Ground Improvement of a Liquefiable Soil by Granular Piles

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Abstract. Liquefaction is a physical process by which soil sediments below the ground water table temporarily lose strength and stiffness, and initiate to behave as a viscous liquid rather than a solid. In addition to earthquake and rapid application of large loads, a soil may liquefy due to construction activities like blasting, and during ground improvement by vibro-flotation and dynamic compaction. Liquefaction phenomenon may cause unrecoverable damage to a building and other civil structures. Hence, it is very important to know about the liquefaction potential of a construction site so that suitable protection measures can be adopted before construction. Granular piles are one of the popular treatment methods to make the soil less prone to liquefaction. In this study, liquefaction potential of a project site at Darbhanga, Bihar has been evaluated (by method developed by Youd et al., 2001) from the borehole data where ground water table is at a shallow depth. Once the liquefiable depths in sub-strata are identified, granular piles along with the shallow foundation have been selected based on the soil characteristics determined. The design of granular piles consists of their diameter, total length, number of piles and their arrangement at site. Use of granular piles will not only improve soil strength significantly but also will provide drainage of water at high pore-water pressure which may be generated during an earthquake event. To present the benefit of granular piles over conventional RCC piles, a comparative design and cost assessment of granular piles with RCC piles were also performed. It has been observed that a significant reduction of construction cost and settlement control may be achieved by granular piles over RCC piles. This kind of study will help in selecting an appropriate liquefaction measure and its design, leading to safer construction of the structure.

Keywords: Liquefaction· Bearing Pressure· Ground Improvement· Granular Piles.

1 Introduction

1.1 The Liquefaction Phenomenon

Soil liquefaction, which is usually known as sudden loss of shear strength in soil due to ground shaking followed by a rapid increase in pore water pressure, generally occurs in loose to very loose saturated granular soils. The ground shaking, predominantly due to cyclic earthquake motions, quickly causes dislodgement of the grain to grain contact of the individual soil grains. This cyclic disturbance causes a substantial loss in shear strength of soil which could result in instability or bearing capacity failures.

Liquefaction may cause any one or a combination of more than one of the following hazards at a vulnerable site: (i) lateral spreading, (ii) flow failures, (iii) loss of bearing strength and increase in settlement of the super-structure, (iv) increased lateral pressure on retaining walls, and (v) ground oscillation which may alter ground motions in terms of amplitude, frequency content and duration. All or any of these may lead to ground failure and a subsequent failure of the super-structure. Liquefaction causes decrease or loss in vertical pile load capacity from both skin and end bearing resistances, depending upon the zone of liquefaction along the pile depth. It also reduces the lateral load carrying capacity of piles. Some of the key recent studies on liquefaction may be found in the literature [1-4], where some researchers (Amini and Qi, 2000 [1]) conducted comprehensive experimental studies by stress-controlled undrained cyclic tri-axial tests to compare the behavior of stratified and homogeneous silty sands during seismic liquefaction conditions for various silt contents and confining pressures in the range of typical field conditions. The homogeneous and heterogeneous soils are important factors for dynamic liquefaction mechanism which is explained perfectly by Chakrabortty and Popescu, 2012 [2]. Owen and Moretti (2011) [3] suggested that liquefaction develops most readily in loosely packed coarse silt to fine sand that is saturated with groundwater and at shallow depths. However, several new liquefaction phenomena have been observed in connection to twenty-first-century earthquakes, for example liquefaction in areas of moderate seismic intensity, liquefaction of gravelly soils, liquefaction of deep-level sandy soils, re-liquefaction in aftershocks, and liquid-like behavior of unsaturated sandy soils (Huang and Miao, 2013 [4]).

Hence, it is very important to know about the liquefaction potential of a construction site so that suitable protection measures can be adopted before construction. Granular piles are one of the popular treatment methods to make the soil less prone to liquefaction. In this study, liquefaction potential of a project site at Darbhanga, Bihar has been evaluated (by method developed by Youd et al., 2001 [5]) from the borehole data where ground water table is at a shallow depth. Once the liquefiable depths in sub-strata are identified, granular piles along with the shallow foundation have been selected based on the soil characteristics determined. To present the benefit of granular piles over conventional RCC piles, a comparative design and cost assessment of granular piles with RCC piles were also performed. It has been observed that a significant reduction of construction cost and settlement control may be achieved by granular piles over RCC piles.

1.2 Evaluation of Liquefaction Potential

Evaluation of soil liquefaction potential and its consequent hazard require an engineering skill with good judgment from past experiences besides testing and analysis.

Significant developments have been accomplished in the past few decades in evolving tools to help evaluating the soil liquefaction potential, however still some characteristics of the liquefaction problem continue to remain ambiguous. An extensive variation of approaches from researchers and experts have been in use to perform the soil liquefaction analysis. The “current standard-of-practice” for evaluating soil liquefaction potential during earthquakes can be found in the paper titled “Liquefaction Resistance of Soils: Summary of Report from the 1996 National Center for Earthquake Engineering Research (NCEER) and 1998 NCEER/National Science Foundation (NSF) Workshops on Evaluation of Liquefaction Resistance of Soils” (Youd et al., 2001) [5]. The SPT-based and CPT-based liquefaction analysis procedures summarized in their paper will hereafter be referred to as the Youd et al. (2001) [5] procedures. Lately, Youd et al. (2001) [5] approach has been examined and liquefaction susceptibility assessment of silts and clays from the CPT-based correlation has been under scrutiny, mostly because of (1) increased volume of field based records, (2) enhanced assessment of peak ground accelerations at sites, and (3) better understanding of the liquefaction behavior of silts and clays [6].

2 Site Description

For the present paper, a liquefaction susceptible site located in Darbhanga, Bihar has been studied where an electrical powerhouse or sub-station is going to be constructed. Before construction, geotechnical investigations were carried out. At two borehole locations in the site, standard penetration tests (SPTs) were carried out, whereas, triaxial and direct shear tests were performed at laboratory for determination of the shear strength properties. For soil classifications, grain size distribution analysis, liquid limit and plastic limit values were also determined. The soil properties or characteristics are described below for the two boreholes:

Table 1. Soil characteristics from borehole 1 (BH-1) location.

N-VALUE	DEPTH METERS	SAMPLE NO	SOIL DESCRIPTION	I.S. CLASSIFICATION	GRAIN SIZE ANALYSIS				LIQUID LIMIT W_L	PLASTIC LIMIT W_P	DRY DENSITY gm/cc	WATER CONTENT %	SHEAR PARAMETER		
					GRAVEL %	SAND %	SILT %	CLAY %					TEST METHOD	C kg/cm^2	Ø DEGREE
	G.L.	DS													
4	1.50	SPT	FILLED UP SOIL												
	2.00	UDS	CLAY OF MEDIUM PLASTICITY	CI	0.0	14.0	64.0	22.0	45.0	26.0	1.47	18.90	TST	0.45	4^0
6	3.00	SPT													
	4.00	UDS	LOOSE CLAY, SAMPLE NOT COLLECTED												
10	4.50	SPT													
	5.00	UDS	CLAY OF MEDIUM PLASTICITY	CI	0.0	16.0	64.0	20.0	40.0	25.0	1.49	20.30	TST	0.50	4^0
7	6.00	SPT													
	7.00	UDS	SILTY SAND	SM											
9	7.50	SPT	SILTY SAND	SM	0.0	72.0	28.0	0.0	N	P	1.50	20.80	DST	0.0	30^0
	8.00	UDS	SILTY SAND												
15	9.00	SPT													
	10.00	UDS	SILTY SAND												
11	10.50	SPT	SILTY SAND	SM	0.0	80.0	20.0	0.0	N	P					
	11.00	UDS	SILTY SAND	SM											
12	12.00	SPT													
	13.00	UDS													
15	13.50	SPT	SILTY SAND	SM	0.0	86.0	14.0	0.0	N	P					
	14.00	UDS	SILTY SAND	SM	0.0	68.0	32.0	0.0	N	P					
16	15.00 / 15.45	SPT													

Table 2. Soil characteristics from borehole 2 (BH-2) location.

N-VALUE	DEPTH METERS	SAMPLE NO	SOIL DESCRIPTION	I.S. CLASSIFICATION	GRAIN SIZE ANALYSIS				LIQUID LIMIT W_L	PLASTIC LIMIT W_P	DRY DENSITY gm/cc	WATER CONTENT %	SHEAR PARAMETER		
					GRAVEL	SAND %	SILT %	CLAY %					TEST METHOD	C kg/cm^2	Ø DEGREE
	G.L.	DS													
	1.50	SPT													
6	2.00	UDS	CLAY OF MEDIUM PLASTICITY	CI	0.0	15.0	65.0	20.0	42.0	25.0	1.48	18.50	TST	0.50	3^0
	3.00	SPT													
5	4.00	UDS	LOOSE CLAY, SAMPLE NOT COLLECTED												
	4.50	SPT	CLAY OF MEDIUM PLASTICITY	CI	0.0	17.0	61.0	22.0	43.0	26.0					
8	5.00	UDS													
	6.00	SPT													
8	7.00	UDS	CLAY OF MEDIUM PLASTICITY	CI	3.0	20.0	57.0	20.0	40.0	25.0	1.56	20.40	TST	0.65	3^0
	7.50	SPT													
12	8.00	UDS	CLAY OF LOW PLASTICITY												
	9.00	SPT													
20	10.00	UDS	CLAY OF LOW PLASTICITY WITH GRAVELS	CL	10.0	24.0	53.0	13.0	34.0	23.0					
	10.50	SPT													
13	11.00	UDS													
	12.00	SPT	CLAY OF LOW PLASTICITY WITH GRAVELS	CL	13.0	26.0	49.0	12.0	33.0	22.0					
11	13.00	UDS													
	13.50	SPT													
16	14.00	UDS													
	15.00	SPT	CLAY OF LOW PLASTICITY	CL	17.0	30.0	44.0	9.0	28.0	20.0	1.64	22.10	TST	0.50	5^0
16	/ 15.45														
	15.45														

The ground water table depths at borehole 1 and 2 locations have been observed at 4.0 m and 1.5 m below the existing ground levels, respectively.

3 SPT N-Value Based Liquefaction Potential Analysis

As per [7], the site at Darbhanga is located in the highest risk prone earthquake zone V of the country and is suspected to have liquefaction potential. Hence, it is of utmost importance to evaluate the liquefaction potential of it prior to any construction. Also, if the design of foundations need liquefaction hazard mitigation measures to provide safety against liquefaction, then that component is also required to be incorporated in the final design.

In the present study, SPT values of 2 boreholes for Darbhanga site have been determined and the same are utilized for the liquefaction potential analyses as per the method prescribed by [5].

Abbreviations used for the analyses are: γ = Bulk unit weight of soil, σ_{vo} = Total overburden stress, σ'_{vo} = Effective overburden stress, N_m = Measured SPT value at field, C_R = Correction factor for drilling rod length, C_E = Correction factor for hammer energy ratio, C_S = Correction factor for sampling method, C_B = Correction factor for borehole diameter, C_N = Correction factor to normalize Nm to a common reference effective overburden stress, $(N_1)_{60}$ = Corrected standard penetration resistance, r_d = Stress reduction coefficient to account for flexibility in soil profile, CSR = Calculated Cyclic Stress Ratio generated by the earthquake shaking, FC = Fines Content (in percentage) of soil passing through US sieve no. 200, $(N_1)_{60}$ (c.s.) or $N_{1,\ 60,\ cs}$ = Value of $(N_1)_{60}$ adjusted to equivalent clean-sand value, CRR $_{(m\ =\ 7.5)}$ = Cyclic resistance ratio for M_w = 7.5 earthquakes, MSF or K_m = Magnitude Scaling Factor, K_σ = Correction factor for soil layers subjected to large static normal stresses, K_α = Correction factor for soil layers subjected to large static shear stresses, CRR = Cyclic Resistance Ratio, and FoS = Factor of Safety against liquefaction potential. The results for the liquefaction potential analyses for the two boreholes are described below:

Table 3. Liquefaction analysis for Borehole location: 1.

Ground water depth from top = 4.0 m, Maximum Horizontal Acceleration (MHA) assuming Zone V area of IS 1893: 2002 = 0.36 (g), Design Moment Magnitude = 7.5, MSF or K_m factor = 1.0.

Depth (m)	γ (kN/m³)	σ_{vo} (kPa)	σ'_{vo} (kPa)	N_m	C_E	C_B	C_S	C_R	C_N	$(N_1)_{60}$	r_d	CSR	FC	$(N_1)_{60}$ (c.s.)	CRR (m = 7.5)	K_σ	K_α	CRR	FoS	Remarks
1.5	17.15	25.719	25.719	4	0.8	1.1	1.2	1.05	1.70	7.5	0.990	0.232	86	14.0	0.151	1	1	0.151	0.65	Liquefiable
3.0	17.15	51.439	51.439	6	0.8	1.1	1.2	1.05	1.39	9.3	0.979	0.229	86	16.1	0.172	1	1	0.172	0.75	Liquefiable
4.5	17.58	77.815	72.91	10	0.8	1.1	1.2	1.05	1.17	13.0	0.969	0.242	84	20.6	0.223	1	1	0.223	0.92	Liquefiable
6.0	17.78	104.48	84.858	7	0.8	1.1	1.2	1.05	1.09	8.4	0.958	0.276	84	15.1	0.161	1	1	0.161	0.58	Liquefiable
7.5	17.78	131.14	96.807	9	0.8	1.1	1.2	1.05	1.02	10.1	0.943	0.299	28	16.1	0.171	1	1	0.171	0.57	Liquefiable
9.0	17.78	157.81	108.76	15	0.8	1.1	1.2	1.05	0.96	15.9	0.923	0.313	28	22.7	0.253	0.98	1	0.248	0.79	Liquefiable
10.5	17.78	184.47	120.7	11	0.8	1.1	1.2	1.05	0.91	11.1	0.894	0.320	20	15.6	0.166	0.95	1	0.157	0.49	Liquefiable
12.0	17.78	211.13	132.65	12	0.8	1.1	1.2	1.05	0.87	11.6	0.857	0.319	20	16.1	0.171	0.92	1	0.157	0.49	Liquefiable
13.5	17.78	237.8	144.6	15	0.8	1.1	1.2	1.05	0.83	13.8	0.811	0.312	14	16.6	0.177	0.9	1	0.158	0.51	Liquefiable
15.0	17.78	264.46	156.55	16	0.8	1.1	1.2	1.05	0.80	14.2	0.761	0.301	32	21.4	0.234	0.87	1	0.205	0.68	Liquefiable

Table 4. Liquefaction analysis for Borehole location: 2.

Ground water depth from top = 1.5 m, Maximum Horizontal Acceleration (MHA) assuming Zone V area of IS 1893: 2002 = 0.36 (g), Design Moment Magnitude = 7.5, MSF or K_m factor = 1.0

Depth (m)	γ (kN/m³)	σ_{vo} (kPa)	σ'_{vo} (kPa)	N_m	C_E	C_B	C_S	C_R	C_N	$(N_1)_{60}$	r_d	CSR	FC	$(N_1)_{60}$ (c.s.)	CRR (m = 7.5)	K_σ	K_α	CRR	FoS	Remarks
1.5	17.20	25.807	25.807	6	0.8	1.1	1.2	1.05	1.70	11.3	0.990	0.232	85	18.6	0.198	1	1	0.198	0.86	Liquefiable
3.0	17.20	51.614	36.899	5	0.8	1.1	1.2	1.05	1.65	9.1	0.979	0.321	85	16.0	0.170	1	1	0.170	0.53	Liquefiable
4.5	17.20	77.422	47.982	8	0.8	1.1	1.2	1.05	1.44	12.8	0.969	0.366	83	20.4	0.220	1	1	0.220	0.60	Liquefiable
6.0	18.43	105.06	60.915	8	0.8	1.1	1.2	1.05	1.28	11.4	0.958	0.387	83	18.8	0.199	1	1	0.199	0.51	Liquefiable
7.5	18.43	132.7	73.838	12	0.8	1.1	1.2	1.05	1.16	15.5	0.943	0.397	77	23.6	0.266	1	1	0.266	0.67	Liquefiable
9.0	18.43	160.34	86.761	20	0.8	1.1	1.2	1.05	1.07	23.8	0.923	0.399	77	33.6	$(N_1)_{60}$ (c.s.) > 30	1	1	$(N_1)_{60}$ (c.s.) > 30	$(N_1)_{60}$ (c.s.) > 30	Non-Liq.
10.5	18.43	187.97	99.685	13	0.8	1.1	1.2	1.05	1.00	14.4	0.894	0.395	66	22.3	0.247	1	1	0.247	0.62	Liquefiable
12.0	18.43	215.61	112.61	11	0.8	1.1	1.2	1.05	0.94	11.5	0.857	0.384	61	18.8	0.201	0.97	1	0.194	0.50	Liquefiable
13.5	19.64	245.08	127.36	16	0.8	1.1	1.2	1.05	0.89	15.7	0.811	0.365	61	23.9	0.271	0.93	1	0.252	0.69	Liquefiable
15.0	19.64	274.54	142.11	16	0.8	1.1	1.2	1.05	0.84	14.9	0.761	0.344	53	22.9	0.255	0.9	1	0.229	0.67	Liquefiable

From the above analyses, it can be inferred that the entire length of soil up to the depth of exploration in borehole location 1 is liquefiable or susceptible to liquefaction. On the other hand, except the soil strata between the depths 9.0 m to 10.5 m, the entire soil strata up to the depth of exploration in borehole location 2 is liquefiable or susceptible to liquefaction. Hence, there is a need to provide liquefaction resistant foundation design as well as liquefaction measures to minimize its adversity if it occurs during the design life of the structure.

4 Recommendations to Mitigate Liquefaction Hazard

4.1 Approaches for Liquefaction Hazard Mitigation

The basic approach for earthquake disaster mitigation can be broadly classified into two major categories: (i) preventing or minimizing the probability of liquefaction (ground improvement) and (ii) minimization of damages in the event of liquefaction (structural improvement). Among the various techniques or remediation methods against liquefaction, more widely used methods for liquefaction mitigation are: vibro methods (vibro-rod, vibro-compaction, and vibro-replacement), deep dynamic compaction, Compaction grouting, deep soil mixing, jet grouting, and drainage through granular piles or stone columns. All other methods are costly to very costly except the drainage through granular piles method. Also, granular piles are effective in mitigating liquefaction damage due to the reinforcement effect and drainage facility.

Granular Piles (GP) function as drains and permit rapid dissipation of earthquake-induced pore pressures by virtue of their high permeability. The generated pore water pressure due to repeated loading may get dissipated almost as fast as they are generated. In addition, they tend to dilate (through bulging) as they get sheared during an earthquake event. Seismic forces which tend to generate positive pore pressures in these deposits cause an opposite effect of dilation in dense granular piles. One of the chief benefits of ground treatment with granular piles is the densification of in situ ground by which the in situ properties (shear strength) of the ground get enhanced to mitigate the seismic risks, especially, liquefaction potential. In addition to it, granular piles provide increased bearing capacity and significant reduction in settlement, besides achieving cost economy. These benefits of granular piles make them a natural choice for liquefaction hazard minimization.

In order to substantiate the above statement, design calculations have been done to compare cost of construction of granular piles and conventional RCC piles for borehole 2 location of Darbhanga site where the sub-station building is going to be constructed.

4.2 Design of Granular Piles

The sub-station building has a load bearing area of = 28 m X 22 m = 616 sq.m., and the super-structure design load is expected to be of 1850 Ton, which makes the load intensity as 3 T/sq.m.

The diameter of granular piles to be installed at site are assumed to be of 0.3 m for the preliminary design. Hence, cross-sectional area of each granular pile becomes, A_p = 0.07065 sq.m. The passive earth pressure co-efficient, K_p is 2.94 for the stone ballasts. Generally, the critical length of granular pile is 5 times the diameter of pile, which makes it 1.5 m for the present case. Critical length of a granular pile is that length beyond which the piles will not have any significant contribution in minimizing the liquefaction hazards in terms of the additional bearing capacity they provide. From the soil samples collected from the borehole, the unit weight of soil is found to be 18.0 kN/m^3.

The design load carrying capacity of each granular pile is obtained from the empirical equation provided by Hughes and Withers, 1974 [8]:

$$Q_d = K_p * (8\ C_u + O_e + O_v) * A_p \qquad (1)$$

From the above equation, Q_d is obtained as = 10.16 T for each of the granular piles. Now, assuming a FoS of 2.5, the safe bearing capacity of each granular pile, Q_{safe} becomes 4.06 T.

Hence, total no. of granular piles required to be installed = 1850 / 4.06 = 456 no's (approx.).

Finally, the approximate spacing between granular piles, if installed in a zig-zag triangular pattern becomes 1.25 m.

4.3 RCC Pile Design

For comparison with the granular piles, let's take the RCC pile diameter as 0.35 m and length of pile required = 12.0 m (as higher SPT values obtained beyond 9.0 m depth from the ground). Here, undrained cohesion = 50 kN/m^2 (from Table 2).

Hence, load carrying capacity of each pile = $c_{ub} * N_c * A_b + \alpha * c_u * A_s$ = 50*9*(3.14*0.352) + 0.7*50*(3.14*0.35)*12.0 = 504.85 kN = 50.48 T.

Safe bearing capacity (assuming a FoS of 2.5) = 20.19 T.

Hence, total no. of piles req. = 1850 / (0.8 * 20.19) (assuming a pile group efficiency of 80%) = 115.

4.4 Cost Comparison between Granular Piles and RCC Piles:

The cost comparison between the granular and RCC piles designed is shown below.

Table 5. Cost comparison between granular piles and RCC piles.

Type of pile	Length of each pile, m	Total no. of piles req.	Total running length, m	Cost per unit length*, Rs.	Total cost, Rs.
Granular piles	1.5	456	684.0	300.00/-	2,05,200/-
RCC piles	12.0	115	1380.0	740.00/-	10,21,200/-

* Cost of construction per unit length is computed as per IS code recommendations and Delhi Schedule Rate (DSR).

Hence, total savings in cost of construction of granular piles over RCC piles = [(10, 21,200 - 2, 05,200)/10, 21,200]*100 = 79.9 %.

Additionally, other advantages of granular piles over RCC piles are:

- Overall settlement control,

- Liquefaction mitigation through drainage, soil compaction and reinforcement,
- Use of natural materials like stone ballasts over steel and concrete, thus reducing carbon foot print,
- Ease of construction at site, does not require skilled labours and heavy machineries like rigs/ cranes.

5 Conclusions

In the present study, a liquefaction susceptible site is analyzed and liquefaction potential of the same has been evaluated. Accordingly, both granular piles and conventional RCC piles have been designed for the design load of a sub-station building. The cost comparison between the two shows that granular piles can be a better alternative for the studied case.

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