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Numerical Assessment of the Permeability for the Pleistocene Sand Gravel Deposits Considering the Subsurface Stratigraphy of Kansai International Airport

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Synopsis

A series of elasto-viscoplastic finite element analyses is performed to assess the long-term deformation including the interactive behavior of the reclaimed Pleistocene foundation due to the adjacent construction of the offshore twin airport. Attention is paid to the modeling of permeability for the Pleistocene sand gravel layers considering the sedimentation environment because the performance of excess pore water pressure is strongly dependent on the extent of distribution as well as the change of thickness of those permeable sand gravel layers. The mechanism for the propagation and dissipation of excess pore water pressure due to construction of the adjacent reclamation is discussed through the numerical procedure using the concepts of “mass permeability” and “standard hydraulic gradient” for the Pleistocene sand gravel layers. The mode of advance in settlement of the Pleistocene clay layers associated with the process of the generation/dissipation and propagation of excess pore water pressure is also carefully discussed. The proposed procedure is validated by comparing the calculated performance and the in-situ measured results. The calculated performance can well describe the actual behavior of the Pleistocene deposits due to construction of the adjacent 1st and 2nd phase islands of Kansai International Airport (KIX).

Keywords: elasto-viscoplastic finite element analysis, mass permeability, standard hydraulic gradient

1. Introduction

The development of coastal areas accomplished in Japan has been outstanding. Kansai International Airport (KIX) was constructed in Osaka Bay as two man-made reclaimed islands to minimize noise and pollution in residential areas as well as to meet the increasing demand for air transportation. Such a large-scale offshore reclamation in Osaka Bay is accompanied with large and rapid settlement of deep Pleistocene clay deposits (Mimura et al., 2003). Long-term settlement of the Pleistocene marine foundations due to huge reclamation load has been of great concern in this project. The seabed deposits of Osaka Bay have been formed due to the soil supply from the rivers and the alternating deposits of KIX have been formed due to sedimentation of clayey soils during transgression and of sandy to gravelly soils during regression on the sinking base of Osaka Bay. The Pleistocene clay deposited in Osaka Bay exhibits the behavior of the quasi-overconsolidated clay without definite mechanical overconsolidation history. Itoh et al. (2001) summarized on the basis of the data from elastic wave exploration and in-situ boring logs that the Pleistocene sand gravel deposits are not always distributed uniformly in thickness, consistently and that the amount of fine contents included in them is significant. The most serious problem originating from these sand gravel deposits is the "permeability" that controls the rate of consolidation of sandwiched Pleistocene clays. In the sense, the modeling for the quasi-overconsolidated Pleistocene clay and the evaluation of permeability for the Pleistocene sand gravel deposits are the significant factors to assess the long-term behavior of the reclaimed Pleistocene foundation due to the reclamation of the offshore twin airport. Mimura and Jang (2004) proposed a concept of compression in which viscoplastic behavior is assumed to occur even in the quasi-overconsolidated region less than p_c for the Pleistocene clays in Osaka Bay. The procedure has been found to be versatile and able to describe the long-term settlement monitored in the reclaimed islands in Osaka Port (Mimura and Jang, 2005a). In the present paper, the numerical procedure to evaluate the permeability of the Pleistocene sand gravel layers at KIX in terms of the finite element analysis is proposed by introducing the concept of "mass permeability" and "standard hydraulic gradient". The validity of the procedure is carefully discussed by comparing the calculated performance with the in-situ measured results.

2. Proposal of numerical procedure

2.1 Concepts of "mass permeability" and "standard hydraulic gradient"

Mimura and Jang (2005a) reported only when the permeability of sand gravel layers is considered perfectly drained, one-dimensional analysis can be adopted because the process of propagation and dissipation of excess pore water pressure in these sand gravel layers can be ruled out. However, the sand gravel layers sandwiched by the Pleistocene clay layers at KIX were recognized not to function as perfect drainage layers through the in-situ measurement of excess pore water pressure. Two or three-dimensional analyses in which the finite permeability of the Pleistocene sand gravel layers plays a significant role are hence required to assess the long-term behavior of the reclaimed Pleistocene foundation of KIX. The influential factors to evaluate the permeability of sand gravel layers are the thickness, the horizontal continuity and the fine contents in them. The permeability of them is different with places even if they are categorized as the geologically identical ones. But, it is impossible to evaluate the individual permeability of sand gravel layers at every point. It is also very difficult to confirm how the sand gravel layers among the Pleistocene marine foundation are distributed in practice. The concept of "mass permeability" is proposed to evaluate the permeability not for the individual element but for the whole geologically identical layer in one body. It is regarded as the macroscopic capability of permeability for the individual sand gravel layers by considering the horizontal continuity, the change in thickness and the degree of fine contents in them. Mimura and Jeon (2011) evaluated the mass permeability of the Pleistocene sand gravel layers at KIX using the simple representative foundation model with the horizontally even layers. The distribution of sand gravel layers not only in the loading area but also in the area outside of the reclaimed area should be considered to assess the mechanism of the propagation/dissipation of excess pore water pressure in the coupled stress-flow analysis. In the sense, on the basis of the assumption that the hydraulic gradient derived in the representative foundation model having the horizontally even layers with constant thickness is regarded as the standard one for the individual Pleistocene sand gravel layers, the evaluated mass permeability can be the representative of the capacity of permeability for the individual Pleistocene sand gravel layers at KIX. The standard hydraulic gradient is hence applied to the geologically genuine foundation model that has been developed to consider the actual stress level not only for the monitoring point but also for the considered area for the numerical analysis.

2.2 Numerical procedure to consider the concept of "standard hydraulic gradient"

The coupled stress-flow finite element equations used in the present study are established on the basis of Biot's formulation (Christian, 1968) in the following form:

$$\begin{bmatrix} [K_1] & [K_2] \\ [K_2]^T & [K_3] \end{bmatrix} \begin{bmatrix} \{\Delta u\}_j \\ \{p_w\}_j \end{bmatrix} = \begin{bmatrix} \{\Delta F\} + [K_2]\{p_w\}_{j-1} \\ \{0\} \end{bmatrix} \tag{1}$$

in which, the nodal displacement increments $\{\Delta u\}$ and the pore water pressure $\{p_w\}$ are taken as the primary unknowns of the problem. Subscript j means the calculated step for time increment, $\{\Delta F\}$ is the generalized nodal load increments and $[K_1]$, $[K_2]$ and $[K_3]$ denote the stiffness matrix. Here, $[K_3]$ associated with the pore water flow is expressed by obeying isotropic Darcy's law. Fig. 1 schematically shows the elements surrounding i element for finite element analysis. Then, the divergence of the flow rate q in the i element is expressed as follows:

$$\begin{aligned}\Delta tq &= \Delta t[(q_j - q_l) + (q_k - q_n)] \\ &= \left(\sum_{i=1}^{4} B_i\right)p_w - \sum_{i=1}^{4}(B_i p_{wi}) = [K_3]p_w\end{aligned} \tag{2}$$

where, the coefficient B composing the stiffness matrix $[K_3]$ is defined by considering the equivalent coefficient of permeability as follows:

$$B_1 = \frac{\Delta t}{\gamma_w} \times (k_{hi} \times S_y) / \left\{(AL1 - A1) \times \frac{k_{hi}}{k_{hj}} + A1\right\} \tag{3}$$

Here, k_{hi} and k_{hj} are the coefficient of permeability for i and j elements respectively, γ_w is the unit weight of water, the other notations are shown in Fig.1. In the present analysis, the hydraulic gradient is the factor controlling the pore water flow and the evaluated hydraulic gradient in the representative foundation model is defined as

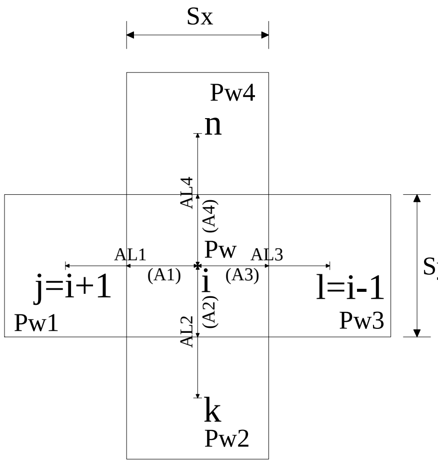


Fig. 1 Schematic elements for the freedom of excess pore water pressure with adjoining four elements

"standard hydraulic gradient". In order to apply the standard hydraulic gradient to the corresponding sand gravel layers of the geologically genuine foundation model, the flow rate q and hydraulic gradient i generated due to the reclaimed load are assumed to be identical with horizontally even and geologically genuine foundation models. The coefficient of permeability in the geologically genuine foundation model is then defined as follows:

$$q = k_s \times i_s \times Sy_s = k_{geo} \times i_{geo} \times Sy_{geo}\,,\quad i_s = i_{geo} \tag{4}$$

then,

$$k_{geo} = k_s \times Sy_s / Sy_{geo} \tag{5}$$

in which, subscripts s and geo mean the standard values in the representative foundation model and the values in the geologically genuine foundation model respectively. In terms of the geologically genuine foundation model, substituting the coefficient of permeability of Eq. (5) into Eq. (3) yields

$$\begin{aligned}B_1 &= \frac{\Delta t}{\gamma_w} \times \left(\frac{k_{si} \times Sy_s}{Sy_{geo}}\right) / \left\{(AL1 - A1) \times \frac{\dfrac{k_{si} \times Sy_{si}}{Sy_{geoi}}}{\dfrac{k_{sj} \times Sy_{sj}}{Sy_{geoj}}} + A1\right\} \\ &\approx \frac{\Delta t}{\gamma_w} \times (k_{si} \times Sy_s) / \left\{(AL1 - A1) \times \frac{k_{si}}{k_{sj}} + A1\right\}\end{aligned} \tag{6}$$

Eventually, the derived coefficient B controlling the pore water flow in terms of the coupled stress-flow becomes the same with the standard values in the representative foundation model. Therefore, it means that the evaluated permeable capacity for the representative foundation model can be intactly applied to the geologically genuine foundation model by assuming the same flow rate q and hydraulic gradient i for both foundation models. Due attention should be paid to the fact that this assumption is only considered in horizontal position for the individual Pleistocene sand gravel layers.

3. Framework of numerical assessment

3.1 Elasto-viscoplastic model

The elasto-viscoplastic constitutive model used in this paper was proposed by Sekiguchi (1977). Sekiguchi (1982) et al. modified the model to a plane-strain version. The viscoplastic flow rule for the model is generally expressed as follows:

$$\dot{\varepsilon}_{ij}^{p} = \Lambda \frac{\partial F}{\partial \sigma_{ij}^{'}} \tag{7}$$

in which F is the viscoplastic potential and Λ is the proportional constant. Viscoplastic potential F is defined as follows:

$$F = \alpha \cdot \ln\left[1 + \frac{\dot{v}_0 \cdot t}{\alpha} \exp\left(\frac{f}{\alpha}\right)\right] = v^p \tag{8}$$

in which α is a secondary compression index, $\dot{v}_o$ is the reference volumetric strain rate, f is the function in terms of the effective stress and v^p is the viscoplastic volumetric strain. The concrete form of the model is shown in the reference (Mimura and Sekiguchi, 1986). The resulting constitutive relations are implemented into the finite element analysis procedure through the following incremental form:

$$\{\Delta\sigma'\} = [C^{evp}]\{\Delta\varepsilon\} - \{\Delta\sigma^R\} \tag{9}$$

where $\{\Delta\sigma'\}$ and $\{\Delta\varepsilon\}$ are the associated sets of the effective stress increments and the strain increments respectively, and $[C^{evp}]$ stands for the elasto-viscoplastic coefficient matrix. The term $\{\Delta\sigma^R\}$ represents a set of ‘relaxation stress’, which increases with time when the strain is held constant. The pore water flow is assumed to obey isotropic Darcy's law. In relation to this, it is further assumed that the coefficient of permeability, k, depends on the void ratio, e, in the following form:

$$k = k_0 \cdot \exp\left(\frac{e - e_0}{\lambda_k}\right) \tag{10}$$

in which k_o is the initial value of k at $e = e_o$ and λ_k is a material constant governing the rate of change in permeability subjected to a change in the void ratio. Note that each quadrilateral element consists of four constant strain triangles and the nodal displacement increments and the element pore water pressure is taken as the primary unknowns of the problem. The finite element equations governing those unknowns are established on the basis of Biot's formulation (Christian, 1968, Akai and Tamura, 1976) and are solved numerically by using the semi-band method of Gaussian elimination.

3.2 Foundation model and hydraulic boundary

The differential settlement of the individual Pleistocene clay layers as well as the excess pore water pressure at various depths, both in the clay and the sand gravel layers, have been measured at a lot of points of KIX. Fig. 2 shows the plan view of KIX together with the location of representative monitoring points on the 1st phase island. A series of elaso-viscoplastic finite element analyses is carried out along the representative section shown by A-A’ at monitoring point 1 in Fig.2. Fig.3 shows the representative foundation model assumed to be horizontally even layer that have a constant thickness and continuous layer based on the boring data at the monitoring point 1. Fig.4 shows the geologically genuine foundation model having the inclined base and layers that is constructed based on

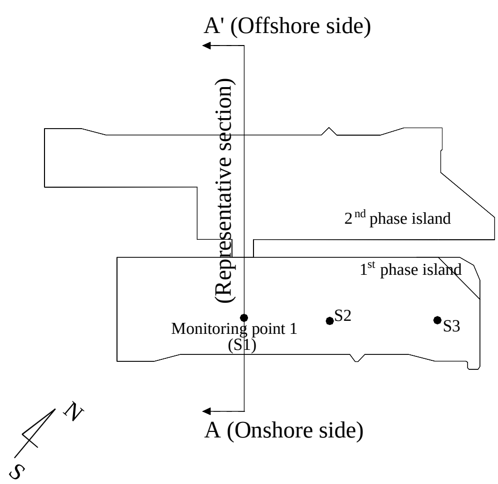


Fig. 2 Plan view of KIX (1st and 2nd phase) and the location of the monitoring points

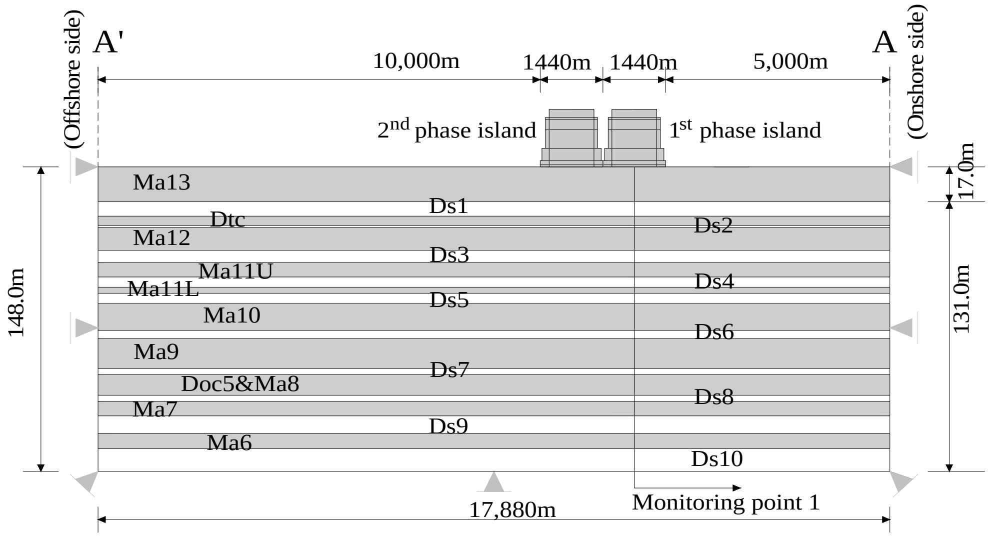


Fig. 3 Representative foundation model of KIX for finite element analysis at representative section

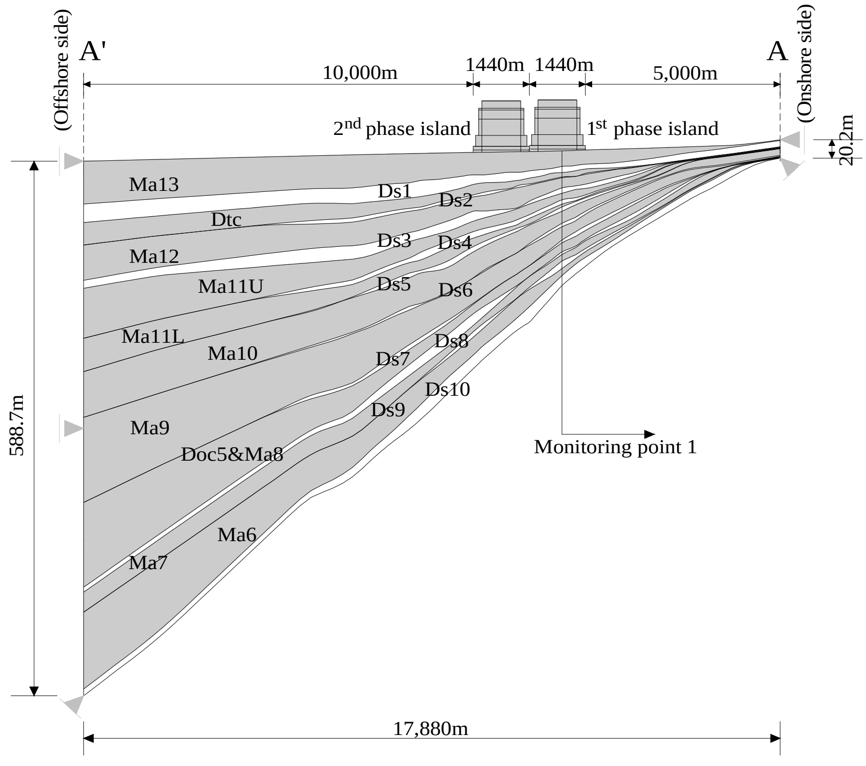


Fig. 4 Geologically genuine foundation model of KIX for finite element analysis at representative section

the soil exploration and geological survey data (Kitada et al, 2011, Inoue et al, 2011).

The clay layers increase in thickness towards the offing and the sand gravel layers drastically change in thickness horizontally. The continuity of the individual layers is still guaranteed even for the geologically genuine foundation model in the present study. Here, Ma and Ds denote marine clay and Pleistocene sand gravel layer respectively. Ma13 is the Holocene marine clay whereas others are the Pleistocene origin. For the Holocene clay deposit, Ma13, sand drains are driven in a rectangular configuration with a pitch of 2.0 to 2.5 meters to promote consolidation. The lateral boundary of the clay layers is assumed to be undrained while the one of the sand gravel layers is assumed to be fully drained. Mimura and Jang (2005b) reported that when the distance to the boundary is set to be about 10 times of the loading area, the effect of the hydraulic boundary condition can be ruled out. Based on the findings, the same condition is satisfied even for the foundation models used in the present study. The distance to the offshore and onshore boundary is set to be 10,000m and 5,000m respectively. The present two foundation models are divided into finite element mesh consisting of 8,580 nodal points and 8,378 elements.

3.3 Loading condition and soil parameters

Fig. 5 shows the reclaimed stress measured at center of the foundation of the 1st and 2nd phase islands respectively. The prescribed final overburden due to airport fill construction amounts to about 430kPa at the 1st phase island and about 530kPa at the 2nd phase island respectively. The 2nd reclamation is started after about 13years from the 1st reclamation. In the present analysis, the permeable capability evaluated from the concept of “mass permeability” for the Pleistocene sand gravel layers is applied for the present finite element analysis. On the basis of the findings by Itoh et al. (2001), the relatively high permeable capability are assumed for Ds1,3 10 because they have been evaluated as gravelly, horizontally continuous and having enough thickness. On the other hand, very low permeable capability is assumed for Ds6 and 7 that have been evaluated to have insufficient thickness with high degree of fine contents and poorly continuous. The other layers have been evaluated as the ordinary permeable ones. All soil parameters used for the present analysis are also exactly the same with that used by Mimura and Jeon (2011).

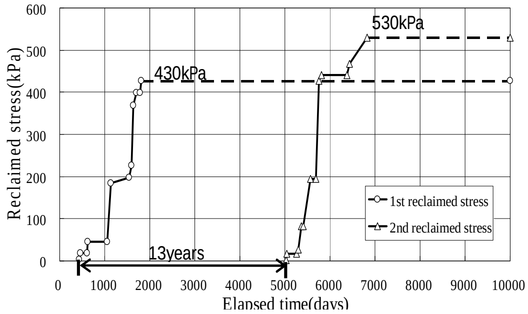


Fig. 5 Reclaimed stress with time for the 1st and 2nd phase reclamations

4. Results and discussions

4.1 Performance of excess pore water pressure

The calculated distribution of excess pore water pressure before and after the construction of the 2nd phase island is shown in Fig.6 for two foundation models respectively. As shown in Fig.6, the similar distribution tendency of excess pore water pressure can be seen for two foundation models. It should be noted that a large amount of excess pore water pressure still remains undissipated in the middle Pleistocene clay layers, Ma10, 9 and Doc5&Ma8 as well as sand gravel layers, Ds6 and 7 before the construction of the 2nd phase island because of poor permeability of sand gravel layers, Ds6 and 7. In contrast, the excess pore water pressure in the upper and lower Pleistocene layers such as Dtc, Ma12,11,7,6 and Ds1,3,9,10 is monotonically dissipated with time because of high permeability of sand gravel layer, Ds1,3 and 10. At the completion of the 2nd phase reclamation, a large amount of excess pore water pressure is concentrated in the upper and middle Pleistocene layers such as Ma12, 10, 9 and Doc5&Ma8 beneath the foundation of the 2nd phase island. Here, a due attention should also be paid to the fact that the increased excess pore water pressure beneath the foundation of the 2nd phase island is propagated to that of the 1st phase island. Since the permeability of the upper and lower Pleistocene sand gravel layers is higher than the one of the middle layers, a larger amount of excess pore water pressure in the upper and lower Pleistocene layers is propagated compared to the one in the middle layers of the foundation of the 1st phase island. The calculated horizontal distribution of excess pore water pressure in the representative Pleistocene sand gravel layers (Ds3, 6, 10) are shown in Fig. 7 at the time before and after the construction of the 2nd phase reclamation for both foundation models. In the present study, the identical permeable capability for the individual Pleistocene sand gravel layers in two foundation models is applied by considering the concepts of “mass permeability” and “standard hydraulic gradient”.

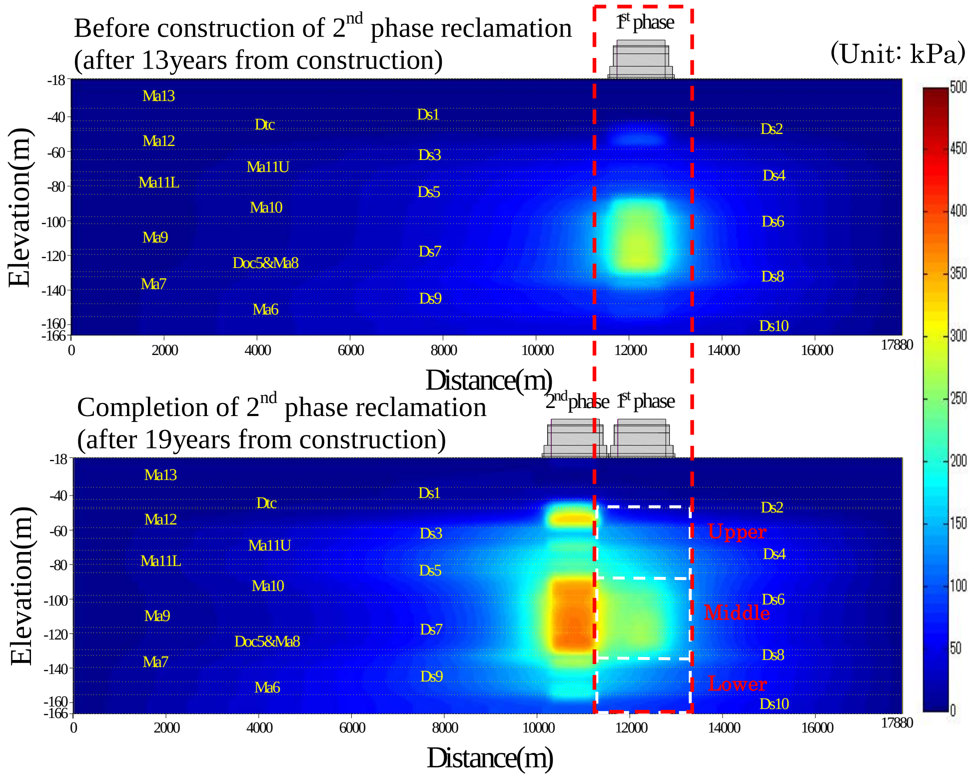


Fig. 6(a) Contour of excess pore water pressure for representative foundation model at before and completion of 2^{nd} phase reclamation

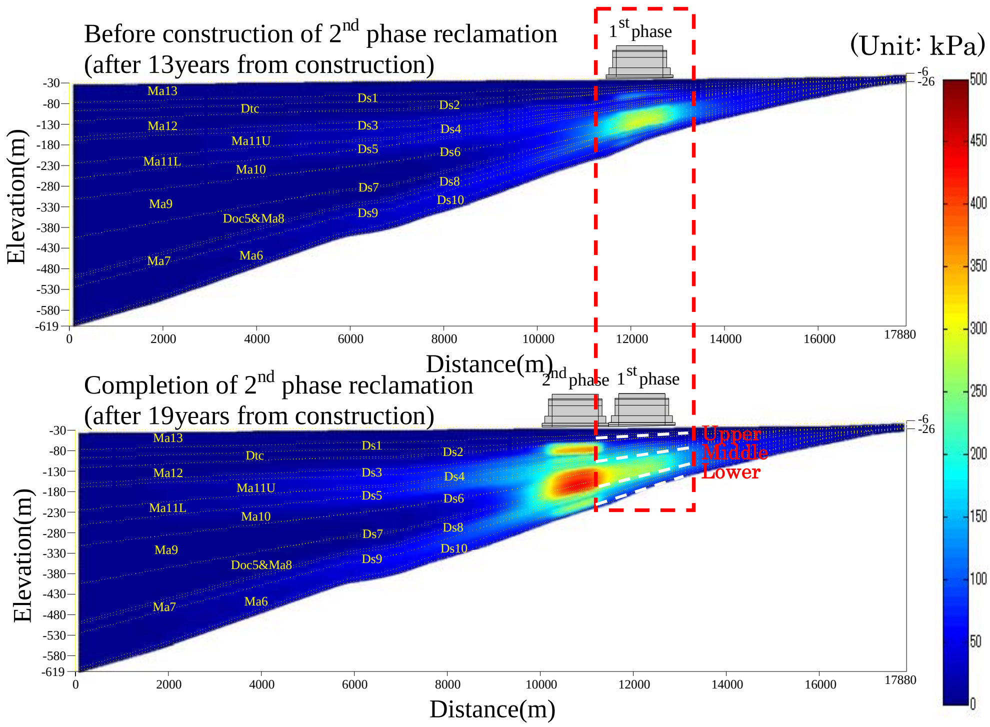
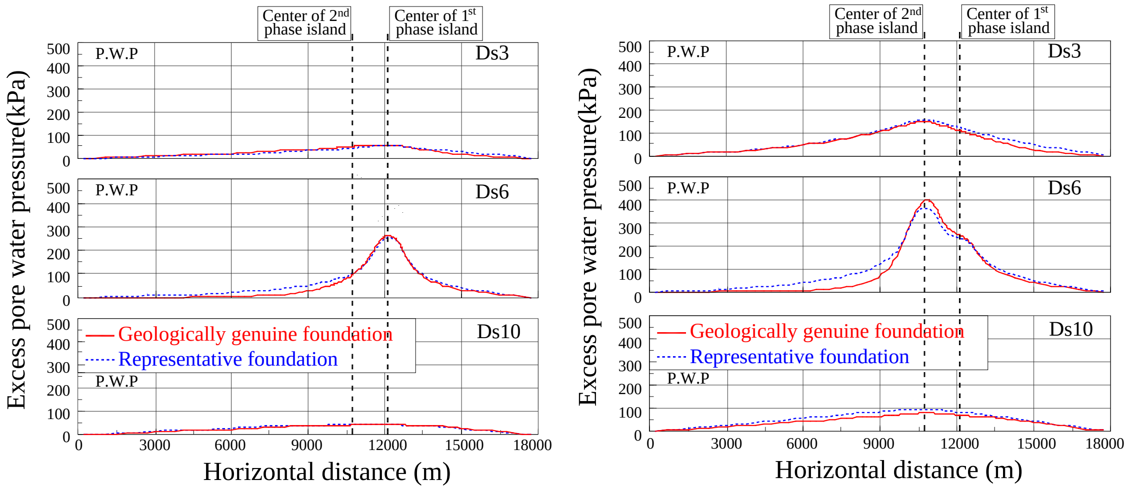


Fig. 6(b) Contour of excess pore water pressure for geologically genuine foundation model at before and completion of 2^{nd} phase reclamation



(a) Before the 2nd phase reclamation (b) Completion the 2nd phase reclamation

Fig. 7 Horizontal distribution of excess pore water pressure for the representative Pleistocene sand gravel layers (Ds3, 6, 10) in a horizontal position

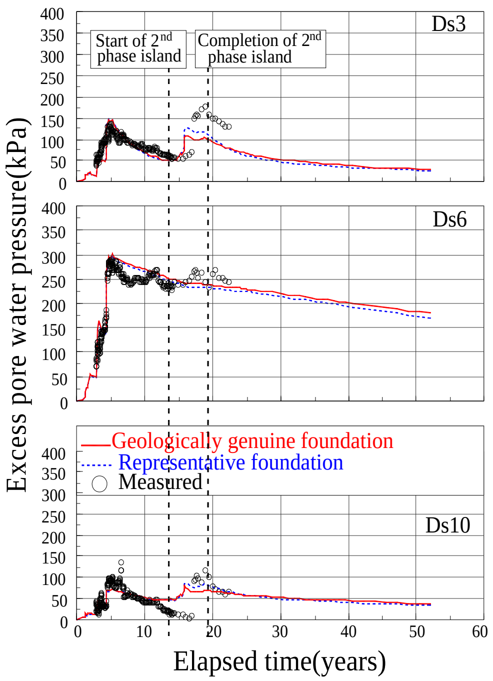


Fig. 8 Comparison of measured and calculated excess pore water pressure with time for the representative Pleistocene sand gravel layers

However, in Fig.7, it should be noted that the distribution of excess pore water pressure near the 1st phase island almost shows a good match for two foundation modes by applying the concept “standard hydraulic gradient” whereas the one of the other region shows the discrepancy distribution with the stress level. The stress level beneath the foundation of the 1st phase island is almost the same for two foundation models because the representative model was developed based on the monitoring point 1 whereas the one beneath the foundation of the 2nd phase island is different each other due to the change in thickness for the geologically genuine foundation model. It is noteworthy that although the identical permeable capability for the individual Pleistocene sand gravel layers was applied, the calculated results of excess pore water pressure could show the difference with the stress level. The calculated excess pore water pressure – time relations for two foundation models are shown in Fig. 8 together with the measured results for the representative Pleistocene sand gravel layers at the monitoring point 1. It is noteworthy that the excess pore water pressure in the upper (Ds3) and lower (Ds10) Pleistocene sand gravel layers is increased but the one of the middle layer (Ds6) is not increased due to the construction of the 2nd phase island.

4.2 Performance of settlement

The long-term settlement associated with the phenomenon of propagation of excess pore water pressure is another serious problem for KIX. The calculated settlement - time relations for two

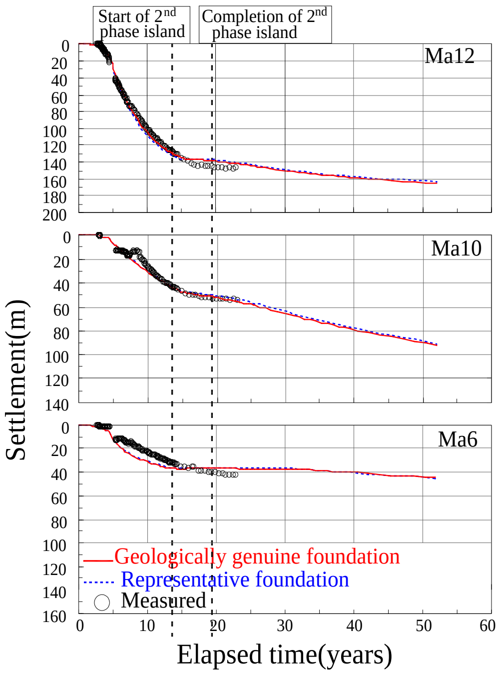


Fig. 9 Comparison of measured and calculated settlement with time for the representative Pleistocene clay layers

foundation models are shown in Fig. 9 together with the measured results for the representative Pleistocene clay layers (Ma12, 10, 6) at the monitoring point 1. As seen from Fig. 8, when the excess pore water pressure increases or the dissipation of excess pore water pressure is hindered due to the construction of the 2nd phase island, the settlement is also retarded or slight upheaval can occur (see Fig.9). It is also found that the calculated performance at the monitoring point 1 shows a good match for two foundation models by applying the concept of "standard hydraulic gradient" and can also well describe the whole process of deformation.

5. Conclusions

The long-term deformation of the reclaimed Pleistocene foundation of the offshore twin airport was numerically evaluated through the elasto-viscoplastic finite element analyses considering the concepts of "mass permeability" and "standard hydraulic gradient" for the Pleistocene sand gravel layers. The concept of "mass permeability" was evaluated as the representative permeable capacity of sand gravel layers of KIX. The representative permeable capacity of sand gravel layers was applied to the geologically genuine foundation model by introducing the concept of "standard hydraulic gradient" for the coupled stress-flow analysis. The concept of mass permeability for the sand gravel layers was found to well function to assess the process of excess pore water pressure generation/dissipation/propagation and long-term settlement in the reclaimed foundations of KIX. The concept of standard hydraulic gradient was also found to well reproduce the representative permeable capacity by comparing the calculated results for two foundation models. The validity and objectivity of the proposed concepts will be investigated by applying them to the additional review sections including the monitoring points S2 or S3 shown in Fig. 2.

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海上埋立による地盤挙動に対する更新統砂礫層の排水性能の評価

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要　旨

長期沈下が懸念されている関西国際空港の基礎地盤は，更新統粘土と砂礫の互層が厚く堆積する構造を有しており，隣接する二つの空港島の建設によって両空港島基礎地盤内に複雑な相互作用を引き起こしている。本稿では，こうした更新統層の地盤挙動を弾粘塑性有限要素法によって解析し，隣接埋立による関西国際空港基礎地盤の相互作用を含む長期挙動を包括的に議論する。解析の際,更新統砂礫層のマクロな透水性を評価するため,“mass permeability”と“standard hydraulic gradient”という概念を導入することによって，不均質性を持って広がっている関西国際空港の更新統砂礫層の透水性能を評価する解析スキームを提案する。適用した解析手法の妥当性については，水平地盤モデルと実地盤モデルの結果と実測値を比較することによって検証する。

キーワード:弾粘塑性有限要素法，更新統砂礫層，mass permeability，standard hydraulic gradient