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ABSTRACT

A rail-cum-road bridge with approach embankments is to be constructed on the Brahmputra river near Bogobil in Assam (India). The project site is located in Brahmputra's alluvial plain and lies in an area of very high seismic hazard (zone V of Indian seismic zone map as per IS:1893-1984) and traverse through the liquefiable ground. A two-level earthquake design criterion has been used: safety-evaluation ground motion of PGA of 0.6g, and a functional-evaluation ground motion of 0.1g. The liquefaction analysis has been carried using a "simplified procedure" originally developed by Seed and Idriss and progressively revised, extended and refined by others.

The soil stratigraphy beneath the embankment consists of about 10m of recent alluvium of very loose to medium dense sand and silty soil overlying up to about 30m of medium to very dense deposits of sand and silty sand. Below this layer a dense to very dense sandy layer is encountered.

The liquefaction assessment has been carried out for maximum embankment height of 21m which is to be constructed on the existing ground surface. The analysis indicated that the liquefaction is not likely to occur for functional evaluation motion. Soil strata under the embankments are liable to liquefy due to ground motion with PGA of 0.6g up to depths of about 14-18m with a total soil settlement of up to about 260 mm. Embankment stability has also been considered when the soil strata underlying the embankment undergo liquefaction. For this analysis, liquefiable soil layers have been assigned only the residual strength. For static case (i.e., no earthquake), the factor of safety ranges from 1.18 to 1.39.

The bridge and abutments are founded on well foundations seated at about 55m below the river bed, conforming to minimum scour depth and grip length requirements. The average and normalized N-values at the bridge foundation levels are 86 and 39, respectively and, therefore, the bridge foundations are not susceptible to liquefaction.

1. INTRODUCTION

A rail-cum-road bridge with approach embankments that traverse liquefiable ground is to be constructed on the Brahmputra river near Bogobil in Assam (India). The project site is located in Brahmputra's alluvial plain and lies in an area of very high seismic hazard (zone V of Indian seismic zone map as per IS:1893-1984).

The soil stratigraphy beneath the embankment consists of about 10m of recent alluvium of very loose to medium dense sand and silty soil overlying on about 30m of medium to very dense deposits of sand and silty sand. Below this layer a dense to very dense sandy layer is encountered. The sub-surface conditions revealed that the North side of approach embankments for rail and road have similar type of foundation soils, and therefore, in the analysis, both sites have been clubbed together. The subsurface conditions of rail and road embankments on South side have some variations, and therefore, the liquefaction evaluations for the rail and road embankment at the South side are carried out separately. The sub-surface condition at the bridge foundation site is also somewhat different, and therefore, it is analyzed separately.

A two level earthquake design criterion(Jain and Murty2000) is used for analyses. A peak ground acceleration value of 0.60g is recommended as safety evaluation ground motion. This motion corresponds to a M7.0 event close to the bridge site. Due to this motion, the bridge and embankments may undergo repairable damage. It is further recommended that functional evaluation of the bridge be carried out for a peak ground acceleration of 0.10g caused by an earthquake of magnitude 6.5±0.25 (functional evaluation ground motion). Such a motion should cause only minimal damage (i.e., essentially elastic performance; permanent deformations not apparent), and full access to normal traffic should be available almost immediately following this motion. This paper summarizes the liquefaction evaluations of bridge and abutment foundations and approach embankments of upto 21m height.

2. CURRENT METHODS FOR EVALUATION OF LIQUEFACTION POTENTIAL

The liquefaction susceptibility of a soil depends on; (a) Geologic age and origin, (b) Fines content and plasticity index, (c) Saturation (d) Depth below ground surface (e) Soil penetration resistance (Kavazanjian, 1997).

Simplified and semi-empirical charts and procedures are widely used for estimating liquefaction potential of level, saturated sandy sites. All of these methods are based on the few dozen case histories that have been compiled of sites that have been subjected to earthquakes, and did (or did not) experience liquefaction. Most of the procedures assume that the maximum horizontal ground surface acceleration and its duration characteristics are known (Ohsaki, 1970; Kishida, 1969; Seed and Idriss, 1971; Castro, 1975; Yegian and Oweis, 1976; Christian and Swinger, 1975; Seed 1976), although Yegian and Whitman (1978) proposed a correlation based directly on the magnitude of the earthquake and its distance to the site. These methods without exception are based on the use of the standard penetration test (SPT) data as a measure of the cyclic shear strength of the soil at the site. Therefore, these methods are affected by the large uncertainty associated with this field test. Some of the authors mentioned above have included ranges or "gray" areas in their proposed correlation, to define in an approximate way the uncertainty of the method, while others have used probabilistic procedures.

In recent years additional in-situ devices (such as the flat plate dialatometer, cone penetration test, electrical measurements, shear wave measurements) have been employed by various investigators (Robertson and Campanella, 1985, 1986; Baldi et. al., 1985; Arulanandan, 1977; Arulmoli et. al., 1985, Arulanandan and Muraleetharan, 1988 a,b; Bierschwale and Stokoe, 1984).

Due to the difficulties in obtaining and testing undisturbed representative samples from most potentially liquefiable sites, in-situ testing is the approach preferred by most engineers for evaluating the liquefaction potential of a soil deposit. Liquefaction potential assessment procedures involving both the SPT and CPT are widely used in practice (e.g., Seed and Idriss, 1982; Ishihara, 1985; Seed and De Alba, 1986; Shibata and Teparaska, 1988; Stark and Olson, 1995). The most common procedure used in engineering practice for the assessment of liquefaction potential of sands and silts is the *Simplified Procedure*. It was originally developed by Seed and Idriss (1982) and progressively revised, extended, and refined (Seed et al., 1983; Seed at al., 1985; Seed and De Alba, 1986; Liao and Whitman, 1986). The procedure may be used with either SPT or CPT data. This procedure with SPT values has been used in the present analysis.

3.SUBSURFACE CONDITIONS

Several shallow and deep borings have been drilled at the site varying in depth from 10m to over 100m depth. The strata encountered from boreholes are discussed (by Dayal and Jain 2000). Generalized subsurface conditions used for liquefaction assessment of the foundation soils embankment sections are given in Table 1 through Table 3 and the bridge site in Fig.1. These figures includes average N-values, corrected or normalised N-values $(N_1)_{60}$, percentage fine and soil classification as per IS standard for depth intervals of 3m upto maximum depth of boring.

Boring logs have revealed that similar subsurface profiles have been encountered below the rail and road embankment on the North side, and accordingly, a generalised subsurface has been prepared for North side, combining the borehole data of both rail and road embankment locations. Table 1 shows the generalised subsurface profile utiliused for liquefaction evaluation for the North side of the rail and road embankments.

At the South side different soil profiles have been encountered beneath rail and road embankment locations, and therefore, these have been treated separately. Table 2 and 3 show the generalised sub-surface profiles utilised for the liquefaction evaluation of the rail and road embankment foundations, respectively. Fig.1 shows the generalised sub-surface profile utilised for the liquefaction potential evaluation of the bridge and abutment piers.

4. LIQUEFACTION EVALUATION

4.1 Bridge and Abutment Foundations

Table 4 summarizes the results of liquefaction analysis for the bridge foundation. Under the functional evaluation earthquake motion (PGA value of 0.1g), there is no likelihood of liquefaction. Under the safety evaluation earthquake motion (PGA value of 0.60g), the soil below the seismic scour depth and at the base of the well foundations are not susceptible to liquefaction. In case there is no scour near the bridge pier, earthquake motion with PGA of 0.60g may cause liquefaction upto a depth of about 7.5m from the bed level of the river. Since the well foundation is being designed to withstand this earthquake motion with a much larger scour depth, liquefaction over a depth of 7.5m is not going to adversely affect seismic safety of the bridge foundation.

4.2 Soil Underlying the Embankments

A maximum of 21m high embankment is to be constructed at Bogibil site on a very loose to medium dense sandy soil. The embankment is to be constructed by silty sand compacted to about 95% modified Proctor dry density with the side slope of 2.5(H):1(V) with 3m wide benches at 6m (vertical) intervals. .As per the screening tests for liquefaction, the Bogibil embankment foundation soil is susceptible to liquefaction, and therefore, detailed investigation and analysis were carried out for liquefaction potential. In this paper a worst case scenario has been considered, assuming the maximum embankment height. For lower height of embankment the consequences of liquefaction should be significantly lower.

The results of liquefaction are summarized in Table 5. It is seen that the site is not susceptible to liquefaction for functional evaluation PGA of 0.1g; this meets the functional requirements of the project. However, liquefaction is likely to occur upto 14m to 19m depth for PGA value of 0.60g; the maximum settlement of the foundation soil due to liquefaction is estimated as 260mm. This much settlement is considered to be acceptable for the embankment of this project under the safety evaluation earthquake conditions.

4.3 Slope Stability Analysis of the Embankments

Static and pseudo-static slope stability analyses of the embankments have been performed using the computer program STABLE5 assuming that the liquefaction will not occur. Under static load (self load; no earthquake inertia force), the embankment has a factor of safety of 1.99. Several pseudo-static trial analyses were performed with different values of seismic coefficient; it was found that a seismic coefficient of 0.275g gave a factor of safety of 1.0. To assess the seismic performance of the embankment, two alternate methodologies were considered: pesudo-static analysis with factor of safety concept, and the permanent deformation evaluation using the Newmark's sliding block concept. Finally, for the liquefaction situation, when the soil underlying the embankment undergoes liquefaction as predicted by the liquefaction analysis, post-earthquake stability evaluation of the embankment has been carried out.

4.4 Pseudo-Static Analysis Criteria

As per the criterion suggested by Terzaghi (1950), the embankment should have a factor of safety greater than 1.0 for 0.20g coefficient. In the present case, the embankment has a factor of safety of 1.0 for 0.275g, and hence, this criteria is met. However, such a criteria is rather primitive as Terzaghi (1950) himself were to recognise: "the concept it conveys of earthquake effects on slopes is very inaccurate, to say the least." As per Marcuson (1981) criterion, the embankment should be checked for a coefficient of about 0.20g to 0.30g corresponding to safety evaluation motion. With 0.275g giving a factor of safety of 1.0, this criterion is also being met.

Following the Hynes and Franklin (1984) criteria, when the yield coefficient (corresponding to factor of safety of 1.0) is one-sixth of the peak average acceleration of the potential failure mass, the embankment will undergo less than 1.0m permanent displacement. In the present case, the peak average acceleration will be less than 0.6g, and hence, this critieria is also satisfied.

4.3 Permanent Seismic Deformation Analysis by Newmark's Sliding Block Model

Following the Makdisi and Seed (1978) approach a permanent displacement of upto about 5 - 15mm is obtained. Ambraseys and Menu (1988) equation gives permanent displacement of 39mm. As per equation by Yegian et al (1991), the permanent displacement comes out to be 30mm. A permanent displacement of the order of 40mm is nominal and quite acceptable. However, this analysis assumes that the soil strata underlying the embankment do not undergo liquefaction.

4.4 Post-Earthquake Stability Analysis of Embankments

The embankment is to be constructed with granular material soil and compacted to at least 95% of modified Proctor density; hence, it is not very susceptible to liquefaction. Due to safety evaluation earthquake motion, it will undergo some deformation which can be repaired. However, liquefaction potential analysis showed that the soil underlying the embankment is prone to liquefaction for some depth. Implications of such liquefaction occurrence on the stability of embankment are considered here. Once the soil underneath the embankment liquefies, it will not be able to transmit any significant part of the shear waves to the embankment, and therefore, the soil in the embankment will not experience any significant inertia force. However, the concern would be if the embankment has adequate factor of safety against sliding failure due to its self-weight when supported on liquefied soil strata. A limit equilibrium analysis is performed using the residual strength for the potentially liquefiable soil beneath the embankment toe. The residual strength is evaluated using the Seed and Harder (1990) relationship between corrected "clean sand" blow count and undrained residual strength. Static analysis (self weight; no earthquake inertia force) gives a factor of safety of 1.39 (North embankment), 1.18 (South embankment for rail), and 1.31 (South embankment for road). These values being greater than 1.1 are adequate and imply that the embankments will be stable under their own weight even after the underlying soil strata have liquefied.

A very conservative calculation for permanent displacement was also carried out with the assumptions: (a) liquefaction occurs very early during the ground shaking so that the embankment undergoes most of the strong shaking while the soil is already liquefied (in real practice, it will require some strong shaking and duration before the soil actually liquefies), and (b) the embankment still undergoes strong shaking with peak ground acceleration of 0.60g (i.e., it is assumed that the liquefied layers of sand are able to transmit the shear waves). These assumptions imply that the permanent displacements of the embankment be evaluated using peak ground acceleration of 0.60g but with residual strength of the liquefiable layers. Slope stability analyses show that with residual strength of the liquefiable layers, the yield acceleration (corresponding to factor of safety of 1.0) is 0.1g (North embankment), 0.048g (South embankment for rail), and 0.08g (South embankment for road). Permanent displacements thus obtained are shown in Table 6. The maximum permanent displacement is upto about 400 mm for North rail and road embankments, about 700 mm for South road embankment, and about 1300 mm for South rail embankment. Under safety evaluation earthquake motion, permanent displacement of upto about 400 – 700 mm will be considered quite acceptable, while value of about 1300 mm would be considered somewhat on the higher side. However, as mentioned above, these calculations are based on two very conservative assumptions. Moreover, even with displacement of about 1.0m to 1.5m the embankment can be repaired and traffic restored within a reasonable time frame. Therefore, no ground improvements are recommended for liquefaction mitigation for the project.

5. Summary and Conclusions

1) A rail-cum-road bridge with approach embankments is to be constructed on the Brahmputra river near Bogobil in Assam (India) which lies in an area of very high seismic hazard and traverse through the liquefiable ground.
2) A two-level earthquake design criterion has been used: safety-evaluation ground motion with PGA of 0.6g, and a function-evaluation ground motion of PGA value of 0.10g.
3) The liquefaction analysis has been carried using a "simplified procedure" originally developed by Seed and Idriss (1982) and progressively revised, extended and refined by others for maximum embankment height of 21m. The analysis indicated that the liquefaction is not likely to occur for functional evaluation motion. Soil strata under the embankments are liable to liquefy due to ground motion with PGA of 0.6g up to depths of about 14-18m with a total soil settlement of up to about 260 mm.
4) The post liquefaction analyses have carried out for the embankment assuming residual shear strength of the soil. The factor of safety for the static condition is found to be1.18 to 3.39 and the yield strength range from 4.8% to 10% and maximum spreading of about 1300mm for PGA of 0.6g.
5) The bridge and abutments are founded on well foundations seated at about 55m below the river bed, are not susceptible to liquefaction.

Table 1 Average Soil Profile for Northern Embankment

Depth (m)	N	$(N_l)_{60}$	% Finer than 75μ	Soil type
3	8	15	8.1	SP-SM
6	14	20	8.4	SP-SM
9	23	26	7	SP-SM
12	27	26	6.4	SP-SM
15	29	25	6.4	SP-SM
18	35	28	7	SP-SM
21	42	31	7	SP-SM
24	51	35	7	SP-SM
27	57	37	7	SP-SM
30	60	37	7	SP-SM

SP-SM = Poorly graded sand and silty sand.

Table2 Average Soil Profile for Southern Embankment (Rail)

Depth (m)	N	$(N_l)_{60}$	% Finer than 75μ	Soil type
3	6	11	55	Clay
6	13	17	6	SP-SM
9	17	18	6	SP-SM
12	24	22	7	SP-SM
15	31	26	6	SP-SM
18	42	32	5	SP-SM
21	48	34	6	SP-SM
24	54	36	6	SP-SM
27	65	41	7	SP-SM
30	74	44	7	SP-SM

SP-SM = Poorly graded sand and silty sand.

Table 3 Average Soil Profile for Southern Embankment (Road)

Depth (m)	N	$(N_l)_{60}$	% Finer than 75μ	Soil type
3	7	13	9	Clay
6	12	15	6	Clay
9	18	20	11	SP-SM
12	25	23	8	SP-SM
15	31	26	6	SP-SM
18	38	30	6	SP-SM
21	44	32	5	SP-SM
24	47	31	6	SP-SM
27	53	33	7	SP-SM
30	57	34	-	SP-SM

SP-SM = Poorly graded sand and silty sand.

Table 4 Summary of Liquefaction Potential Analysis at the Bridge Site

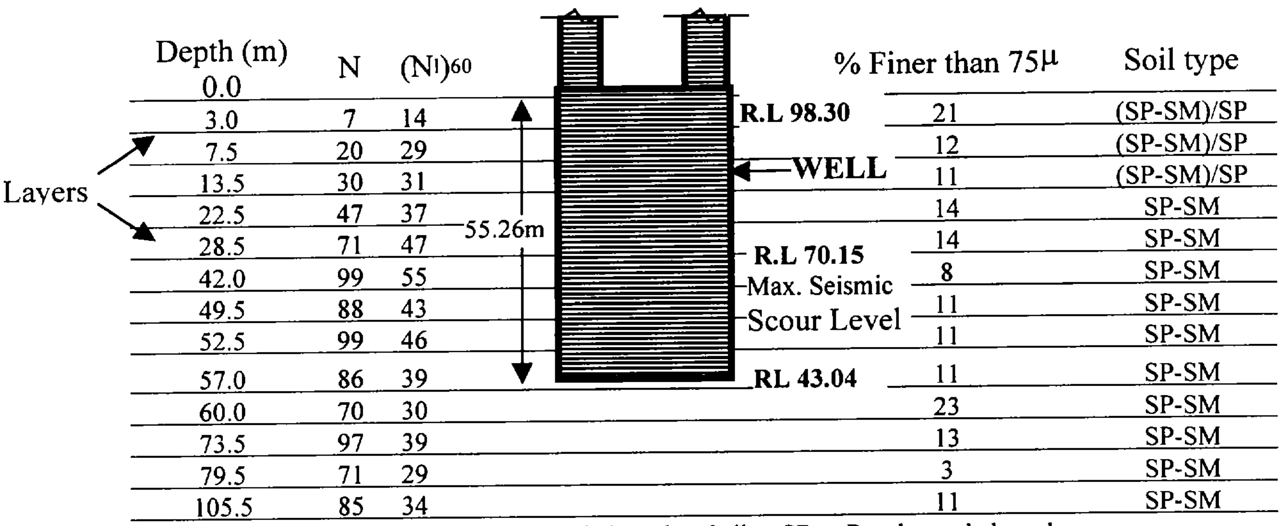
Sl. No.	Level	Depth of Liquefaction (m)	Soil Settlement (m)
		Functional Evaluation Motion (0.1g)	
1.	River Bed Level	-	-
2.	Seismic Scour Level	-	-
3.	Base of Well Level	-	-
		Safety Evaluation Motion (0.6g)	
4.	River Bed Level	7.50	0.10
5.	Seismic Scour Level	-	-
6.	Base of Well Level	-	-

Table 5 Summary of depth of liquefaction and vertical Settlement of foundation soil for embankment

Sl. No.	Details	Depth of Liquefaction (m)	Soil Settlement (m)
		Functional Evaluation Motion (0.1g)	
1.	North Embankment (Rail & Road)	-	-
2.	South Rail Embankment	-	-
3.	South Road Embankment	-	-
		Safety Evaluation Motion (0.6g)	
4.	North Embankment (Rail & Road)	18.75	0.26
5.	South Rail Embankment	14.25	0.19
6.	South Road Embankment	14.25	0.18

Table 6 Post-Liquefaction Evaluation of Embankments Using Residual Strength of Liquefiable Layers

Details	North Bank Embankment	South Bank Embankment	
		Rail	Road
FOS for Static case (no earthquake)	1.39	1.18	1.31
Yield Acceleration (for FOS = 1.0)	0.1g	0.048g	0.08g
	Permanent Displacement Considering PGA of 0.60g		
Ambraseys and Menu (1998) Equation	353 mm	1310 mm	497 mm
Yegian et al (1991) Equation	317 mm	1190 mm	502 mm
Makdisi and Seed (1978)	50-400 mm	200-1000 mm	80-700 mm



SP-SM = Poorly graded sand and silty, SP = Poorly graded sand

Fig. 1 Average Soil Profile along Bridge Alignment

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