GEOTECHNICAL ASPECTS RELATED TO KIA PROJECT

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Introduction

Japan's rapid economic and social development has been accompanied by an ever-growing demand for air transport. The nation's airport capacity, however, has not been developed sufficiently to meet this demand. Problems of noise and restrictions on facility expansion have severely restrained any substantial increase in takeoffs and landings.

To solve these problems, construction of the Kansai International Airport (KIA) began in January 1987. The airport is being built on a man-made island in Osaka Bay (**Fig.1**), 5 kirometers off the mainland, optimally located to avoid noise problems. The airport will have a 3,500m-long main runway able to accomodate approximately 160,000 takeoffs and landings per year (**Fig.2**).

Construction of the airport island was completed in December 1991, and construction of the airport facilities including the passenger terminal building is progressing on schedule. When the airport opens in the summer of 1994, it will be Japan's first 24-hour international airport, allowing airlines to devise flexible scheduling to and from all parts of the world.

With the completion of the airport, the Kansai region which has the second largest economic activities in Japan comparable to Tokyo metropolis area will occupy an important position as a gateway to Japan and as a major hub of international air transportation.

Construction of the airport island

The water depth of the construction site varies from 16.5m to 19m. The seabed slopes with a gradient of about 1/750. Seabed soil at the airport site was evaluated by drilling 65 borings of 100 - 400m in depth. In the soil exploration program, a series of laboratory tests and geologic tests was carried out on all of the soil samples obtained. As the result of these geological studies, the sedimentation environment and age of the soil layers were traced. The typical soil profile of the seabed is illustrated in **Fig.3**.

The subsoils immediately below the seabed bottom surface (mudline) of the airport islalnd consist of a thick, weak alluvial clay layer which should be improved so as to ensure the stability of seawalls and embankments, while they are being constructed, and to minimize the residual or differential settlement which will occur after the airport is put into operation.

In the construction of the surrounding seawalls and the reclaimed land within the walls, the sanddrain method (SD) and the sand compaction pile method (SCP) were applied to most of the area, with the deep mixing method (DM) used at two corners of the seawall.

The man-made airport islalnd is to be enclosed by several types of seawall, when combined, total approximately 11.2km in linear length. As shown in **Fig.4**, it consists of four sections, each of which has different functions in accordance with orientation, land use and construction schedule. The seawall construction work finished after 20 months.

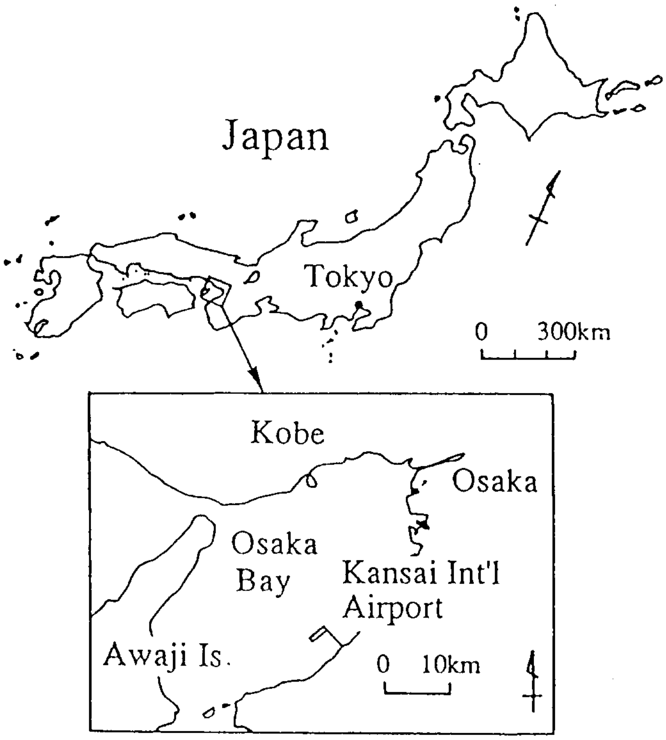

Fig.1 Location of Kansai International Airport (KIA).

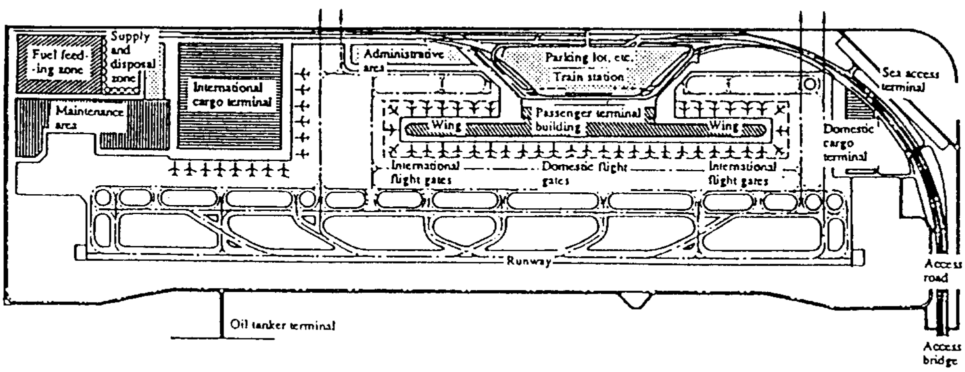

Fig.2 Overall layout of KIA (Phase 1).

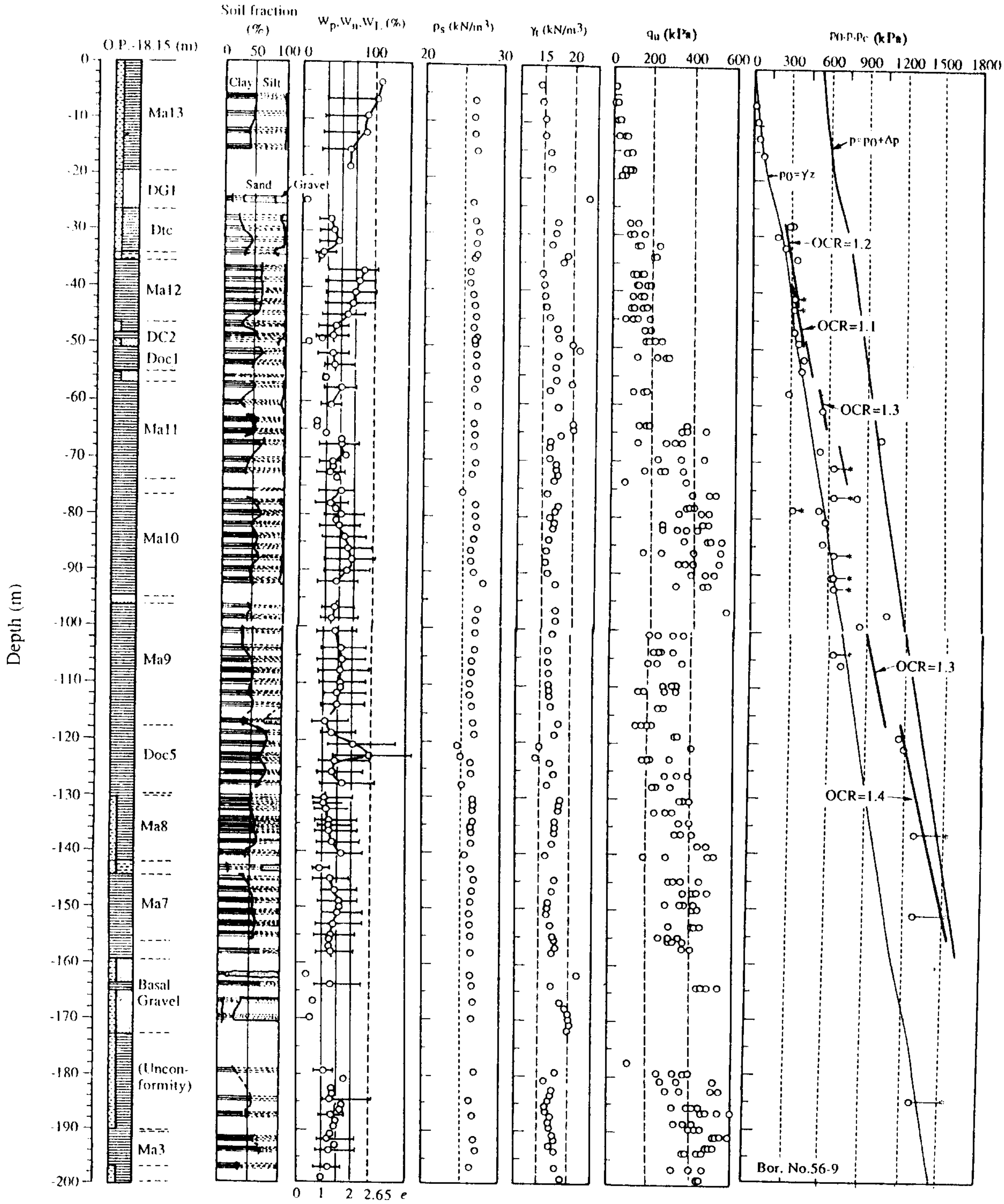

Fig.3 Soil profiles and index properties of seabed soils.

The surface elevation of the airport island is specified to be not lower than +4.0m above seawater level, 50 years after the opening of the airport. The depth and volume of the reclamation soil had to be determined by considering the settlement of the seabed subsoil. The total amount of settlement in these subsoil was estimated to be approximately 11.5m after 50 years. Accordingly, soil reclamation of about 33m in depth and about 180 million m^3 in volume was required to form the island.

After all sanddrain piles of about 1 million were driven to the alluvial clay stratum of seabed, the relamation work started at the end of 1988 when the entire seawall emerged above the water surface (**Fig.5**). The earth and sand used for reclamation were transported by berges from several excavation sites in the hill area located at the distance of 10km to 30km from the airport island. The volume of soil handled per month during peak period reached approximately 5.5 million m^3. The reclamation works completed by the end of 1991.

Settlement of seabed due to the reclamation load

Accuracy in predicting future settlement is the most important factor in KIA project; since a large error in settlement prediction would lead to a large change in the amount of fill material required for maintaining the specified final surface elevation (+3.2m above the sea level) of the airport island.

In the design, the settlement prediction for the alluvial clay stratum where the sanddrain is executed extensively was carried out by the m_v-method, and the settlement-time curve was calculated by Barron's theory. The result of calculation, shown in **Fig.6**, indicates that the settlement of approximately 6.0m would occur in the alluvial clay beneath the reclaimed land and almost stop at 6 months after full surcharge. The observed settlement is also plotted in the figure. Accordance between the observed and calculated settlements is seen quite well, when we take the soil parameters which have been obtained by soil tests in the laboratory.

The overburden stress due to the load of reclaimed soil is as much as 500 kPa and exceeds the maximum past pressure to the depth of about 150m, whereby the pleistocene layers are also expected to consolidate. In the settlement analysis for the pleistocene layer, the determination of drainage layers is rather complex, because of the existence of many alternating clay and sandy or gravel layers. As also shown in Fig.6, the compression of the pleistocene layers has become remarkable after beginning of reclamation above the sea level. In June 1993 it has reached about 4.5m and keeps a rate of 4cm per month. A final estimation of the amount of compression in these layers is some 5.0m at the opening of airport (September 1994) and 6.0m at ten years later. It means that the total amount of settlement in the seabed as the summation of compressions of alluvial stratum and pleistocene layers plus that of fill material itself (about 0.5m) is estimated as approximately 11.5m at the opening of airport and 12.5m at ten years later, respectively (refer **Fig.7**).

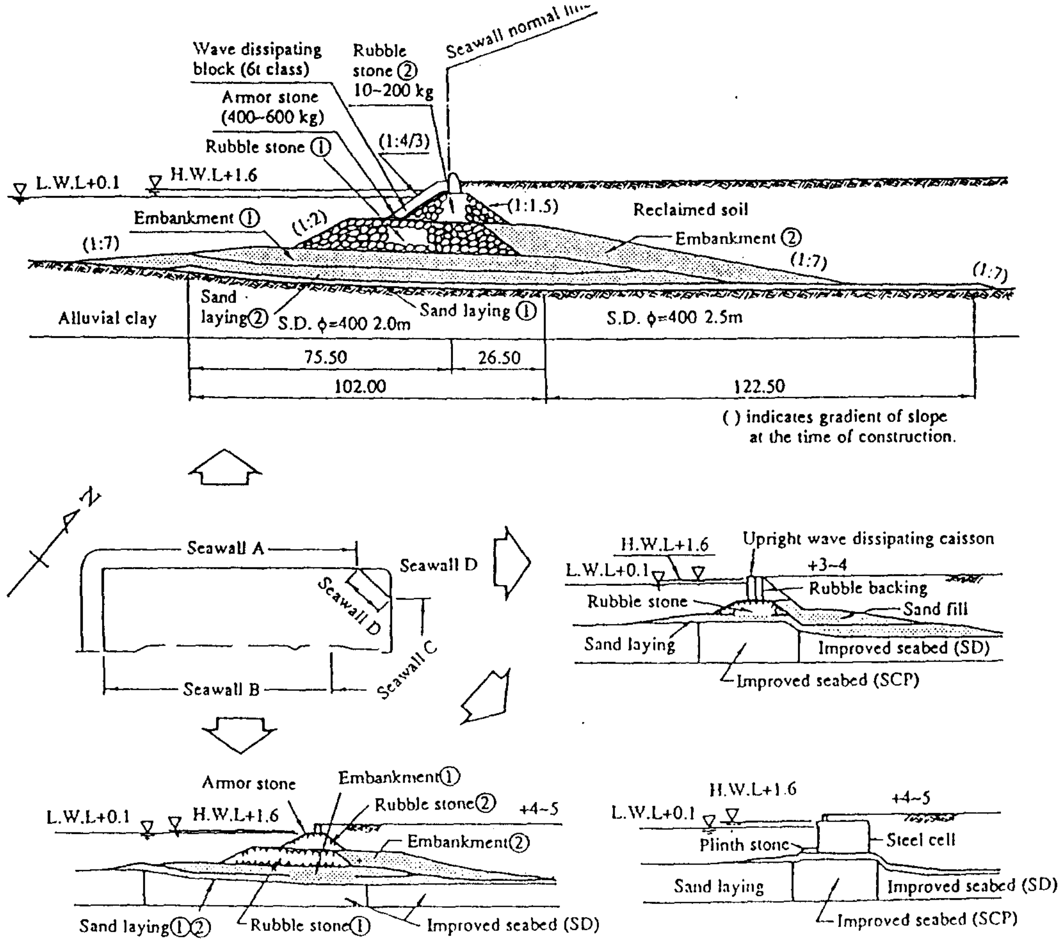

Fig.4 Seawall structures and their locations.

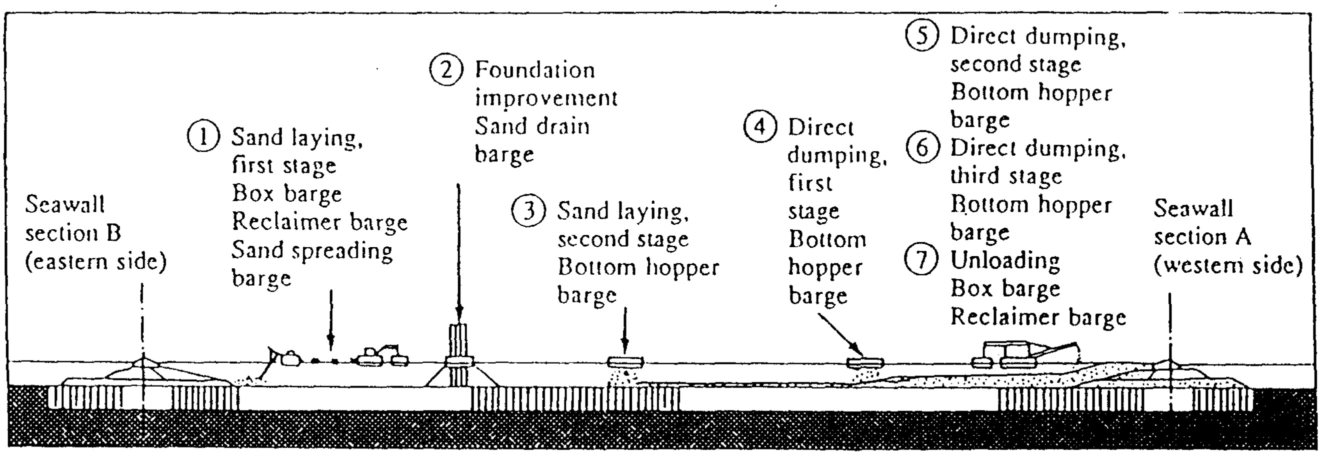

Fig.5 Conceptional view of reclamation work.

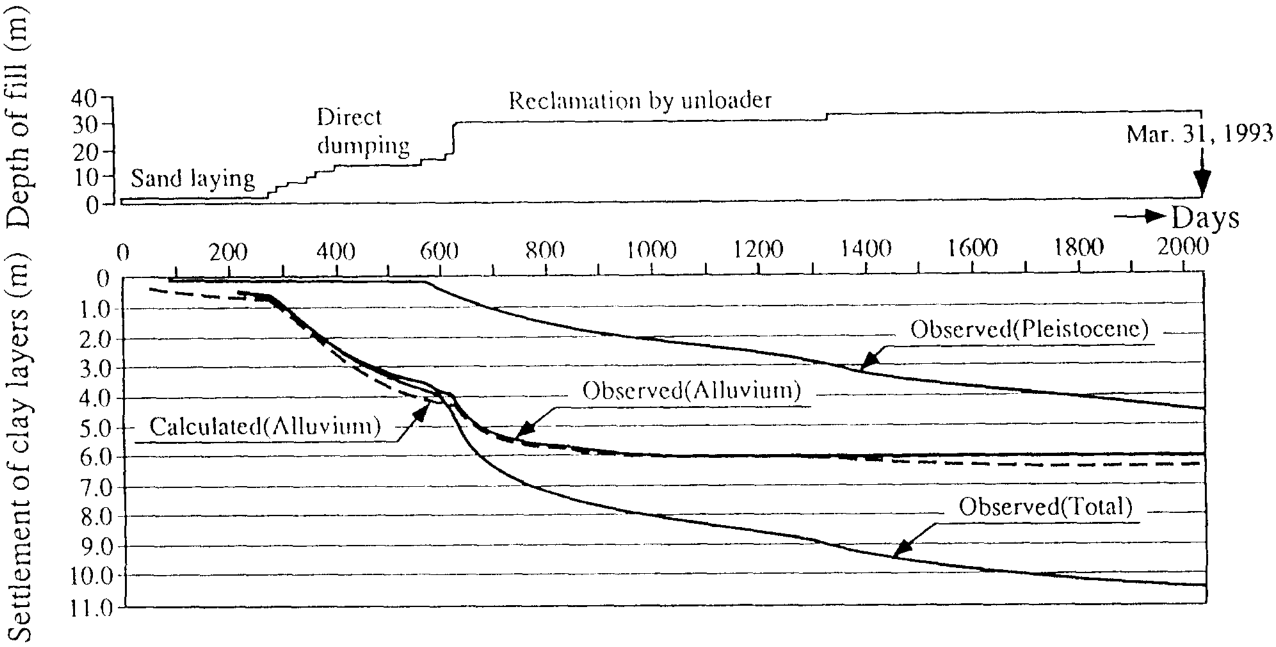

Fig.6 Settlement behavior of airport island.

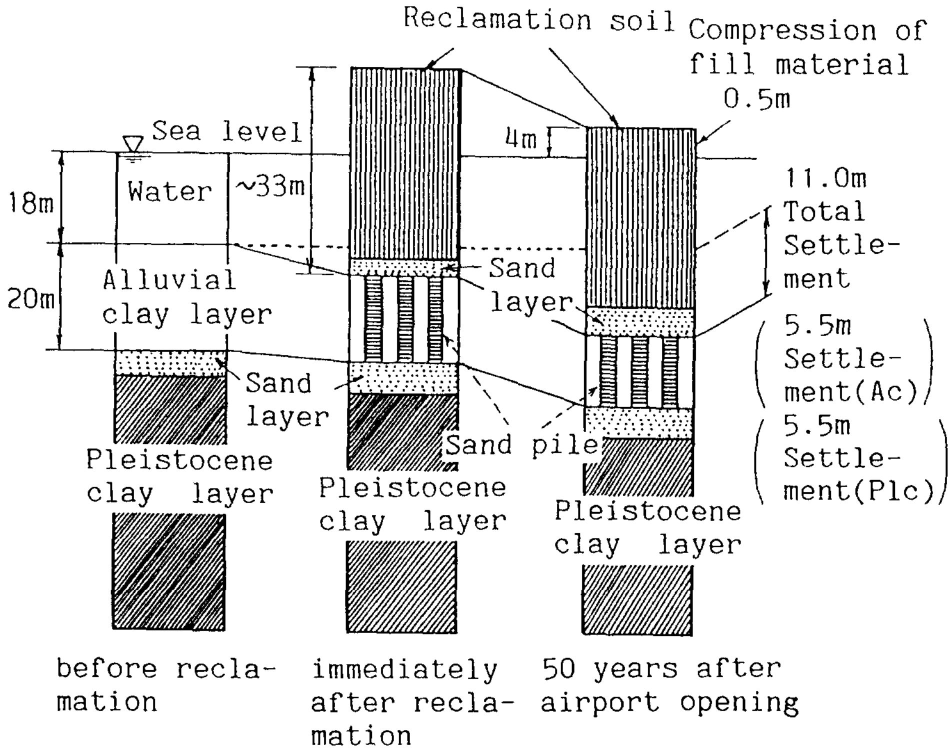

Fig.7 Settlement of seabed soils due to reclamation.

No one has experienced such a large amount of settlement in any past reclamation works. The main reason is considered as due to unknown behavior of the seabed materials, especially the compression behavior of young aged clays called the Upper Pleistocene Clays, Ma12 and Ma11. The measured re-cord of settlement layer by layer ascertains the fact that the main part of compression is occurring in the upper-most layers of young pleistocene.

Disaccord between theory and practice

There exist some remarkable phenomena indicating anomaly of compressive behavior of pleistocene clays which cannot be explained logically by the conventional theory of soil mechanics. Namely, the principal disaccords are summarized as follows:

1. As is clear from **Fig.8** which indicates the comparison between calculated and observed settlement in the layers of pleistocene clay, accordance in compression only of Ma12 in the upper part is fairly satisfactory. Respecting the middle part of pleistocene layers, however, the observed consolidation is clearly slower than the calculated values. Namely, the measured amount of settlement in toal layers is only about 55% of the estimation. Especially, the delay of consolidation is remarkable in Ma9 whose thickness is the largest occupying 18.3m in average.
2. Respecting the lower part of pleistocene layers, on the other hand, the observed value is about 25% larger than the calculated one. The strata included in these lower part are situated in large depth as more than120m below the mudline and their compressibility is very small. Therefore, the precision of measuring settlement is somewhat uncertain.
3. According to Fig.8, only three sand/gravel layers of large thickness can be assumed as having full drainage capacity of water squeezing out from clay stratum in between during consolidation; namely, the layer between Ac and Dtc, the layer between Ma8 and Ma7, and the layer underneath Ma7. In other thin sand layers, as shown in **Fig.9**, there remains excess pore pressure of remarkabe intensity which is comparable with that in adjacent clay strata. We are forced to adopt a virtual assumption of having complete drainage capacity for all sand/gravel layers in the settlement estimation, in order to explain the up-to-date record of the seabed behavior. However, this is not confirmed in terms of actual behavior of pore water pressure .
4. There is no correspondence between the settlement-time curve of individual clay stratum and the excess pore water pressure-time curve in itself. Settlement proceeds with about constant rate even while pore pressure increases or remains constant.
5. Measured amount of 'total' settlement of pleistocene clay layers is in good agreement with that calculated, as long as the record after full surcharge concerns. However, it should be recalled in mind again that the estimation is based on an irrational model where all sand/gravel layers, even of extreamely thin thickness, are treated as having complete drainage capacity.

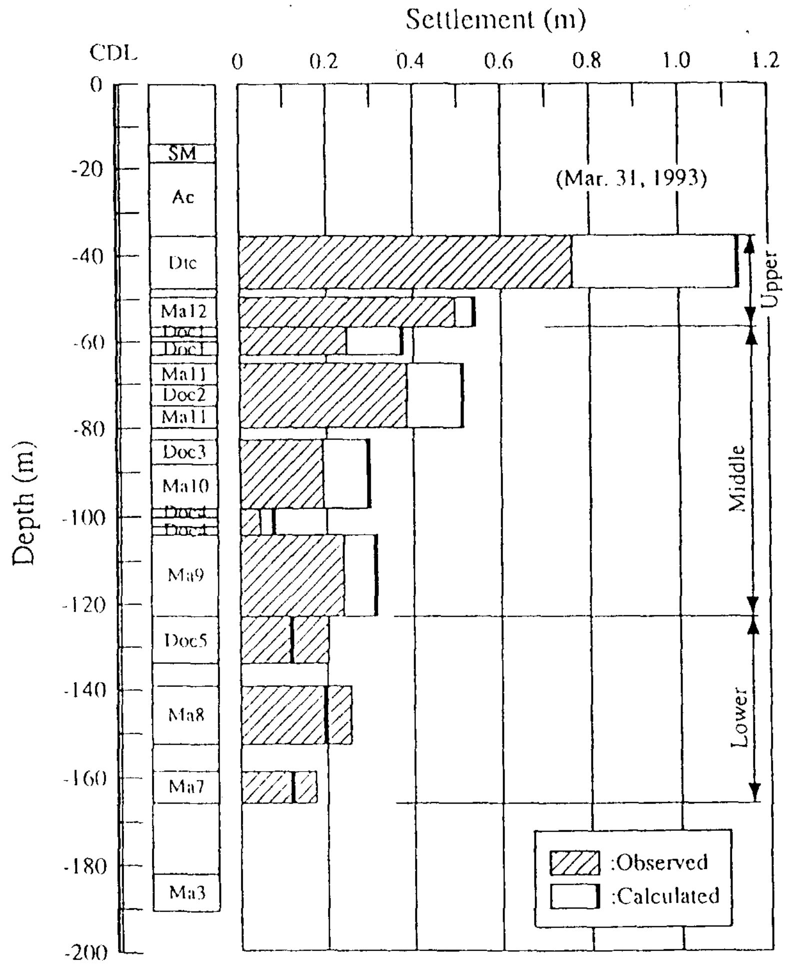

Fig.8 Comparison between calculated and observed settlement in the layers of pleistocene clay.

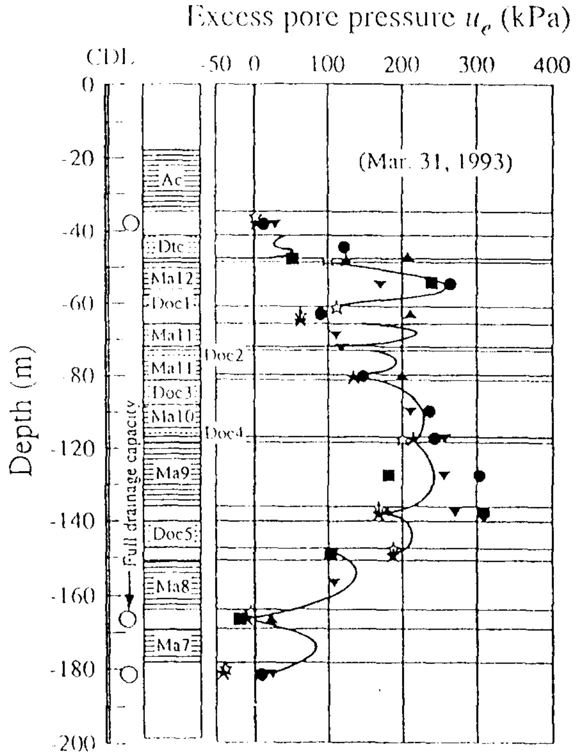

Fig.9 Distribution of excess pore water pressure in seabed soils.

Deformation behavior of aged clay

According to the geological research for the seabed in Osaka Bay, it becomes clear that the Upper Pleistcene Clays, Ma12 and Ma11, are classified as the young aged clays due to delayed consolidation because of their small over-consolidation ratio (refer Fig.3). **Fig.10**(a) is a schematic diagram showing consolidation behavior of such a quasi-overconsolidated clay subjected to vertical loading under the condition of one-dimensional compression and (b) shows the corresponding effective stress path. Here is proposed a concept of stress overshooting when this type of soil caused by aging and cementation is loaded in the field. The proposal is summarized as follows:

1. When the stress at the point B after sedimentation is appropriate to cause cementation between soil particles, a clay of delayed consolidation at the ploint D shows a locking behavior with stress overshooting until reaching the point E. Thenceforward, an abrupt destructuring of chemical bond established by cementation causes a large strain to the point G.
2. If the stress at the point B is too small, the chemical bond between soil particles cannot be produced due to insufficient aging period. If the stress at B is too large, on the other hand, the ordinary overconsolidation overcomes the stress overshooting.
3. Remarkable overshooting can be seen when the loading is directly applied to an in-situ soil under delayed consolidation at the point D. Soil sampling at field diminishes the memory of delayed consolidation. Therefore, the consolidation curve obtained in the laboratory becomes moderate likely to that of overconsolidated soil.

In Fig.10(b) the process of delayed consolidation is shown by the effectiive stress path BD along which there occurs an increase in K_0-value. By loading the clay shows a locking behavior DE and the effective stress path rises up nearly equal to p = const. The chemical bond established during delayed consolidation destructures at the point E and, at the same time, there occur a sudden vertical strain and excess pore water pressure u_d. Thus, EF_1F_2 is the effective stress path just after destructuring of soil particles due to loading. The point F_2 is situated on the K_0*-line which shows larger K_0-value than that in normally consolidated state. The effective stress path in subsequent loading is on the new normally consolidated line F_2G and the point G corresponds to the end of primary consolidation.

Dynamic response of access bridge

The access bridge is a 3.75km-long dual purpose (vehicular and rail) route connecting the airport island with the coastal area which has been relaimed and developed to provide support for the airport services.

The central portion over open water (2,700m in length) is a steel truss, with an upper vehicular expressway and lower rail tracks. Each of the two approaches to the access bridge (airport island and mainland) have separate steel box girder structures for the road and rail facilities.

The design of the bridge substructure required a total of 29 piers on steel pile foundations with the standard span length of 150m.

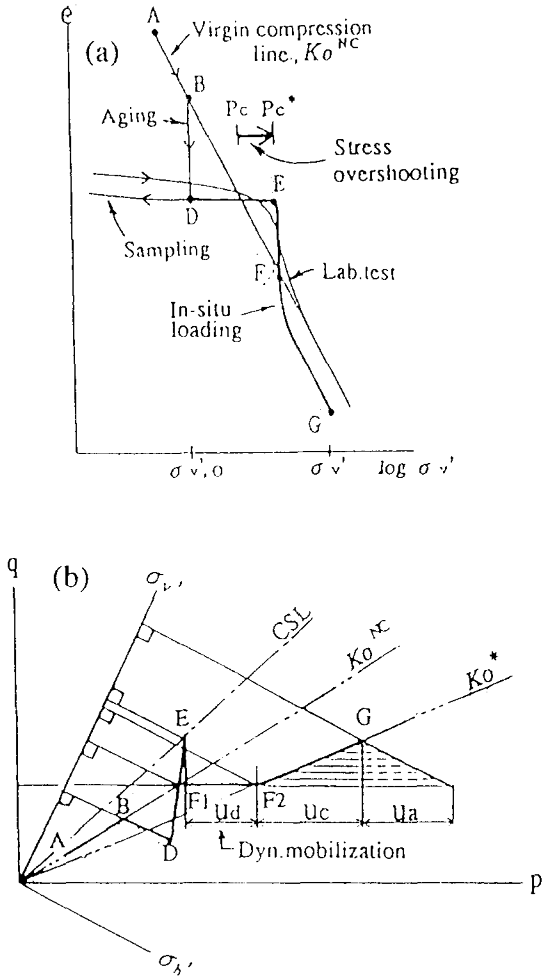

Fig.10 Schematic diagram illustrating deformation behavior of aged clay.

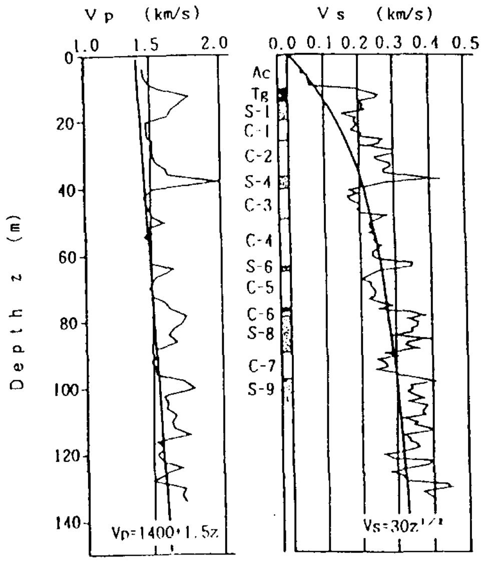

Fig.11 PS-logging data at the boring site in Osaka Bay.

Fig.11 indicates an example showing distribution of celerities with depth obtained by PS-logging at the boring site in the seabed of Osaka Bay with the water depth of 15.2m. From this figure, the following empirical equations can be established for clay layers with an exception of the Upper Pleistocene Clays (C-1, C-2). Namely, for the celerity of longitudinal wave v_p ,

$$v_p = 1400 + 1.5z$$

and for the celerity of shear wave v_s,

$$v_s = 30z^{0.5}$$

where z denotes the depth from mudline in meter and the dimension of celerities is expressed by m/s.

The shear modulus of ground G can be obtained from the celerity of shear wave v_s as

$$G = \rho v_s^2 = 900\rho z$$

where ρ denotes the bulk density of ground. Thus, it is known that the shear modulus G of the seabed in Osaka Bay approximately increases proportionally with the depth from mudline.

Respecting the strain-dependency of the shear modulus G and the damping ratio h, Hardin-Drnevich model is used in the analysis (refer **Fig.12**).

In the ground model for KIA project shown in Fig.11, the alluvial clay stratum (Ac) just below the mudline which has a thickness of 10.5m is divided into four thin layers in order to grade up the accuracy of dynamic response analysis.

An acceleration reord of EW-component of Tokachi-oki earthquake (Hokkaido) in 1968 (M=7.8) shown in **Fig.13** is used as the input seismic wave in the research. The wave form at the seismic base in the observed site, Hachinohe (a city at the north part of Japanese main island), is back calculated, and then put into the seismic base ground of KIA. The maximum acceleration of the wave obtained in this manner is 105.3gal.

The distribution of dynamic response values with depth under mudline is illustrated in **Fig.14**, in which they increse gradually with decrease in depth in the pleistocene layers. It is known that the seismic responses amplify themselves remarkably in the top alluvium. The maximum shear strain becomes very large near the mudline, whereas it is quite small below the terrace sand/gravel deposit (Tg).

Structural figures of the bridge pier to be analysed are indicated in **Fig.15**. Analysis is performed for the section of bridge axis direction. The bridge pier is made of steel box whose section is 5mx5m, filled with concrete until the height of 5m above the sea level after setting into the seabed. The foundation structure of pier is 7x8pile group (ϕ=1,5m) designed as friction piles whose tips reach a depth of 69.2m below the sea level in the clay layer, C-4.

In the research, two-dimensional analysis is performed, so that input parameters are evaluated as values per unit meter of thickness. The ground and the pier are modeled as solid elements, whereas the piles are modeled as beam elements in FEM. **Fig.16** illustrates the analytical model together with FEM-mesh. The base of FEM-mesh is at the lower surface of C-5.

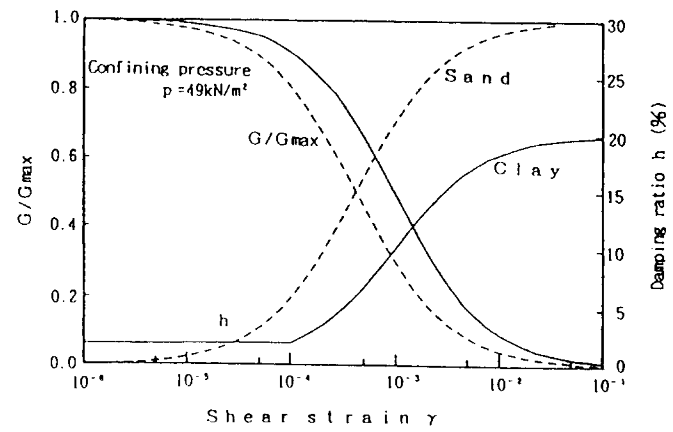

Fig.12 Normalized shear modulus and damping factor *vs.* shear strain for ground model.

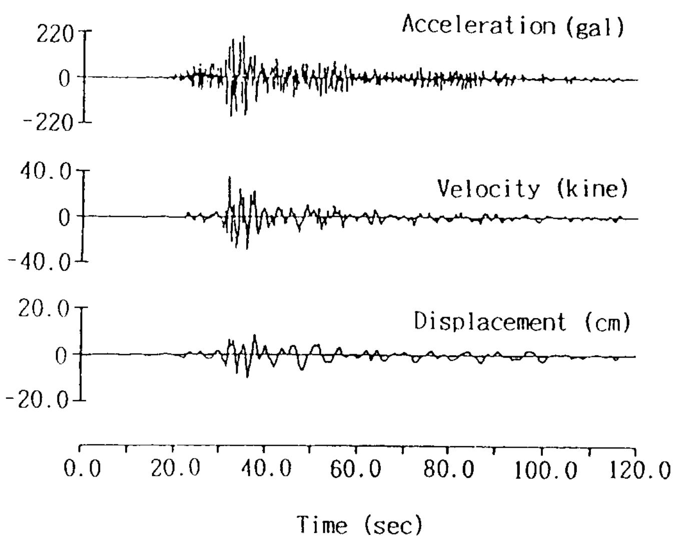

Fig.13 Characteristics of input seismic wave.

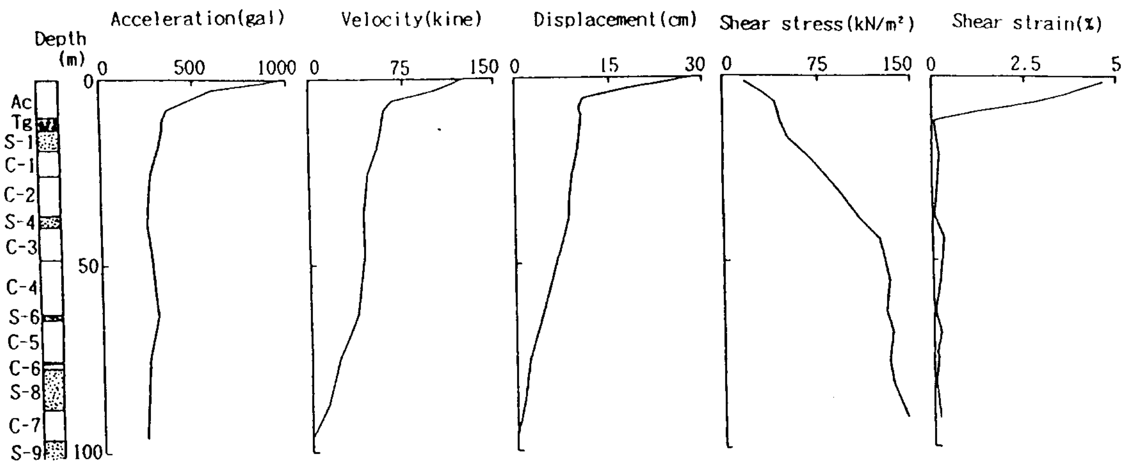

Fig.14 Distribution of dynamic response values with depth under mudline.

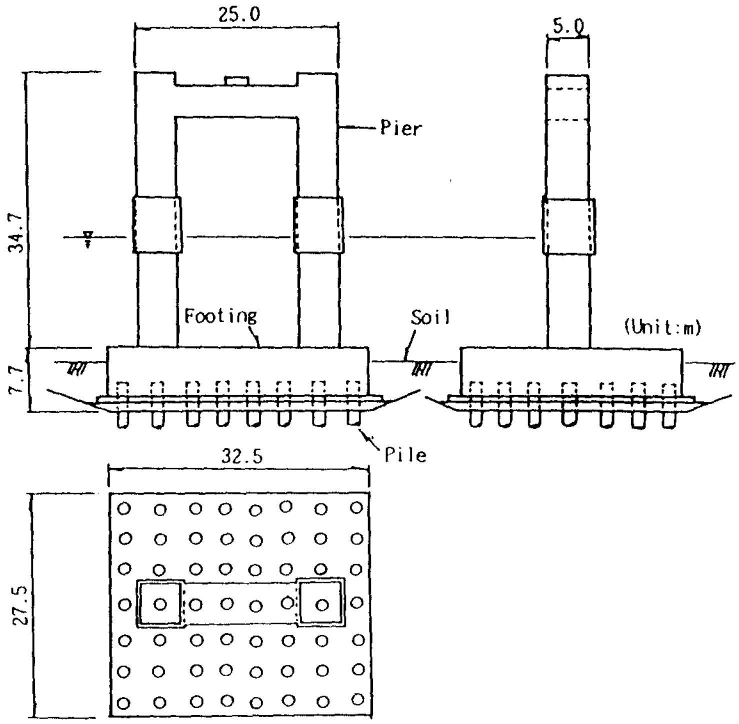

Fig.15 Structural figures of the bridge pier.

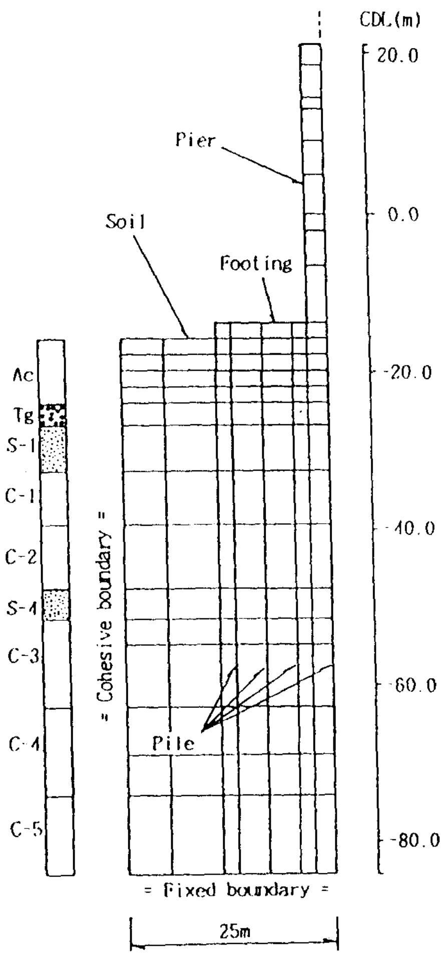

Fig.16 Analytical model of soil-structure interaction system.

In the seismic response analysis, the input wave shown in Fig.13 is modified as having the maximum acceleration of 100gal at the top surface of S-9 layer and brought into the lower surface of C-5. **Fig.17** indicates distribution of the maximum acceleration response at the nodal points of pier, pile and ground together with those of the maximum bending moment acting on the foundation pile. The amplification of acceleration is remarkable at the upper part of pier. The value at the top of pier reaches as large as about 4.6 times that at the base. The amplification in pile and ground is rather small, especially in the sand/gravel layers existing at the intermediate depth.

Passenger terminal building (PTB)
Since the foundation of the passenger terminal bulding with total length of 1700m (**Fig.18**) was designed as not to be supported on piles but on a spread foundation, the building will settle for long period with the foundation. It is expected that the weight of the main terminal building differs by 78.4kPa at most from that of the excavated material removed for construction of the underground level and the spread foundation as shown in **Fig.19**. Due to this difference, the settlement at the center of the building will be smaller than that at the perimeter of the building, causing differential settlement. It is expected that, after 50 years, this differential settlement will reach approximately 60cm, including that due to variation in soil parameters, and exceed the maximum degree to which level adjustment by jacks is possible.

In order to reduce the difference between the weight of the building and that of excavated materials, iron ore was placed beneath the building. A 2.5m-thick iron ore layer was formed so as to offset about 50% of the differential settlement. It is expected that, as a result of this measure, the differential settlement will be reduced to approximately 23cm.

In order to offset the remaining differential settlement, jack-up systems are introduced at the base of whole columms of the building. The details of the column base is shown in **Fig.20**. Normally, the column in this figure is bolted to the foundation. When level adjustment becomes necessary, the anchor bolts are loosened, and the column is lifted by jacks to adjust the level by installing or removing filler plates.

Closing remarks
In September of 1994, Japan will open the new major international airport (KIA) in Osaka Bay, located five kirometers offshore. KIA involves the creation of a large scale man-made island in an average of about 18m-deep water. The reclamation work for the 511ha island was carried out up to the end of 1991. A 3.75km-long bridge combined vehicular expressway and railroad will provide the main access to the airport island. The bridge construction over the sea finished in the beginning of 1992.

The KIA construction has been a challenging civil engineering project in view of its size, concept as a marine airport and the severe geotechnical conditions. Internatioal cooperations in construction of the airport facilities such as PTB and the control tower, *etc.* are being performed in full success.

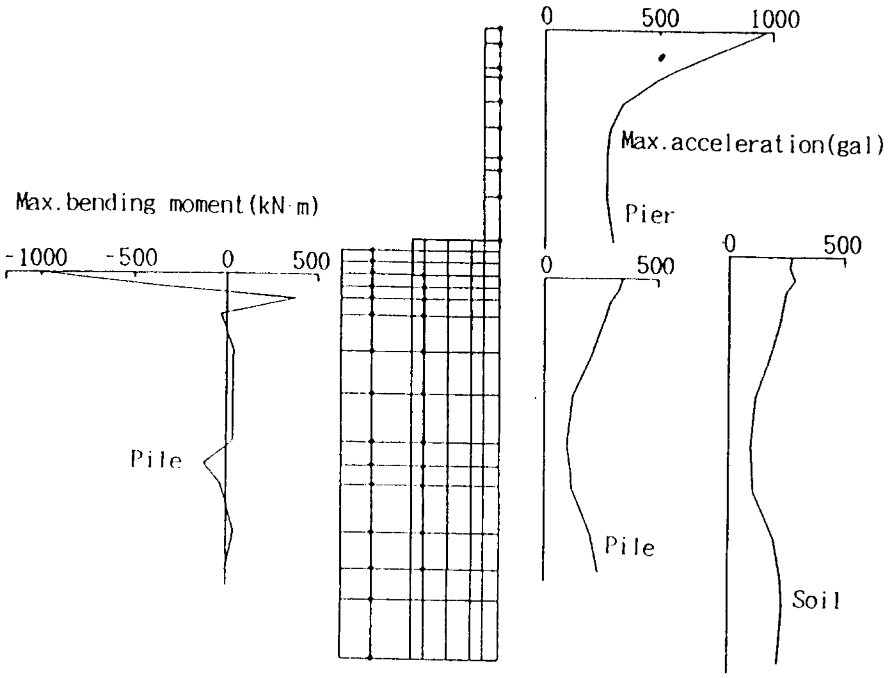

Fig.17 Distribution of the maximum acceleration response and the maximum bending moment.

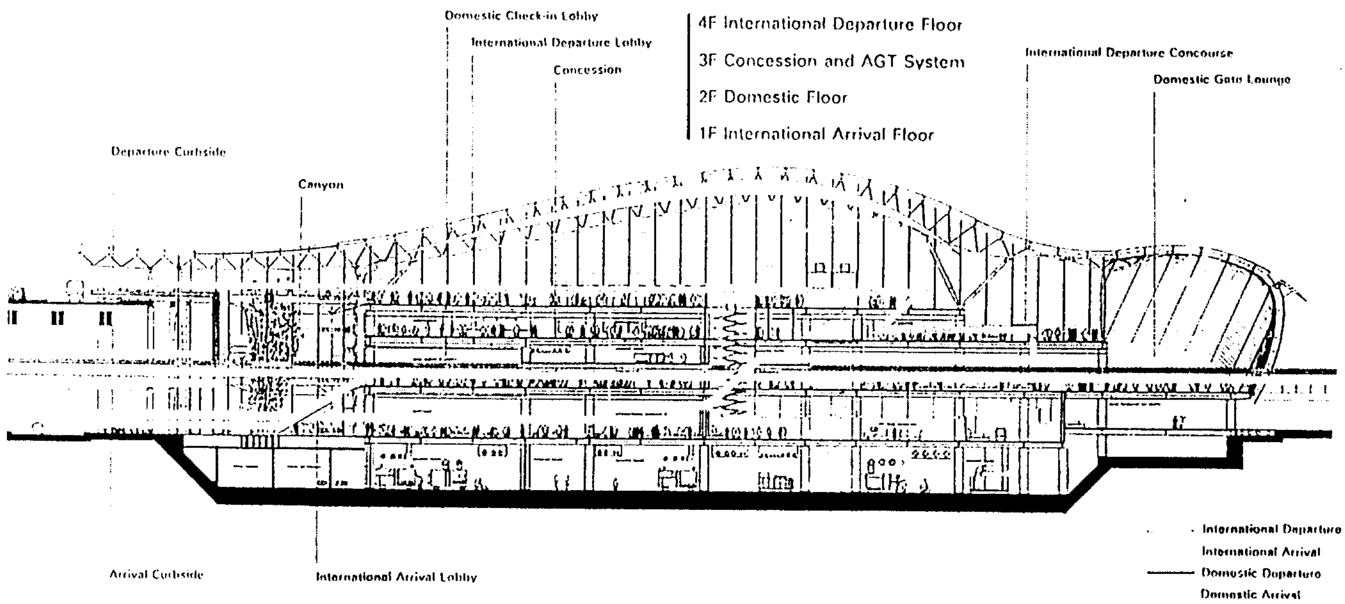

Fig.18 Terminal building levels and passenger circulation.

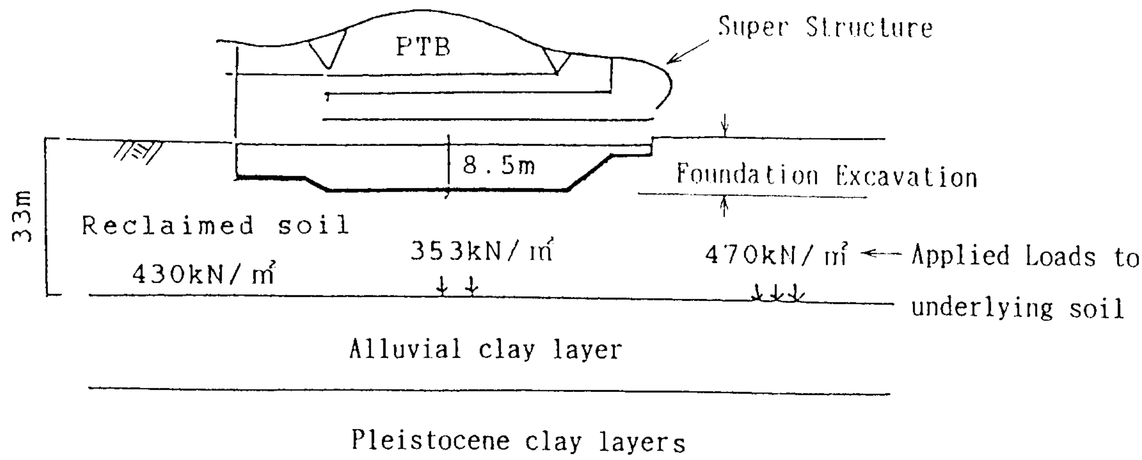

Fig.19 Conceptional diagram of load application for PTB.

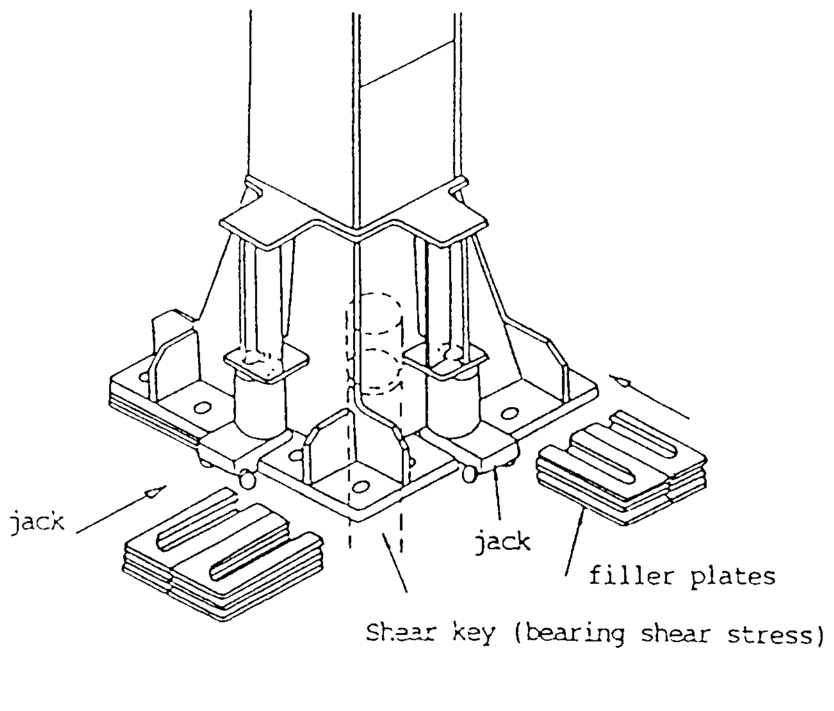

Fig.20 Concept of level adjustment at the bottom of columns.