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Seismic liquefaction performance of strip foundations: Effect of ground improvement dimensions

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ABSTRACT

Evidence from field case studies, as well as from experimental and theoretical research, suggest that the detrimental effects of seismic liquefaction on the performance of surface foundations on level ground may be mitigated in presence of a non-liquefiable soil crust of adequate dimensions and shear strength. This paper refers to the case where the non-liquefiable crust is not natural, but it has been artificially created by ground improvement, and focuses upon the effect of the size (thickness and width) of the improved area on the seismic settlement and the degraded post-seismic bearing capacity of the foundation. Size effects are evaluated numerically, starting from the reference case of an infinitely extending improved soil crust and progressing to more realistic cases of ground improvement of gradually decreasing lateral extend of improved soil. Guidelines are provided for a cost-effective design of ground improvement, based on the rate of seismic settlement reduction with increasing dimensions of the improved ground.

1. Introduction

Modern seismic codes request that structures located in areas of high liquefaction potential are not founded at shallow depth, as the particular foundation option is prone to excessive seismic settlements and temporary bearing capacity loss. Nevertheless, there is sufficient evidence from field case studies, as well as from experimental and theoretical research (e.g. [1–6]), which shows that the above detrimental effects may be mitigated in presence of a non-liquefiable surface layer (crust) of adequate dimensions and shear strength. This layer may be either natural (e.g. clay or dense gravel above the water table) or artificially created by ground improvement. In the latter case, that is considered herein, the thickness and width of the improvement must be optimally determined, in order to ensure viable foundation performance and cost-effective design.

The reviewed literature, briefly presented in the following sections, reveals only a limited number of studies which address the practical problem of the required ground improvement dimensions (e.g. [7–9]). A common feature of these studies is the assumption that ground improvement extends to the entire liquefaction depth, so that only the width of the improved zone needs to be specified. Nevertheless, there are cases of thick liquefiable layers where the above assumption will lead to over-conservative and costly improvement solutions, as the foundation performance may be also ensured by partial with depth improvement of the liquefiable layer. For instance, the centrifuge test results of Liu and Dobry [10] show that footing settlements are reduced by about 60% when ground improvement is extended to 50–55% of the liquefiable layer thickness, while the reduction increases marginally to 66% when the improvement depth is further extended to the entire thickness of the liquefiable layer. In qualitative terms, the same trend is also exhibited by the large majority of the case studies compiled by Sitar and Hausler [5], which show that extending the ground improvement to more than about 60% of the liquefiable layer thickness has a minor additional effect upon the accumulation of footing settlements.

Working towards a more general performance based design approach for shallow foundations on liquefiable sand, the seismic response of strip foundations on level ground has been analyzed numerically, by means of elasto-plastic dynamic analyses which may take consistently into account the effects of excess pore pressure build up and liquefaction in the foundation soil. The first stage of that study ([11,12]) focused upon the evaluation of the seismic settlements ($\rho_{dyn}{}^{inf}$) and the degraded post-seismic bearing capacity of the foundation soil ($q_{ult,deg}{}^{inf}$) for the reference case of a non-liquefiable cohesionless and highly permeable soil crust of infinite lateral extent. It should be acknowledged that this is a rather unusual case when

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considering an artificially created crust of improved ground, but it may be applicable at sites with a natural non-liquefiable top soil layer, such as a layer of dense gravel or sand combined with a deep water table.

The second stage of the study, which is presented herein, examines the more common case, where the liquefiable soil is improved over a prescribed thickness (H_{imp}) and width (B_{imp}) around the foundation. To reduce the number of required numerical analyses, and also to remain consistent with the previous research stage, the effect of ground improvement dimensions on seismic settlements (ρ_{dyn}) and degraded bearing capacity ($q_{ult,deg}$) is evaluated with reference to the results for the infinitely extending non-liquefiable crust. An additional benefit from this practice is that the interpretation of the results may focus on the dimensions of ground improvement (B_{imp} & H_{imp}), accepting in fact that the effect of the remaining (soil, excitation and foundation) problem parameters is included in the adopted relations for the reference case of ground improvement of infinite width. On that basis, correction factors are established, in the form of analytical relations and design charts, for the computation of ρ_{dyn} and $q_{ult,deg}$ in terms of the associated values for the reference case $\rho_{dyn}{}^{inf}$ and $q_{ult,deg}{}^{inf}$. Additionally, guidelines are provided for a cost-effective design of ground improvement, for level ground conditions, based on the rate of seismic settlement reduction with increasing volume of the improved ground.

To aid independent reading of the paper, the analytical expressions for the reference seismic settlements and degraded bearing capacity values ($\rho_{dyn}{}^{inf}$ and $q_{ult,deg}{}^{inf}$) are briefly outlined in Appendix A.

2. Review of previous studies

Assessing the dimensions of ground improvement around shallow foundations appears to be a rather complex task, with relatively limited references in the literature. In the guidelines for the seismic design of oil tanks, issued by the Japanese Fire Defense Agency [7], it is recommended that soil improvement should be extended to a distance SL on both sides of the footing, equal to two thirds of the improvement depth within the 5–10 m range (Fig. 1). Tsuchida et al. [8], also referred by Hatanaka et al. [1], suggest that the width of the improved area should be such that the inertia forces from the structure are counterbalanced by the passive resistance of the non-liquefiable compacted area. This is schematically explained in Fig. 2, where the liquefaction affected area is assumed to be located beyond the active failure line (defined by the active failure angle α_2).

Attention is finally due to the work of Iai et al. [9], adopted by many contemporary guidelines (e.g. [13–16]), which relates the required improvement width to the spatial distribution of excess pore water pressures below the shallow foundation. More specifically, based on laboratory data, it is assumed that excess pore pressure build-up within the compacted area leads to significant shear strength loss only when the excess pore pressure ratio, $r_u = \Delta u/\sigma'_{vo}$, exceeds 0.50. Furthermore, based on shaking table tests and seepage flow analyses, the area of reduced shear strength along the perimeter of the improved zone,

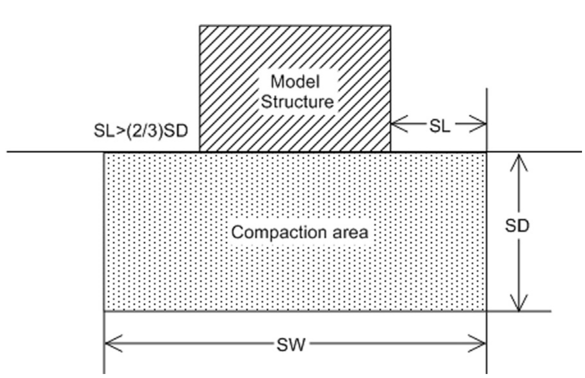

Fig. 1. Specification of minimum improvement width for tank foundations (JDFA, [7]).

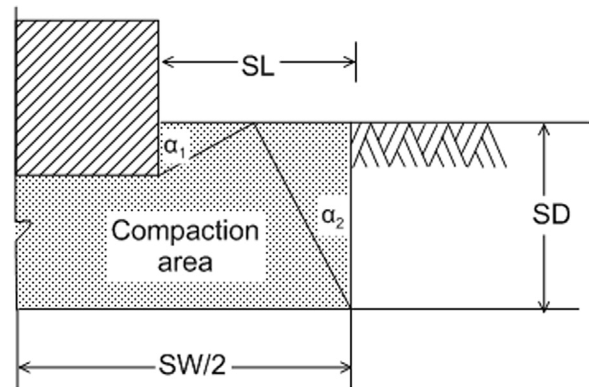

Fig. 2. Specification of minimum improvement width based on soil friction angle (Thuchida et al. [8]).

where r_u values exceed 0.50, is defined as a rectangle (ABCD in Fig. 3**a**). Special emphasis should be given to triangle ACD, which exhibited unstable response in shaking table tests and should be thus considered completely liquefied. Along the vertical interface of the improved and the unimproved soil, both dynamic and static earth pressures are applied. The static pressure component corresponds to an earth pressure coefficient $K_o = 1$, while the (positive and negative) dynamic pressure components are computed as noted in Figs. 3**a** and **b**. The bearing capacity of a shallow foundation on partially improved ground is solely attributed to the shear resistance which is mobilized along failure surface EFG. The least horizontal pressures (i.e. static minus dynamic) from the liquefied sand, applied on the GG' interface, contribute to the stability of the structure and may be also included in the stability analysis.

As mentioned in the introduction, all above guidelines request that the liquefiable layer is improved throughout its depth, overlooking essentially more recent evidence (referenced in the introduction), which show that the foundation performance may be ensured even with partial with depth ground improvement. Hence, the present study aims to remove this limitation and establish the means for assessing the liquefaction performance of strip foundations under conditions of limited width and also partial with depth ground improvement.

3. Numerical methodology outline

Similar to the reference numerical study for a non-liquefiable crust of infinite lateral extent ([11,12]), 2-dimensional dynamic effective stress analyses with parallel water flow were performed, with the finite difference code FLAC v7.0 [17]. The NTUA-SAND Critical State constitutive model ([18,19]) was adopted for the monotonic and cyclic response of the liquefiable soil layer and the non-liquefiable crust. The general problem configuration is presented in Fig. 4: a strip footing of width B and average contact pressure q rests on a liquefiable uniform sand layer of total thickness H_{total}, initial relative density $D_{r,o}$ and permeability coefficient k_{sand}. The soil below the foundation is locally improved against liquefaction with vibro-compaction and gravel drain installation, so that the free field excess pore pressures during shaking increase up to approximately 20% of the associated vertical effective stress ($r_u = 0.20$). To simplify the numerical model, this area was simulated as an equivalent-uniform cohesionless soil mass of thickness H_{imp} ($< H_{total}$) and width B_{imp} ($\geq B$), with increased relative density ($D_{r,imp} > D_{r,o}$) and permeability ($k_{eq} > k_{sand}$). The lateral and vertical dimensions of the foundation soil were specified following sensitivity analyses, so that boundary effects on the foundation response were essentially eliminated [11].

The imposed excitation consisted of N harmonic cycles with period T_{exc} and acceleration amplitude α_{max}. Furthermore, the seismic excitation was applied at the base of the liquefiable sand layer, implying