Precautionary Measures Addressing Potential Geological/Geotechnical Failure: A Case Study of the Lord Shiva Statue Construction

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Abstract

This study delves into the geotechnical considerations surrounding the construction of a 108-foot-tall Lord Shiva Statue in Kalamasi, Changunarayan Municipality-8, Bhaktapur, Nepal. Thirteen exploratory drills were undertaken to assess soil and rock composition as well as groundwater levels. Findings revealed a mixture of silty clay, occasional gravel and sand, transitioning into highly weathered sandstone. The absence of engineering oversight in the excavation for the statue's foundation resulted in slope failure and settlement due to misinterpretation of soil data. 2D-ERT (electric resistivity tomography) tests were conducted to identify geological and geomorphological disturbances, unveiling lineaments and fractured rocks near the surface within the foundation site. The borehole log misinterpreted core observations, rendering the evaluation of bearing capacity for a raft foundation insufficient. Consequently, settlement analyses using Phase2 2D finite element analysis were conducted, leading to the proposal of a combined raft and pile foundation to mitigate potential failure by minimizing settlement, while considering wet and seismic conditions.

Keywords: geological/geotechnical failure, Lord Shiva statue, 2D-ERT, Phase2

I. Introduction

The Kalikadevi Kalpeshwordham Temple Project conducted a thorough geotechnical investigation for the proposed Kalpeshwordham temple in Kalamasi, Changunarayan Municipality-8, Bhaktapur, situated along the popular tourist route to Nagarkot. The project aims to erect one of the tallest statues of Lord Shiva, standing at 108 feet, on a gentle slope. The investigation involved soil explorations, laboratory testing, and the formulation of foundation design recommendations. Rotary drilling was conducted around the vicinity of the proposed statue area, resulting in the completion of 13 boreholes and the acquisition of soil/rock properties through in-situ and laboratory tests.

The geotechnical investigation relied on exploratory drilling boreholes and the execution of Standard Penetration Test (SPT) and Dynamic Cone Penetration Test (DCPT) to depths of up to 12.0 meters. Field operations were conducted under the coordination and direct supervision of the Kalpeshwor Project team, while laboratory testing was undertaken by technical staff at a soil laboratory in Kathmandu, in alignment with the project client. This comprehensive geotechnical report presents the findings of the field and laboratory investigations carried out for the proposed 108-foot-tall statue of Lord Shiva at Kalpeshwordham, Kalamasi.The objective of this site investigation is to evaluate the current soil profiles and engineering properties of the subsurface conditions on-site, aiming to furnish the designer with insights into appropriate footing types, foundation depths, and geotechnical design parameters in order to protect the potential geological and geotechnical failures obtained from non-engineered excavation which led to slope failure after exploratory drilling. To ensure a secure design and mitigate potential disasters, 2D Electric Resistivity Tomography (ERT) and settlement analyses were conducted, taking into account seismic and wet conditions. These analyses aimed to offer an appropriate foundation design that minimizes settlement of the terrain.

Borehole	Northing	Easting	Location	Accessibility	Land use	Drill Rig	Drill Depth
BH01	N: $27^0$41' 1.805"	E: 85^0 30' 7.836"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH02	N: $27^0$41' 2.343"	E: 85^0 30' 7.886"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH03	N: $27^0$41' 2.074"	E: 85^0 30' 7.915"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH04	N: $27^0$41' 2.265"	E: 85^0 30' 8.283"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH05	N: $27^0$41' 2.714"	E: 85^0 30' 8.300"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH06	N: $27^0$41' 2.813"	E: 85^0 30' 8.106"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH07	N: $27^0$41' 3.192"	E: 85^0 30' 7.975"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH08	N: $27^0$41' 2.972"	E: 85^0 30' 7.898"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH09	N: $27^0$41' 2.838"	E: 85^0 30' 7.874"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH10	N: $27^0$41' 2.578"	E: 85^0 30' 7.802"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH11	N: $27^0$41' 2.484"	E: 85^0 30' 7.714"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH12	N: $27^0$41' 2.368"	E: 85^0 30' 7.690"	Kalpeshwordham	Good	Grass Land	[redacted]	12 m
BH13	N: $27^0$41' 2.361"	E: 85^0 30' 7.886"	Kalpeshwordham	Good	Forest	[redacted]	12 m

Fig. 1. Location of study area and borehole coordinates.

II. Materials and Methods

The materials and methods encompassed various stages to ensure comprehensive site evaluation and foundation design. A desk study was initially conducted, reviewing existing literature and data pertinent to the project. This was followed by on-site investigations involving exploratory techniques such as core drilling and Standard Penetration Test (SPT) and Dynamic Cone Penetration Testing (DCPT) to gather soil samples and assess subsurface conditions. Additionally, 2D Electric Resistivity Tomography (ERT) was employed to provide insight into the site's geological composition.

Soil samples obtained from drilling were subjected to laboratory testing for detailed analysis. Natural moisture content, consistency limits, particle size analysis, specific gravity, bulk density and direct shear test based on IS code were carried out for the determination of index, physical and strength parameters of the soil layers. The evaluation of soil bearing capacity was carried out using Standard Penetration Test (SPT) N-values (Skempton, 1986, Liao and Whitman, 1987). The Standard Penetration Test (SPT) was performed for each borehole at an interval of 1.5 m and soil samples were carried out. A split spoon sampler, 50 mm in diameter was driven by the blows of a standard hammer weighing 63.5 kg and falling freely from a height of 760 mm. The number of blows required to give a tube penetration of 300 mm was taken as SPT N-value of the soil tested at a specified borehole depth. The SPT were carried out as per ASTM standards. The Dynam ic Cone Penetration Test (DCPT) was performed for each borehole in the gravelly and clayey layers. A cone having an apex angle 60° was attached to the drill rods and was driven into the soil by the blows of a standard hammer weighing 63.5 kg and falling freely from a height of 760 mm. The number of blows required to give a tube penetration for every 100 mm was continuously measured up to the depth till refused. The number of blows required for 300 mm penetration is noted as the dynamic cone resistance, Ncd and continuously recorded with depth. The DCPT was carried out using 50 mm diameter cone without bentonite slurry in accordance with IS: 4968, 1976 Part II. Furthermore, the computation of pile capacity was conducted to ensure the stability and safety of the proposed foundation design. These methodologies collectively facilitated the development of an informed and robust foundation design strategy aimed at minimizing settlement and mitigating potential risks.

Two ERT profiles have been crossed each other at the center of the profile line (Fig. 2). Wenner-Schlumberger array method was applied in ERT at the electrode spacing of 5.0 m with 60 number of electrodes covering the length of 295.0 m. The Wenner-Schlumberger array is one of the important arrays used in 2D imaging technique. It is a combination of the Wenner and Schlumberger arrays (Pazdirek and Blaha, 1966). The advantage of this array is that it can be used with the electrodes system arranged with a constant spacing, hence it is easier than Schlumberger array. The purpose of using this array is that they are moderately sensitive to both horizontal and vertical structures (Loke, 2012: 172).

Fig. 2. Location of two ERT profiles (ERT-1 and ERT-2) at Kalpeshwordham.

III. Results

1. In-Situ Testing Results

The vertical rotary wash boring was carried out using the rotary type rig up to the depth of 12 m for all thirteen boreholes. The drilling was performed on the flat surface on the grass land along the downslope of Kapileshwordham temple at Kalamasi, Changunarauan Municipality-8, Bhak tapur.

Generally, soil strata are represented by occasional gravel together with clayey silt and silty clay with presence of sand layer up to 6.0 m. Below this depth grey coloured to dark grey coloured, medium-grained, highly weathered micaceous sandstone followed by fractured and fresh micaceous sandstone when the rotary drilling were conducted up to 12.0 m. The same rock type of Kulikhani Formation is represented by all borehole logs at depths varying between 6.5 m to 7.0 m respectively (Fig. 3).

The corrected SPT N-values are ranged from 9.90 to 20.90 for BH13 and 22 for BH04 as the maximum values, and their corresponding allowable bearing capacity are ranged from 40.69 kPa to 76.69 kPa for BH13 and 90.23 kPa for BH04 using Meyerhof (1956). The estimated allowable bearing capacity has been measured at the depth of 1.5 m to 4.5 m which is quite lower and increased gradually up to 4.5 m for predicted foundation width of from 0.5 m.

Although the bearing capacities are increased below the depth of 4.5 m, the foundation for 108 feet tall Lord Shiva statue is not adequate up to the depth of 6.0 m on account of execution of receiving lower field SPT values along with presence of silty clay to clayey silt soil layers together with the presence of occasional gravels and considerable sands.

The DCPT value below 4.5 m is suddenly increased due to presence of stiff completely weathered rock, the raft foundation at this layer would be not sufficient due to the proposed heavy structure owing to presence of weak micaceous minerals present in rock strata.

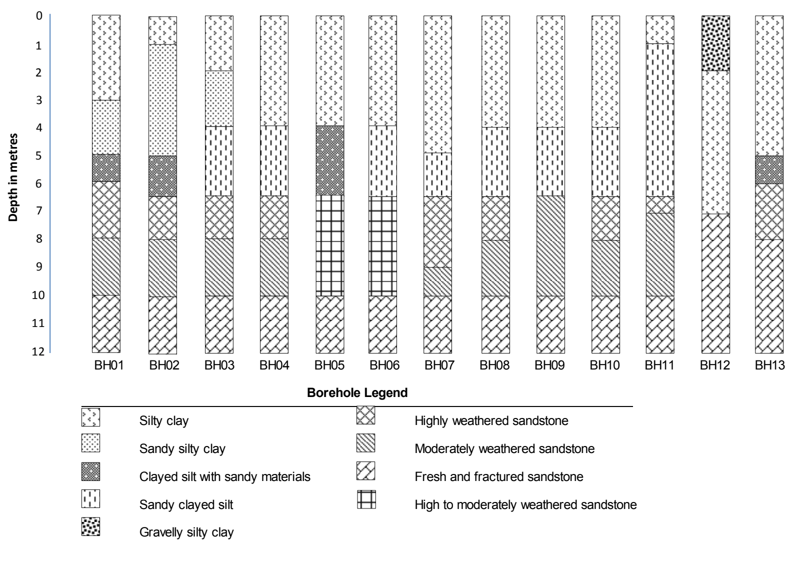

Fig. 3. Results of borehole logs at Kalpeshwordham, 108 feet Lord Shiva statue project premises.

2. Laboratory Testing Results

Laboratory tests samples taken from SPT adjacent to borehole locations revealed medium natural moisture contents. The fine-grained soils pass through Sieve No. 200 are higher in percentage ranged between more than 50% for all SPT-1 and SPT-2 soils of all thirteen boreholes. the maximum percent of clay present in BH12 (at 1.5 m depth) is 39.97% and the minimum clay of 6.68% is present in BH10 at the depth of 4.5 m respectively. Similarly, the maximum silt is governed by BH01 at the depth of 6.0 m as 67.88% while the minimum silt of 21.9% is governed by BH09 at the depth of 3.0 m. On the other hand, the difference of presence of maximum and minimum sand are not varied much as in fines. For example, the maximum sand of 39.9% is present in BH09 at the depth of 3.0 m while the minimum sand of 19.13% is present at BH12 at the depth of 6.0 m. The range of gravel is ranged from 0% extracted from some boreholes up to 18.17% as the maximum recorded value in BH10 at the depth of 3.0 m (Tables 1 and 2).

The consistency limits for all boreholes from their corresponding SPT samples reveal higher liquid limits and higher plasticity index governing natural moisture content/liquid limit (NMC/LL) > 0.85 showing non-liquefiable ground.

Table 1. Summary of geotechnical properties of soil taken from SPT sample (BH01-BH07).

Borehole No.	Testing Depth (m)	NMC %	Grain Size Analysis/Hydrometer				Atterberg Limits			Loose Bulk Density gm/cc	Specific Gravity
			Gravel %	Sand %	Silt %	Clay %	LL, %	PL, %	PI, %		
BH01	1.50	27.8	13.66	25.11	24.81	36.42	42.00	10.08	31.92	1.593	2.658
	3.00	17.6	19.66	29.11	20.00	31.23	35.05	5.57	29.48	1.610	2.651
	4.50	12.02	2.77	23.77	67.59	5.87					2.665
	6.00	15.25	4.44	19.83	68.88	6.85					2.672
BH02	1.50	26.24	12.33	25.42	26.03	36.23	42.50	8.44	34.06	1.590	2.649
	3.00	15.28	18.77	28.35	21.94	30.94	39.60	7.56	32.04	1.608	2.661
	4.50	12.44	4.56	23.87	64.12	7.45					2.660
	6.00	14.61	6.56	20.37	64.53	8.54					2.671
BH03	1.50	27.34	11.19	25.43	26.30	37.08	42.00	7.38	34.62	1.590	2.658
	3.00	17.75	16.23	28.91	24.85	30.01	35.05	5.57	29.48	1.603	2.653
	4.50	11.53	1.67	24.35	64.21	9.77					2.665
	6.00	14.37	7.56	21.89	60.18	10.37					2.671
BH04	1.50	26.59	10.79	28.80	22.94	37.47	39.70	4.93	34.77	1.590	2.658
	3.00	17.73	12.21	32.41	24.47	30.91	36.27	4.63	31.64	1.604	2.653
	4.50	11.79	3.96	22.73	64.42	8.89					2.665
	6.00	14.51	8.89	19.17	61.60	10.34					2.671
BH05	1.50	27.18	10.00	25.54	28.23	36.23	40.22	2.63	37.59	1.602	2.656
	3.00	16.77	19.66	29.11	20.00	31.23	33.15	2.86	30.29	1.594	2.665
	4.50	11.88	2.77	23.77	67.59	5.87					2.664
	6.00	14.32	4.44	19.83	68.88	6.85					2.673
BH06	1.50	23.92	0.00	30.05	35.72	34.23	41.95	5.05	36.90	1.607	2.657
	3.00	17.28	0.00	34.48	35.07	30.45	35.28	3.34	31.94	1.604	2.651
	4.50	11.48	0.00	26.54	67.59	5.87					2.664
	6.00	13.75	0.00	24.27	68.88	6.85					2.677
BH07	1.50	25.25	0.00	31.11	31.17	37.72	41.75	4.75	37.00	1.582	2.655
	3.00	16.57	0.00	32.79	31.65	35.56	34.90	3.69	31.21	1.601	2.650
	4.50	11.77	3.45	24.31	63.01	9.23					2.663
	6.00	13.22	5.50	20.03	64.11	10.35					2.667

Table 2. Summary of geotechnical properties of soil taken from SPT sample (BH08-BH13).

Borehole No.	Testing Depth (m)	NMC %	Grain Size Analysis/Hydrometer				Atterberg Limits			Loose Bulk Density gm/cc	Specific Gravity
			Gravel %	Sand %	Silt %	Clay %	LL, %	PL, %	PI, %		
BH08	1.50	26.87	8.66	30.07	25.06	36.21	39.70	4.93	34.77	1.589	2.664
	3.00	16.03	9.44	31.69	28.86	30.01	35.15	5.08	30.07	1.602	2.663
	4.50	11.51	4.45	23.34	62.98	9.23					2.667
	6.00	15.52	0.00	24.44	63.30	12.26					2.686
BH09	1.50	26.69	0.00	35.55	29.47	34.98	42.50	4.34	38.16	1.595	2.659
	3.00	16.13	0.00	39.92	27.10	32.98	31.83	0.00	31.83	1.601	2.664
	4.50	13.38	0.00	27.74	63.72	8.54					2.664
	6.00	14.21	0.00	24.49	63.08	12.43					2.662
BH10	1.50	26.87	8.88	25.54	30.26	35.32	41.50	0.00	41.50	1.592	2.656
	3.00	16.22	18.77	28.99	21.93	30.31	34.10	0.00	34.10	1.615	2.654
	4.50	11.64	1.77	23.77	67.78	6.68					2.678
	6.00	14.08	2.55	20.77	67.80	8.88					2.673
BH11	1.50	26.86	0.00	24.77	36.69	38.54	41.75	4.56	37.19	1.602	2.660
	3.00	16.81	0.00	31.02	30.11	38.87	43.00	4.93	38.07	1.601	2.654
	4.50	11.51	0.00	23.65	64.01	12.34					2.668
	6.00	14.02	0.00	20.73	65.71	13.56					2.669
BH12	1.50	27.06	3.13	19.98	36.92	39.97	41.50	0.00	41.50	1.599	2.658
	3.00	16.3	1.12	28.84	34.48	35.56	34.90	0.00	34.90	1.608	2.662
	4.50	11.17	1.55	19.62	64.40	14.43					2.665
	6.00	14.02	2.00	19.13	63.45	15.42					2.673
BH13	1.50	27.12	9.98	28.83	25.12	36.07	42.09	0.00	42.09	1.594	2.658
	3.00	16.84	11.00	31.93	24.80	32.27	35.00	4.15	30.85	1.615	2.656
	4.50	11.53	4.44	23.29	62.28	9.99					2.665
	6.00	14.03	7.77	20.00	59.97	12.26					2.672

2. 2D ERT Results

Figure 4 shows the tomogram of Wenner-Schlumberger array of the first ERT profile (see Fig. 1). Here, high range of resistivity are present in the ERT tomogram. The top layer which has resistivity of 150 t0 1600 Ohm-m indicates dry to moist silt sandy soil. The thickness of the top layer varies < 1 m to 12 m. The left portion of the ERT tomogram consists thick soil than right portion of the ERT tomogram. Below this top layer soil, there is presence of low resistivity to high resistivity zone which represents weathered to hard rock. The low resistivity < 200 Ohm-m present from chainage 0+000 to 0+150 below the top layer soil consists of weak/shear/fault zone. The rock might be highly weathered or highly fractured and in saturated condition. The weak zone again observed at chainage 0+180 to 0+190. The mid portion of the ERT tomogram shows high resistivity than its neighbouring resistivity zone. At chainage 0+160, below the thin top layer it consists of weathered rock (resistivity 800 to 1600 Ohm-m) up to the depth of 10m, fractured rock (resistivity 1600 to 3200 Ohm-m) to the depth of 25m, again weathered rock encountered to the depth of 65 m and at last very high resistivity zone (> 6400 Ohm-m) is present.

The location of ERT of second profile (ERT-2) is shown in Fig. 5. Here the top layer of ERT tomogram consists of resistivity varying from 100 to 1600 Ohm-m representing dry to moist silt to sandy soil and residual soil. The thickness of this top layer silt sandy soil is about 10 m. Below this layer there is presence of low to high resistivity zone showing weak zone to fractured rock. From chainage 0+025 to 0+150 there is low resistivity (< 200 Ohm-m) indicating weak/shear/fault zone. Other weak zones are present in chainage 0+165 to 0+185 and chainage 0+250 to 0+260. Here series of weak zones are present along the ERT profile. Fractured rock is present in chainage 0+160, and 0+185 to 0+250. The columnar lithology reveals that from top to bottom it co nsists of about 10.0 m of weathered rock, 20.0 m of fractured rock and rest are weathered rock. Usually, it comprises weathered to fractured rock at the mid portion of the ERT profile.

IV. Identification of Problems

The current site condition reveals that the vertical excavation to a depth of 7.0 meters to meet the plinth level for the Lord Shiva statue foundation was conducted without proper consultation with geo-engineers and structural engineers, leading to immediate slope failures. The designed plinth level, determined by structural analyses, was not adequately supported during excavation, causing the sides of the excavation to collapse. Figures 6 and 7 illustrate the aftermath of this non-engineered vertical box cut, highlighting significant cracks, wall collapses, and displaced materials. These failures underscore the necessity of adhering to standard excavation practices and involving geo-engineering expertise to prevent such issues.

Figure 6 shows the displaced material indicated by an arrow, resulting from the non-engineered excavation at the site. Numerous tension cracks have appeared on the top of the slope adjacent to the box cut, which seem to be progressing. Figure 7 depicts the wide and deep tension cracks

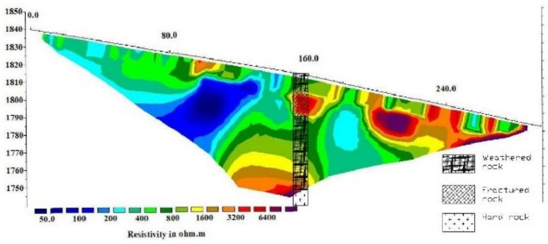

Fig. 4. Columnar litho log extracted from ERT-1 representing weathered rock to fractured rock at most part and hard rock at the bottom

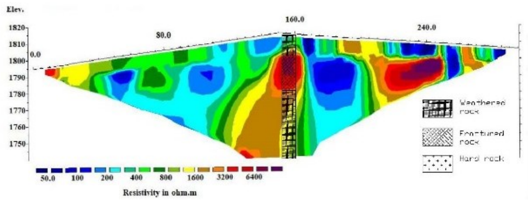

Fig. 5. Columnar litho log extracted from ERT-2 representing weathered to fractured rock at most of the part.

visible at the ground surface, indicating significant instability in the earthworks. These unstable conditions necessitate careful consideration and engineering to ensure the stability of the statue's foundation. Proper geotechnical measures and adherence to standard excavation practices are crucial to address these issues and safely support the superstructure of the statue.

Additionally, the vertical cut exposed highly fractured rocks on the wall surface, contradicting the results of the exploratory core drilling conducted at 13 sites. Figure 8 illustrates the s-type micro fold starting from the ground surface in the box cut, and Fig. 9 shows geomorphological disturbances, marked by yellow lines, in a Google Earth photo, which align with the ERT results shown in Figs. 4 and 5. This discrepancy suggests potential inaccuracies in the drilling results, possibly due to improper core collection and inadequate initial geological studies. Consequently, these issues complicate the foundation design process based on borehole samples, SPT, and DCPT data. Therefore, a careful and thorough observation is essential for designing the foundation, incorporating all available results to ensure accuracy and stability.

V. Precautionary Measures

Considering the geomorphological distribution, geological sub-surface strata, borehole log, ERT log, and visual inspection of the vertical cut, it is evident that the site for constructing the 108-foot tall Lord Shiva statue is situated in a locally disturbed area. While the geotechnical investigation and bearing capacity evaluation based on SPT (Table 3) suggest that a raft foundation could be designed, the presence of weathered to fractured rock and possible lineaments, as indicated by geomorphological observations and S-type microfolds (Fig. 8), raises concerns about settlement issues. These factors underscore the need for detailed analysis and modeling. Therefore, it is crucial to consider a combined raft and pile foundation to minimize vertical and horizontal settlement and prevent potential foundation failure due to weak layers.

The dimensions of the raft foundation are 18.40 m in length, 15.815 m in breadth, and 0.60 m in thickness, with the water level near the ground surface, simulating field conditions for Phase 2 FEM analysis. A uniformly distributed load (UDL) of 120 kN/m² is applied to the raft to represent the structural load. Observations indicate that the settlement value from plastic calculations was higher than from consolidation settlement, suggesting that the quick rate of loading results in greater settlement than a slow rate of loading. For design purposes, particularly for structures intended for long-term use, the consolidation settlement should be considered. Additionally, the loading effect is more pronounced directly below the raft and diminishes with depth.

Fig. 6. Slope failure of the box cut with the presence of clear demarcation of displaced mass indicated by an arrow.

Fig. 7. Wide tension crack appeared on the top of the box cut.

Fig. 8. S-type microfold marked by red line present on the vertical cut.

Fig. 9. Straight lineament/local fault line shown by yellow lines.

Table 3. Summary of corrected SPT/DCPT N-value (BH01 - BH13) along the proposed area.

Borehole No.	Depth, m	SPT Blow Number	Corrected SPT, N-60	Borehole No.	Depth, m	SPT Blow Number	Corrected SPT, N-60
		N1				N1	
BH01	1.5	10	11	**BH08**	1.00	13	14.3
	3.00	15	16.5		3.00	14	15.4
	4.50	19	20.9		4.50	16	17.6
BH02	1.50	11	13.2	**BH09**	1.00	14	15.4
	3.00	14	15.4		3.00	15	16.5
	4.50	18	19.8		4.50	15	16.5
	6.00	**3**		**BH10**	1.00	13	14.3
BH03	1.50	12	13.2		5.00	17	18.7
	3.00	16	17.6		3.00	17	18.7
	4.50	18	19.8	**BH11**	1.50	14	15.4
BH04	1.50	14	15.4		3.00	16	17.6
	3.00	16	17.6		4.50	16	17.6
	4.50	20	22	**BH12**	1.50	15	16.5
BH05	1.50	15	16.5		3.00	15	16.5
	3.00	15	16.5		4.50	18	19.8
	4.50	17	18.7	**BH13**	1.50	9	9.9
BH06	1.50	13	14.3		3.00	14	15.4
	3.00	16	17.6		4.50	19	20.9
	4.50	17	18.7				
BH07	1.50	15	16.5				
	3.00	16	17.6				
	4.50	17	18.7				

The dimensions of the raft foundation are 18.40 m in length, 15.815 m in breadth, and 0.60 m in thickness, with the water level near the ground surface, simulating field conditions for Phase 2 FEM analysis. A uniformly distributed load (UDL) of 120 kN/m² is applied to the raft to repre sent the structural load. Observations indicate that the settlement value from plastic calculations w as higher than from consolidation settlement, suggesting that the quick rate of loading results in g reater settlement than a slow rate of loading. For design purposes, particularly for structures inten ded for long-term use, the consolidation settlement should be considered. Additionally, the loading effect is more pronounced directly below the raft and diminishes with depth.

The excavated site condition reveals that the plinth level is at a depth of 7.0 m, where a 60.0 cm thick raft foundation has been proposed (adopted from Structural Analysis Report). Settlement analysis for the proposed raft foundation, conducted, considered wet conditions with the groundwater table as indicated by the borehole log, as well as seismic conditions. Table 3 shows that the total vertical settlement of the ground can reach up to 55.0 mm, which is at the critical limit of tolerance but still permissible. This analysis underscores the importance of accounting for both groundwater and seismic factors in the design to ensure the stability and safety of the foundation.

Proceeding with trials and iterations, piles were embedded below the raft and analyzed under actual wet conditions and seismicity. The numerical modeling involved eight piles of 700 mm diameter placed at the corners and middle of the raft, and one 850 mm diameter pile at the raft's center, where the statue's center of gravity lies. The depths chosen were 12.0 m for the 700 mm piles and 16.0 m for the 850 mm pile. Table 3 and Fig. 10 represent that the total vertical settlement of the ground decreased significantly to 20.0 mm, compared to the settlement with the raft foundation alone. In this paper, the remaining case studies with different loading and conditions are not presented and all results were tabulated in Table 3.

Table 4. Results of Total Settlement applied for different loading.

S. N.	Load Cases	Total Settlement, mm
1.	Model Analysis for natural state in Dry Condition	0.00
2.	Model Analysis for natural state in Wet Condition (with Probable Water Table)	35.00
3.	Model Analysis for natural state in Wet Condition & Seismicity	50.00
4.	Model Analysis for excavation in Wet Condition & Seismicity	30.00
5.	Model Analysis for excavation with Raft Foundation in Wet Condition & Seismicity	55.00
6.	Model Analysis for excavation with Raft & Pile Foundation in Wet Condition & Seismicity	20.00

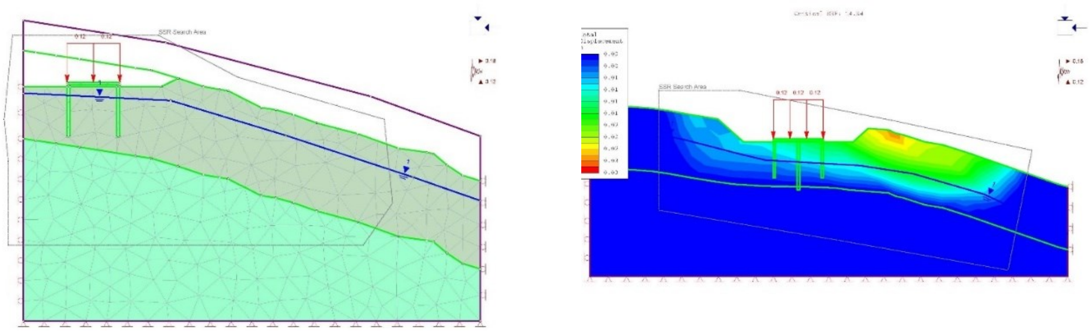

Fig. 10. Phase 2 analysis results for excavated ground with raft & pile foundation considering actual site condition of groundwater level and seismicity.

IV. Conclusions

The geotechnical investigation, which included drilling 13 rotary boreholes to a depth of 12.0 meters, confirmed consistent subsurface geology around the site for the proposed 108 feet tall Lor d Shiva statue. The findings showed similar soil types and rock layers at comparable depths, with clayey silt, silty clay, and sand up to 6.0 meters, followed by highly weathered to fractured micaceous sandstone. The non-engineered box cut of the area contradicted the interpretation of borehole log (soil formation) with presence of highly fractured rocks with the presence of S-type microfold advanced towards the ground surface. Furthermore, despite the sufficient bearing capacity, the presence of weak, weathered rock necessitates a combined raft and pile foundation to mitigate settlement issues and prevent foundation failure. The site is not susceptible to liquefaction, and the proposed combined foundation is designed to address the complex geological conditions, ensuring stability and safety for the statue.

Laboratory tests and Phase2 2-Dimensional finite element analysis further supported the need for a combined foundation. While a raft foundation alone could result in settlements up to 55.0 mm, considered marginally critical, the integration of raft and pile foundation significantly reduces this to 20.0 mm. This solution incorporates 9 piles, strategically placed and sized, to distribute the load effectively and provide a safer foundation. These findings emphasize the importance of detailed geotechnical analysis and tailored foundation design for large structures in complex geological settings.

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