Bridge Foundations in Strata with Potential of Liquefaction

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Abstract When liquefaction occurs, the soil loses its stiffness and strength and can not only deform but also flow laterally. In-situ testing is relied upon to assess the liquefaction potential of soils due to the difficulties in obtaining and laboratory testing of undisturbed samples. The standard penetration test (SPT) and the cone penetration test (CPT) are the two most frequently used field investigations for determining the characteristics of soils. In this paper, SPT values have been used as they remain by far the most popular and economical method of subsurface investigation in India. The methods suitable for design offices in the Indian context have received special attention in this paper which examines the effects of liquefaction on piled bridge foundations.

Keywords Liquefaction · Inertial effects · Kinematic effects · Evaluation

1 Introduction

IS 1893 Part 1 (2016) defines liquefaction as a state primarily in saturated cohesionless soils, wherein the effective shear strength is reduced to negligible value for all engineering purposes when the pore pressure approaches the total confining pressure during earthquake shaking. In this condition, the soil tends to behave like a fluid mass.

IS 1893 Part 1 (2016) stipulates the types of strata that should be investigated to evaluate its potential for liquefaction.

The Caltrans Geotechnical Manual (2014) suggests that soils with the characteristics shown in Table 1 are not liquefiable. $(N_1)_{60}$ is defined in a subsequent section of this paper.

Empirical and semi-empirical approaches for determining potential of liquefaction used in design offices cannot account for all the effects attributable to the

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Table 1 Soil not Liquefiable

$(N_1)_{60}$	% Fines
>30	≥ 5
>25	≥ 15
>21	≥ 35

characteristics of the soil, the topography, the ground motion and the structural arrangement. Nevertheless, for design office applications such approaches are invaluable especially if there is a consensus in a large body of experts and investigators that a particular approach is acceptable with the present state of knowledge.

A review of the design codes applicable to piles in liquefiable soils is available at Ghosh et al. (2012).

A comprehensive compilation of the SPT-based procedures including case history databases is recorded by Idriss and Boulanger (2010).

Wells (caissons) and piles are the most common types of foundations used in bridge structures. They have the advantage of transmitting loads to competent strata below when the upper strata have been subjected to liquefaction during strong ground shaking. Liquefaction results in loss of surface friction and stabilising support of the surrounding soil over the depth affected by the phenomenon. Well foundations, due to their large size and stiffness, are able to better cope with the effects of liquefaction.

2 Liquefaction Basics

The shear strength of cohesionless soil, τ, depends mainly on the angle of internal friction and the effective stress acting on the soil grains and can be expressed as

$$\tau = \sigma' \tan \varphi \tag{1}$$

$$\sigma' = \sigma - u \tag{2}$$

where

- τ shear strength
- σ' effective normal stress
- σ total normal stress
- u pore pressure
- φ angle of internal friction.

When saturated loose cohesionless soils are subjected to earthquake loading, they tend to settle due to the densification of soil. The duration of the cyclic stress application is so short compared to the time required for water to drain that excess pore pressure progressively builds up. When the pore pressure equals the total stress, thereby reducing the effective stress to zero, the soil will experience a sudden degradation of strength and stiffness.

The most popular and accepted method of analysis of soil substrata for liquefaction potential is available at Youd et al. (2001). This method is suited up to a depth of 23 m below ground. Extrapolation beyond this depth could be of uncertain validity, AASHTO (2012).

The earthquake magnitude to be considered for liquefaction potential analysis is slightly different in international codes. AASHTO (2012) stipulates the basis as a 975-year return period (i.e. 5% possibility of exceedance in 50 years). IS 1893 Part 1 (2016) is not based on Probabilistic Seismic Hazard Analysis at present but stipulates specific values of horizontal ground accelerations to be adopted in the four seismic zones of the country.

3 Piled Bridge Foundations

The diameter of piles in bridge foundations is usually restricted to the range 800–1500 mm for reasons of economy and construction convenience, though occasionally large diameter (2000–2500 mm) or raked piles have been adopted in India. As a matter of interest, for the 6.15 km long rail-cum-road Padma Bridge in Bangladesh, some foundations consist of racked (inclination 1+1: 6V) steel tubular driven piles, 6 nos, each of 3 m diameter and 128 m depth. These were necessitated by the susceptibility to deep sour and high seismicity of the area. Since the surrounding soil over the liquefied height can no longer be depended upon to provide lateral support to the pile, the resulting deformations (including *P*-delta effects) and bending effects can be quite significant. Example of a bridge foundation on piles is shown in Fig. 1. This part of the analytical process relates to the “inertial effects” of liquefaction.

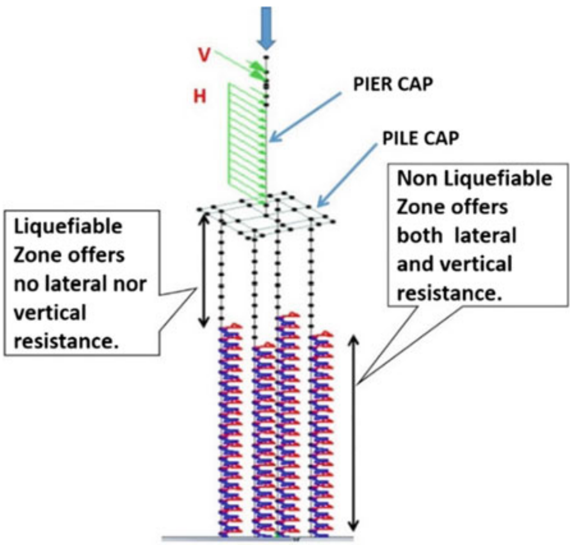

Fig. 1 Three-dimensional model of bridge pier and piled foundation

The diameter of pile must be selected with care so that apart from its vertical load carrying capacity the serviceability of the structure is not affected. The potential consequences of liquefaction associated with pile foundations include loss of vertical load capacity, loss of lateral stiffness and capacity, lateral loading due to lateral soil displacements and down-drag on piles due to post-liquefaction reconsolidation of soil.

Figure 2 from Bhattacharya et al. (2005) depicts deflected shapes of pile and pier in some structural configurations.

The problems concerning liquefaction have another dimension. Lateral spreading arises if soils subject to liquefaction are situated on a slope or near a river channel or sea which may cause movement of the liquefied soil perpendicular to the waterfront, thereby aggravating the induced effects further. In these conditions, the presence of non-liquefied soil strata overlying the liquefiable soil makes the situation even more onerous. The non-liquefied crust would exert passive earth pressure on the foundation. For shallow slopes, a simplified prescriptive approach is indicated in JRA (1996, 2002) based on back calculations from the observed damages in the Kobe earthquake of 1995. The equivalent static forces acting on the bridge foundations due to the ground flow can be estimated as (a) passive earth pressure of the upper non-liquefiable layer, plus (b) 30% of the overburden pressure of the lower liquefiable layer, as shown in Fig. 3. This part of the analytical process relates to the "kinematic effects" of liquefaction.

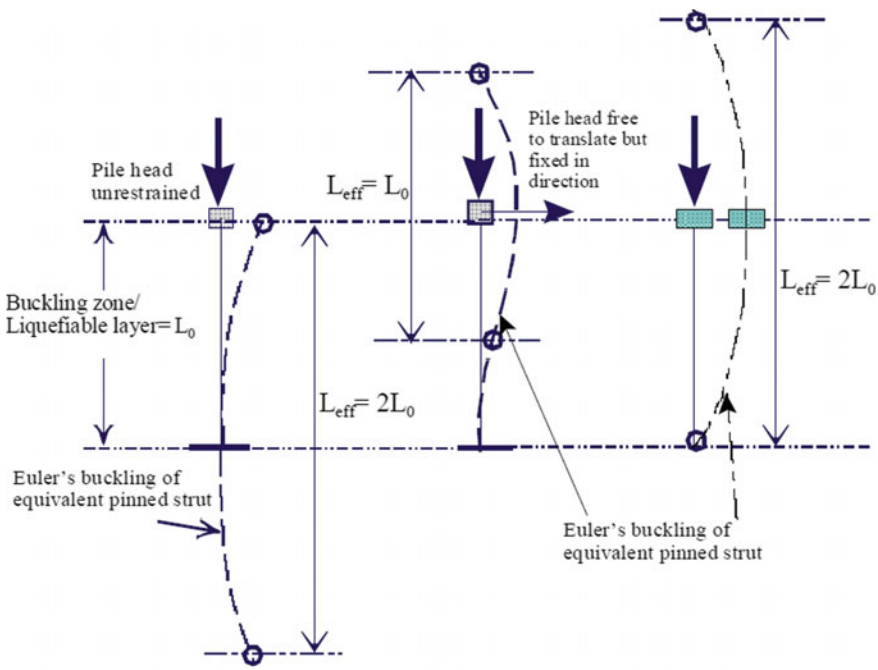

Fig. 2 Effective lengths for buckling considerations

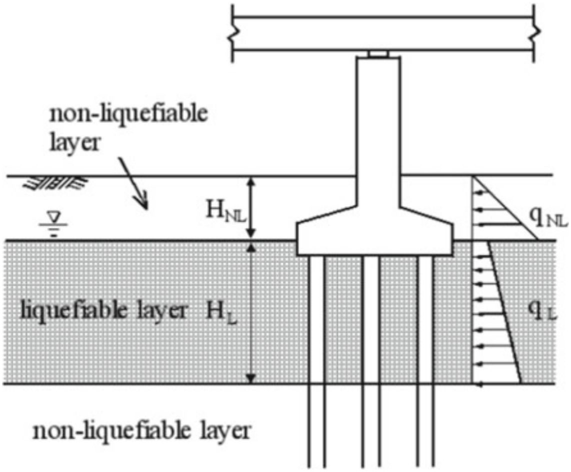

Fig. 3 JRA (1996) code of practice showing the idealisation for seismic design of bridge foundation

Showa Bridge, Fig. 4, is an example of failure due to lateral spreading, resulting in drag down the slope in the 1964 Niigata Earthquake. The bridge has a 24.8 m wide deck and a total length of 303.9 m (13.75 m + 10@ 27.64 m + 13.75 m). Each of the pier foundations consists of nine piles in a single row. One of the investigators, Bhattacharya et al. (2005), identified buckling as a possible failure mechanism of the piled foundations.

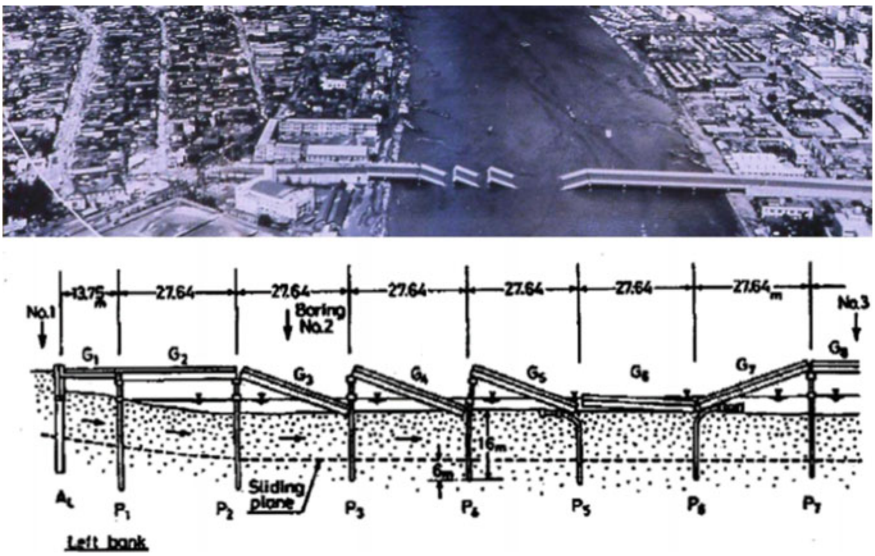

Fig. 4 Schematic diagram of the fall of the girder in Showa Bridge, Bhattacharya et al. (2005)

Following the 1964 collapse, a law was passed to prevent bridge piers being founded on a single row of piles, Bhattacharya et al. (2005).

The centrifuge test results of Haigh (2002) suggest that the pressure distribution in Fig. 3 is under-conservative in the transient phase but gives reasonable predictions for residual sliding.

The existing simplified methods cannot account for the "inertial effects" and the "kinematic effects" of liquefaction as coupled phenomena. This is justified by the fact that peak "inertial" loads are likely to occur before ground flow, Kavazanjian et al. (2011). The response of the structure to liquefaction is checked separately for the peak inertial load and the kinematic load (where applicable) without superimposing or adding the two.

Piles must be checked for buckling instability due to both inertial and kinematic effects as well as out-of-straightness which would increase lateral deflections, thereby reducing the buckling load. As per JRA (1996, 2002), the effective length of the pile should in no case exceed 50.

In the case of stiff piers including plate piers, it becomes difficult to avoid plastic hinging in the piles. Pile integrity and ductile behaviour should be ensured in such cases. The potential hinge locations should be provided with proper ductile reinforcement, for instance, in accordance with IS 13920 (2016) at the following locations: (a) at the pile heads just below the pile cap, (b) at the depth where maximum bending moments develop in the pile and (c) at the interfaces of soil layers having markedly different shear deformability. For the locations (b) and (c), longitudinal as well as confining reinforcement of the same amount as that required at the pile head should be provided.

The above issues should find specific provisions in IS 1893 Part 1 (2016) and RDSO Guidelines (2015).

4 Quantification of Liquefaction Potential

The quantification of liquefaction potential is carried out in accordance with Youd et al. (2001). The estimation of two parameters is required to evaluate liquefaction potential:

(A) CSR or critical (or cyclic) stress ratio—demand on soil layers during the seismic event
(B) CRR or critical (or cyclic) resistance ratio—capacity of the soil to resist liquefaction.

A factor of safety, FOS (= CRR/CSR), of greater than 1 is usually associated with non-liquefiable soil. A higher FOS may be warranted if uncertainty exists about the quality of data. The Indian code IS 1893 Part 1 (2016) recommends a value of 1.2.

Sample calculations in accordance with RDSO Guidelines (2015) of a real-life project by use of worksheets are shown in Tandon et al. (2015).

The evaluation is done in the following steps.

4.1 Determination of Design Groundwater Level

Caltrans (2014) suggests that if the groundwater table is at a level greater than 15 m, the site should be considered non-liquefiable.

It is usual to decide this on the basis of water table in the area as per local records. Since the water table in the area at the time of carrying out subsurface investigations could be at a lower level, it is recommended that the values obtained (e.g. soil density) should be modified to take this into account while evaluating CSR. The groundwater table affects the soil density, and hence, σ'_{V}, the effective vertical stress in the evaluation of CSR. IS 1983 Part 1 (2016) and RDSO Guidelines (2015) are silent on the subject. Eurocode 5 (2004) suggests that free-field site conditions (ground surface elevation and water table elevation) prevailing during the lifetime of the structure should be adopted.

4.2 Making a Realistic Stratification of the Soil from the Subsurface Investigation Data

The soil characteristics such as SPT values, unit weight and fines content (passing IS standard sieve no: 75 microns in the India context) are identified.

4.3 Normalisation of Field SPT Values

First, we must normalise the field SPT values, N, to (N_{60}) and then to $(N_1)_{60}$ for further processing, where

(N_{60}) SPT blow count of same soil for hammer with efficiency of 60%.

$(N_1)_{60}$ Value of (N_{60}) normalised for effective overburden pressure at ground level, i.e atmospheric pressure, 98 kPa.

The observed field test blow count values N need correction factor C_{60} to be applied to enable them to be converted to (N_{60}).

$$(N_1)_{60} = C_N C_{60} \tag{3}$$

C_{60} $C_{\mathrm{HT}}\ C_{\mathrm{Hw}}\ C_{\mathrm{SS}}\ C_{\mathrm{RL}}\ C_{\mathrm{BD}}$, where
C_{HT} Energy ratio correction
C_{Hw} Hammer weight correction
C_{SS} Sampling method correction
C_{RL} Rod length correction
C_{BD} Borehole diameter correction and

$$C_N = \left(P_{\mathrm{a}}/\sigma'_{\mathrm{V}}\right)^{1/2} = 9.79\left(1/\sigma'_{\mathrm{V}}\right)^{1/2} \tag{4}$$

where P_{a} is the atmospheric pressure.

A total of six numbers of corrections are applied on observed N value to arrive at $(N_1)_{60}$. There is no provision in the Indian codes for these corrections based on SPT equipment and methods employed in the country. Brief specification of the SPT equipment and methods is available in IS 2131 (1981).

The standard specifications for the SPT equipment recommended in Youd et al. (2001) is as per ASTM D1586 (2011), which is summarised in Table 1, and which is also included in RDSO Guidelines (2015).

In IS 1893 Part 1 (2016), it has been suggested that if the SPT values have been conducted as per IS 2131 (1981), the value of C_{60} may be taken as 1.

The RDSO Guidelines (2015), however, suggest that in the absence of test-specific energy measurement, the corrections, C_{60}, should in fact be carried out.

4.4 *Evaluation of Cyclic Stress Ratio (CSR)*

$$\mathrm{CSR} = 0.65(a_{\max}/g)\left(\sigma_{\mathrm{V}}/\sigma'_{\mathrm{V}}\right) r_d \tag{5}$$

r_d Stress reduction factor which depends on depth below ground level.
$a_{\max}/g$ (Ratio of peak horizontal ground acceleration/acceleration due to gravity). For Zone IV, for instance, MCE = 0.24, IS 1893 which is suggested for liquefaction in case PGA is not available. Some codes like AASHTO (2012) recommend a value corresponding to a 975-year return period (5% probability in 50 years).
$\sigma_{\mathrm{V}}/\sigma_{\mathrm{V}}'$ (Total vertical stress/effective vertical stress) should be evaluated for all the potentially liquefiable layers in the substrate. It would vary from approximately 2–1 depending upon where groundwater table is considered. It would be equal to 2 for groundwater table at ground level and equal to 1 if groundwater table is considered at the level lower than that where liquefaction is to be determined.

In Eq. (5), the flexibility of the soil profile is accounted for by r_d which can be calculated from Eqs. (6) and (6a).

$$r_d = 1 - 0.00765\,z \text{ for } z < 9.15 \text{ m} \tag{6}$$

$$r_d = 1.174 - 0.0267\,z \text{ for } 9.15 < z < 23 \text{ m} \tag{6a}$$

The depth z below the ground surface should be measured up to the centre of the concerned layer.

4.5 Making Correction for Fines

Seed and Idriss (1982) concluded that liquefaction potential in a soil layer increases with decreasing fines content and plasticity of the soil.

In accordance with IS 1893 Part 1 (2016), fines are defined as per cent by weight passing the IS Standard Sieve No. 75μ.

The corrections for fines content can be done following the equations developed by Idriss with the assistance of Seed for correction of $(N_1)_{60}$ to an equivalent clean sand value, $(N_1)_{60cs}$:

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \tag{7}$$

where $(N_1)_{60cs}$ is the blow count corrected for fines content, and α and β are coefficients that depend on the fines content.

$$(N_1)_{60cs} = \alpha + \beta(N_1)_{60} \tag{7a}$$

where α and β = coefficients determined from the following relationships:

$$\alpha = 0 \text{ for FC } \leq 5\% \tag{8a}$$

$$\alpha = \exp\left[1.76 - \left(190/\text{FC}^2\right)\right] \text{ for } 5\% < \text{FC} < 35\% \tag{8b}$$

$$\alpha = 5.0 \text{ for FC} \geq 35\% \tag{8c}$$

$$\beta = 1.0 \text{ for FC} \leq 5\% \tag{9a}$$

$$\beta = \left[0.99 + \left(\text{FC}^{1.5}/1.00\right)\right] \text{ for } 5\% < \text{FC} < 35\% \tag{9b}$$

$$\beta = 1.2 \text{ for FC} \geq 35\% \tag{9c}$$

4.6 Calculation of Cyclic Resistance Ratio $(CRR)_{7.5}$

The database from sites where liquefaction effects were or were not observed, a base curve for "clean sand" for magnitude 7.5 earthquakes, was arrived. The curve was approximated to Eq. (10)—credited to AF Rauch of the University of Texas (1998)—which can be used more conveniently in worksheets.

$$\begin{aligned} \mathrm{CRR}_{7.5} = & \frac{1}{34 - (N_1)_{60\mathrm{cs}}} + \frac{(N_1)_{60\mathrm{cs}}}{135} \\ & + \frac{50}{\left[10(N_1)_{60\mathrm{cs}} + 45\right]^2} - \frac{1}{200} \end{aligned} \tag{10}$$

4.7 Calculation of the Magnitude Scaling Factor (MSF)

The next step is to calculate the CRR for the particular site by evaluating the magnitude scaling factor (MSF) for the particular site.

The data available for various parts of India for past earthquakes have been plotted in IS 1893 Part 1 (2016) which is shown in Fig. 5. If site-specific investigations have not been carried out, Fig. 5 can be used for determining the earthquake magnitude, (M_w), applicable to the site.

MSF is determined from Eq. (11).

$$\mathrm{MSF} = 10^{2.24}/M_\mathrm{w}^{2.56} \tag{11}$$

4.8 Calculation of CRR

The CRR for the particular site is evaluated by multiplying $\mathrm{CRR}_{7.5}$ with MSF as shown in Eq. (12).

$$\mathrm{CRR} = \mathrm{CRR}_{7.5}(\mathrm{MSF}) \tag{12}$$

4.9 Evaluation of Factor of Safety, FOS

The factor of safety with respect to potential of liquefaction is finally arrived at by using Eq. (13).

$$\mathrm{FOS} = \frac{\mathrm{CRR}}{\mathrm{CSR}} \tag{13}$$

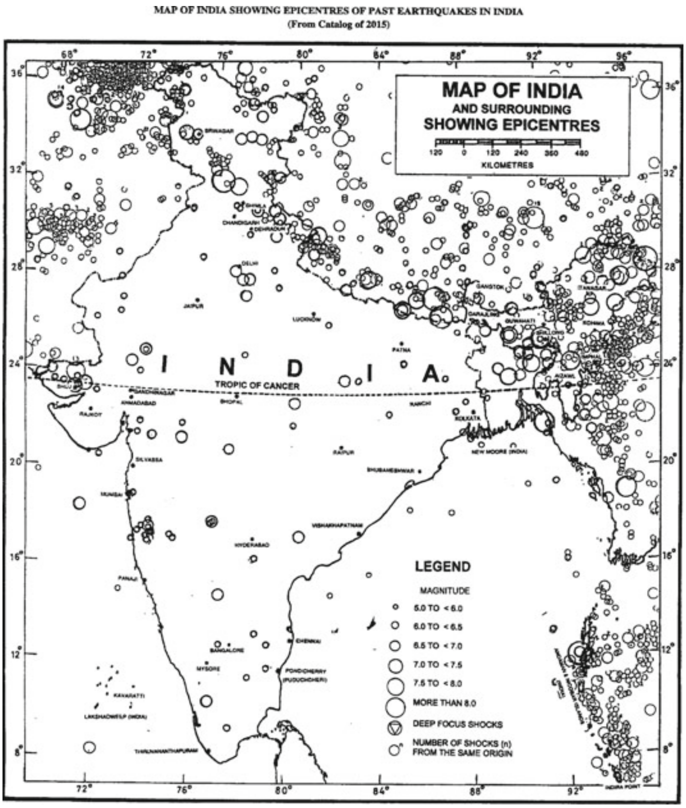

Fig. 5 Epicentres of past earthquakes (from IS 1893)

5 Conclusions

The paper highlights the methodology that should be employed in bridge design offices in the Indian context while determining the potential of liquefaction using SPT field tests. The frequently used code provisions have been reviewed. Some differences between the two codes/guidelines, i.e. IS 1893 Part 1 (2016) and RDSO Guidelines (2015), have been highlighted. Some of the missing provisions in these codes have been discussed.

In many situations, the liquefied layer is overlain by non-liquefied strata. The bridge site may be located on sloping ground or near a waterfront, in which case lateral spreading would occur creating significant lateral forces on the piles. A simplified approach to account for the same has been identified. Both inertial effects and kinematic effects of liquefaction have been discussed in the paper.

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