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G. Armijo
GEOCISA, Spain

P. Sola
GEOCISA, Spain

C. Oteo
Laboratorio de Geotecnia, CEDEX, Spain

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Application of Deep Compaction Techniques to Liquefaction Prevention

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G. Armijo and P. Sola
GEOCISA, Spain

C. Oteo
Laboratorio de Geotecnia, CEDEX, Spain

SYPNOSIS: This article analyzes the application of dynamic compaction, vibroflotation and vibroreplacement (stone columns) to liquefaction prevention. The ground types to which they can be applied, the depths that can be reached and the degree of improvement that can be obtained are all studied. Finally, and on the basis of the above, basic guidelines are given for the design of ground improvement with these techniques and for the aforementioned purpose.

INTRODUCTION

When the existing methods of analysis indicate that liquefaction is a possibility, or the safety margins are not sufficient, measures must be taken to prevent the phenomenon itself, or to prevent the consequences thereof.

The measures that can be applied, whether individually or collectively, to prevent the occurrence of liquefaction, consist of excavating and replacing the dangerous layers, increasing their density or allowing for the dissipation of the pore pressure generated therein.

In view of the above, this article reviews the application of dynamic compaction, vibroflotation and vibrosubstitution (stone columns) to liquefaction prevention.

APPLICATION OF DEEP COMPACTION TECHNIQUES

The most effective deep compaction techniques are the vibratory ones, especially dynamic compaction (DC), vibroflotation (VF) and vibroreplacement (VR) or the formation of stone columns. The aim of the first two techniques is to densify the ground in depth, whereas the third one combines this with an improvement in the drainage conditions.

Soils to which they can be applied

Fig. 1 shows the envelopes for the granulometric curves of soils that liquefied in past earthquakes, together with those for soils to which the three techniques mentioned can be applied. It can be deduced from this figure, that the first two do not yield good results when the fines content (FC) $\geq$ 15 - 20%. In this case VR is more suitable.

Maximum depth that can be reached

Almost all the data available concerning cases of liquefaction, concerns layers lying above 16 and 17 m and the most frequent occurence of the phenomenon taking place between 3 and 7 m. It may be deduced from this and existing experience, that the VF and VR methods, which reach a depth of 20 m in normal conditions, reach such depths without problems. However, this is not the case with the DC, with which it is only possible to reach depths ranging from 8 to 12 m., when normal equipment is used.

On the basis of the data obtained by the authors of this article, the depth of influence of DC (D) would be approximately expressed by the following formula: $D=n(W.H)^{0.5}$, where W is the weight in tons of the mass that is dropped, H is the height from which it falls in metres and n is a coefficient that, for ground lying below the water table, ranges from 0.5 (clean sand) to 0.35 (silty soils and silt with PI $\leq$ 10%).

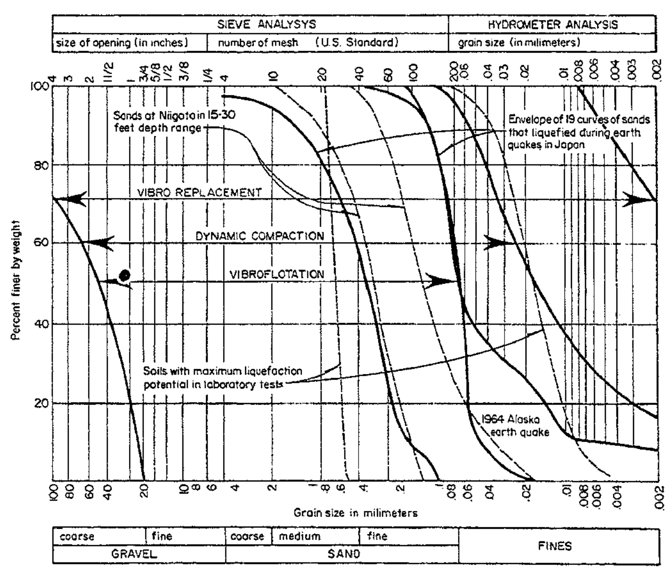

Figure 1. Range of soils treatable by vibratory techniques and range of liquefaction.

Degree of ground improvement that can be obtained

Dynamic compaction: The amount of energy applied per unit of surface area (E_e) is the main influence on the degree of improvement that can be obtained. Fig. 2 shows the improvement that can be achieved in terms of N (SPT) and qc (CPT), for different values of E_e.

With regard to the finest soils that are susceptible to liquefaction, such as fine silty sands and silts, not only the impact points and the number of blows applied to each one of them, should be taken into account, but also the position of the water table (WT) and the waiting time between each passing, in order to facilitate the dissipation of the pore pressure generated.

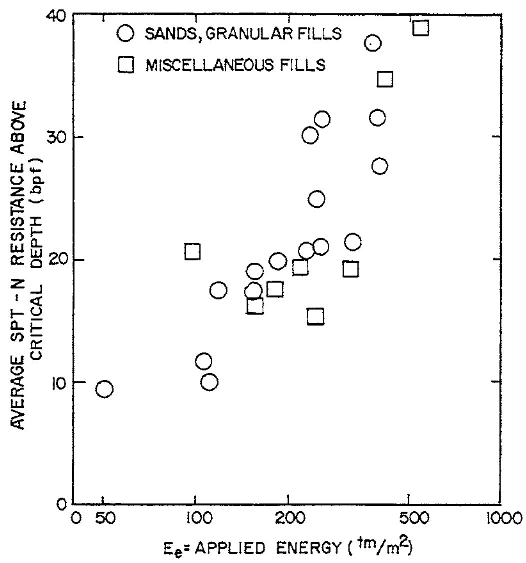

Figure 2. Observed Trend beetween SPT-N Value and Applied Energy per Unit Area (Mayne et al., 1984)

Vibroflotation: As long as the vibrator is working for long enough, the compaction that is achieved by applying this technique to soils lying below the WT, depends on the grain-size of the soil and the frequency with which the equipment is used and the power thereof. The farther away the vibrator is, the less effective the compaction; nevertheless, this lack of evenness is of no consequence if the increase brought about is sufficient at the point lying halfway between two consecutive vibration points. For clean sands, this increase can be estimated from Fig. 3. In the case of sands with a fines content of up to 15 or 20% and with a PI $\leq$ 10%, the same curve can be used as in the previous figure, as long as no increase takes place in the values for the corrected penetration resistance (N_1 and q_{c1}), as a function of the FC, for later calculation of the liquefaction resistance (τ_1/σ'_v).

Vibroreplacement: As in the case above, the densification that is obtained depends on the grain-size of the soil, the separation between the centres of the stone columns (2b) and the diameter (2a) of the columns (normally ranging between 0.6 and 1.0 m). The method given by Van Impe (1989) makes it possible to estimate the improvement obtained in terms of the relationship q_c after / q_c before, taking into account the factors mentioned (Fig. 4). However, our experience, shows that for fine soils (FC > 80-90%) the average improvement measured with the static penetrometer (CPT) between columns, is not adequate. It is better to use methods based on surface wave spectral analysis and to measure the efficiency of the treatment in terms of the relationship between the coefficients of compresibility before and after (m_v before / m_v after. Fig. 5 shows the values of the effectiveness obtained in a real situation (Medinaceli, Spain).

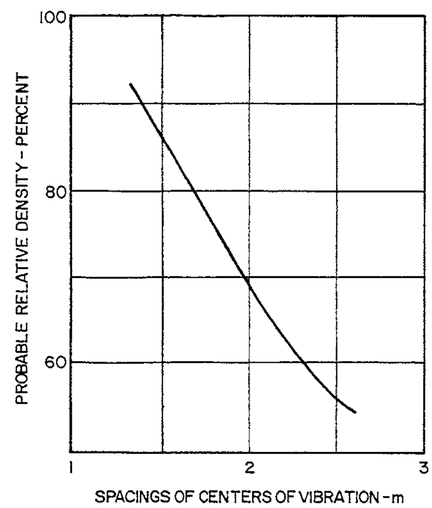

Figure 3. Relative Density of Clean Sand at Points Midway Between Centers of Vibration as a Function of Probe Spacing

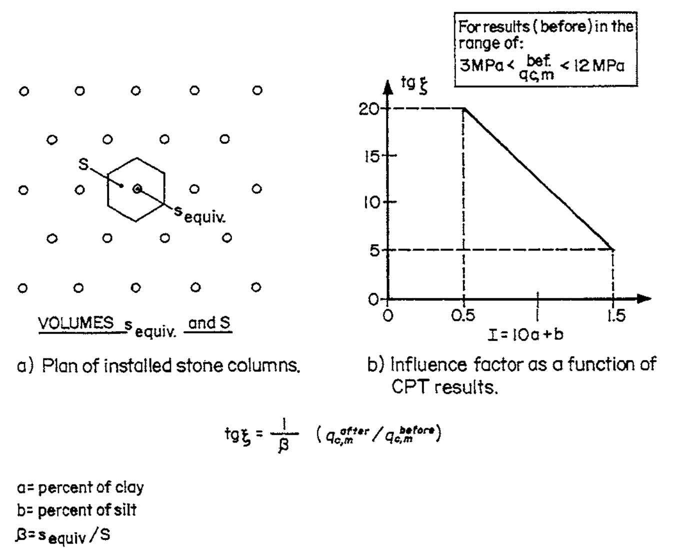

Figure 4. Method for the evaluation of the soil improvement by vibroreplacement (Van Impe, 1989).

The treatment gives rise to changes in the coefficient of apparent compressibility (m_v) and the number of cycles required to cause liquefaction (N_1). These parameters are basic for an analysis of the pore pressure generation-dissipation process brought about by the design earthquake, for a specific a/b relationship.

The variations of m_v can be obtained as approximations, from Fig. 6 after correlating the values of $q_{c\ after}$ with the relative density (D_r), using expressions such as those given by Jamiolkowski et al. (1985). Finally, the value of N_1 can be determined by applying the concept of the number of equivalent cycles and the liquefaction potential evaluation methods that are indicated in the next section, as a function of the values of $q_{c\ after}$ mentioned, or of N or V_s equivalents.

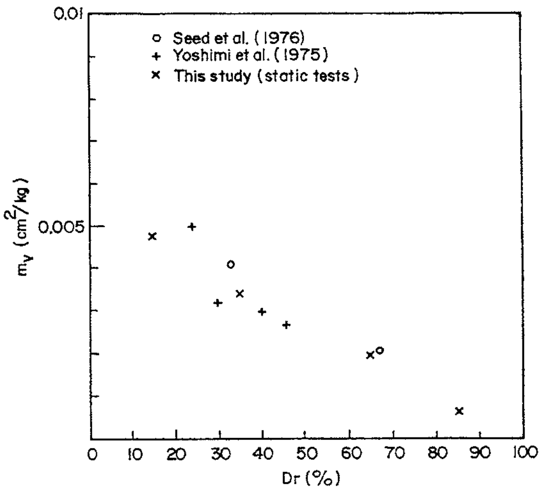

Figure 5. Aproximate relationship between coefficient of compressibility and relative density.

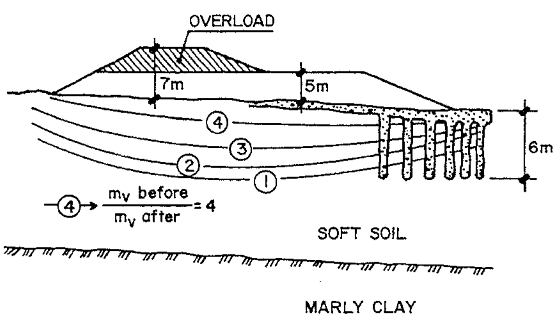

Figure 6. Improvement obtained under an embankment with short stone columns (Oteo and Sopeña, 1983).

GROUND IMPROVEMENT IN THE FACE OF LIQUEFACTION RISK

Deep compaction

When assessing the dynamic improvement caused by deep soil compaction, the following two aspects should be considered: a) Of the existing methods used in analysis of the liquefaction potential, the most advisable ones from a perspective of the usual practical application, are those referred to as "field methods", which correlate the behaviour observed in past earthquakes with the corrected penetration strength (N_1 or q_{c1}) or the corrected propagation velocity of shear waves V_{s1}; b) According to the experiences of the authors of this article, the τ_1/σ'_v of sandy soils compacted using vibratory techniques, varies in the same way with N_1, q_{c1} and V_{s1} as that of natural deposits in similar soils. This is verified basically for values of N_1 ranging from 20 to 30, although this trend may not be valid for low values of N.

As a result of the above, it is possible to use the field methods to calculate the safety factor (SF) in the original situation and to evaluate the degree of soil improvement, in terms of N, q_c or V_s, which must be done in order to increase this factor to safe values, now transformed into a coefficient of dynamic guarantee of improvement (DGI). These methods, for which a detailed analysis is given in the work by Armijo et al. (1994-a), are basically divided into two types: those which evaluate the τ_1/σ'_v and compare them with the cycle stress induced by the earthquake (τ_d/σ'_v), and those which establish critical values of the penetration strength (N_{crit} and $q_{c\ crit}$) and compare them with the "in situ" measurements.

Drainage Improvement

The effect that stone columns have on improving the drainage conditions can be studied using both experimental and analytical models. The former require the use of tests on large shaking tables.

As a result of the above, analytical models are more accesible and, in general, provide good results when the hypotheses upon which each one is based are taken into account. Of the existing analytical models, the most widely used are those of Seed and Booker (1977) and Millea (1990). Out of these two, the first one has been most extensively examined, and can be applied either through the graphs provided by the authors or by means of the computer program called GADFLEA.

The Seed and Booker model links the relationship between the pore pressure generated by the earthquake and the effective confinement pressure ($r_u = u_g/\sigma'_o$) with an adimensional time factor (T_{ad}) and with the relationship between the diameter and the distance between the stone columns (a/b), and between the number of equivalent cycles to the design earthquake and the number of cycles needed to bring about liquefaction (N_e/N_1). The latter parameter, together with m_v and a, which in addition to the duration of the main part of the earthquake (td), is included in T_{ad}, are related to the improvement that is achieved in the soil situated between the stone columns. In this case, the DGI coefficient must be calculated from r_u, by means of the expression given by Tokimatsu and Yoshimi (1983).

GROUND IMPROVEMENT DESIGN FOR PREVENTING LIQUEFACTION

Field methods for evaluating liquefaction potential must be used to determining the dynamic efficiency of the treatment DGI. These methods make it possible to obtain directly the above-mentioned coefficient, through the relationships τ_1/τ_d, N/N_{crit} or $q_c/q_{c\ crit}$, in cases where only the densification effect is considered, and indirectly, through N_1, when drainage improvement conditions are also considered.

However, it is no easy matter to evaluate the liquefaction potential using these methods, because such an assessment involves an important number of variables for both the ground and the earthquake under consideration, and the influence of each of those variables has to be taken into account in a different way. It is extremely complicated to aprioristically establish which method is the most suitable. From a practical perspective, this means that the problem either has to be dealt with by real specialists, or it is necessary to use a computer program of the expert system type, such as the one proposed by Armijo et al. (1994-b).

Furthermore, in designing a treatment to prevent liquefaction, it should be remembered that empirical methods are used when evaluating the degree of ground improvement in terms of N, q_c or V_s, and that such methods are of an approximate nature. Therefore, the values predicted on the basis of these methods, must be verified "in situ", by means of test areas that are representative of the real conditions.

In view of all the aforementioned, it is advisable that the ground treatment be designed so that values of $N_1 > 20$ (or the equivalents in terms of q_{c1} or V_{s1}) in methods DC and VF, are obtained after application. For densities associated with these values, the sands begin to show evidence of dilatant behaviour. Therefore, tolerable errors in the estimation of τ_1/σ'_v or τ_d/σ'_v, will not give rise to considerable changes in the strains caused by the earthquake. In such conditions, a GDI of about 1.3 or 1.5. would be sufficient. For lower values of N_1, the GDI must be in the 1.75 - 2.0 range.

In the case of soils treated with VR, the limit value indicated for N_1, refers to the equivalent soil that would be obtained when considering the overall effect of both densification and the stone columns. In practice, when working with GDI values of about 1.3, and with a view to ensuring that the results yielded by the Seed and Brooker method are sufficiently close, a network of stone columns must be designed in such a way that it can be guaranteed that r_u will not rise above 0.5.

CONCLUSIONS

The DC and VF methods can be effectively applied to most potentially liquefiable soils, with the exception of those with an FC > 15 to 20%. In the latter case the VR method ought to be used, because this combines the beneficial effect of increasing the overall strength, with an improvement in the drainage conditions.

The VF and VR methods can, without difficulty, reach depths of over the maximum for which liquefaction took place in previous earthquakes. However, the equipment that is normally used in the DC system cannot reach these maximum depths, being only capable of reaching the depths at which liquefaction most commonly occurs, i.e., 3 to 7 m.

If one is to aprioristically determine the degree of ground improvement that can be achieved with these techniques, a series of empirical studies has to be performed, which takes into account not only the properties of the soils, but also the E_e in the case of the DC, the distance between the vibration points in the case of the VF and this latter parameter and the diameter of the stone column, in the case of the VR system. Nevertheless, all these methods are only approximate in nature, and the values obtained from them must be subjected to "in situ" verification.

The DGI must be obtained using the field methods for analyzing the liquefaction potential. These methods allow either for a direct evaluation of the coefficient mentioned, when only the densification effect is considered, or for an indirect evaluation, when the improvement of drainage conditions is also being taken into account. In the latter case, the Seed and Booker method can be used in designing the stone column network, because this method provides results that are sufficiently accurate, as long as the design is made for r_u that are below 0.5, and a correct evaluation has been carried out for the values of m_v and N_1 in the ground lying between the stone columns; this can be done by following procedures such as those indicated by the authors of this article.

Given the approximations involved in the indicated methods for predicting the degree of improvement to be obtained and the liquefaction potential, ground treatment should be designed to obtain values of $N_1 > 20$ after application. In these conditions it is enough to take a DGI of about 1.3 to 1.5, and if such conditions are not fulfilled, the DGI ought to range from 1.75 to 2.0.

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