, and shear are capped at 2.4 MPa, 0.02 MPa, and 0.0035 MPa, respectively [31]. Hence, the stress values derived from the calculations, as presented in Table 10, indicate that the analysed structure is stable under compressive loads but poses risks under tensile and shear stress.

Table 10: Stresses on Masonry Wall

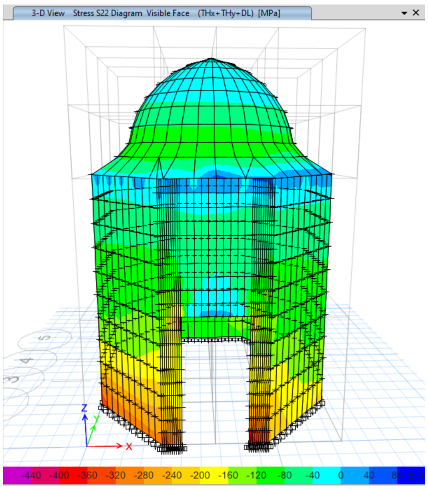
Component		**Stress (MPa)**
S11	C	0.18
	T	0.25
S22	C	0.01
	T	0.47
S12	C	0.1
	T	0.14

The stress contour on the wall due to the (DL+TH) is shown in

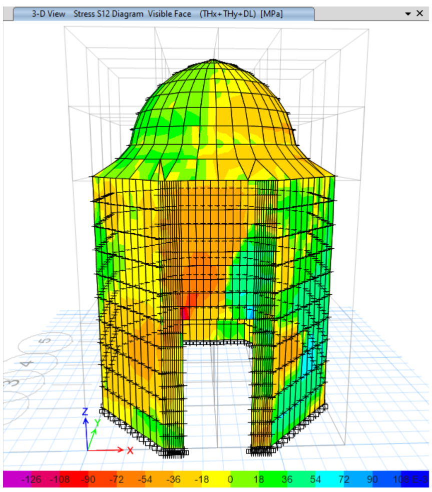
Figure 6. The contour shows the opening is the most vulnerable part of the structure followed by the base and the dome-wall connection.



(a) S11



(b) S22



(c) S12

Figure 6: Stress Concentration Diagram

4. Conclusion

Both qualitative and quantitative assessments were carried out in this study to assess the seismic vulnerability of Bindhyabasini Temple. From the evaluation, it can be concluded that the temple is vulnerable to future earthquakes of intensity greater than VI MMI scale.

From the FEMA-154 RVS form, the Level 1 score was found to be 0.1 which is less than a minimum acceptable score of 0.2 for URM. It indicates that plan and vertical irregularity are concerns for the performance of the building. For such buildings, detailed investigation need to be carried out. The empirical method used for the Vulnerability Index suggests a moderate susceptibility to seismic events with an Iv of 80. Moderate structural damage and heavy non-structural damage are expected at Intensity VII (MMI Scale).

The average Rebound Number for the majority of the walls was found to be in the range of 30 to 40, which indicates the good quality of the wall. However, the ultrasonic pulse velocity test exposed the poor quality of materials in the dome, a critical structural element. The result of the quantitative assessment shows that the structure is safe in compression but is unsafe in tensile and shear stress. Stress concentration around the door opening and the dome-wall connection is high. The Drift Ratio of the temple does not exceed the allowable drift value of 0.004.

The vulnerability checklist [6] was completed, revealing that the majority of items assessed showed compliance. Parameters related to lateral force resisting systems such as:- horizontal band, vertical reinforcement at corners, corner stitches, diagonal bracing and lateral restrainers were absent resulting to non-compliance in the checklist.

Based on these findings, it is recommended that targeted strengthening measures be considered, particularly at the identified vulnerable spots, to enhance the temple's resilience against future seismic events. This study underscores the necessity for continuous monitoring and periodic reassessment of the temple's structural health to ensure its preservation and safety.

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