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Dynamic Consolidation of Liquefiable Sands

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SYNOPSIS The Subsoils underneath a tank were improved by dynamic consolidation to density levels required to resist earthquake induced liquefaction. The paper presents seismic analysis of the refinery region, simple liquefaction potential analysis using modified depth reduction factors and performance of the tank resting on the densified ground. The degree of uniformity of compaction achieved by the dynamic consolidation has been discussed.

INTRODUCTION

Soil investigation consisting of laboratory and field tests including a footing load test at the refinery site at Bongaigaon, Assam, indicated satisfactory subsoil conditions for building and unit foundations. North eastern region of India accommodating the refinery site is a seismically active area (Hazra et.al, 1962) and thus, although suitability of the subsoils under the static loading conditions was established by the investigation, stability of the soils under random earthquake loadings was a major concern in determining the foundation requirements of the refinery structures. The analyses presented herein clearly demonstrated susceptibility of the Bongaigaon soils to liquefaction. Systematic study by Kishida (1966) of the damage caused by liquefaction during the Niigata Earthquake of 1964 revealed that a higher degree of damage was caused to the structures supported on shallow foundations than those supported on Piles. His investigation also revealed that if piles of appropriate lengths (15m to 20m) have their tips resting in strata with standard penetration N value greater than 25, the degree of damage suffered by such structures was little or none. Accordingly, costly and long delivery imported tall vessels and other unit equipments for the Bongaigaon Refinery were carried on R.C.C. driven Cast-in-Situ piles having an average length of 16.0m with their tips embedded in dense sand mixed with gravels and cobbles.

The adoption of piled foundations for the storage tanks covering an area of approximately 40,000 m^2 would have been very expensive and would have delayed the project schedule. Therefore, to resist liquefaction, the soils underneath the tanks were strengthened by various methods to achieve 80 per cent relative density which was checked in the field as per the following criteria:

N = 10 blows/30 cm penetration at surface and 25 blows/30 cm penetration at 10.0m depth with linear variation in between. This paper describes the method of Dynamic Consolidation adopted to compact soils underneath a 22m dia. and 9.6m high tank. The degree of densification achieved with depth and performance of the tank as observed during its hydrostatic testing are presented and discussed.

LIQUEFACTION STUDIES

Liquefaction is the result of interaction between the earthquake induced forces and the soil properties as they relate to resistance to liquefaction.

Soil Parameters

Typically, the soil stratification below the tank consisted of coarse to fine sand to a depth of 10.5m followed by dense to very dense sands mixed with gravels and cobbles down to a depth of 18.5m. The water table at

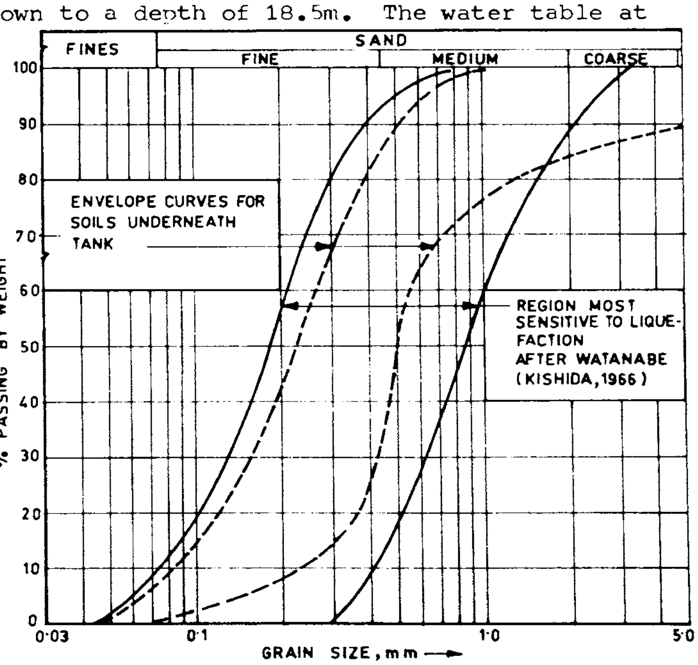

Fig. 1. Gradation Curves for Sands Below Tank

the time of treatment was 4.6m below the ground surface and its chances of rising to about 1.0m depth after incessant heavy rains to which the area is normally subjected, were rated high. Gradation envelope curves for the sands encountered at the tank site within the 10.0m depth are shown in Fig.1 which also plots the grain size limits found most sensitive to liquefaction by Watanabe (Kishida, 1966) in the Niigata Earthquake. The positioning of the curves clearly demonstrates their susceptibility to liquefaction.

Main soil parameters of interest are mean grain size, D_{50}, initial confining pressure and the relative density, D_r. The relative density was indirectly assessed from the standard penetration resistance values using the relationship, $N = 25\ D_r^2$ (Nishiyama, 1977).

Seismic Analysis of the Region

Relevant seismic parameters are the maximum horizontal acceleration at the refinery site and the duration of shaking expressed as average equivalent number of cycles, Neq.

Seismotectonic map of the region revealed that the refinery site lies in the Shillong massif forming a part of the Assam Plateau which is bordered on all sides by faults. The earthquakes experienced by the region around Bongaigaon had magnitudes of 5, 6, 7 and greater than 7 with their epicentral distances from the site as 20, 60, 80 and 55 km respectively. General frequency distribution for the entire north eastern region indicated, however, only eleven shocks of magnitude between 7 to 8 out of a total of 160 shocks experienced since 1822. Majority of the shocks were reported to be of shallow type (Arya et. al, 1973).

Considering the Socioeconomic importance of the project, expected life of the structures and probability of occurrence of shocks during the life time of the structures, it was deemed adequate to choose magnitude M = 7 commensurating with a return period of 79 years. The most proximate potential epicentral distance of the site was reasonably taken as 50 km as also the focal depth of 32 km.

Since no measured records of the ground motion at the refinery site were available, it was determined from the following expression (Krishna et. al, 1969):

$$\frac{a}{g} = \frac{2{\cdot}925\ \frac{10^{(M-5)}}{h}}{1+4{\cdot}5\ \frac{10^{(M-5)}}{h}} \cdot e^{-0{\cdot}26\left\{\frac{D}{h}\right\}^{3/2}} \qquad (1)$$

Where 'a' is the maximum induced ground acceleration due to an earthquake of magnitude M having apicentral distance D and focal depth h and g is the acceleration due to gravity. The validity of this expression for firm soils was established by the authors from a close comparison of the computed accelerations with those recorded at a total of thirteen sites in India as well as in the United States. This expression is applicable for a limiting epicentral acceleration of 0.65 g having atten-uation varying as an exponential function denoted by $(D/h)^{3/2}$ and a constant α = 0.26 determined from the ground motions experienced during the parkfield earthquake of 1966. Maximum ground acceleration of 0.38g was computed from this expression for the refinery site. The value of average equivalent number of stress cycles depends upon the duration of ground shaking and thus on the magnitude of the earthquake. Seed et. al (1971) suggested Neq = 10 for an earthquake of magnitude M = 7.0.

Evaluation of Liquefaction Potential

The average equivalent shear stress, Te, induced by earthquake was compared with the shear resistance, Tr, of the site at various depths. Simplified equation to calculate Te was modified using depth reduction factor, rd, determined from the dynamic response analyses by one wave propagation theory (Nishiyama,1977). This analysis predicted liquefaction to depths of 10 to 11m consistent with the general soil characteristics of the site as against unrealistically large depths predicted from Seed's rd factors, see Fig. 2 (Bhandari, 1980).

DYNAMIC CONSOLIDATION

Some Considerations in the Dynamic Compaction of Soils

The dynamic consolidation method was initially developed to compact ballast fills and natural sandy gravel soils. However, now it is considered an effective tool to compact saturated clays also although the number of passes increases with percentage of fines present in the material. The effectiveness of deep compaction in soft saturated clays is, in part, attributed to the expulsion or compression of the gas microbubbles under initial successive blows. Continued tamping results in the development of high pore pressures causing liquefaction of the soil. Further increases in the pore pressures split solid 'tissues' creating fissures in the soil mass which form preferential drainage paths for the escape of pore water as well as for the adsobed water rendered free by successive impacts. Closer contact between particles and adsorbed water layer are restored (thixotropic recovery) subsequent to

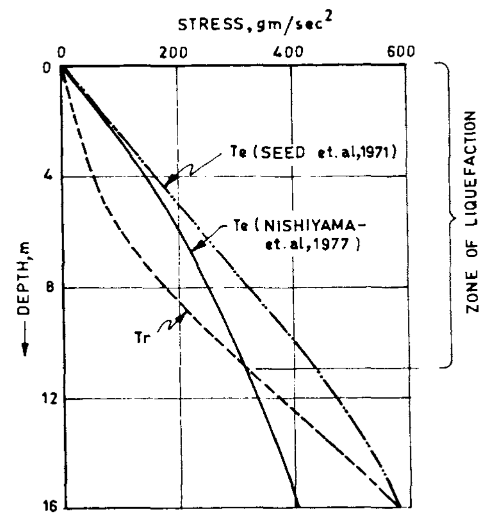

Fig. 2. Liquefaction Potential Analysis

the final pass of the weight. Lukas (1980) cited two-fold increase in the pressure modulus seventy days after pounding to bear out the thixotropic phenomenon for a saturated clayey silt.

Base area of the pounder rather than its height of fall has significant effect on the amplitude of vibrations delivered to the ground. Undesirable kneading action confined only to the upper soil layer is avoided by suitably proportioning the base dimensions so that the depth of imprint is less than half the width of the pounder.

The impact generates two types of body waves viz. pressure wave and the shear wave. The third and the most important wave form, for the present purposes, is the surface Rayleigh wave, the vertical component of which compacts the soil in depth close to the source of vibrations; the horizontal component becoming significant as the distance increases. Pounding induces high amplitude low frequency vibrations (2-12HZ). Mitchell (1976) listed safe distances to different types of structures. In important cases, however, these are determined by measurement of the seismic velocity. Wiss (Lukas, 1980) indicated that no damage to a residential structure is caused if velocity of the waves at the building location is less than 2 in/sec. Marked increase in energy efficiency is observed when impact velocity exceeds the velocity of wave transmission. Menard et. al (1976) derived the following empirical relationship between the depth of compaction D(m), the mass of the weight W(tons) and the height of fall H (m):

$$W.H. = D^2 \tag{2}$$

The Technique and Discussion of Results

Fig.3 shows that the initial standard penetration N values of sands underneath the tank were non-uniform and in certain depths, the available density was less than that required to counter liquefaction. The pounding was

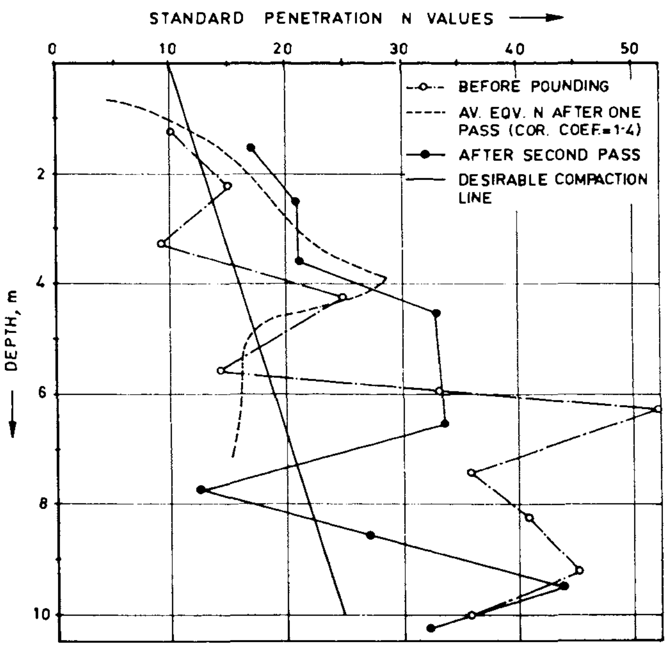

Fig. 3. Standard Penetration N Versus Depth

undertaken on the tank area plus a ring of 1.5m radius beyond the actual tank periphery. The upper 6.0m depth of the medium dense sand was compacted by dropping 7 ton weight measuring 2m x 2m at the base having a fall of about 11.0m. The compaction points were located at 5.0m interval in a square grid pattern and each point was pounded by ten blows in quick succession. At the end of the operation, the whole site was levelled and a settlement of 14.4cm was noted. After a pause of a few days, post treatment improvement was checked by conducting four dynamic cone tests which furnished a continuous depthwise record of resistance. A comparison of the pre and post treatment equivalent N values plotted in Fig.3 indicates sufficient improvement to 4 to 4.5m depth so as to satisfy the densification requirements. This depth, however, is only 51 per cent of the depth predicted by Eqn.2. The data presented by Lukas (1980) indicate that the depth to which improvement occurs is on the order of 65% to 85%. It is observed that with the pattern and weight of the pounder used in the present case, the depth of improvement could be accurately predicted if it is taken as proportional to cubic root of the energy transmitted. Also, as the depth of improvement is almost identical to the depth of water table (4.6m), it appears that the energy delivered by one pass of the pounder was just sufficient to drive out/compress the pore air and in addition, perhaps cause collapse of the empty voids to result in the settlement noted above. It was clear that to achieve compaction to the required depth of 6.0m, additional effort was necessary. Therefore, another tamping pass identical in all respects to the initial pass was given after a two-week rest period (forced due to equipment breakdown) with the hammer points located in between the previous points reducing therby the interval between them to 2.5m. It is apparent from Fig.3 that the energy delivered by the second coverage was able to effectively penetrate to and thus densify a depth of approximately 6.0m. At the same time, however, transmission of the additional energy caused loosening of the soil between 6 to 10m depth where originally it was observed in a dense to very dense state. It is likely that the pounding shears the dense sand and causes it to dialate thereby resulting in reduction of the N values. An additional settlement of 13.5cm was observed subsequent to ground levelling thus making a total of about 28.0cm settlement after two coverages of the pounder.

From the cone penetration tests, it was observed that the upper 60cm to 75cm depth where the confining pressure was low, showed only marginal improvement. In order to develop necessary liquefaction resistance, this depth was compacted by rolling in layers to 90 per cent of the maximum proctor laboratory density before laying the steel plates on the compacted sand pad.

Settlement Observations

The hydrotesting of the tank was done in five stages by filling water upto a height of about 8.4m, the balance of the 9.6m height being occupied by the floating pontoon. The water was fed to several tanks at the same time resulting in rather slow rate of filling. Total time taken for completion of the hydrotest was nearly 135 days, reference Fig.4(a), which also included nearly 80 days filling suspension period due to hose breakdown. The levels were taken on four peripheral points during the test with a tilting level and a levelling staff. The observed settlements at each point were: N=23mm, S=21mm, E=19mm, W=21mm. The maximum differential settlement between two successive points yields an angular distortion of approximately 1/3900 which is well within the limits of 0.3 to 0.35 per cent stated by Belloni (1974) for the safe operation of the floating roof tanks. This observation combined with small absolute recorded settlements and the observed low rate of settlement as apparent from Fig.4(b) point out to the effectiveness and relative uniform improvement in the soil characteristics that can be achieved with dynamic consolidation.

Before the treatment, the settlements based on the SPT N values were estimated as 49mm and 52mm at two peripheral points, E and W, respectively. These estimates were obtained for a tank located eastward close to the treated tank area. Method of computations is detailed elsewhere (Bhandari, 1979). A comparison of these settlements with those recorded during the hydrotest at the corresponding points revealed a reduction of nearly 60 per cent on account of the treatment by dynamic consolidation.

CONCLUSIONS

1 Seismic analysis of the region predicted that the refinery site is likely to be rocked by an earthquake of magnitude 7 with maximum horizontal acceleration of 0.38g and duration of shaking equivalent to 10 cycles. Depth reduction factors based on the dynamic response analyses predicted liquefaction depth of 10 to 11m consistent with the general soil characteristics.

2 Dynamic consolidation is an effective method of compacting the Bongaigaon sands to the density levels required to resist earthquake induced liquefaction. However, if originally dense to very dense sand is subjected to pounding, its structural arrangement may be severely damaged to cause significant reductions in the standard penetration resistance.

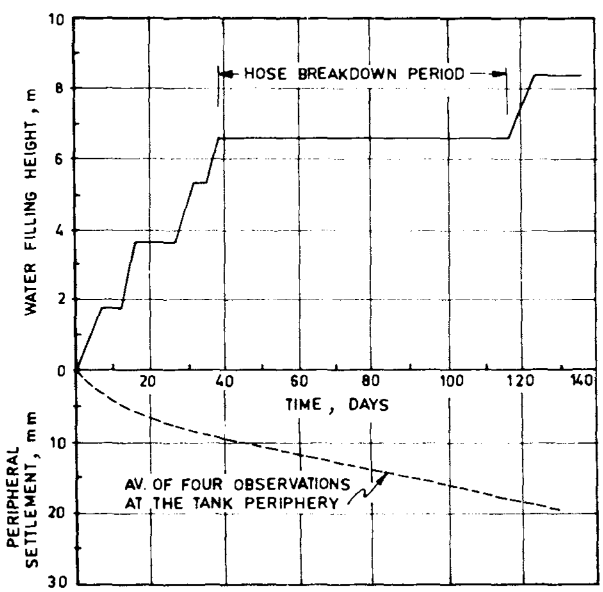

Fig.4. Time-Load-Settlement Relation For Tank

3 Performance of the tank during its hydrotesting has shown that carefully applied hammer blows compact the sands to relatively uniform densities resulting in low peripheral differential and total settlements. Reduction in settlement due to pounding was observed to be on the order of about 60 per cent.

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