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REPORT OF THE PORT AND
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# 7. Development of Design Method for Concrete Pavement on Reclaimed Ground - Its Application to Tokyo International Airport -

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## Synopsis

When airports are constructed on reclaimed ground, the design of the pavement should consider problems of the subsoil. Differential settlement of the ground will affect the behavior of the pavement in the future. Paving of the West Terminal area for the Tokyo International Airport expansion project is presented as a typical example.

This paper describes a system for managing airport concrete pavement on soft ground. Both the extent and schedule of rehabilitation works are calculated in two cases:

1) The gradient of the pavement surface deviates from the criterion because pavement subsidence fully follows subsoil settlement.
2) Evidence of distress appears on the surface, as the pavement does not follow the settlement fully.

This analysis was applied to the expansion project. In the former case, rehabilitation was found to be necessary a maximum of three times and over 60% of the whole area for a 20 year period. In the latter case, pavement management systems for 10 and 20 years were studied including economic considerations.

Rehabilitation is inevitable for the pavement with this ground condition after it is open to traffic. Rehabilitation will be needed not only to reinstate the pavement elevation, but also to strengthen the structure. The use of a thin bonded concrete overlay, for which construction procedures are not established, is discussed. Furthermore, the applicability of a lift-up method for rehabilitation of prestressed concrete pavement was studied. The validity of these procedures were confirmed during large-scale experimental pavement repair at the Tokyo International Airport.

**Key Words:** Airport Concrete Pavement, Pavement Management System, Rehabilitation, Differential Settlement, Reclaimed Ground

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# 7. 埋立地盤上におけるコンクリート舗装設計法の開発 —東京国際空港への適用—

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## 要 旨

埋立地盤上に空港を建設する場合，舗装設計においては地盤に起因する問題が生ずる。特に，建設後の不同沈下が舗装の挙動に影響を及ぼす。その点についてここでは東京国際空港の沖合展開事業地区を例にとって論ずる。

この論文では，軟弱地盤上の空港コンクリート舗装のマネージメントシステムが示されている。舗装の補修時期と範囲は以下に示す2つの場合に対して計算できる。

1）舗装が地盤の不同沈下に十分に追随して，表面勾配に関する基準から逸脱する場合。

2）舗装が地盤の不同沈下に十分には追随できないために，コンクリート舗装が破損する場合。

この解析は沖合展開事業に適用された。前者においては，著しい不同沈下が予想される地区で，20年間で60％程度の範囲で3回にわたって補修が必要になるものと推定された。後者では，解析期間を10年，20年として，経済性を考慮に入れたマネージメントシステムを策定した。

軟弱地盤上に舗装を建設する場合には，供用後の補修が必須のものとなる。この補修は，舗装高さの回復・調整のほか構造強化のためにも必要となる。ここでは，工法がまだ確立されたものとはなっていないコンクリートによる薄層付着かさ上げ工法が論じられた。さらに，プレストレストコンクリート舗装が沈下した場合の補修工法として開発されたリフトアップ工法の実用性についても考察された。これらは、東京国際空港で実施された実規模の試験舗装において検証された。

キーワード：空港コンクリート舗装，舗装マネージメントシステム，補修，不同沈下，埋立地盤

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## Contents

# 1. Introduction

Many civil engineering projects were executed in coastal areas where pavement is required for highways, airports, and yards. The ground in these areas is generally poor, which affects the pavement conditions. The anticipated subsoil settlement will cause some distress, such as surface cracks, distortion, or joint faulting in the pavement. However, the basis for designing pavement on such foundations is not described in the current airport pavement design method, which assumes relatively good soil conditions.

Two extreme approaches can be considered for pavement construction on poor ground conditions: the first is to construct pavement only where such troublesome soils can be avoided, while the other is to construct and manage the pavement without any particular consideration of the soils. Procedures between these extremes can also be considered.

Surface settlement will occur and, in the worst case, will cause differential settlement unless the first extreme is adopted. This differential settlement will cause problems in drainage and in vehicle operations. This is the argument for asphalt pavement that can follow the subsoil settlement fully.

On the other hand, concrete pavement is necessary for areas such as parking lots, aprons, and yards, regardless of the ground condition. Concrete pavement behaves in two ways, depending on its ability to follow the settlement:

1) The pavement surface grdient deviates from the prescribed limits, as the pavement fully follows the settlement, and
2) Unacceptable indications of distress appear on the surface, as the pavement does not fully follow the settlement.

This paper describes a system for managing airport concrete pavement constructed on reclaimed land that is susceptible to future differential settlement. The second stage of the Tokyo International Airport expansion project is discussed as a case study. First, the design principle for concrete pavement on the soft ground is developed. Then, both the extent and schedule of the rehabilitation effort are forecast. Finally, a system for the managing concrete pavement on soft ground is described. A typical countermeasure for settled concrete pavement is an overlay, with concrete overlays being preferred considering the loading conditions. A bonded concrete overlay is required for thinner overlays, although it has been seldom used in airports in Japan because of the insufficient reliability in its structure. New construction techniques to overcome this problem are described. In addition, a lift-up method has been developed to restore the pavement elevation. In this method, the prestressed concrete slab is raised by use of jacks, and the voids below the slab are grouted with a cement paste. The applicability of this method to actual airport construction is also presented in this paper.

# 2. Management Strategy on Soft Ground

Differential settlement of the subsoil affects the behavior of pavement constructed on reclaimed ground. Concrete pavement may behave in the following two ways (1):

1) It follows the settlement fully, or
2) It does not follow the settlement fully.

In the former case, the specified surface gradient may not be maintained. In the second case, pavement distress such as cracking may be caused by traffic loads because voids can be formed beneath the concrete slabs.

The surface condition of concrete pavement in Switzerland (2), is reported to be good in spite of differential settlement. Prestressed concrete (PC) pavements constructed 20 years ago at some reclaimed land sites in Japan are also in relatively good surface condition with few cracks, even though the pavement settled unevenly (90 cm, maximum). However, it cannot be determined from these limited instances whether the pavement followed the subsoil settlement exactly. Therefore, above two cases should be considered for managing pavement.

According to the current design procedure for airport concrete pavement in Japan, the slab thickness is determined in consideration of the load-induced stress and an appropriate safety factor (3). The stress is calculated using the Westergaard equation for the internal stress, while the safety factor depends on the anticipated aircraft coverage during the specified period. Problems arising from subsoil settlement are not anticipated in this procedure. Therefore, the thickness of the slab, which must sustain both the loss of support from subsoil settlement and the aircraft loads, cannot be determined.

Under these conditions, the primary policies of pavement management are as follows:

1) Residual settlement of the ground must be estimated accurately,
2) Pavement must have sufficient load-carrying capacity for the predicted differential settlement and the aircraft loads, and
3) Rehabilitation strategies for the pavement must be established against not only the grade change, but also loss of pavement support.

The pavement surface gradient must be kept within the specified range when pavement follows the settlement fully (4), and the rehabilitation procedures for restoring the pavement elevation are important. The necessity for rehabilitation works in the future will be able to be judged because quantitative forecasting of differential settlement will be possible (5).

An overlay is one possible way to accomplish this. Concrete overlays are preferable because of concrete's good ability to resist load-induced deformations. In addition, a lift-up method was developed to shorten the time that the facilities are closed (6). In this method, the PC slabs are raised by jacks, and the void under the slabs is grouted with a cement paste.

The pavement structure must have a sufficient bearing capacity to sustain the aircraft loads when the pavement does not follow the foundation settlement **Figure 1** shows a flow chart for calculating the slab thickness. First, the stress due to aircraft loads and the tem-perature variations in the slab must be calculated for the assumed slab thickness. Then, both fatigue and ultimate failure of the slab have to be examined. Joint failure must also be checked by using the Pavement Rehabilitation Index (PRI) to decide the necessity of re-habilitation (7). Finally, the pavement structure can be determined in accordance with the management strategy.

Based on the study, a pavement management system (PMS) for concrete pavement on reclaimed ground was assembled. First, rehabilitation that is necessary to correct the pavement surface gradient is presented. This method can be applied to all the concrete pavement structures. Then, rehabilitation necessary to restore pavement distress is presented for two cases: one where rehabilitation is unnecessary for 10 years, and the other for managing pavement for 20 years based on economics.

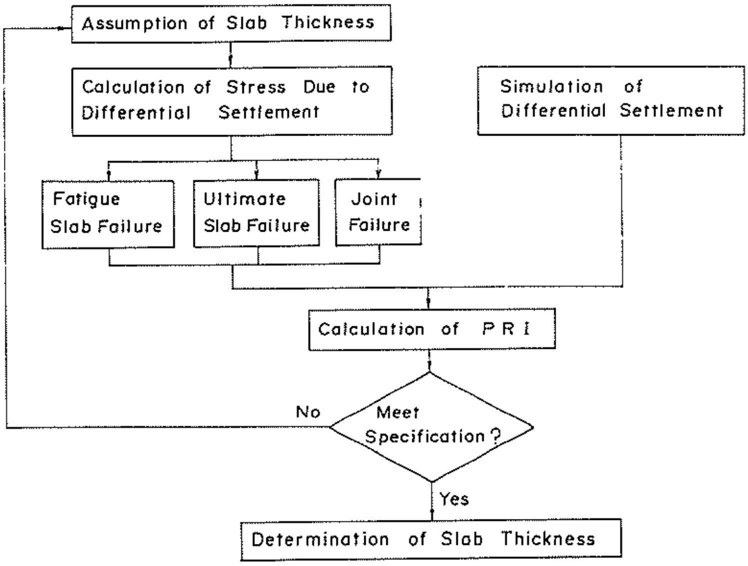


**Figure 1 Flow Chart of Slab Thickness Determination**

## 3. Offshore Expansion Project of Tokyo International Airport

### 3.1 Tokyo International Airport

Tokyo International Airport (Haneda Airport), which is located at the mouth of the Tama River in Tokyo, is the most important facility in the Japanese domestic aviation network. The annual volume of air passengers at Tokyo International Airport reached 40 million in 1990 This is approximately half of the number of annual domestic air passengers in Japan, and is still increasing steadily. With the increase in passengers and cargo, Tokyo International Airport has become seriously congested. It is impossible to cope with the increasing demand without essential expansion of the present airport. The offshore expansion project was therefore planned and implemented.

**Figure 2** shows a master plan of the Tokyo International Airport offshore expansion project. The project is undertaken in three construction stages to keep pace with the reclamation work The first stage of construction included the construction of Runway A and was completed in March 1986. In the second stage, the west terminal area, including aprons, a passenger terminal building, and other facilities will be completed. The concrete pavement for the apron is the subject of this paper. The East Terminal area and Runways B and C will be constructed in the third stage. After completion, the landing and take-off capacity will increase from 180,000 at present to 230,000 per year.

In the second stage, about 100 hectares of aprons are scheduled to be completed. 80 gates are planned there for loading and unloading passengers, baggage and cargo, maintaining aircraft, staying overnight, and other operations as shown in **Table 1.** The maximum number of aircraft at each gate will be fourteen per day. Other design requirements for concrete pavement in this section are as follows:

1) Design aircraft : B-747-200 B
2) Design coverage : 20,000

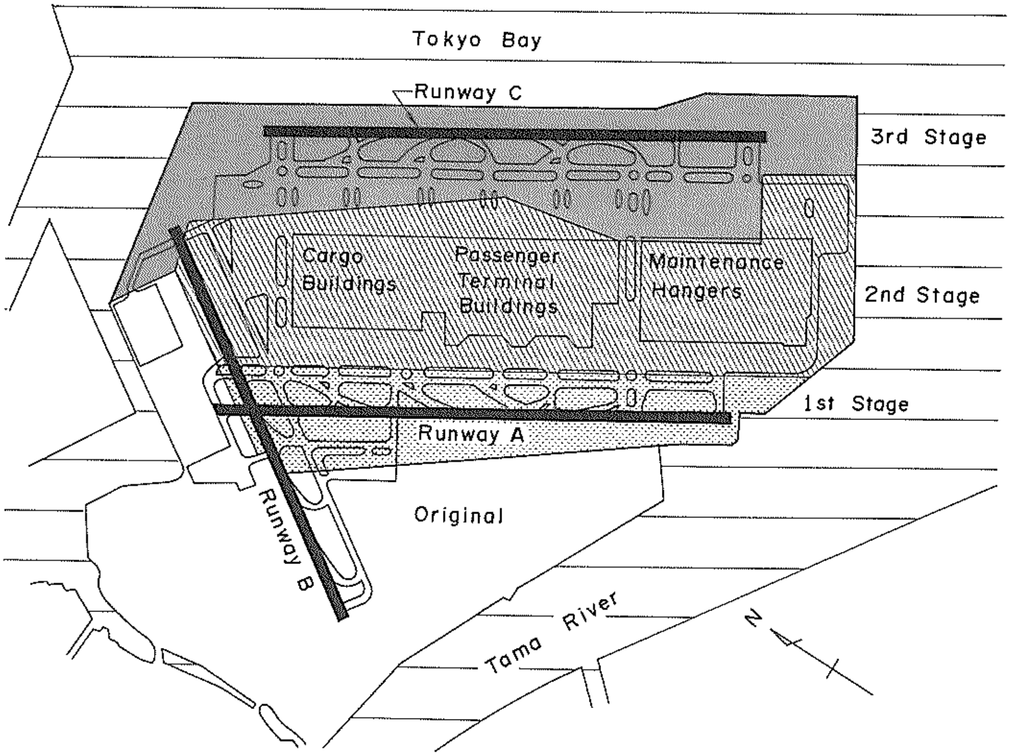


**Figure 2 Master Plan of the Tokyo International Airport Offshore Expansion Project**

**Table 1 Aircraft Operating Condition**

| Section | No. of Aircraft | Planned Surface Gradient (%) |
|---|---|---|
| Passenger Processing | 14 | 0.6 – 1.0 |
| Cargo Handling | 4 | 0.6 |
| Maintenance | 2 | 0.6 – 0.9 |

3) Subgrade reaction coefficient : 69 MN/mm³

This table also shows the surface gradient of the pavement which was initially planned to meet the specification (between 0.5 and 1.0%) (4).

## 3.2 Soil Characteristics

**Figure 3** (8), which describes the representative soil profiles at the site, shows that the boundary between a clay layer ($AC_1$) and a sand layer ($AS_1$) is both uneven and complex. Severe ground settlement is inevitable because the time necessary for full consolidation cannot be provided before the airport facilities are constructed. Therefore, considerable differential settlement will be unavoidable after completion, as

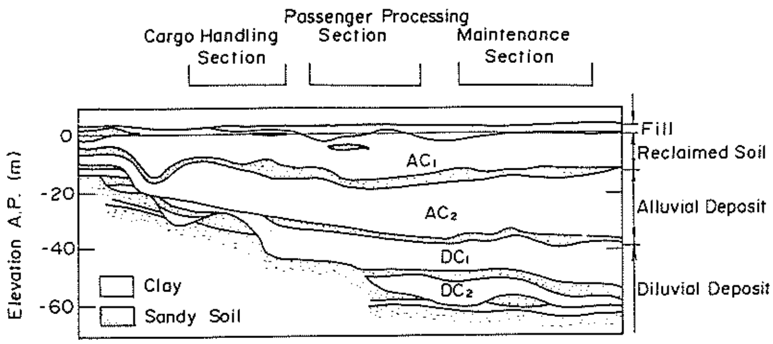


**Figure 3 Representative Soil Profile**

well as during construction.

The amount of differential settlement due to subsoil consolidation can be estimated analytically as mentioned above. The procedure described in references 5 and 9 was used here. The calculated differential settlement is used as described below. First, numerical simulation of the future settlement of the construction site is carried out, in which the soil investigation results are used as input data. Then, the differential settlement is calculated in two ways (**Figure 4**):

1) Differential settlement is defined by the gradient between two neighboring points that are used to check the surface gradient, and
2) Differential settlement is defined by the difference of the settlement at two adjacent points 30 m apart. The surface configuration is assumed to be represented as a cubic curve for expressing voids beneath the concrete slab.

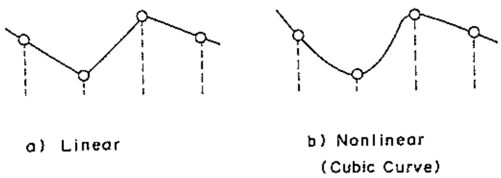


**Figure 4 Definition of Differential Settlement**

## 4. Examination of Surface Gradient of Pavements

When the pavement follows subsoil settlement, the pavement surface will similarly subside. The maximum change of the surface gradient increases with time most dramatically just after construction, as shown in **Figure 5**. Some repair is necessary for surface gradients that deviate from the range specified in the design standard (i.e., 0.5 - 1.0%). The extent and the timing of rehabilitation necessary in the future are described in this section.

The need for rehabilitation is defined here by the percentage of the area where repair is required to the whole area. The initial allowable gradient change (Ga) is also specified as the smaller difference between the initially planned gradient and 0.5 or 1.0%. **Figure 6** indicates changes in the need for rehabilitation over time. The need for rehabilitation differs significantly depending on the allowable gradient change because the planned surface gradient is not uniform over the whole apron area.

This will be the case when the specification must be observed strictly. However, repairs are requested when any part of the apron area cannot function well, not immediately after the actual gradient deviates from the specification. For example, aircraft cannot be towed when the gradient becomes much larger than 1.0%, and drainage problems will occur when the gradient is less than 0.5%. Therefore, the critical gradient for commencing rehabilitation is assumed to be 1.2% for the former and 0.3% for the latter.

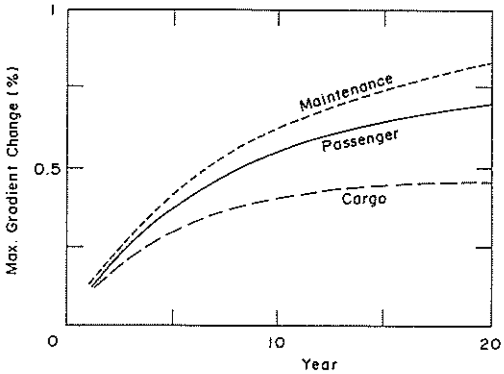


**Figure 5 Maximum Gradient Change**

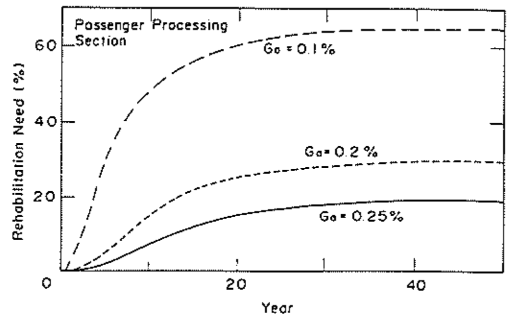


**Figure 6 Changes of Rehabilitation Need with Time**

Therefore, pavement with surface gradients that deviate from the specified range (0.5 - 1.0%) will be completely repaired when the gradient deviates from these critical values at even one point. Using **Figure 5**, the time at which rehabilitation will be required can be determined for various allowable gradient changes, as shown in **Table 2**. The extent of the repair work, which can be decided by using **Table 2** and **Figure 6**, is described in **Table 3**.

Rehabilitation may be required again, even in areas where the surface gradient has been restored beforehand, because the foundation will subside continuously. Therefore, two cases are summarized in the table: namely, when further rehabilitation works will and will not be required for areas repaired previously. However, the exact amount cannot be determined with present technology although the actual amount must surely be between these extremes.

Table 2 Time for Rehabilitation Works

| Section | Ga (%) | Time for Works (Year) | | |
|---|---|---|---|---|
| | | 1st | 2nd | 3rd |
| Passenger Processing | 0.25<br>0.20<br>0.10 | 7.0<br>5.5<br>3.4 | 10.4<br>11.9<br>7.7 | <br><br>19.1 |
| Cargo Handling | 0.25<br>0.20<br>0.10 | 5.7<br>4.6<br>3.0 | 15.8<br>9.4<br>6.6 | <br>17.9<br>13.1 |
| Maintenance | 0.25<br>0.20<br>0.10 | 16.4<br>9.1<br>4.9 | | |

Table 3 Rehabilitation Need

| Section | Ga (%) | Rehabilitation Need (%) | | |
|---|---|---|---|---|
| | | 1st | 2nd | 3rd |
| Passenger Processing | 0.25<br>0.20<br>0.10 | 4.5<br>5.6<br>18.0 | 8.3 (3.8)<br>18.5 (12.9)<br>43.6 (25.6) | <br><br>60.1 (16.5) |
| Cargo Handling | 0.25<br>0.20<br>0.10 | 2.7<br>5.6<br>18.4 | 16.8 (14.1)<br>16.2 (10.6)<br>39.9 (21.5) | <br>28.8 (12.6)<br>56.4 (16.5) |
| Maintenance | 0.25<br>0.20<br>0.10 | 2.4<br>4.3<br>18.1 | | |

Figure in the parenthesis means rehabilitation need in new portions.

## 5. Examination of Pavement Distress

### 5.1 Distress in Concrete Slabs

In contrast with the above discussion, the situation where the pavement does not follow subsoil settlement is considered in this section. Some distress such as cracks and joint faulting will result in this case.

**(1) Type of Failure**

Rehabilitation is required when the surface distress becomes severe as well as when the surface gradient deviates from the specification. In Japan, this is judged by the Pavement Rehabilitation Index (PRI), which is calculated from three kinds of surface distress. The necessity for rehabilitation is also decided based on the condition of the type of distress. For this latter criterion, rehabilitation is required when the cracking index exceeds 11.1 cm/m$^2$ or the joint failure rate exceeds 5.7% (7). To utilize these criteria, the following three types of failure are considered in this paper:

1) Ultimate failure of the concrete slabs
2) Fatigue failure of the concrete slabs

3) Joint failure

A concrete slab is considered to have failed when the total stress due to the loads and temperature changes exceeds the flexural strength of the concrete. Miner's law is used to assess fatigue failure, where both load-induced and thermal stresses are taken into consideration.

Joint failure is assumed to occur when the stress in either the dowel bars or the nearby concrete exceed their strength. Based on the analysis described later, the joint will not fail for 20 years after construction.

**(2) Computation of Stress**

The finite element method was used to compute the load-induced stress of the slab (10). In this method, the concrete pavement is modeled as several slabs connected by vertical springs at the joints, under the Winkler foundation condition. The method is applicable to concrete pavement where voids exist beneath the slabs. Based on the preliminary analysis, the slab was supported at the four corners in the worst situation. Accordingly, the concrete slab is considered to deform against differential settlement as shown in **Figure 7**. In the analysis, both one landing gear (four wheels) and two gears (eight wheels) on one slab were considered.

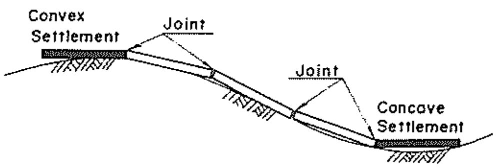


**Figure 7 Deformation of Concrete Pavement**

The stress calculation consists of two steps. The stress caused by the weight of the slab is computed in the first step, and the stress due to the aircraft load is calculated in the second step. In the former, the creep characteristics of the concrete were taken into account because the subsoil settlement increases gradually over a long period of time. Two loading conditions were studied for the stress due to aircraft: one at the slab center and the second at the joint.

**Figure 8** shows the relationship between the maximum load-induced stress and the differential settlement. In the figure, two kinds of settlement (concave and convex)

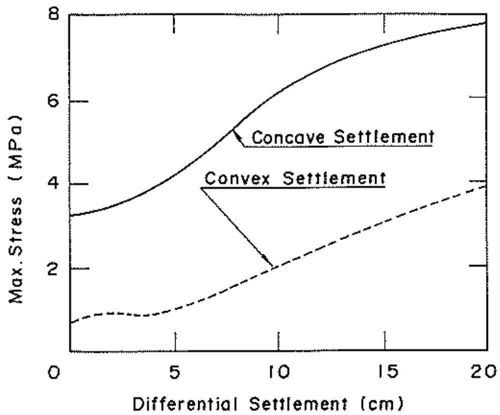


**Figure 8 Maximum Concrete Stress**

are considered for the standard design condition. In the standard condition, 38 cm thick jointed concrete slabs are placed on the foundation with a reaction coefficient of 69 MN/m$^3$. This thickness is clearly insufficient for larger subsoil settlements.

Thermally-induced stress is calculated by the use of Iwama's equation, which was adopted for concrete road pavement design in Japan (11). The fatigue curve used for road pavement design is also used with some modifications for the stresses due to slab weight and temperature changes. The stress in the dowel bars is calculated using the procedure described in reference 12.

**(3) Critical Differential Settlement**

The critical differential settlement is defined as the settlement at which the concrete slab fails and is shown in **Table 4** for ultimate failure. **Figure 9** shows the critical differential settlement for a fatigue failure case of a 38 cm thick slab with 14 aircraft loadings per day. The critical settlement increases with increasing slab thickness for ultimate failure and with decreasing aircraft loadings for fatigue.

The predominant type of failure may be judged by comparison of **Table 4** and **Figure 9**. Thus, the critical differential settlement of the concrete slab may be determined.

**Table 4 Critical Differential Settlement for Ultimate Failure**

| Slab Thickness (cm) | Differential Settlement * (cm) |
|---|---|
| 38 | 0.8 |
| 40 | 2.0 |
| 45 | 3.2 |
| 50 | 3.8 |

* Differential settlement is defined by difference of settlement between two points 30 m apart.

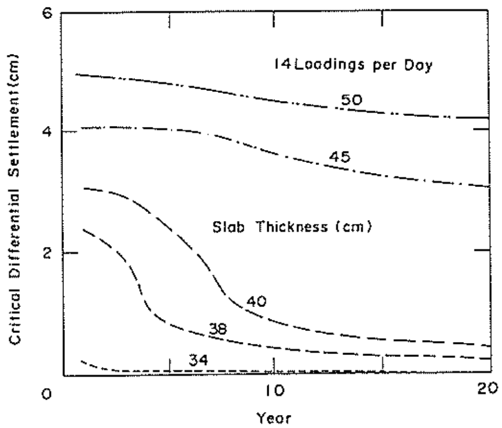


**Figure 9 Critical Differential Settlement for Fatigue Failure**

**(4) Calculation of Distress Severity**

Cracks are caused when the surface gradient exceeds the critical gradient. The cracking index is determined as described below. First, the surface gradient at any point should be calculated based on the numerical simulation results for the subsoil settlement. Then, the area for which the surface gradient exceeds the critical gradient is determined by transformation of the critical differential settlement. Finally, the cracking index is calculated using the crack length defined by the following equation, which is based on the assumption that cracks occuring two directions:

$$\text{CRACK LENGTH} = 2L(Sr/S) \qquad (1)$$

where, L : the length of the slab,
Sr : the area that exceeds the critical gradient, and
S : the area of the concrete slab.

**Figure 10** shows that the required rehabilitation works increase with time. This figure suggests that thicker concrete slabs are distinctly advantageous for decreasing repairs.

## 5.2 Framework for Management System

The management system for the concrete pavement at Tokyo International Airport is assembled in this section, based on the above considerations. Because the method for judging rehabilitation from the pavement surface gradient can be applied to any type of pavement, rehabilitation required due to surface distress is described herein. Two examples are considered; one where rehabilitation is not required for 10 years, and the other where the pavement management system is used for a 20 year period, based on optimizing economic considerations.

The execution of rehabilitation is not practical for extremely small areas. Two types of realistic strategies are considered below.

First, rehabilitation should be undertaken when surface distress is apparent on 10% of the area of each gate. The critical differential settlement at which the pavement fails 10 years after construction is considered here. The thickness of the concrete slab corresponding to ultimate failure can be obtained using **Table 4** together with **Figure 10**, while that for fatigue failure can be determined using **Figures 9** and **10**. For ultimate failure, slab thickness of 38, 38 and 39 cm are required for the passenger processing section, the cargo handling section, and the maintenance section, respectively. For fatigue failure, thickness of 40, 38 and 41 cm are necessary for these locations. Therefore, it is concluded that 40, 38 and 41 cm are necessary for these locations, respectively.

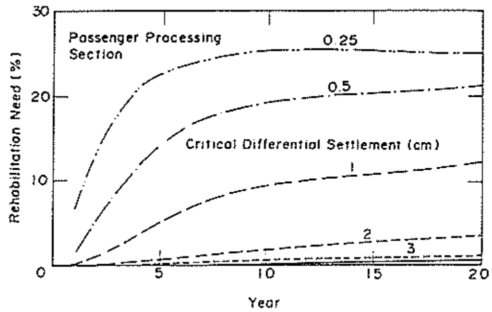


**Figure 10 Rehabilitation Need and Year**

Next, rehabilitation work will be executed when 1% of the area at a gate is deteriorated, which may be more practical for the busy Tokyo International Airport. The initial thickness of the constructed concrete slab is considered for the following three cases:

CASE 1: concrete pavement 38 cm thick,
CASE 2: concrete pavement without any rehabilitation for 10 years, and
CASE 3: concrete pavement without any rehabilitation for 20 years.

Using the same procedure, initial slab thicknesses of 43 and 45 cm were obtained for CASE 2 and CASE 3, respectively. The effect of these results on the rehabilitation schedule is shown in **Table 5**. Increasing the slab thickness by 5 cm can reduce the number of re-habilitation works: three times while eleven times for the standard resign case.

The present worth of each case was calculated based on current pavement construction costs with a 6% discount rate. The results show that there is little difference in these three cases when only reconstruction is considered for ease of calculation. This is based on the fact that the costs for only construction and the rehabilitation are considered in this analysis. Therefore, other items such as the difficulty in obtaining budgets, the easiness of aircraft operation, and the reliability of the rehabilitation work, should be taken into account to establish the economically optimum strategy.

**Table 5 Time and Area of Rehabilitation Works**

| Year | Percentage of Units (%) | | |
|---|---|---|---|
| | CASE 1 | CASE 2 | CASE 3 |
| 1 | | | |
| 2 | | | |
| 3 | | | |
| 4 | | | |
| 5 | 2 | | |
| 6 | 8 | | |
| 7 | 4 | | |
| 8 | 2 | | |
| 9 | 2 | | |
| 10 | 2 | | |
| 11 | 2 | 1 | |
| 12 | | | |
| 13 | 1 | | |
| 14 | | | |
| 15 | 1 | 1 | |
| 16 | | | |
| 17 | 1 | | |
| 18 | | | |
| 19 | 1 | | |
| 20 | | 1 | |

## 6. Establishment of Rehabilitation Procedures

Because the Tokyo International Airport expansion project is developed on reclaimed land, settlement, and differential settlement in particular, due to subsoil consolidation is inevitable. Thus, some rehabilitation of the pavement will become necessary in the future. Countermeasures for settled concrete pavement are described in this section.

The existing methods are reconstructing a new concrete pavement and adding an overlay on the settled pavement. Reconstruction is considered unrealistic for Tokyo Inter-national Airport because the facilities cannot be closed for long periods of time. The overlay method is applicable for this case to shorten the period that the airport is closed. Compared with an asphalt overlay, a concrete overlay is particularly suitable to the apron area because of its good resistance to load-induced deformation. Although a thinner overlay is preferable not only to strengthen the pavement structure, but also to adjust the surface gradient, it has been seldom used at airports in Japan because of the insufficient reliability in its structure.

In addition, a lift-up method has been developed for prestressed concrete (PC) pavement to adjust unevenness of the pavement (6). The principle is to raise the settled slab using many jacks, and then grouting the void below. To assure the effectiveness of this method for actual rehabilitation, lift-up tests were performed on large-scale experimental pavement at Tokyo International Airport.

### 6.1 Thin Bonded Concrete Overlay

#### (1) Bond at the interface

The thin bonded concrete overlay has been rarely used for airports because the bond between the concrete overlay and the concrete slab is insufficient. This has caused problems such as separation that effect the required overlay thickness, as can be understood from composite pavement theory (13, 14).

The bond between two layers is designated as the slab integrity (R), where R = 0% for an unbonded overlay and R = 100% for a fully bonded overlay. The moment of inertia of the integrated concrete overlay slab is expressed as:

$$I_R = I_0 + R(I_{100} - I_0)$$

$$= (h_1^3 - nh_2^3)/12 + R\{h_1(h_1/2 - a)^2 + nh_2(h_2/2 - b)^2\} \quad (2)$$

where,

- $I_R$ : the moment of inertia of the slab with slab integrity R,
- $I_0, I_{100}$ : the moment of inertia of the slab with slab integrity of 0 and 100%, respectively,
- R : the slab integrity,
- $h_1, h_2$ : the thickness of the upper and lower layer, respectively,
- n : ratio of the elastic moduli (= $E_1/E_2$),
- $E_1, E_2$ : the elastic modulus of the upper and lower layer, respectively, and
- a,b : the distance between the interface and the neutral axis in the upper and lower layer, respectively.

The equivalent thickness of the composite slab (h*) can be calculated by Equation (3).

$$h = (12I_R)^{1/3} \quad (3)$$

Finally, the maximum concrete stress due to the design aircraft can be computed. As an example, the required overlay thickness is calculated for a B-747-200 B on a 34 cm thick slab of concrete pavement. **Figure 11** is the obtained result, which shows that

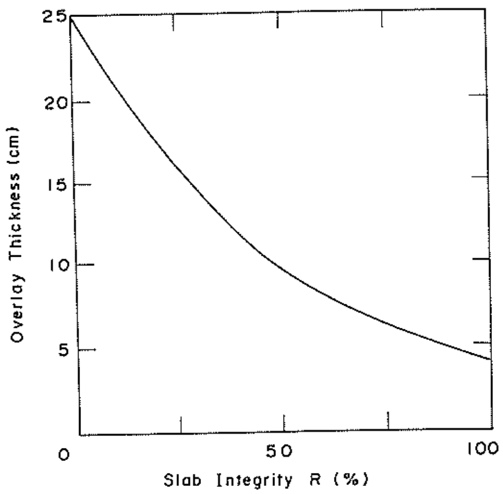


**Figure 11 Overlay Thickness and Slab Integrity**

a 25 cm thick overlay is necessary for R = 0% although only 4 cm is required for 100% bond.

Thus, the establishment of a bonding procedure is essential not only to prevent separation at the interface, but also to allow thinner concrete overlays. The following two items need to be considered to ensure a sufficient bond at the interface:

1) Surface treatment of the existing slab
2) The bonding medium

These were studied through two kinds of experiments: laboratory and field tests.

**(2) Laboratory Test**

**Table 6** shows the results of the preliminary laboratory tests (15). From the table, shotblast cleaning of the existing concrete surface is found to be essential. Both the tensile strength and the shear strength of the interface are significantly increased by this cleaning. Moreover, wetting the surface when placing the overlay is not as

**Table 6 Bond Strength at Interface**

| Surface Treatment | | Tensile Strength | | Shear Strength | |
|---|---|---|---|---|---|
| | | Age (days) | | Age (days) | |
| Shotblast | dry/wet | 28 | 91 | 28 | 91 |
| No | Wet | 0.51 | 0.91 | 0.64 | 0.76 |
| Yes | Wet | 1.51 | 1.90 | 1.90 | 2.96 |
| Yes | Dry | 1.82 | 2.08 | 2.28 | 3.16 |
| Monolithic | | 3.98 | 3.77 | 10.8 | 12.8 |

(unit: MPa)

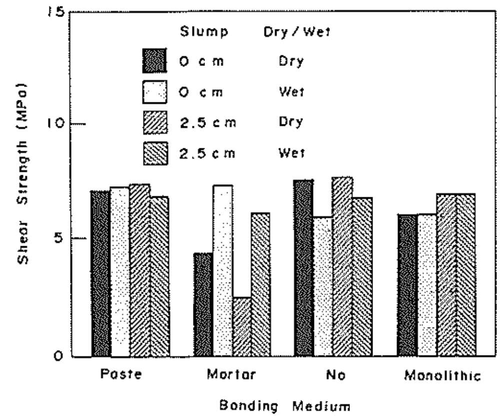


**Figure 12 Shear Strength Affected by Bonding Medium**

favorable as keeping it dry.

Next, a suitable bonding medium was studied. **Figure 12** shows the shear strength at the interface where the existing concrete surface was shotblast cleaned in all cases. The strength with cement paste is almost the same as that with cement mortar.

The strength is strongly affected by the surface moisture condition, in contrast to **Table 6**. This difference may be related to differences in the water cement ratio of cement mortar, which was 48% in the first series and 37% in the second series. Moreover, the application of cement mortar is more complicated than that of cement paste. Therefore, cement paste is recommended as the most practical bonding medium.

However, a bond strength that is the same as that of a monolithic specimen cannot be obtained by shotblasting alone as shown in **Table 6**. Thus, a procedure for improving the bond at the interface was developed that uses grooves cut on the existing concrete pavement. Such grooves are generally applied in runway pavement.

Two types of grooves were used: one was designated type A and consisted on a groove width of 6 mm and depth of 6 mm, spaced every 25 mm. The other was

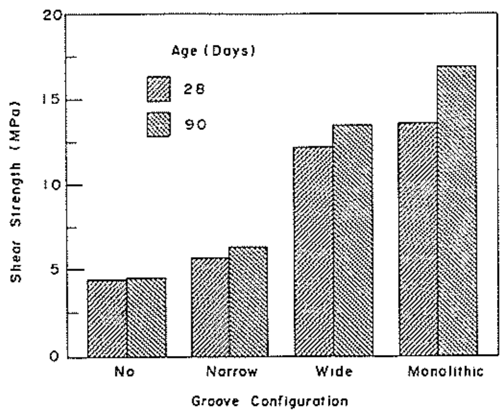


**Figure 13 Effect of Grooving on Shear Strength**

designated type B and consisted of grooves with a width of 33 mm, depth of 15 mm and a spacing of 58 mm. The effectivness of these grooves are explained in **Figure 13,** which shows the shear strength. Specimens with both groove types and a larger shear strength than the un-grooved specimens. The specimen with the smaller grooves (type A) and about 40% of the strength of the monolith specimen, while the specimen with larger grooves (type B) provided 90% lof the strength. Thus, the effectiveness of the grooves was definitely demonstrated in the laboratory tests.

The typical indication of distress in the bonded overlay pavement is separation at the interface, as described above. Because this may be caused by environmental action, the overlay specimens shown in **Figure 14** were placed outdoors to observe the effect. Both shotblasting and cement paste were used for the surface preparation of the lower layer. The upper layer was laid four weeks after placing the lower layer. In one specimen, a 30 mm wide and 15 mm deep groove was provided every 60 mm.

**Figure 15** shows the relationship between the temperature change and the actual strain in the upper layer at a distance of 120 mm from the edge and 40 mm from the surface. The movement of the overlay layer was definitely affected by the lower layer in both cases, and the specimen with the grooves was more restrained. This means that the bond at the interface can be improved by cutting grooves on the existing concrete slab.

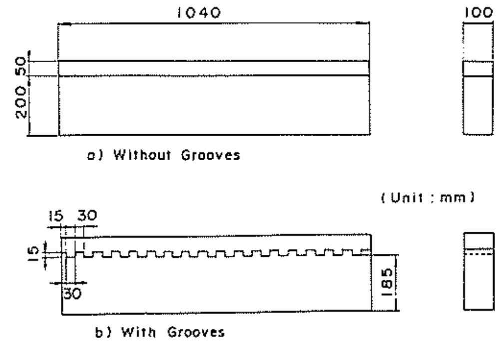


**Figure 14 Speciment Exposed Outdoors**

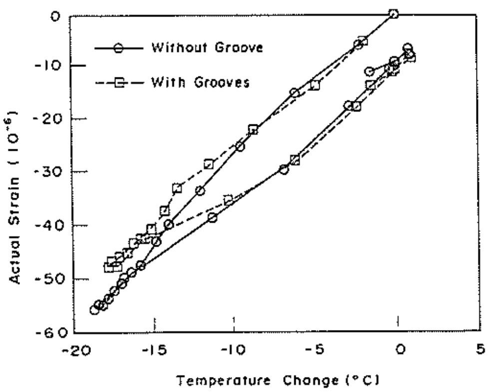


**Figure 15 Difference in Temperature Induced Strain**

### (3) Field Test

The effect of the micro texture from shotblasting together with the macro texture from the grooves was clarified by the laboratory tests. Based on the results, experimental prototype overlays were made to establish the bonding procedure. Two kinds of experiments were conducted: one to observe the response to environmental action, and the other to study the effect of repetitive loadings.

For the first experiment, a 50 mm thick concrete overlay was placed on an existing concrete pavement that was constructed at Tokyo International Airport more than 20 years ago. To prevent pot holes, concrete with a 0.5% volume of 60mm long steel fibers were adopted as an overlay material. The design flexural strength of the base concrete (before mixing with steel fibers) is specified as 4.9 MPa.

The procedure that was effective in the laboratory test was examined: i.e., the existing concrete slabs were cleaned by shotblasting, the grooves were cut, and the cement paste was applied to a dried surface. Four different types of grooving were evaluated, as shown in **Table 7**. Two basic types of grooving were used: edge grooving and interior grooving (**Figure 16**). Five grooves were provided in the edge portion of the slab, as shown in the figure. The interior grooves were cut with a spacing of 1,000 mm for type A, and 500 mm for type B. These tests where conducted to ensure a good bond at the interface for the interior portion. Both groove types were 30 mm wide and 20 mm deep.

**Table 7 Grooving of Test Pavement**

| Section | Grooving | |
|---|---|---|
| | Edge area | Interior area |
| 1 | No | No |
| 2 | Yes | No |
| 3 | Yes | Type A |
| 4 | Yes | Type B |

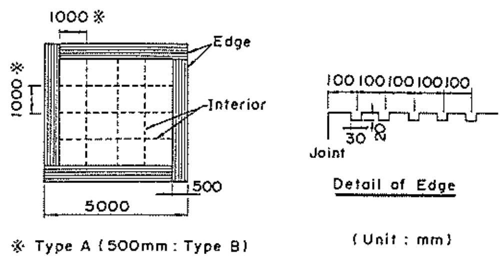


**Figure 16 Experimental Thin Bonded Concrete Overlay Pavement**

Hairline cracks were observed in some slabs eight months after construction. The cracks were tightly closed and could only be found by careful inspection after spraying with water. Separation at the interface was not observed even in the edge portions. Moreover, none of the specimens failed at the interface in the in-situ direct pull test. Thus, it was concluded that the bonding procedure used in the experiment was effective.

Two years after construction, the structural condition of the overlay pavement was evaluated by using a Falling Weight Deflectometer (FWD). **Figure 17** shows the maximum deflection measured at the center of a 450 mm diameter loading plate with a load amplitude of 196 kN. The effect of the grooves was definite for both the corner loading and the joint loading cases, with Section 2 through 4, and especially Section 4, showing much smaller deflections. The grooves clearly improve the bond at the interface.

However, there is scatter in the effect, as shown in **Figure 18**, which indicates the calculated slab integrity and load transfer effectivness at the joint. A procedure developed by the authors (16) was used to analyze the deflection data measured at the corner. The interior grooves may influence the bond strength at the interface not only at the interior portion, but also at the edge, as indicated by the data in Section 4.

A second experiment was conducted to investigate the influence of repeated loadings on the bond at the interface. In this experiment, only the edge portion was grooved for practical reasons. Both shotblast cleaning and cement paste were used.

The experimental pavement consisted of three square slabs with sides of 7.5 m. A 50 mm thick, steel fiber reinforced concrete layer was placed five months after the 380

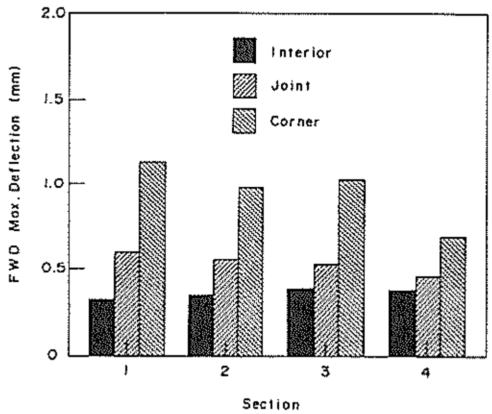


**Figure 17 FWD Maximum Deflection in Test Sections**

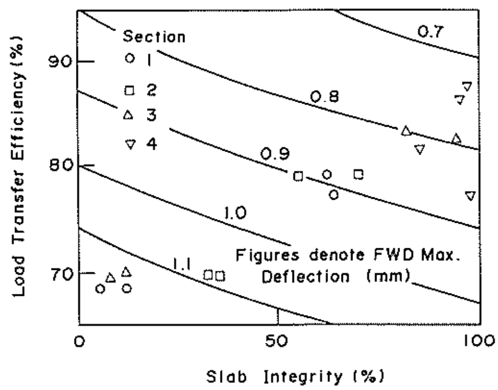


**Figure 18 Difference in Slab Integrity**

mm thick slabs were cast. The concrete mix design was the same as the first experiment. The repeated loading test was carried out using a loading cart with the same wheel configuration as the main gear of a B-747-200 B aircraft. In this test, a load of 809 kN was applied 5,000 times, and the structural condition was evaluated by FWD.

**Figure 19** shows the relationship between the maximum FWD deflection and the number of loadings. Deflections measured both at the slab center and at the joints did not vary drastically with loadings. Thus, the bond at the interface was adequate.

Based on these test results, the following recommendations are given to improve the bond at the interface:

1) Shotblast cleaning is used,
2) Cement paste is applied,
3) The surface is dried, and
4) Relatively large grooves are cut on the surface of both the edge and interior areas.

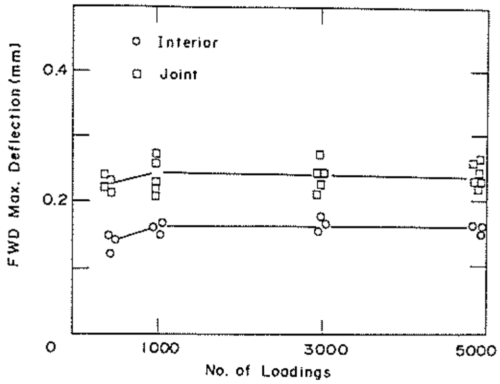


**Figure 19 Effect of Repetitive Loadings on FWD Maximum Deflection**

## 6.2 Lift-up Method of PC Pavement

### (1) Construction of Experimental PC Pavement

Experimental PC pavement with a width of 55.5 m, length of 100 m and thickness of 18 cm was constructed in the offshore expansion project area of Tokyo International Airport, as shown in **Figure 20** (17). The PC slabs were designed in accordance with class III PC theory (18). However, the PC slabs in Section A had a larger prestress to investigate the effect of prestress on the lift-up method.

The coefficient of the foundation reaction was designed as 69 MN/m$^3$. Vinyl sheets were laid on the base course to reduce friction between the base course and the PC slabs. Polypropylene – laminated (PPL) sheets were then placed over the vinyl sheets to prevent adhesion between the PC slabs and the cement paste used for grouting. Therefore, lift-up could be repeated many times corresponding to successive settlement.

Strands with a nominal diameter of 17.8 mm composed of 19 wires were used as prestressing tendons. High early strength cement was used to produced concrete with the specified compressive strength of 34 MPa in seven days. Prestressing was conducted in three successive stages. The longitudinal tendons were tensioned to 20% of the required stress one day after placing the concrete for the first stage, and 50% at four days of age for the second stage. After completing this prestressing for each PC slab, the design prestress was applied both longitudinally and transversely in the final stage. The amount of effective prestress in Section A for the longitudinal direction was

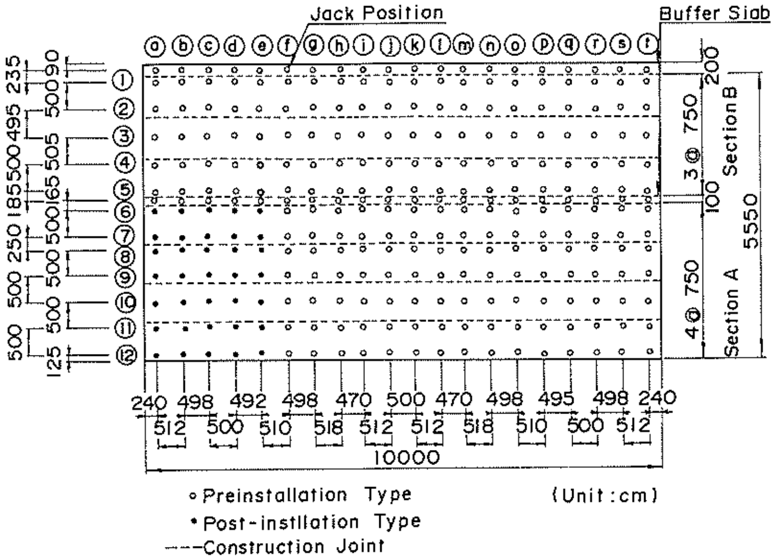


**Figure 20 Plan of Experimental PC Pavement**

4.5 MPa while that for the transverse direction was 3.8 MPa. The effective prestress for these directions in Section B was 3.8 MPa and 3.4 MPa, respectively.

**(2) Lift-up Work**

Lift-up of the test pavement was executed by using hydraulic jacks placed at intervals of about 5 m, as shown in **Figure 20.** A total of 280 installation fittings for the jacks were used: 245 pre-installation type sets and 35 post-installation type sets. The pre-installation type fittings were placed when the concrete was cast. For this type, the vinyl film and PPL sheets were laid after setting the reaction plates on the base course, as shown in **Figure 21.** Post-installation type fittings were installed by making a 160 mm diameter hole in the PC slab and excavating the base course materials for placing a concrete bed. The installation fitting for the jack was then attached to the holes in the PC slab.

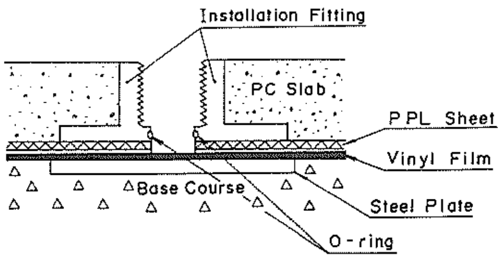


**Figure 21 Pre-installation Type Fitting**

The lift-up procedure consisted of attaching the hydraulic jacks, raising the PC slab by using the jacks, and grouting the void beneath the PC slab (**Figure 22**). To verify this procedure, the following tests were planned based on the small-scale tests (6):

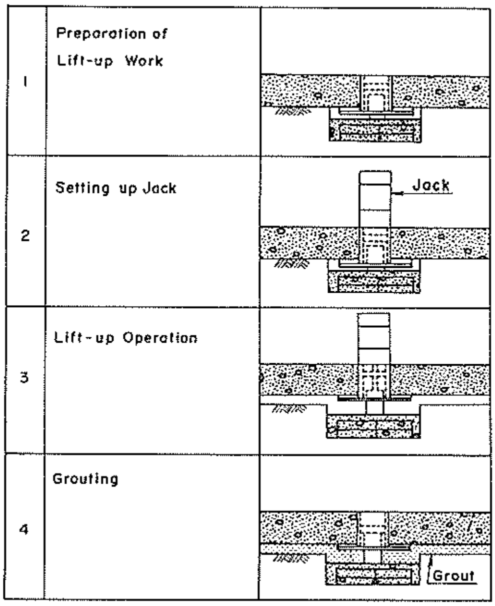


**Figure 22 Lift-up Work**

1) A large-scale lift-up test
2) A verification test for lift-up

The first test was intended to establish the lift-up procedure and to verify the accuracy of the work. Grouting under the raised PC slabs was omitted. In the second test, execution time and the confidence of work at each step were assessed by conducting a series of lift-up works. The grouting system was also checked.

The entire PC slab was lifted 5 cm high on one side of the pavement (from the slab end to 35 m inside), as shown in **Figure 23**. In this test, a large area across the whole width of 55.5 m was raised in parallel. In order to lift the whole area simultaneously, a large number of jacks are needed; however, a limited number of jacks should be used from an economical point of view. Therefore, jacks must be shifted after the height reaches the required level. In this experiment, the jacks were moved to another row

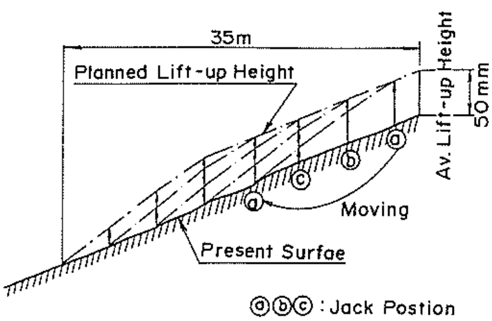


**Figure 23 Large-scale Lift-up Test**

when the row height reached 50 mm. The time required to shift jacks one row was about 30 minutes.

As shown in **Figure 24**, the difference was minimal between the planned height and the actual height, which was measured after the completion of lift-up. **Figure 25** shows a comparison between the actual load and the calculated load using the finite element method. The small difference between loads confirmed the validity of the calculation method for planning lift-up work.

In the second lift-up test, for which the total lift-up height was planned as about 150 mm, PC slabs were raised within 30 m from one end. However, the maximum

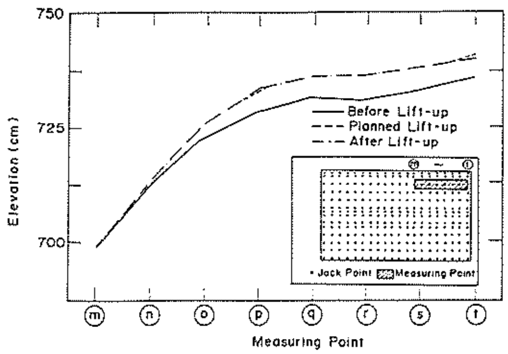


**Figure 24 Comparison of Lifted-up Height**

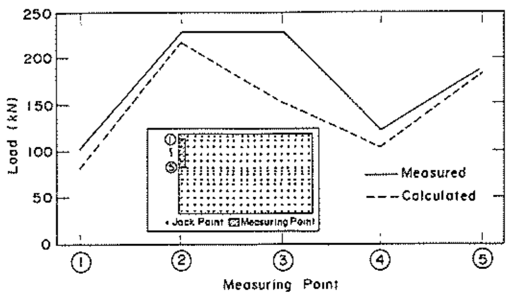


**Figure 25 Comparison of Lift-up Load**

raised height for a series of lift-up is considered to be 50 mm, which resulted from the small-scale tests. Therefore, the lift-up operation must be conducted in three stages, as shown in **Figure 26**. In this test, grouting was carried out at each stage.

The adopted grouting system is shown in **Figure 27**. Cement paste from the mixer was temporarily stocked in an agitator before being transported to the pouring containers by pumps. The void was grouted by hydraulically pouring the cement paste through the pre-arranged holes in the PC slab from a height of about 1 m.

**Table 8** shows the roughly estimated execution time for each lift-up operation. This table shows that most of the working time is for attaching the jacks, and the lifting operation itself does not need to take a long time. Therefore, the execution time can be shortened somewhat if workers become more proficient.

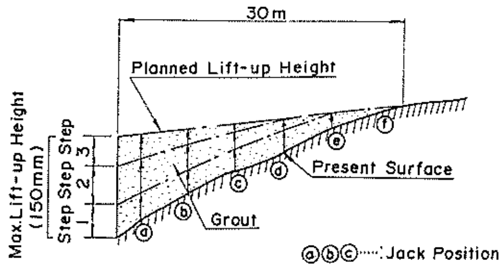


**Figure 26 Lift-up Test with Grouting**

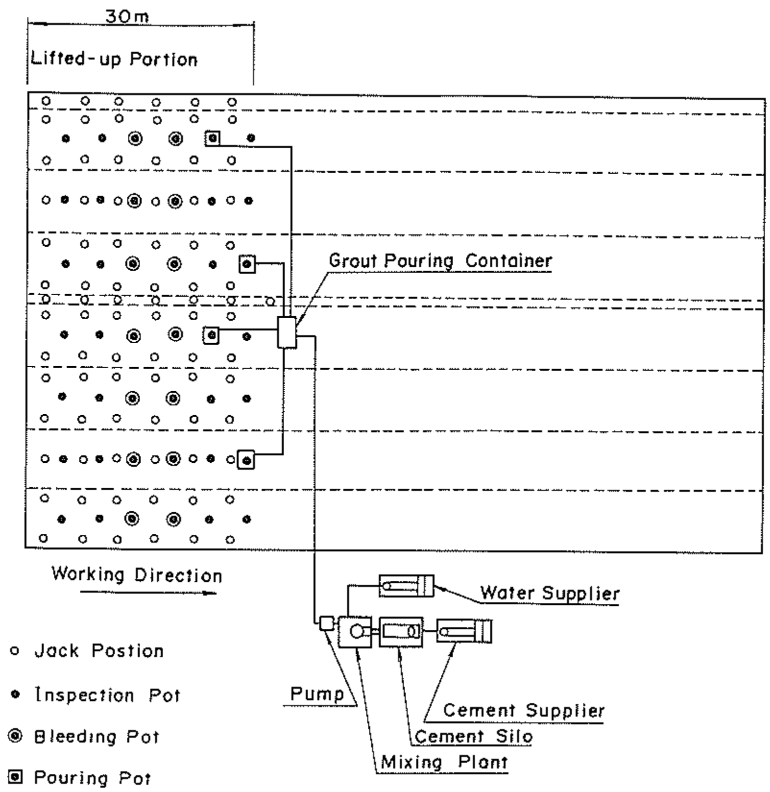


**Figure 27 Grouting System**

**Table 8 Execution Time of Lift-up Works**

| Item | Necessary Time per 900 $m^2$ |
|---|---|
| Jack Installation | 80 min. (15 Workers and 42 Jacks) |
| Lift-up | 60 min. |
| Grouting | 10.5 $m^3$ per hour |

## 7. Concluding Remarks

The Tokyo International Airport offshore expansion project is now under construction on reclaimed ground. The influence of subsoil settlement on the behavior of the pavement was considered both where the pavement follows the settlement fully, and where it does not. In the former case, the surface gradient of the pavement may deviate from the specification. To maintain the requirements, rehabilitation will be necessary a maximum of three times and over 60% of the whole area for a 20 year service life. Where the pavement does not follow the settlement, it may fail structurally due to voids beneath the concrete slabs. Management systems for the two analytical periods of 10 and 20 years were considered.

Rehabilitation will be necessary to keep the pavement surface within the specifications. Two procedures are considered: the thin bonded concrete overlay and the lift-up method. In the former, both shotblast cleaning and cement paste must be applied to improve the bond at the interface. Moreover, grooves are required for both the edge and interior areas. This procedure was verified through laboratory tests and two types of experimental pavement. The durability of the bonded concrete overlay pavement against repetitive loadings was also assured. In addition, the lift-up method was verified through various tests and can also be applied to actual repairs. The validity of the finite element analysis for planning the lift-up process was established, and the grouting system was demonstrated to supply the cement paste satisfactorily.

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