論 文

Consideration of Ground Deformation Characteristics in Vacuum Consolidation and Application for Design

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ABSTRACT: Experience from vacuum consolidation practice revealed discrepancy between actually measured and predicted settlement and strength increment obtained by conventional designs, which can be overestimated or underestimated depending on the depth involved. It addresses the necessity of proper consideration of ground deformation under this peculiar loading. By considering the change in lateral earth pressure condition associated with inward compression of the ground under application of vacuum load, effective stress increment in the soil at any treatment depth and their distribution are established, which indicates a variation of deformation characteristics along depth. Based on that, equations have been derived for approximately estimating the vacuum induced displacement and strength increment with appropriate deformation at the corresponding depth being taken into account. On the other hand, considering the influence of vacuum consolidation beyond the treatment boundary, an approximation method is proposed to evaluate the influence zone and predict the displacement at various distances within that zone. In this paper, applicability of the proposed approach and equations are examined using data from several actual vacuum consolidation cases. As a result, some modification for the input in proposed equations are suggested for a better agreement between calculated and measured data. It suggests about the usefulness of the approach at least in preliminary design.

Keywords: Soft ground improvement, vacuum consolidation, deformation characteristics, active lateral earth pressure, effective stress, negative pore water pressure

1 Introduction

Vacuum consolidation technique initially proposed by Kjellman in 1952 [6] is based on the idea of applying vacuum suction to an isolated soil mass to reduce the atmospheric pressure in it, and by that way, without changing the total stress, the soil effective stress increases due to decrease in pore water pressure and soil consolidation takes place. With lots of effort having been put in development and improvement, the technique has become an effective ground improvement method. In the design of vacuum consolidation, soil deformation under vacuum loading is commonly referred as isotropic compression regarding soil effective stress increments being identical in all directions due solely to decrease in pore water pressure according to principle of vacuum preloading established so far in number of publications (Cognon et al., 1996 [3] and Liu 1996 [7]…). Consequently, a "*q-constant line*" stress path on the *q-p'* space is often used for representing the process uniquely irrespective of depth. However, practical experiences revealed various aspects needed to be clarified, including soil deformation characteristics and strength increment during vacuum consolidation.

Imai et al. [4] are among the first who pointed out the problem of variation of vacuum induced ground deformation characteristics along the treatment depth and suggesting correction factors for vertical deformation at different depths. They proposed that, in vacuum consolidation free lateral contraction is allowed only near the surface where nearly isotropic compression may take place, while near the bottom of improved area, lateral displacement is restrained due to large confinement by the surrounding soil, thus compression should be likely one-dimensional. However, how the confinement controls the deformation variation had not been clarified yet.

Mitachi et al. [9] also stopped at acknowledging the variation of deformation with depth for a reason that the effect of vacuum increment becomes not essential in

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relation to very large initial effective stress at a great depth, so that deformation condition would be remained almost one-dimensional as initial.

Acknowledging the need to design vacuum consolidation as close as possible to the actual deformation characteristics, which had not been evaluated by any previous study by that time, the authors of this paper proposed an approach, which takes into account the change in lateral earth pressure induced by lateral compression of the treated soil under vacuum consolidation in estimating the vacuum induced vertical and horizontal effective stress increments. Based on that, the stress increment ratio ($\Delta\sigma'_h/\Delta\sigma'_v$) can be established for any depth to numerically evaluate the continuous variation of ground deformation condition. Therefore, displacement and strength increment could be estimated in proper deformation at any given depth, based on elastic stress-strain relationship for the purpose of simplification and approximation. Furthermore, the approach was also extended to evaluate the influence of vacuum consolidation beyond the treated area onto the surrounding ground. The authors of this paper had discussed the theoretical considerations of this approach with Prof. Imai, who was an expert in this field and being the Chairman of Japan Vacuum Consolidation Association at that time. Consequently, this proposed approach was introduced for the first time in September 2005 in Imai's paper [5] published in Japanese.

In December 2005, Chai et al. [1] published a study of the ground deformation induced by vacuum consolidation. Although lateral earth pressure variation was considered, however, they did not come to establish the stress increment, explaining that deformation in vacuum consolidation is mostly plastic and the direction of deformation is mainly influenced by the stress state. Therefore, evaluation of deformation along depth had been made with assumptions, and the suggested correction factor for vertical displacement was obtained by semi-empiricall method. Dealing with lateral displacement, they suggested calculating that simply by multiplying the lateral strain to the half width of treated area. That lateral strain is averaged as obtained by subtracting the vertical strain from total volumetric strain (assumed the same as in 1-D consolidation). However, that study reported that the displacement predicted in that way is overestimated especially for the near surface layer.

Since in Imai paper [5] our proposed approach was introduced theoretically without verification the necessity of examination on its applications is addressed due to various assumptions and simplifications used in development. Thoroughly examination by comparing calculated and measured data from several actual vacuum consolidation cases demonstrated the suitability of the approach in predicting settlement and strength increment in treated area, while it revealed a possible cause of largely overestimating of lateral displacement, which led to modification of the equation of lateral strain for a better prediction. The paper is organized with reviewing the approach in Sections 2 and 3, and its application examination in Section 4.

2 Considerations on ground deformation characteristics in vacuum consolidation

Evidences from vacuum consolidation design practice suggest about the variation of soil deformation characteristics along the vacuum treated depth. From a point of view that soil deformation characteristics can be represented through the ratio of effective stress increment, $\Delta\sigma'_h/\Delta\sigma'_v$, if the vacuum induced negative pore pressure were the only stress component contributed to the stress increment as commonly considered, $\Delta\sigma'_v = \Delta\sigma'_h = -\Delta u$, then $\Delta\sigma'_h/\Delta\sigma'_v = 1$ and the deformation would be isotropic all over the treatment depth. However, if a change in lateral earth pressure associated with vacuum induced lateral compression had been incorporated into the horizontal effective stress increment ($\Delta\sigma'_h$), it could be the reason that makes $\Delta\sigma'_h$ differed from $\Delta\sigma'_v$, and thus causes continuous variation of $\Delta\sigma'_h/\Delta\sigma'_v$ along depth. Once $\Delta\sigma'_h$ and $\Delta\sigma'_v$ are able to be determined, calculations of displacement and strength increment at any depth with appropriate deformation become possible.

2.1 Soil stress conditions under vacuum consolidation

Under applying vacuum pressure to an isolated soil mass, the initial soil stress conditions inside and outside the treated area are changed. First, the vacuum causes pore pressure decreased (assuming $-\Delta u = P_v$), aud

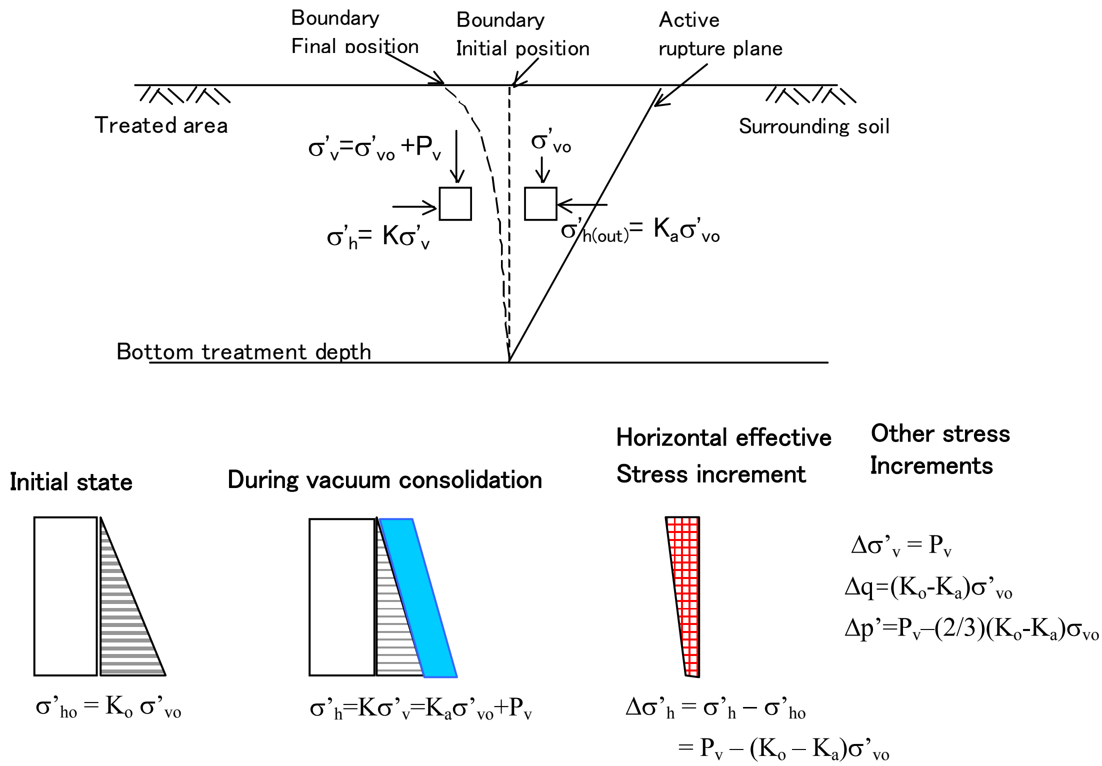

Fig. 1 Conceptual consideration of lateral pressure condition in vacuum consolidation

vertical and horizontal effective stresses are increased by the same amount for the soil inside the treated area, but not for the surrounding soil. Second, as the treated soil is compressed laterally, the imaginary vertical boundary between treated and surrounding soils is moved (deflected) away from its initial position, allowing the soil in surrounding area laterally decompressed spontaneously as in extension. In such circumstance, the lateral stress condition in either sides of that boundary keeps changing, toward a passive state inside the treated soil and toward an active state in the surrounding soil, until new equilibrium in horizontal forces is established and that vertical boundary stops displacing. Figure 1 depicts soil elements inside and outside the vertical boundary of vacuum treated area and vertical and horizontal stresses acting on them at the equilibrium state of lateral earth pressure.

Inside the treated soil mass, with lateral pressure coefficient $K>K_o$ by the time of equilibrium, the vertical and horizontal effective stresses are as following:

$$\sigma'_v = \sigma'_{vo} + P_v \qquad (1)$$

$$\sigma'_h = K\,\sigma'_v. \qquad (2)$$

Meanwhile, if a fully active lateral state (tensile crack had been initiated) is assumed in the surrounding soil, hen the horizontal effective stress $\sigma'_{h(a)}$ is related to the unchanged vertical effective stress σ'_{vo} by active lateral earth pressure coefficient K_a:

$$\sigma'_{h(a)} = K_a\,\sigma'_{vo} \qquad (3)$$

Now, the equilibrium state of total horizontal stress can be expressed by following equation:

$$K\sigma'_v + (u_o - P_v) = K_a\,\sigma'_{vo} + u_o \qquad (4)$$

where u_o is initial pore water pressure.

Thus, lateral earth pressure coefficient K and horizontal stress on a soil element in treated area at the equilibrium state can be determined as followings:

$$K = (K_a\,\sigma'_{vo} + P_v) / \sigma'_v \qquad (5)$$

$$\sigma'_h = K\sigma'_v = K_a\,\sigma'_{vo} + P_v \qquad (6)$$

At any treatment depth, vertical and horizontal stress increments are established by following equations and depicted in Fig.1:

$$\Delta\sigma'_v = \sigma'_v - \sigma'_{vo} \quad = P_v \qquad (7)$$

$$\Delta\sigma'_h = \sigma'_h - \sigma'_{ho} \quad = P_v - (K_o - K_a)\,\sigma'_{vo} \qquad (8)$$

2.2 Varying deformation characteristics along treatment depth

Ratio of effective stress increment $\Delta\sigma'_h / \Delta\sigma'_v$ denoted hereafter as I is used for characterizing deformation condition:

$$I = \Delta\sigma'_h / \Delta\sigma'_v = 1 - (K_o - K_a)\,\sigma'_{vo} / P_v \qquad (9)$$

Eq.(9) detects the variation of deformation condition along depth continuously. It implies that the true isotropic condition ($I = 1$ or $\Delta\sigma'_h = \Delta\sigma'_v$) generally does not exist, but a nearly isotropic condition is valid near the surface when σ'_{vo} is very small.

Now, mean and deviator stress increments can be established and expressed through I as:

$$\Delta p' = (\Delta\sigma'_v + 2\,\Delta\sigma'_h) / 3$$
$$= P_v - (2/3)(K_o - K_a)\,\sigma'_{vo}$$
$$= [(1+2I)/3]\,P_v \qquad (10)$$
$$\Delta q' = \Delta\sigma'_v - \Delta\sigma'_h$$
$$= (K_o - K_a)\,\sigma'_{vo}$$
$$= (1 - I)\,P_v \qquad (11)$$

Dependency on overburden σ'_{vo} (or depth) of horizontal, mean and deviator effective stress increments expressed in Eqs.(8) ~ (11) explain why vacuum consolidation is not always necessarily isotropic, therefore the *q-constant* stress pass should not uniquely represent the process at every depth. Figure 2(a) illustrates possible stress paths on *q-p'* space: a near *q-constant* line for a near surface depth (I), a near *K_o-line* for a near bottom depth (III) and any one declined from those two for a middle depth (II).

To define the depth of resuming lateral constrain deformation, where a stress condition of $K = K_o$ or $\sigma'_h/\sigma'_v = \sigma'_{ho}/\sigma'_{vo} = \Delta\sigma'_h/\Delta\sigma'_v$, hence $K = K_o = I$ is required, the above Eq.(9) is rearranged for the overburden with assigning I the value of K_o as follows:

$$\sigma'_{vo\,(Ko)} = P_v - (1-K_o)/(K_o-K_a) \qquad (12)$$

Figure 2(b) illustrates the distribution curves of K and I with depth, in which the intersection of these curves represents the K_o-condition. The depth of lateral constrain, $z_{(Ko)}$, is where the overburden $\sigma'_{vo(Ko)}$ satisfies Eq.(12), which can be approximated by Eq.(12b) assuming constant unit weight γ' over the treatment depth:

$$z_{(Ko)} = P_v\,(1 - K_o) / \gamma'\,(K_o - K_a) \qquad (12b)$$

For most treated soil with γ'= 6 kN/m^3, K_o = 0.5, K_a = 0.33, if P_v = 50, 60, 70 or 80 kN/m^2, the depth $z_{(Ko)}$ is approximately 25, 30, 35 or 40m, respectively. Although $z_{(Ko)}$ is rather dependent of vacuum pressure, it might be approximated to bottom drain depth ($z_{(Ko)} \approx H_d$) in cases of deep treatment.

2.3 Strain and displacement in the treated soil mass

Elastic stress-strain relationship (Hook's Law) is used for estimating strain increment in the treated soil area, which is for the purpose of simplifying but not to indicate that the strain behavior is recoverable:

$$\varepsilon_z = (1/E) * \{\Delta\sigma'_v - 2\nu * \Delta\sigma'_h\}$$
$$\varepsilon_h = (1/E) * \{(1-\nu)\,\Delta\sigma'_h - \nu * \Delta\sigma'_v\}$$

By expressing the Young's modulus E through 1-D volume compressibility $m_v = (1-2\nu)(1+\nu)/(1-\nu)E$ and the Poisson's ratio $\nu = K_o/(1+K_o)$, and assigning $\Delta\sigma'_v$ and $\Delta\sigma'_h$ values obtained by Eqs.(7) & (8), respectively, following equations can approximate the vertical and horizontal strains in the treated soil:

$$\varepsilon_z = m_v P_v * [(1+K_o) - 2K_o I]/[(1+2K_o)(1-K_o)]$$
$$= \alpha_z\, m_v\, P_v \qquad (13)$$
$$\varepsilon_h = m_v\, P_v * [I - K_o]/[(1+2K_o)(1-K_o)]$$
$$= \alpha_h\, m_v\, P_v \qquad (14)$$

where, the values of α_z and α_h expressed by following Eqs.(15) & (16) are understood as vertical and

a) Possible stress paths along vacuum treatment depth

b) K & I profiles along vacuum treatment depth

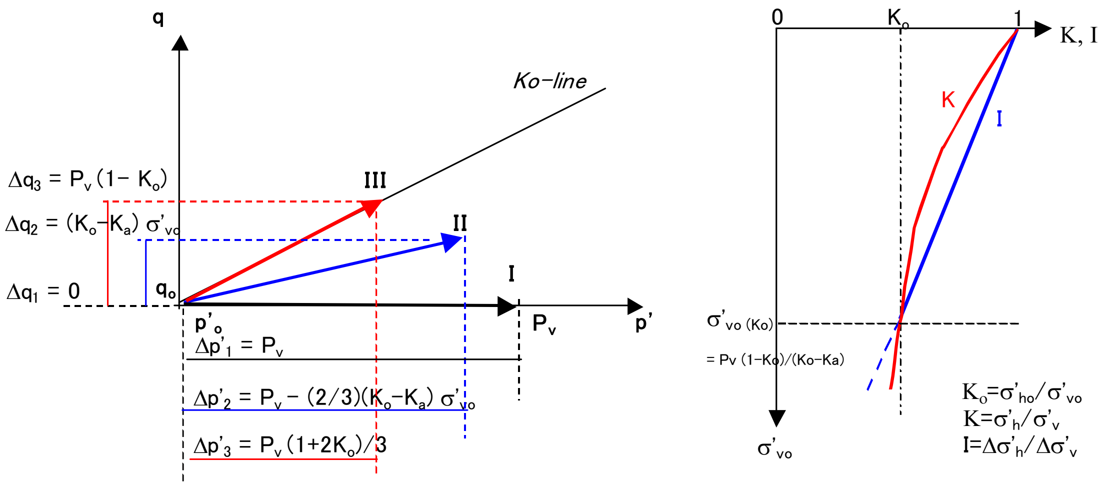

Fig. 2 Stress paths, K and I profiles along depth

horizontal correction factors for the variation of deformation characteristics with depth:

$$\alpha_z = [(1+K_o) - 2K_oI]/[(1+2K_o)(1-K_o)] \quad (15)$$

$$\alpha_h = [I - K_o]/[(1+2K_o)(1-K_o)] \quad (16)$$

The surface settlement S would be summed up from vertical compression of all soil layers subjected to vacuum treatment. For a given layer of thickness Δz, the vertical compression δ_z is approximated by the following equation:

$$\delta_z = \alpha_z m_v P_v \Delta z \quad (17)$$

where α_z, m_v and P_v are estimated for the mid-point depth and assumed constant for the given layer.

If at any given depth z, lateral strain ε_h (Eq.14) is assumed uniform over the half-width L of the treated area, then inward lateral displacement at the boundary is simply approximated as following:

$$\delta_{h(z)} = \varepsilon_{h(z)} * L$$
$$= \alpha_{h(z)} m_{v(z)} P_{v(z)} L \quad (18)$$

2.4 Vacuum induced strength increment and change in void ratio

Soil strength (c_u) is a function of mean effective stress, and its increment Δc_u is directly related to the increment in mean effective stress ($\Delta p'$), which is described by the relationship $c_u/p' = c_{uo}/p'_o$, or $\Delta c_u = c_{uo} * (\Delta p'/p'_o)$, where c_{uo} is initial soil strength and p'_o is initial mean effective stress.

Now, with value of $\Delta p'$ given by Eq.(10), while $p'_o = [(1+2K_o)/3]\,\sigma'_{vo}$, the above relation is written as:

$$\Delta c_u = (c_{uo}/\sigma'_{vo}) * [(1+2I)/(1+2K_o)] * P_v$$
$$= m * \beta * P_v \quad (19)$$

Notice that in Eq.(19), the value $m = c_{uo}/\sigma'_{vo}$ is nothing but the strength increment ratio conventionally used in 1-D consolidation design. And coefficient β involved in Eq.(19) can be considered as a correction factor for variation of deformation condition along depth with regard to 1-D consolidation because:

$$\beta = (1+2I)/(1+2K_o) \quad (20)$$

That means when appropriate deformation is taken into account, the vacuum induced strength increment at any treatment depth differs from that evaluated by the 1-D conventional consolidation design by a factor β. For example, for most treated soils with $K_o = 0.5$, the strength increment near the surface could be 1.5 times greater than that evaluated by the 1-D equation.

Similar to the strength increment, the change in void ratio Δe due to vacuum consolidation is mean effective stress dependent and estimated by using the relationship $\Delta e = -\lambda * \Delta p'/p'_o$, where $\lambda = -0.434\,C_c$. Since $\Delta p'/p'_o$ is varied with deformation characteristics as:

$$\Delta p'/p'_o = \beta * (P_v/\sigma'_{vo}) \quad (21)$$

The equation for evaluating Δe with appropriate deformation through correction factor β is:

$$\Delta e = \beta * 0.434C_c * (P_v/\sigma'_{vo}) \quad (22)$$

3. Influence of vacuum consolidation to surrounding soil

This approach considered the main deformation mechanism in surrounding soil as crack-like failure induced by large tensile strain under reduced effective lateral stress associated with lateral movement of the boundary of a treated soil mass. The deformation can be evaluated if the area subjected to extension, and the tensile strain (ε_{ext}) distribution over that are known.

3.1 Extension zone and active zone

An approximation method for analyzing deformation of surrounding soil is proposed as illustrated in Fig. 3 and described hereafter.

Considering lateral displacement in surrounding soil induced by corresponding lateral compression of the treated soil, an area called *Extension zone* where extension may develop is conservatively assumed to be confined by the soil's natural stable slope surface (line B'R), which is defined by the soil internal friction angle ϕ' and passes the vertical boundary (line BB') at the point of zero lateral displacement B' ($z = z_{(ko)}$). By approximating $z_{(ko)} \approx H_d$, the width of the extension zone at depth z, denoted as $X_{ext\,(z)}$, is estimated as following:

$$X_{ext\,(z)} = [H_d - z]\tan(90^o - \phi') \quad (23)$$

The *Active zone* (area BB'A) where tensile cracks are confined is a part of the extension zone enclosed by the vertical boundary and active failure plane. The width of the active zone at depth z, denoted as $x_{a(z)}$, is estimated according to Rankin's theory:

$$x_{a(z)} = [H_d - z]\tan(45^o - \phi'/2) \quad (24)$$

Eqs.(23) and (24) suggest that for most treated soils with ϕ' varied at 20-30^o, tension cracks would be expected within a distance of 0.6~0.7H_d, though displacement could be observed at distances as far as

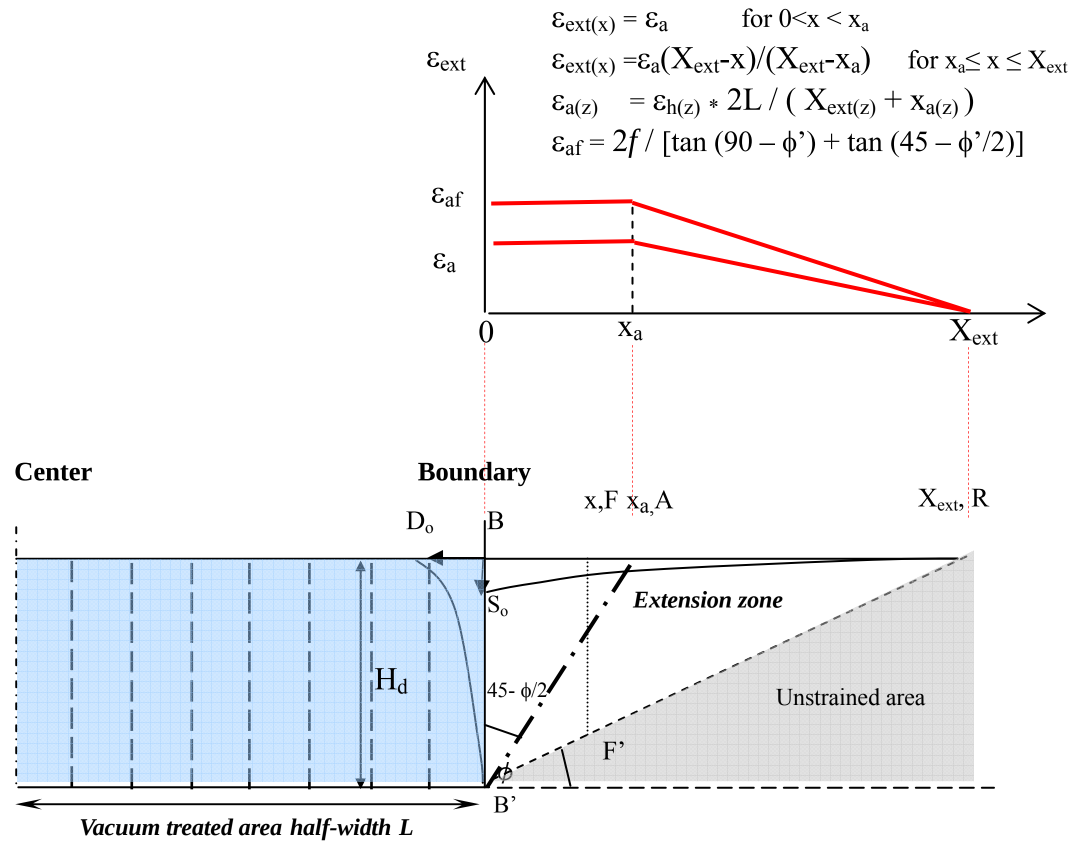

Fig. 3 Consideration of influence of vacuum consolidation on surrounding soil

1.7~2.8 H_d. This seems quite reasonable as experience from actual vacuum consolidation sites indicated that tension cracks are confined at a few meters from the boundary, while lateral displacement could develop at distances as far as about 2-3 times of vertical drain length H_d in the surrounding area.

3.2 Lateral displacement in surrounding soil

A simple bi-linear distribution of lateral extension strain $\varepsilon_{ext(x)}$ in horizontal direction from the boundary ($x = 0$) to the end of the extension zone ($x = X_{ext}$) is assumed: 1) constant within the active zone at strain ε_a, and 2) linearly decreases beyond the active zone from ε_a to zero (see Fig. 3), expressed by following equations respectively:

$$\varepsilon_{ext(x)} = \varepsilon_a \qquad \text{for } 0 \leq x \leq x_a \qquad (25a)$$

$$\varepsilon_{ext(x)} = \varepsilon_a * (X_{ext} - x) / (X_{ext} - x_a) \qquad \text{for } x_a \leq x \leq X_{ext} \qquad (25b)$$

where ε_a is active extension strain developed in accordance with ε_h at the given depth z.

Lateral displacement at any point x is calculated by integration of $\varepsilon_{ext(x)}$ from x to X_{ext}. Lateral displacement at the boundary $D_{(0)}$ shall be maximal as a total extension of the extension zone. A set of equations 26(a, b & c) is given for displacement at different range of x:

$$D_{(0)} = (\varepsilon_a/2)(X_{ext} + x_a) \qquad \text{for } x = 0 \qquad (26a)$$

$$D_{(x)} = (\varepsilon_a/2)(X_{ext} + x_a - 2x) \qquad \text{for } 0 \leq x \leq x_a \qquad (26b)$$

$$D_{(x)} = (\varepsilon_a/2)(X_{ext} - x)^2/(X_{ext} - x_a) \qquad \text{for } x_a \leq x \leq X_{ext} \qquad (26c)$$

As long as the soil is still intact (before cracking), at any depth z the total extension of the extension zone should completely compensate the total lateral compression of the treated soil. Therefore, by equalizing $D_{(0)}$ (eq.26a) to $\delta_{h(z)}$ (eq.18), the magnitude of strain ε_a can be determined as following:

$$\varepsilon_a = \varepsilon_h * 2L / (X_{ext} + x_a) = \varepsilon_h * 2L / [(H_d - z)T] \qquad (27)$$

where $T = [\tan(90^o - \phi') + \tan(45^o - \phi'/2)]$

It should be reminded that, when using equations from 25(a & b) to (27), all involved parameters (including $\varepsilon_{ext(x)}$, ε_h, $D_{(x)}$, X_{ext}, x_a) shall be evaluated for corresponding depth z, though the suffix z indicating depth has been omitted for simplification.

It is supposed that ε_a is increased with increasing ε_h during consolidation, and would ultimately reach an active failure strain (denoted as ε_{af}) as the soil commenced cracking. After cracking the soil beyond the cracks would no longer be strained. Thus, a condition of

$\varepsilon_a \le \varepsilon_{af}$ is applied in predicting lateral displacement.

The active failure strain ε_{af} is soil type dependent. According to practice designs of retaining walls (see Reference [11]), to achieve a full active lateral condition in the soil, the wall should be laterally displaced for a distance proportional to the wall height. The proportion coefficient is soil type dependent, varying from 0.02-0.05 for clayey soils to 0.005 for sand. Applying to the case of vacuum consolidation, where the vertical boundary is assumed as the wall of a height H_d, the above specification can be formulated as:

$$D_{af} = f * H_d \qquad (28)$$

where D_{af} is displacement at the top of the vertical boundary at the time of active failure, and f is the proportion coefficient.

Substituting D_{af} for $D_{(0)}$ and ε_{af} for ε_a in Eq.(26a) and rearranging it for ε_{af}, then:

$$\varepsilon_{af} = 2D_{af} / (X_{ext} + x_a)$$
$$= 2f / [\tan(90-\phi') + \tan(45-\phi'/2)] \qquad (29)$$

Notice that, the value of ε_{af} given by Eq.(29) is not affected by depth, but only by soil type. Since most of the soils being treated by vacuum consolidation are soft silty clay to clayey silt, it is reasonable to select f = 0.02-0.03. Therefore, for $\phi' = 25\sim30^{o}$, the active failure strain ε_{af} is about 2–4%. Anyway, suitable f value shall be adjusted based on site experience for a better design.

3.3 Vertical displacement in surrounding soil

Ideally assuming a condition of no volume change within the extension zone, any deformation in vertical direction would have been solely converted from corresponding deformation in horizontal direction according to principle of deformation compatibility. The "no volume change" condition is 2-dimensionally expressed by equalizing the integration of $D_{(0,z)}$ along depth z (for $z = 0\sim H_d$) (i.e. the area BB'D_o in Fig.4) to the integration of surface settlement $S_{(x)}$ along distance x (for $x = 0\sim X_{ext}$) (i.e the area BRS_o in Fig. 4). Thus, for a given point x on the surface there would be a corresponding point z* along depth, so that the surface settlement at x ($S_{(x)}$) is proportional to the boundary lateral displacement at referent depth z* ($D_{(0,z^*)}$):

$$z^* = x * H_d / X_{ext}$$
$$= x / \tan(90 - \phi') \qquad (30)$$
$$S_{(x)} = D_{(0,z^*)} / \tan(90 - \phi')$$
$$= (\varepsilon_{a(z^*)}/2)(X_{ext(z^*)} + x_{a(z^*)}) / \tan(90-\phi') \qquad (31)$$

Eq.(31) can also be implicitly understood as vertical compression of the extension zone with an average vertical compressive strain $\varepsilon_{av(x)}$ over the thickness of the zone at point x (section FF' in Fig.4) given by:

$$\varepsilon_{av(x)} = (\varepsilon_{a(z^*)}/2)(X_{ext(z^*)} + x_{a(z^*)}) / (X_{ext(o)} - x) \qquad (32)$$

4. Examination on applicability of the proposed approach

Applicability of the proposed approach is examined based on comparison of predicted values with field measurements of vacuum induced settlement, lateral displacement and strength increment using three case histories with different site and treatment conditions.

4.1 Selected case histories

Hazama test embankment: Hazama Case A (Kyushu, Japan):

Accompanying to a project conducted by Hazama Corp. since 1990's to confirm the effectiveness of deep soil vacuum treatment, an experimental embankment over an area of 20m x 20m was constructed with 0.3m sand mat, through which vertical drains of 27m length were installed into soft ground composed of 8m thick surface reclamation fill (6m silty clay + 2m sand) and an underlying 30m thick Ariake alluvial clay. Measurement during vacuum consolidation indicated an average pore water pressure drawdown of 50 kPa within the treatment depth. Settlement at various depths and surface lateral displacement at various distances were measured. Data on soil properties and strength before and after vacuum treatment are also available [8].

Oil storage station project (Port of Tianjin, China) is an actual vacuum improvement for very high water content reclaimed soil in Tianjin - a typical vacuum consolidation practice in China. The case was reported in details in a publication by Chu et al.[2], from which soil profile and initial soil properties, as well as measurement data on vacuum pressure, settlement, lateral displacement and strength profiles were cited for use in this study. However only data from section II (among two sections) were used for comparison. According to that, the soil profile is characterized by a 4-5m thick dredged clay slurry overlying on 10-16m sea bed marine clay, which is divided into sub-layers of silty clay, medium plastic clay and relatively stiff silty clay, and the ground water level at the surface. Due to high water content mostly at or above the liquid limit, and low undrained shear strength, a surcharge of 40kPa (including 2m partially dried clay fill plus 0.3m sand) was applied before installation of vertical drains (20 m in length) for vacuum consolidation. Pore pressure evidence indicated completed compression under surcharge load of the upper soil layer when vacuum consolidation commenced. Average vacuum pressure of 80 kPa was monitored over the treatment depth and maintained during the treatment. Overall vacuum consolidation degree > 80% was suggested.

Sanriku Ota Road test embankment (Tohoku, Japan)

An experimental embankment of 12.5m high over about 700m^2 area was constructed at the site of Sanriku motorway in a road improvement project in Monou town (Ota, Tohoku, Japan). The treatment depth of 13m covers very weak ground consisting of 5m thick peat layer underlain by 3-8m thick very soft clay layers. Thanks to utilizing a newly developed air-water separation system, a high vacuum pressure upto 85kPa was created over the treatment depth and maintained during the entire period of construction of 172 days. Just after 30 days of initial vacuum consolidation to improve the initial soil condition, rapid construction of 12.5m-high embankment was started. It has been successfully completed at an average speed of 13 cm/day under continuing vacuum consolidation support (Sato et al., 2003). In this study, only data obtained for the initial stage of soil improvement (before starting embankment) were subjected to comparison.

4.2 Obtaining input parameters

Parameters required for the equations in this approach can be obtained from initial soil investigation including soil stratification, soil physical and mechanical properties based on standard field tests or index laboratory tests. In this study, wherever direct measurements of some mechanical parameters were not available, they might be estimated empirically from initial index properties or generally assumed according to the soil types for the first approximation.

If friction angle ϕ'is available from test, lateral earth pressure coefficients K_o and K_a can be estimated by Jaky's equation (Jaky, 1944):

$K_o = 1 - \sin\phi'$

$K_a = (1 - \sin\phi') / (1+\sin\phi')$

Value of ϕ' can be approximately estimated from relative density and gradation for sands or from I_p for clayey soils, or at least from assumed value of K_o.

Compression index C_c was estimated empirically from liquid limit w_L in case of Oil Storage Station.

Coefficient of 1-D volume compressibility m_v at initial state can be estimated from C_c and e_o and σ'_{vo} as following:

$m_{vo} = -\lambda / [(1+e_o)\,\sigma'_{vo}]$

$= 0.434\,C_c / [(1+e_o)\,\sigma'_{vo}]$

However, considering non-linear variation, value of m_v for input in stress-strain equations should be averaged from those representing effective stress states at the initial and end of vacuum consolidation. Effects from any loading before vacuum loading (surcharges, fill, very soft surface layer treatment, sand mat, etc…) should also be accounted beforehand.

Conventional strength increment ratio, $m = c_u/\sigma'_v$ used in estimating Δc_u, is selected in accordance with soil types based on conventional consolidation design. For most soft NC clayey soil layers, an average value of $m = 0.30$ could be reasonable.

Finally, it should be noticed that in development of this approach, the vertical effective stress increment was equalized to effective vacuum pressure P_v, which indicated that a consolidation degree of 100% or full dissipation of induced negative pore pressure was achieved. Therefore, for the soil behaviors at a consolidation degree U less than 100% (such as at the time of embankment commencement) relevant equations can be used with introducing the consolidation degree U as a proportional coefficient.

Table 1 summarizes soil and vacuum consolidation conditions and parameters used in calculations for the selected three cases.

4.3 Results and discussions

Vacuum induced displacement and strength increment are calculated using proposed equations in this approach and then compared with measured values in Fig. 4 (a, b, c) for three cases, respectively. For each case, the plots arranged from left to right are settlement, strength before and after treatment and boundary lateral displacement over treatment depth. The values are summarized in Table 2.

Table1　Summary of site conditions and vacuum treatment at three construction sites

Case history	Treated soil layer soil type	Thickness Δz (m)	Unit weight γ' (g/cm^3)	Liquid limit W_L and (w_o)	Void ratio e_o	Compression index, C_c	At-rest coef. K_o	Poison ratio, ν	Friction angle ϕ' (degree)	Strength increment ratio m=su/p	Factor f
Hazama Case A (Kyushu, Japan)	Sand mat	0.3	1.8				0.40	0.29	37		
	Reclaimed soil	6.2	0.5	(75)	2.1	0.37	0.50	0.33	30	0.30	0.02
	Sand	2.1	0.8	(40)	0.9	0.05	0.40	0.29	37	0.30	0.02
	Silt	4	0.5	(80)	2.0	0.50	0.50	0.33	30	0.30	0.02
	Clayey silt	7.7	0.5	(70)	1.8	0.60	0.50	0.33	30	0.30	0.02
	Clayey silt	7	0.5	(95)	2.3	0.80	0.50	0.33	30	0.30	0.02
Oil Storage Station (Tianjin, China)	Clay fill + sand	2	1.9	45	1.10	0.32	0.50	0.33	30		
	Dredged slurry	5.2	0.75	38	1.25	0.25	0.50	0.33	30	0.30	0.02
	Soft silty clay	2.5	0.85	40	1.10	0.27	0.50	0.33	30	0.30	0.02
	Clay	7.7	0.75	47	1.30	0.33	0.50	0.33	30	0.25	0.02
	Med.silty clay	3.8	0.95	35	0.85	0.23	0.50	0.33	30	0.25	0.02
Sanriku Ota Road (Tohoku, Japan)	Sand mat	0.7	1.9				0.40	0.29	37		
	Soft Organic Clay	0.9	0.45	110	2.8	1.50	0.40	0.29	37	–	0.02
	Peat	3.55	0.1	510	7.3	5.00	0.36	0.26	40	–	0.02
	Clay (CH-CL)	5.1	0.58	110	1.8	0.90	0.50	0.33	30	–	0.02
	Clay (CL)	3.3	0.8	50	1.1	0.36	0.50	0.33	30	–	0.02

a) <u>Vacuum induced settlement and strength increment in treated area</u>

Plots of settlement for each case demonstrated reasonable agreement between calculated and measured values at every depth, suggesting about suitability of deformation equations. For Oil Storage Station and Sanriku Ota Road cases, vertical compression under surcharge (sand mat or fill) of the surface layer was judged completed when vacuum consolidation commenced (based on recorded pore water pressure). Accordingly, the amount of compression calculated for that was excluded from total settlement before subjected to comparison. This fact was also accounted for in evaluating effective stresses and m_v value for corresponding soil layers.

Furthermore, for Sanriku Ota Road, settlement was calculated corresponding to vacuum consolidation degree of 65% at the beginning of embankment (as judged from field settlement curve) and subjected to comparison with settlement measured at that time.

Comparison of vacuum induced strength increment was made for Hazama case A and Oil Storage Station, but not for Sanriku Ota case because of lack of measured data on strength at beginning of embankment. The use of an average strength increment ratio m = 0.30 for soft soil layers is seemed reasonable in Hazama case A. However, in Oil Storage Station case, m=0.30 appeared well predicted for upper soft layers, but for deep stiff clay layers the calculation overestimated the strength even at m = 0.25.

b) <u>Displacement in surrounding ground and the need of modifying active extension strain</u>

As lateral extension strain ε_a estimated by Eq.(27) is proportional to the vacuum induced lateral compressive strain ε_h and the ratio L/H_d, it's value appeared to be heavily affected because these parameters could be largely varied from case to case.

For Hazama case A, due to low vacuum pressure (P_v = 50kPa) hence low ε_h and low ratio L/H_d (=0.37), Eq.(27) provided values of $\varepsilon_a < \varepsilon_{af}$. In such case, lateral displacement calculated based on obtained value ε_a appeared reasonable, especially for the surface layer.

However, at higher vacuum pressure and the ratio L/H_d > 1, the value of ε_a calculated by Eq.(27) far exceeded the active failure strain ε_{af}, leading to largely overestimating lateral displacement (even several hundreds percents) as shown in Fig.4 for Oil Storage Station (L/H_d = 5) and Sanriku Ota Road (L/H_d = 2.1). It addressed the need for reconsidering Eq. (27) for ε_a .

Considering that under high vacuum pressure as well as large treatment area (high L/H_d), lateral compression of near surface layer could be so great that active failure state in surrounding soil might be achieved early before consolidation degree reached 100% in the treated soil. Under such condition, Eq.(27) will give an extension strain ε_a higher than active failure strain. Therefore, at the near surface depth z_c, if $\varepsilon_{a(zc)} > \varepsilon_{af}$, it might be interpreted as the active failure occurred at a consolidation degree equal to $U = \varepsilon_{af} / \varepsilon_{a(zc)}$. By taking this consolidation degree into account for the value of ε_h being input in Eq.(27), the resulted extension strain is thus modified as $\varepsilon_{a(mod)}$ expressed by following Eq.(27b).

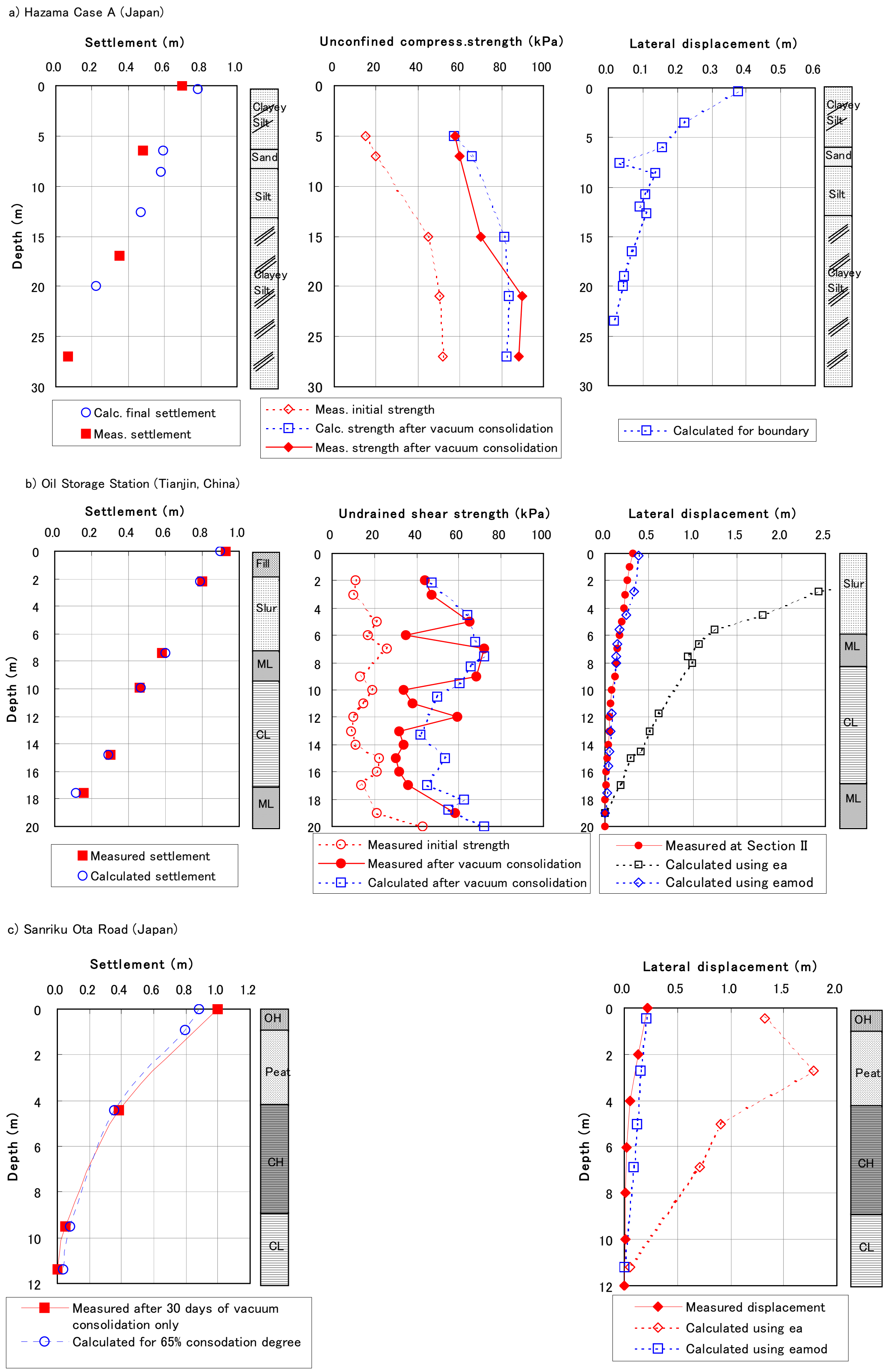

Fig. 4 Calculated and measured settlement, lateral displacement and strength for three cases

Table 2 Calculated results for three vacuum construction sites

Case history	Vacuum consolidation treatment				Calculated results							
					Treated area			Influence in surrounding area				
	Treatment depth Hd (m)	Area halfwidth L (m)	Fill surcharge kPa	Effective Vacuum Pv (kPa)	Mid-layer depth (m)	Layer's compression (m)	Strength increment Δs_u (kPa)	Depth (m)	Extension zone X_{ext} (m)	Active zone x_a (m)	Boundary lateral displacement Do (m)	Maximum settlement So (m)
Hazama Case A (Kyushu, Japan)	27	10	5.4	50	0.15	0.007	–	–	–	–	–	–
				50	3.5	0.193	41.96	0.0	46.2	15.4	0.387	0.224
				50	7.5	0.011	45.72	6.2	33.8	11.3	0.034	
				50	10.7	0.115	36.33	8.3	31.9	10.6	0.144	
	L/Hd = 0.37			50	16.5	0.245	33.33	12.3	24.9	8.3	0.117	
				50	23.5	0.222	30.33	19.7	12.1	4.0	0.050	
Oil Storage Station (Tianjin, China)	20	100	40	80	1	0.110	–	–	–	–	–	–
				80	4.8	0.188	48.15	0.1	32.6	10.9	0.376	0.217
				80	8.65	0.134	45.70	5.1	21.4	7.1	0.166	
	L/Hd = 5.0			80	13.7	0.350	31.86	7.7	12.6	4.2	0.131	
				80	19	0.118	29.81	15.0	3.5	1.2	0.035	
Sanriku Ota Road (Tohoku, Japan)	13	27	13.8	85	0.35	0.026	–	–	–	–	–	–
				85	1.15	0.058	–	0.5	16.7	6.3	0.253	0.190
				85	3.4	0.447	–	2.7	12.3	4.8	0.205	
	L/Hd = 2.1			85	7.6	0.275	–	6.9	10.6	3.5	0.120	
				85	11.9	0.077	–	11.2	3.1	1.0	0.035	

This modified lateral strain would conform to the condition of $\varepsilon_{a(mod)} \leq \varepsilon_{af}$ at the beginning of cracking.

$\varepsilon_{a(mod)} = \varepsilon_h * U * 2L / (X_{ext} + x_a)$ or

$\varepsilon_{a(mod)z} = \varepsilon_{a(z)} * [\varepsilon_{af} / \varepsilon_{a(zc)}]$ (27b)

In Fig.4, for Oil Storage Station and Sanriku Ota Road cases, lateral displacement calculated based on $\varepsilon_{a(mod)}$ (Eq.27b), as well as based on ε_a (Eq.27) are compared with actual measured values, which clearly demonstrate the suitability of using $\varepsilon_{a(mod)}$ (Eq.27b) instead of ε_a for predicting lateral displacement in the cases with $L/H_d > 1$.

On the other hand, Fig.5 compares calculated and measured surface lateral displacement for Hazama case A. In term of horizontal distribution, a tendency of shifting of all measured points including the location of the maximum displacement for a few meters outward the treatment boundary compared to the calculated points can be seen. This problem could be due to the fact that in practice the vacuum pressure was not ideally isolated inside the boundary as assumed in calculation.

Calculated surface settlements at various distances in surrounding ground are shown in Fig.6 for all three cases. The maximum settlement near the boundary was about 20cm for all three cases. Measured data were available only for Sanriku Ota Road, and calculated values appeared in reasonable agreement with them.

5. Conclusions

Being a soil improvement method based on consolidation, the main items to be concerned in vacuum consolidation design are settlement and strength increment in the treated area, as well as deformation in surrounding ground. A conceptual approach has been proposed by the authors to evaluate the frequently observed ground deformation variation over vacuum treatment depth, and how to account for it in vacuum consolidation design. Due to the fact that the approach was developed theoretically with various assumptions and simplifications, it is important to validate its applicability through actual vacuum consolidation cases as demonstrated in this study. So far, following conclusions could be made.

1) The change in lateral earth pressure conditions from both sides of the vertical boundary between treated and surrounding grounds in association with the vacuum induced lateral displacement is incorporated in estimating the effective stress increment under vacuum consolidation assuming fully developed active lateral condition at horizontal equilibrium. The use of stress increment ratio, $I = \Delta\sigma'_h / \Delta\sigma'_v$ for detecting the ground deformation continuously along treatment depth suggests a variation from nearly isotropic for the near surface layer to nearly 1-D deformation for the near bottom layer.

2) Assuming elastic relationships for simplification, strain increments as well as increase in soil strength are evaluated based on stress increments. For convenience,

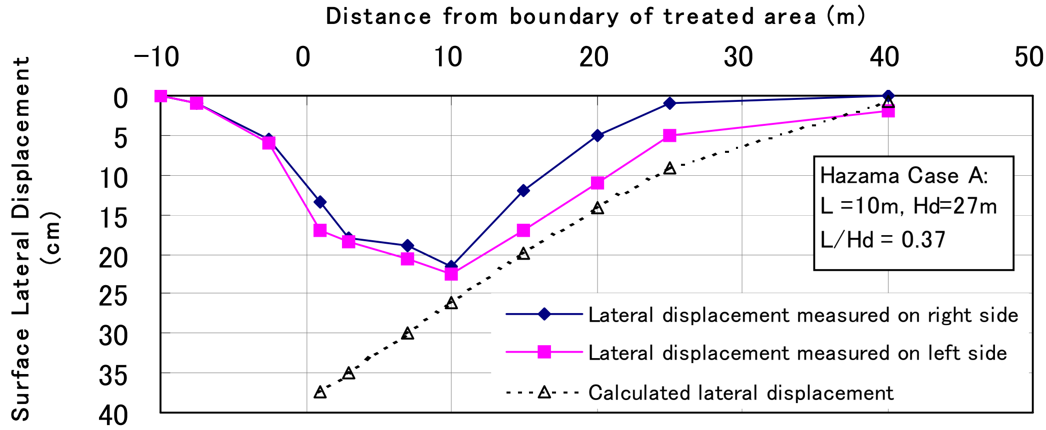

Fig. 5 Comparison of measured and calculated surface lateral displacement for Hazama Case A

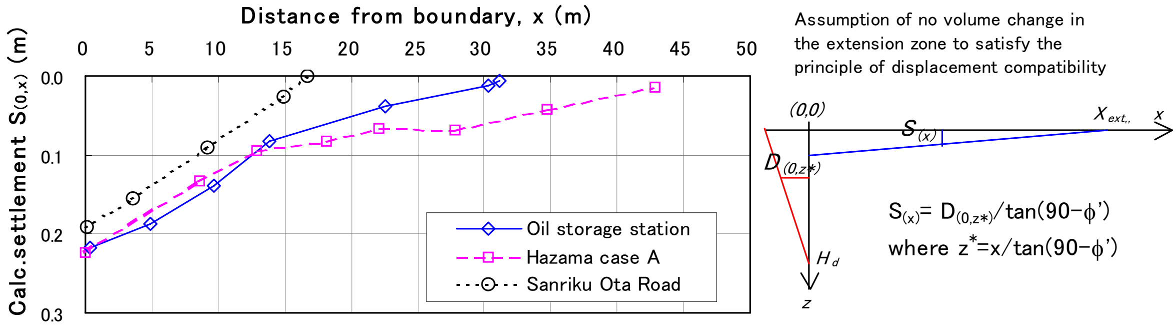

Fig. 6 Calculated vertical displacement in surrounding area for the four cases

derived equations are expressed in conventional formula with introducing correction factor for appropriate deformation at corresponding depth.

3) Considering the crack-like failure induced by large tensile strain under reduced effective lateral stress as a main deformation mechanism in surrounding ground, a method for approximating lateral and vertical displacements within a potential influence zone is proposed assuming the principle of displacement compatibility under condition of no volume change.

4) Applicability of the proposed approach is examined using data from three actual cases with different soil conditions and design treatment conditions. With input parameters for the equations being obtained directly from initial soil investigations or indirectly from empirical relationships, reasonable agreements between calculated and measured data on settlement and strength increment in the treated area are demonstrated for all cases. Also, through examination, a modified equation for lateral strain in surrounding ground is suggested.

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真空圧密工法による地盤変形特性に関する察と設計への応用

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真空圧密工法の施工事例から，従来設計手法による沈下と強度増加の予測値が実測値と一致しない場合があり，この特殊な載荷条件下での地盤変形を適切に考慮すべきであることが分かってきた。真空圧載荷時の地盤収縮による水平土圧の変化を考慮することによって，有効応力増加の深さ方向の分布と地盤変形を求めることができる。そして，真空圧密の領域外への影響を考慮することで，その影響範囲と地盤変位を評価しようとする簡易な手法が提案されている。本論文では，その簡易手法の適用性を数例の実測データを用いて調べた。その結果，提案式への入力に若干の修正を加えれば実測値とさらによく一致し，少なくとも一次設計に対して本手法が有用であることが分かった。

キーワード：地盤改良，真空圧密工法，変形特性，主動土圧，有効応力，負の過剰間隙水圧