JAPAN INTERNATIONAL COOPERATION AGENCY (JICA)

SCIENCE AND TECHNOLOGY COMMISSION OF
SHANGHAI MUNICIPAL PEOPLE'S GOVERNMENT,
PEOPLE'S REPUBLIC OF CHINA

DETAILED DESIGN OF SHANGHAI PUDONG INTERNATIONAL AIRPORT FINAL REPORT

VOLUME I
MAIN REPORT

PART II
BASIC DESIGN

SEPTEMBER 1997

NIPPON KOEI CO., LTD.
NIKKEN SEKKEI LTD.

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PART II - 1
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CHAPTER 1 SITE PREPARATION AND EARTHWORKS PLAN

1.1 Summary

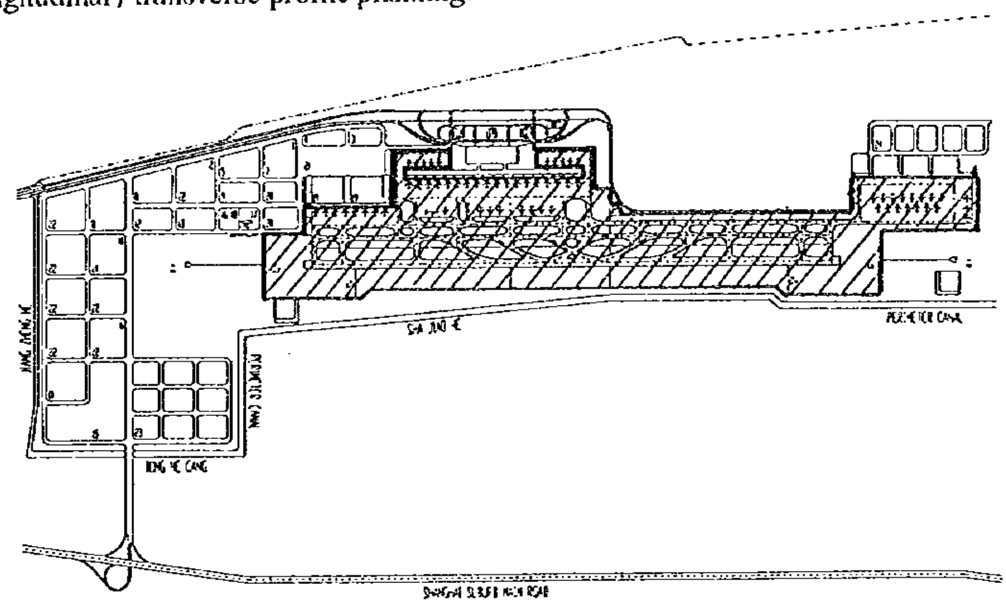
The airport construction site is located in the coastal area on the right bank at the mouth of the Yangzhou River. The topography is flat and there is a heavy concentration of rivers and channels within the site. The elevation is + 3.8 - 4.4 m above sea level.

Along the seaside there are two series of dikes. This area receives annual silting from the Yangzhou River which has resulted in sedimentation up to approximately 20 m.

The Phase One Site is presently a dry land surrounded by the outer dike, while the area designated for the Phase Two Eastern Terminal and Eastern Runway, presently under water, is scheduled to be reclaimed utilizing the induced sedimentation of the Yangzhou River within a few years.

The design area for the site preparation and earthworks plan includes the Flight Area (Basic Facilities Portion) of the Phase One Area, where land surveys and geological soil investigations have been carried out. The plan covers layout design, longitudinal / transverse profile design and earthworks planning.

The General Layout Plan for the Phase Two Area and beyond will be studied after the area has been reclaimed and various investigations have been carried out. However, considering the co-ordination required between the Flight Area, operational aspects and Terminal Facilities, it is considered that there will be no drastic revision of the Phase One longitudinal / transverse profile planning.



FigureII-1.1.1 Design Area

1.2 Layout Planning

1.2.1 Basic Principles

The Basic Principles for the setting out the alignment of the basic facilities are as follows:

• Position of Airport

In the south eastern section of Pudong New Area, about 30 km from Shanghai City center.

• Position of Aerodrome Reference Point

Latitude 31° 8 '43 " North

Longitude 121° 47 ′ 28 Eastern

• Aerodrome Reference Code "Technical Standards for Civil Aviation Transport Airport Flight Area" (Civil Aviation Agency of China) reference code 4E/4F (B-747 and future aircraft)

• Runway 4000 m × 60 m one number

• ILS Facilities CAT-II

1.2.2 Basic Facilities

(1) Runway

The runway will be 4000 m long and 60 m wide to allow landing and take-off of the largest existing aircraft and assumed future aircraft. Overruns 60 m long and having the same width as the runway will be provided at both ends of the runway. Furthermore, the runway and overruns will be provided 7.5 m wide shoulders.

(2) Runway Strip

The Runway Strip is provided to insure safety when aircraft re-try after failure to land and when aircraft deviate from the runway. Normally, the landing area surface is protected by sodding.

The Runway Strip for this airport will be 4120 m long and 300 m wide, because the runway will be designed for precision approach.

In addition, a runway end safety area will be provided at both ends of the Runway Strip to ensure safety for human life and reduce damage to aircraft when they over-run or under-shoot . The safety area will have a length of 90 m from the end of the Runway Strip and a width of 120m , i.e. double the runway width.

(3) Width of Taxiway

Taxiways will be laid out to provide safe and quick operations of aircraft within the airport. The width of taxiways of each type was determined based on the aircraft with the largest wheel base and wheel track among commissioned aircraft and aircraft assumed to be commissioned, the B-777-300.

The shoulders of the taxiways have been designed to have an uniform width of 7.5 m.

• Parallel Taxiways	:	29 m
• Entrance Taxiways	:	31.5 m (14.5 m + 17 m)
• Exit Taxiways	:	34 m
• High-speed Exit Taxiway	:	29m

A Taxiway Strip (an area of 57 m from the centerline of the taxiway, clear of fixed obstructions) will be provided. This Taxiway Strip must be kept clear of obstructions which may endanger aircraft operations.

(4) Apron

All aprons will be designed for nose-in aircraft parking. The type and number of spots for each area are listed below. Aprons will be provided with 7.5m wide shoulders.

• Passenger Terminal Area Apron

The area in front of the passenger terminal will have 18 spots for type E aircraft and the area at the back of the terminal finger will have 10 spots for type D aircraft.

• Open Spots

Spots for type E, D and C aircraft will be arranged according to apron dimensions. There will be a total of 11 spots including 3 of type C, 1 of type D (will be double d for type E use), and 7 of type E spots.

• Cargo Apron

8 of type E spots will be provided in front of the Cargo Building.

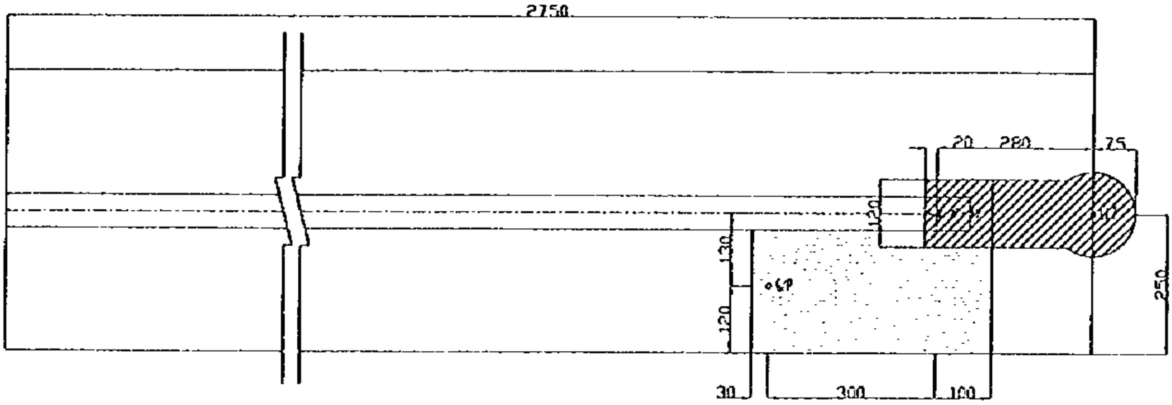
• Maintenance Apron

15 night stay spots will be provided in the maintenance apron and 3 additional type E spots will be provided for engine test purposes.

1.2.3 Glide Slope and Localizer Site

The design conditions for the Glide Slope site and the Localizer site will be based on the "Demands for Magnetic Environment of Radio Navigational Aids" (National Bureau of Chinese Standards).

The Glide Slope is a facility to instruct aircraft on the descending route on the approach. The Localizer is a facility to guide aircraft precisely onto the runway center line. The protected area for this airport is shown in FigureII1-1.2.1.



FigureII1-1.2.1 Protected Area for Radio Navigational Aids

1.2.4 Approach Lighting Site

In the Approach Lighting Site area, maintenance roads, perimeter roads and other facilities will be provided in addition to the approach lights.

The maintenance roads will be laid out on the terminal side and have a turn-around for vehicles at the extreme end of the approach light facilities.

1.3 Longitudinal / Transverse Profile Planning

1.3.1 Limiting Conditions

In determining the Longitudinal / Transverse Profile for the Basic Facilities such as the runway, taxiways and aprons, the following limiting conditions should be considered:

(1) Balancing Earthworks Volumes of the Flight Area

The deficit earth volume for filling of the rivers and channels in the site and the landscaping of the Flight Area shall be balanced with surplus earth from the dredging of the new canals to be provided around the airport , to keep to a minimum the earth volume required to be brought in from outside areas.

(2) Stability of Pavement of the Basic Facilities

The groundwater level within the site area fluctuates between +2.5 m and 3.5 m in elevation and lies 0.5 - 1.5 m below the ground surface. If the pavement body is permanently saturated by groundwater, the pavement will be damaged due to reduced dispersal of weight resulting, in excessive stress in the pavement slab. Therefore, the subgrade level of pavement of the basic facilities must be set higher than the groundwater level.

(3) Drainage of the Airport Site Area

In order to quickly drain rain water from the site, it is necessary to secure a hydraulic gradient sufficient to allow gravity flow (0.8-1.0%) and also to secure safe operations by preventing the basic facilities such as runways from being affected even when standing water occurs due to heavy rains.

1.3.2 Design Elevation of Basic Facilities

(1) Basic Policies

Based on the above mentioned Limiting Conditions for Longitudinal / Transverse profile planning, a technical and economic study was carried out, after which the design elevation of the Basic Facilities has been set out as follows:

- In order to balance the excavation volume from the excavation for the new canals and the widening of the existing canals, the average design elevation of the Flight Area has been set at 4.5 m, and the highest point along the runway centerline has been set at 5.3 m.

• Considering the fluctuations of groundwater level and the residual settlement after construction completion (about 10 cm over 10 years), the pavement level of the basic facilities has been set at 5.1-5.3 m to prevent deterioration of the asphalt - paved structure (assumed to be 1.3m in depth) due to groundwater.

• In order to provide gravity outflow by opening the gates at low water level of the Yangzhou River, a water head of 3.2-4.0 m must be provided over the approximate 4.0 km length of the drain system. Therefore, the upstream end of the storm drainage channels cannot be set lower than 5.3 m in elevation following the site preparation gradient conditions.

Furthermore, most of the Chinese experts also agree on the need to set the design elevation of the airport at about 5.3m in order to secure safe operations of the airport. Based on these considerations, the design elevations of the basic facilities have been set as follows;

• The longitudinal shape of the runway and parallel taxiways is set at +5.3 m at the center point, +5.1 m at both ends and the longitudinal gradient at 0.01%, witch is well within the standard maximum value of 0.8%.

• The design elevations for the aprons in the passenger terminal, cargo and maintenance areas are set at +4.3-5.3 m, considering the drainage gradients.

• The aprons in front of the passenger terminal building, fingers, cargo terminal building and hangers (at point of contact with GSE) is set at +5.0-5.3 m.

(2) Runway Area

Based on the basic policies mentioned above, the longitudinal profile of the runway and parallel taxiways is set at El. +5.3 m in elevation at the center point and El. +5.1 m at both ends. The rapid exit, entrance and exit taxiways will be blended with the runway and parallel taxiways. Next, the transverse profile is set out as follows with some allowance over the standard maximum value in consideration of future consolidated settlement.

• Transverse gradient of the Runway and Taxiways proper	1.3%
• Transverse gradient of the Runway and Taxiways shoulders	1.6%
• Transverse gradient of the Landing Strip	1.3 - 2.0%
• Transverse gradient of the Glide Slope Area	1.0%

(3) Apron Area

Since the Apron Area is very large, considerations have been made for the reduction of embankment volume, the relation with buildings such as the passenger terminal , the prevention of standing water and the prevention of spontaneous movement of parked aircraft. The gradient parallel with the runway is set at 0.01%, equal to the runway longitudinal gradient, while the gradient normal to the runway direction is designed to be a sawtooth pattern with a gradient of 0.5%. Furthermore, measures will be taken to prevent drained water from entering buildings such as the passenger terminal building.

(4) Localizer, Approach Lights Site

The longitudinal gradient of the Localizer site on the extended line of the runway is set at 0.01% in accordance with the longitudinal gradient of the runway. This prevents the Localizer antenna from intruding into the Approach surface and to reduce the required embankment volume. Next, the Approach Lights Site is designed to be at El. +4.5m with an embankment of 0.5m above the existing ground level to prevent flooding of the Approach Lights under storm conditions.

1.4 Earthworks Design

1.4.1 Stripping of Topsoil

The topsoil will be stripped in the pavement area in order to avoid differential settlement caused by the highly compactible top soil (organic soil) . The depth of the stripping of the topsoil will be 30 cm uniformly. The removed soil will be utilized as cover soil for the planting over the Runway Strip.

1.4.2 Treatment of Existing Channels

Water channels within the pavement area will be dredged to be clean of sludge and bench-cut will be done on 1:2 slopes. Channels outside the pavement area will be subject to neither dredging nor mapping and will simply be filled in.

1.4.3 Planting

The planting around the Runway and Taxiways will consist of sodding for 2 m from the shoulder edge and 50% sodding for 3 m beyond this, considering the effect of the jet blast. Additionally, areas around roads and drainage channels which collect rainwater collects will also be completely sodded.

Furthermore, areas other than the areas to receive sodding will be seeded to protect the landscaped surface from rain and wind erosion .

During construction, the parts in contact with the runway and taxiway edges will be set 3 cm lower than the pavement in order to facilitate runoff of rainwater.

1.4.4 Calculation Earthworks Volume for the Flight Area

The Earthworks Volume calculated by laying out a 40 m mesh with design elevations for each facility based on the above longitudinal / transverse profile planning over the existing ground elevations from the survey results and reading the difference is shown in TableII-1.4.1. The following conditions are assumed in the calculations.

- The existing elevations are the surveyed values of the points on the 40 m mesh.
- The volume for one mesh unit is determined by calculating the average of the earthworks information height (shown as Δh below) and multiplying by the mesh area. The earthworks volume for filling up the existing channels is calculated separately. The existing ground levels over the existing channels are derived from the surveyed values of the adjacent points.
- The shapes of the channels are grouped into patterns following the surveyed sections. Sections in the pavement area will be bench-cut and dredged of sludge followed by a 0.5 m thick sand mat and then filled up. Channels outside the pavement area will be simply filled up.
- Excavated earth volumes for new canals, widening of existing channels and existing dikes are the same as those described in the feasibility study.
- The earthworks height at each mesh point is calculated as follows:

Pavement Area : Δh= Design elevation - existing ground elevation - pavement thickness + top soil stripping depth + compaction depth for soil improvement

= Design elevation - existing ground elevation - 1.0 m + 0.3 m + 0.3 m

= Design elevation - existing ground elevation - 0.3 m

Shoulder Pavement Area : Δh= Design elevation - existing ground elevation - pavement thickness + top soil stripping depth + compaction depth for soil improvement

= Design elevation -existing ground elevation - 0.5 m+ 0.3 m+ 0.3 m

= Design elevation -existing ground elevation + 0.1m

Sodding Area Δh= Design elevation - existing ground elevation + compaction depth for soil improvement

= Design elevation -existing ground elevation + 0.02 m

• The ratio of volumetric expansion was assumed to be 0.9.

TableII1-1.4.1 Calculation of Earthworks Volume for the Flight Area

(unit : 10000m^3)

Excavated Volume		Embankment Volume	
Sludge Dredging under Pavement	13	Filling of Existing Channels under Pavement	51
Stripping of topsoil	50	Filling of Existing Channels not under Pavement	31
Bench-cut Volume under Pavement	20	Embankment for Soil Preparation in Flight Area under Pavement	165
Excavation for Soil Preparation in Flight Area	40	Embankment for Soil Preparation in Flight Area not under Pavement	91
Surplus Soil from Drainage Channels and Regulation Pond	30		
		Sand Mat(Purchased Soil)	(8)
Total Excavated Soil	A 153	Total Embankment Soil	B 338

The results of earthworks volume calculation show that the embankment soil volume required is larger then the excavated soil volume. The deficit volume is calculated as follow:

A-B/0.9 =1,530,000 - 3,380,000 / 0.9 = -2,230,000 m^3

As shown above there will be a deficit of approximately 2,230,000 m^3 of soil for filling, this deficit volume will be made up with soil brought from outside areas.

The candidate borrow material soursces are excavated materials from new river channels (approx. 1,980,000 m^3), excavated materials from existing dikes (approx. 270,000 m^3), and other dredged materials from the Yangzhou River. The selection of the soil source will require adjustment of the construction schedule. However, it can be said that the total earth works volume roughly balances if borrow materials from surrounding areas is considered.

CHAPTER 2 IMPROVEMENT OF SOFT GROUND

2.1 Summary of Geographical, Geological and Soil Conditions

2.1.1 Summary of Geographical, and Geological Conditions

(1) Geological Conditions

Prior to construction planning and designing of Pudong International Airport, the following investigations were carried out focusing on the Phase 1 area:

1) Soil investigation in the feasible study stage by the Chinese side (the Runway Area), February 1996
2) Additional soil investigation in the feasibly study stage by the Chinese side (fill material in the surrounding area), May 1996
3) Detailed soil investigation in the Runway Area, August 1996
4) Detailed soil investigation in the Taxiway Area, September 1996
5) Detailed soil investigation in the Apron Area, September 1996

The proposed site for Pudong International Airport consists of sedimentary deposits of the Yangzhou River. After dike construction and excavation work for channels were carried out, the area is used as cultivated land now. The topography is almost flat, with an elevation of 3.5 ~ 4.4 m. The underground water level is generally between El. 2.5 and El. 3.5 m. (affected by the level of river (canal) water).

In the proposed Phase I Construction site, the strata are relatively uniform, and no marked difference is observed. Drilling positions are shown in FigureII1-2.1.1. The geological longitudinal section of the runway in the Phase I area is shown in FigureII1-2.1.2 as a representative example. The stratum structure and description of each stratum are shown in TableII1-2.1.1. Strata ⑤-3 and 4 exist partly in the Apron Area only.

TableII-2.1.1 Strata Structure and Description of Each Stratum

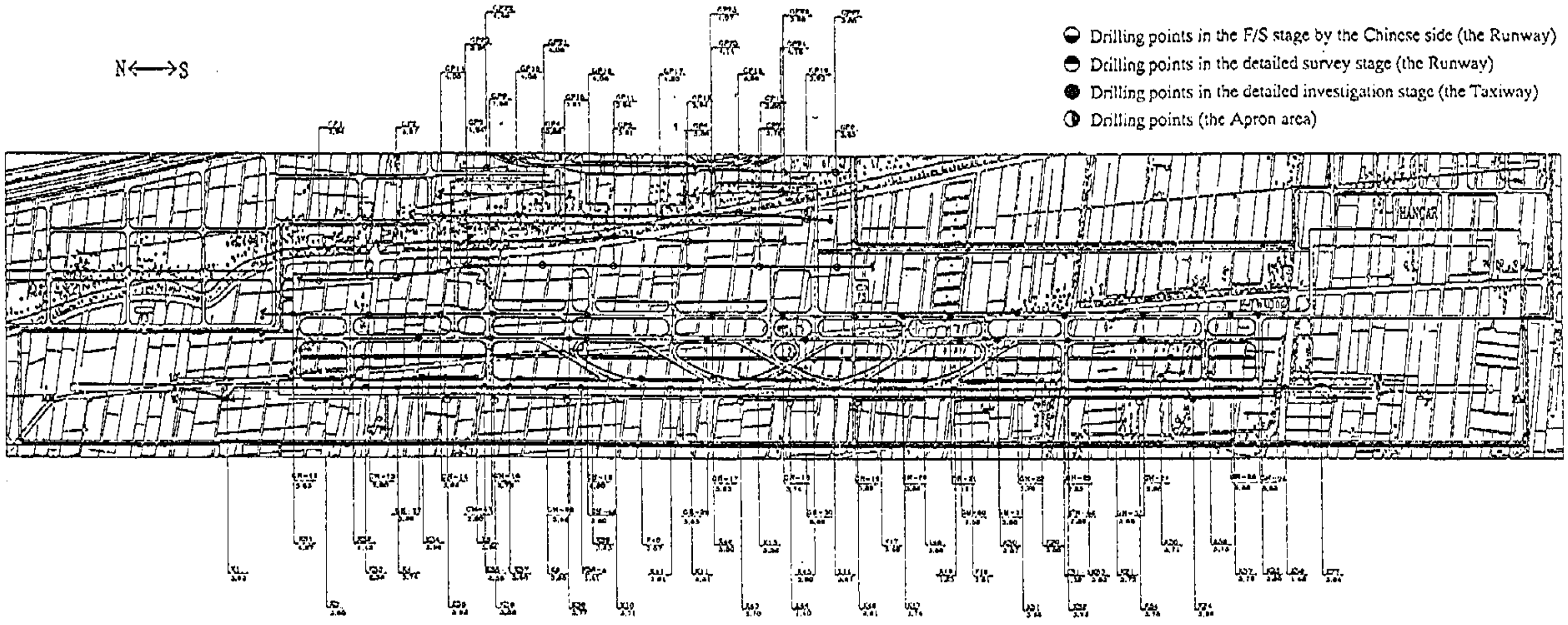
Stratum No.	Elevation of Strata Surface (m)	Thickness of Strata (m)	N Value	Color and Soil Type	Notes
①	3.5 ~4.4	0.4~0.7 Av. 0.5		Topsoil	*Cultivated soil and road banking *Mixed with plant roots and stems
②-1	1.9 ~3.9	0.4~1.2 Av. 0.8	2~5	Brownish yellow, Silty clay	*Contains hard nucleus *Belongs to medum compressible soil
②-2	1.9 ~3.2	0.5~ 1.7 Av. 1.0	2~5	Grayish yellow, Silty clay	*Localized sandy silt. With natural cavities *Belongs to medum compressible soil.
②-3,4	0.8 ~2.1	5.0~ 8.3 Av. 6.5	3~18 Av. 9.3	Gray, Clayey~ Sandy silt	*Soil is uniform andmixedwith shallowlayers of fine sand *Contains small flakes of mica *Belongs to medum compressible soil
③	−6.7 ~-3.7	0.0~2.4 Av. 1.1	1 ~ 4	Gray, Silty clay	*Mixed with shallow layers of silty sand * Belongs to highly compressible soil
④	−8.3 ~ −4.3	8.8~ 12.8 Av. 10.7	1	Gray, Silty clay	*High water content, clay with high plasticity * Soil is uniform and mixed with shallow layers of silty sand. Shells are found on the layer bottom. * Belongs to highly compressible soil.
⑤- 1 ~4	−17.8 ~ −15.8	5.3~11.3 Av.8.1	1	Gray, Clay~ Silty sand	*High water content and high plasticity; contains grayish - white silty clods, semi - decomposed plant roots and stems. *Soil is uniform and belongs to highly compressible soil.
⑦-1	−26.7 ~ −22.8	4. 6 ~ 11.6 Av.7.5	20 ~30	Greenish yellow, Sandy silt	*Between "slight density" and "medium density"; contains mica flakes and stripped figures with ferreous penetration. *Compressibility is medium.
⑦-2	−34.4 ~ −30.4		30 or more	Greenish yellow, sand mixed with silt	*Medium density; contains ferreous spots and mica flakes. *Compressibility is medium. *This layer is the primary water retaining soil in Shanghai area.

(2) Conditions of Channels (Rivers)

The river network is highly developed in the proposed airport site. Major channels (rivers) include the Jiang Zhen river (forming the border in the northern part of the airport), the Xie Jin Hong Gang River (to be the border in the southern part), and the Pudong Canal in the east of the border of the airport. Within the site, channels running longitudinally through the runway are the Sai tong River and Bai Long River. Those crossing the site are listed from the northern part as follows: the Shi Wan, the Ying Xiong, the Chao Yang, the Dong Feng, the Jia, the Wang Jia Lu and the Liu Loon Gang port rivers.

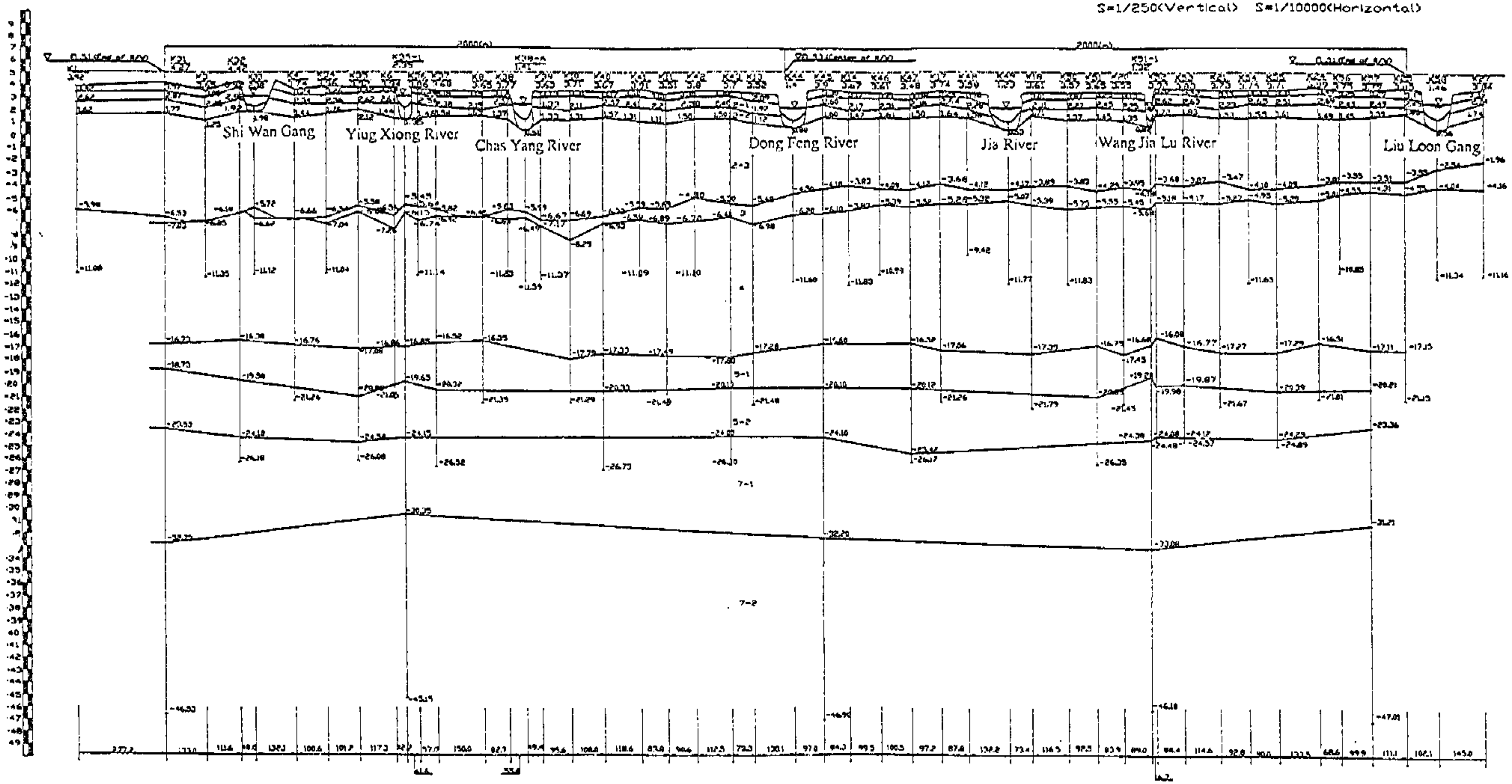
Location of Drilling Survey Positions

S=1/10000



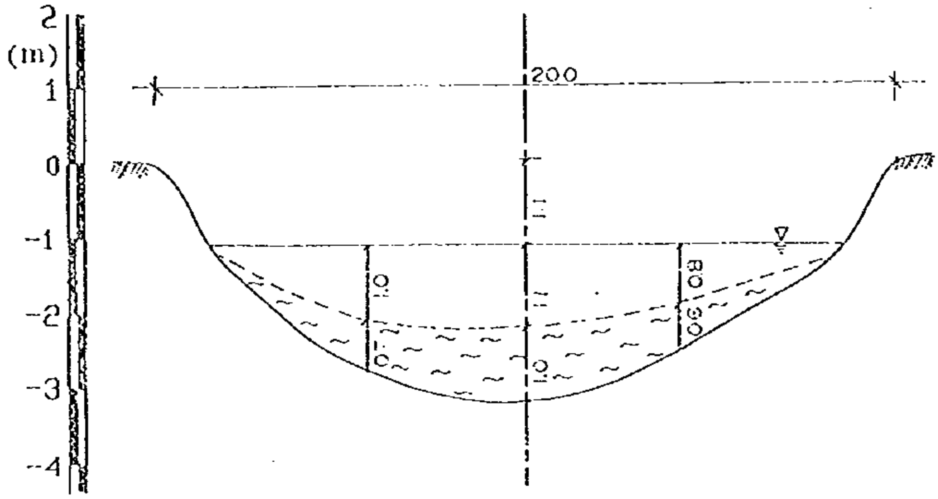
FigureII-2.1.1 Distribution Plan of Drilling Points

Cross-Section of Layers in the Runway Area (8-8' 6-6' 10-10')



FigureII-2.1.2 Longitudinal Geological Cross-Section of the Runway Area

These main channels (rivers) have a width of 15 ~ 30 m. They are used for transportation by boat. The water level is adjusted in the Pudong Canal to a depth of around 2.0 m. The channel bottom is about 0.0 m in elevation (above the sea level). Other than these rivers, small-sized channels with a width of 5 ~ 10 m run longitudinally and latitudinally. They are covered with duck weed. They have about 1.0 m sedimentary deposits of sludge on the river bed (see the figure below). In many places, are filled up and used as cultivated land.



FigureII-2.1.3 Diagrammatic Cross-Section of Channel (River) in Airport Construction Site

2.1.2 Summary of Soil Conditions

TablesII-2.1.2(a) ~ 2.1.2(c) list the physical, shearing force and consolidation properties of each stratum. The values were obtained from the results of the laboratory tests conducted in the detailed-survey stage in the Runway Area.

(1) Physical Properties

Stratum ①is humus (cultivated) soil stratum containing the large amount of plant roots and stems. Soil of Strata ②1, 2, 3, ③, ④ and⑤-1, 2 is classified as clay. The natural water content is W_n = 30 ~ 50%. On the other hand, soil of Strata② -4 and ⑦-1, 2 is classified as silt ~ fine sand. Their sand content ratios are around 20%, 30% and 60% respectively. Among these strata, ①is necessary to be removed as topsoil.

(2) Shearing force

Soil of Strata ②-1, 2, 3, 4, ③, ④ and ⑤-1, 2 is classified as clay. They consist of soft clay with N value of 3 or less. Their shearing force is low. On the other hand, the

TableII1-2.1.2(a) List of Physical Properties of Each Stratum (Average values of the results of laboratory tests in the detailed-survey stage in the Rumway Area)

No of Strata	Name	Thickness of Strata(m)	Specific Gravity	Sand Content Ratio (%)	Water Content Ratio Wn(%)	Wet Density γt(tf/m3)	Dry Density γd(tf/m3)	Void Ratio e	Index of Plasticity Ip	Index of Liquefaction IL
①	Cultivated soil	0.3～0.7	-	-	-	-	-	-	-	-
②-1	Silty clay	0.4～1.2	2.73	3	30.8	1.90	1.45	0.88	16.4	0.50
②-2	Silty clay	0.5～1.7	2.73	4	36.8	1.83	1.34	1.04	14.9	0.98
②-3	Clayey silt	-	-	-	-	-	-	-	-	-
②-4	Sandy silt	5.0～8.3	2.70	27	30.8	1.90	1.45	0.86	-	-
③	Silty clay	0.0～2.4	2.73	3	41.8	1.79	1.26	1.16	14.9	>1
④	Silty clay	8.8～12.8	2.75	0	51.6	1.71	1.13	1.44	20.2	>1
⑤-1	Clay	2.0～5.0	2.74	0	41.5	1.78	1.26	1.18	18.9	0.95
⑤-2	Silty clay	2.9～7.0	2.73	0	32.4	1.87	1.41	0.94	15.2	0.73
⑦-1	Sandy silt	4.6～11.6	2.70	32	25.9	1.96	1.47	0.74	-	-
⑦-2	Gray	-	2.69	57	26.4	1.94	1.53	0.75	-	-

TableII1-2.1.2(b) List of Shearign force of Each Stratum

No of Strata	Standard Penetration N Value	Cone Penetration qc kg/cm2	Bane Test Cu (tf/m2)	Unconfined Compression Test qu (tf/m2)	Direct Shear Test C (tf/m2)	Direct Shear Test φ (°)	Triaxial Compression Test UU Test Cu (tf/m2)	UU Test φu (°)	CU(CD) Test Ccu (tf/m2)	CU(CD) Test φcu (°)	CU(CD) Test C'(Cd) (tf/m2)	CU(CD) Test φ'(φd) (°)
①	-	-	-	-	-	-	-	-	-	-	-	-
②-1	3.0	8.0	-	-	2.5	22.4	6.3	1.7	2.9	22.0	2.4	27.2
②-2	2.1	4.8	-	-	1.8	18.8	3.8	1.5	2.3	21.2	1.8	28.3
②-3	—	—	—	—	—	—	—	—	— —	—	—	—
②-4	10.3	15.6	-	-	0.6	30.7	-	-	0.7	26.3	0.3	31.8
③	1.3	5.4	-	-	1.4	17.2	1.8	0.5	1.9	20.3	1.4	28.0
④	1.1	5.0	3.2	-	1.1	11.0	1.9	0.0	-	-	-	-
⑤-1	-	7.4	4.2	-	1.7	15.5	-	-	-	-	-	-
⑤-2	-	9.6	4.8	-	2.0	21.2	-	-	-	-	-	-
⑦-1	25.5	69.4	-	-	0.8	30.9	-	-	-	-	-	-
⑦-2	40.5	153.1	-	-	0.3	35.1	-	-	-	-	-	-

TableII1-2.1.2(c) List of consolidation Properties of Each Stratum

No. of Strata	Coefficient of Volume Compressibility mv(E-2cm2/kgf) 0.0～0.5	0.5～1.0	1.0～2.0	2.0～3.0	3.0～4.0	4.0～6.0	Coefficient of consolidation Cv(cm2/day) 0.0～0.5	0.5～1.0	1.0～2.0	2.0～3.0	3.0～4.0	4.0～6.0	Consolidation yield stress Pc (kgf/cm2)	Compression Index Cc
①	-	-	-	-	-	-	-	-	-	-	-	-	-	-
②-1	-	-	1.85	-	-	-	129	152	241	-	-	-	1.50	0.208
②-2	-	-	2.52	-	-	-	463	431	353	-	-	-	1.06	0.273
②-3	-	-	-	-	-	-	-	-	-	-	-	-	2.04	0.279
②-4	-	-	1.05	-	-	-	737	546	439	-	-	-	1.50	0.111
③	-	-	3.50	-	-	-	365	453	319	-	-	-	0.98	0.317
④	-	-	4.67	-	-	-	82	95	99	-	-	-	1.29	0.467
⑤-1	-	-	3.07	-	-	-	150	142	134	-	-	-	2.45	0.332
⑤-2	-	-	2.28	-		-	392	331	326	-	-	-	2.92	0.225
⑦-1	-	-	0.91	-	-	-	-	-	-	-	-	-	4.13	0.087
⑦-2	-	-	0.61	-	-	-	-	-	-	-	-	-	-	-

soil of Stratum ② -4 is viscous alluvium. Its N value is relatively as high as around 10. It also shows high shearing force. Strata ⑦-2 has high N value of about 40, and is judged to be a bearing layer for pile foundation etc.

(3) Consolidation Characteristics

Soil of Strata ②-1, 2, 3, 4, ③, ④ and ⑤is classified as clay. They consist of soft clay with N value of 3 or lower. Their coefficients of volume compressibility are relatively high around mv = $5x10^{-2}$ cm^2/kgf. In addition, the coefficients of consolidation are as small as $200m^2/day$ or less. These strata show representative consolidation characteristics. Particularly Strata ④ and ⑤-1, 2 are so thick as to show relatively large settlement. As to Strata ⑦-1 and 2, the coefficients of volume compressibility are as small as mv = $1x10^{-2}cm^2/kgf$ or less. The consolidation yield stress is also high. That the strata show little settlement. Stratum ⑦-2 is also a water-retaining layer, therefore, it is expected to function as a drainage layer.

2.1.3 Characteristics as Fill Material and Subgrade Bearing Capacity

Strata ②-1 and 2 will be used as fill materials. At the same time, they will be also used as the subgrade course. The results of the laboratory tests for compaction and CBR are shown in TableII-2.1.3(a). TablesII-2.1.3(b) and 2.1.3(c) show the results of in-situ tests for density, water content, and CBR. The results of laboratory tests show higher values of both density and CBR than those of in-situ tests. Both strata have high water content in their natural state. Namely, they are loosely sedimented and have the possibility to be improved by compaction.

TableII1-2.1.3(a) Results of Laboratory Tests for compaction and CBR of Fill Materials around the Airport Site

		Compaction Test						CBR Test(submerged)		
		Max. Dry Density			Optimum Water Content			CBR 25		
	Test No.	Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average
Fill Materials around the site	8	1.74	1.78	1.76	17.2	19.0	18.0	5.4	7.9	7.0
Stratum②-1	13	1.75	1.77	1.76	17.2	19.0	18.0	6.5	8.3	7.2
Stratum②-2	13	1.73	1.79	1.77	16.9	18.5	17.6	6.0	8.0	6.7
Sludge on the channel bottom	3	1.74	1.75	1.74	19.5	20.0	19.7	4.6	5.1	4.8

Fill Materials around the site: Test results of the additional survey in the FS (feasibility study) stage strata②-1,2, sludge on the channel bottom: Test results of the detailed survey

TableII1-2.1.3(b) Results of the Field Tests fore Density, Water Content and CBR in Subgrade Parts (Stratum ②-1) in Each Area

		Tests for Density and Water Content						Field CBR Test		
		Dry Density ρ dn (t / m³)			Natural Water Content Wb(%)			CBR 25 (%)		
	Test No.	Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average
Runway Area	20	1.44	1.55	1.48	26.3	31.3	29.2	2.8	4.4	3.6
Taxiway Area	9	1.40	1.53	1.46	28.0	33.9	31.2	1.5	5.4	3.8

Runway Area: Test results during the survey in the FS stage
Taxiway Area: Test results during the detailed survey

TableII1-2.1.3(c) Results of Field Tests for Plate Loading During Detailed Survey in Each Area

Ground Bearing Capacity (kgf / cm²)			Coefficient of Deformation E0 (kgf / cm²)			Coefficient of Expansion E cr (kgf / cm²)			Coefficient of Bearing Capacity k 75 (kgf / cm²)		
Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average	Minimum	Maximum	Average
0.64	1.02	0.89	44.7	105.8	71.7	116	228	157	1.25	2.91	1.72
0.69	1.04	0.90	57.8	129.6	80.4	90	249	144	1.84	3.31	2.54
0.54	0.82	0.63	27.7	40.4	33.9	89	125	100	1.77	2.86	2.39

Upper row: Runway area (11), Middle row: Taxiway area (10), Lower row: Apron area (8) Figures in ( ) are the number of the test

2.2 Settlement Values in Original Ground Conditions and Necessity of Improvement

2.2.1 Method of Settlement Analysis and Analysis Conditions

As mentioned in the previous section, in this area, the upper strata with approximately 30 m depth are alluvial cohesive soils. Therefore, consolidation settlement will take place if banking / paving is carried out. Especially, Strata ③ and ⑤ are so thick that relatively large consolidation settlement will take place and that the consolidation speed will be slow. For settlement analysis, the one-dimensional consolidation analysis method was employed and the Terzaghi equation for degree of consolidation was used. The final settlement values were determined by applying the coefficients of volume compressibility. Principles and conditions assumed in the settlement analysis are as follows:

(1) To conduct consolidation settlement analysis at every drilling point, using the thickness of layers and the results of consolidation tests at every layer so as to reproduce uneven settlement values due to unevenness in layer thickness and the characteristics of consolidation at each point.

(2) To apply even load increase ΔP due to banking and paving in each layer. Because it was assumed that the load area is so large that the same load as the upper layer load would occur in each layer at a depth of around 30 m.

(3) Stratum ① will be excavated and replaced with fill materials because if contains many plant roots and stems.

(4) To adjust the height to the planning height of paving at each drilling point in the runway, taxiways, apron (runway and the taxiways: middle part 5.3 m, both ends 5.1 m in elevation; apron area: 4.3 ~ 5.3 m in elevation).

(5) Strata subject to settlement: Strata ②-1 ~ ⑤-2 (Strata under ⑦-1 have large consolidation yield stress and are in condition of excess consolidation. Therefore, it is judged that they will not subside under paving load.)

(6) To provide drainage on bothsides of the surface and the lower stratum ⑦-2.

Items 5) and 6) mentioned above have been verified by comparing the results of the field tests and those of theoretical analysis.

2.2.2 Settlement Values in the Existing Soil Conditions (Results of consolidation Settlement Analysis)

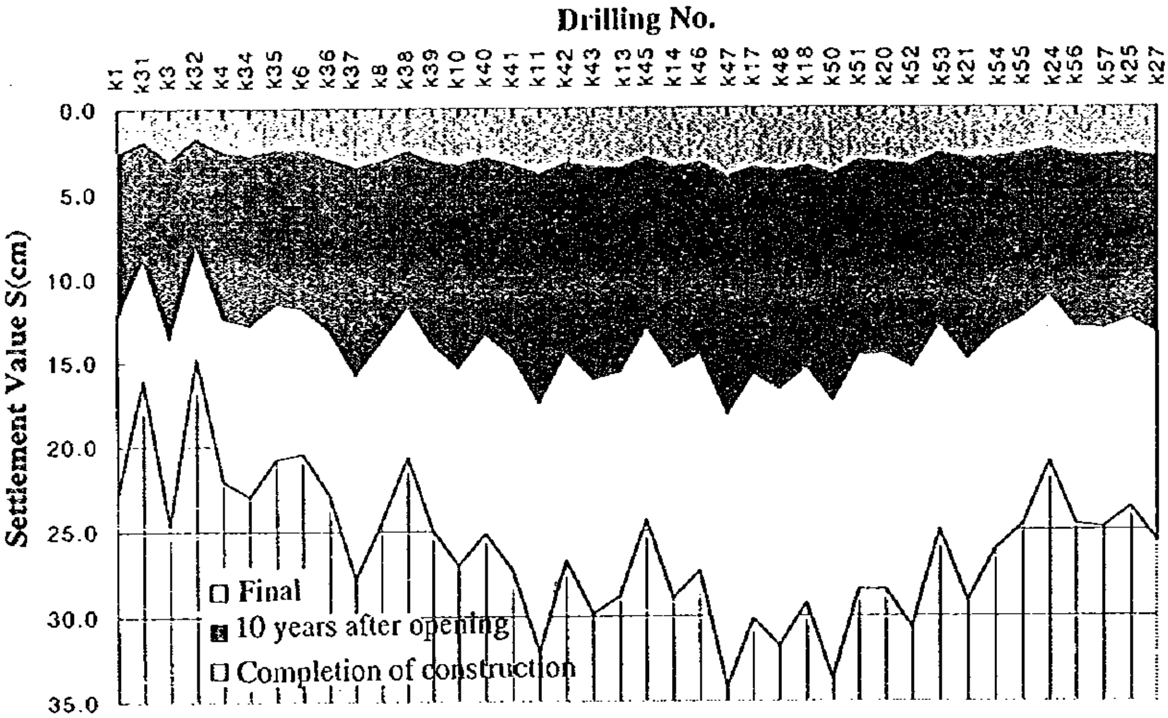
The following table and FiguresII1-2.2.1 ~ 2.2.3 show settlement values in the existing soil condition of the runway, taxiways. and apron. If banking and paving are carried out to the planning height of the runway and taxiways, i.e. 5.1 ~ 5.3 m, and that of the apron, 4.5 ~

5.3 m, the final settlement values will be 15 ~ 34 cm. FigureII-2.2.1 shows the relation between time and settlement values at the typical drilling point No. K11. Eighty percent of these settlements occur in the Strata④ and ⑤. The thickness of the subject layers is as thick as over 35 m, and the coefficient of consolidation is small. As a result, nearly 40 years are required to reach 90% in consolidation degree. The consolidation degree for ten years after airport opening will be only 50% plus. Settlement values (residual settlement values) in this period will range from 4 cm to 14 cm. The settlement values in each distance between drilling points (100 ~ 200 m) will be 0 ~ 4 cm in the same period. All of these values are low.

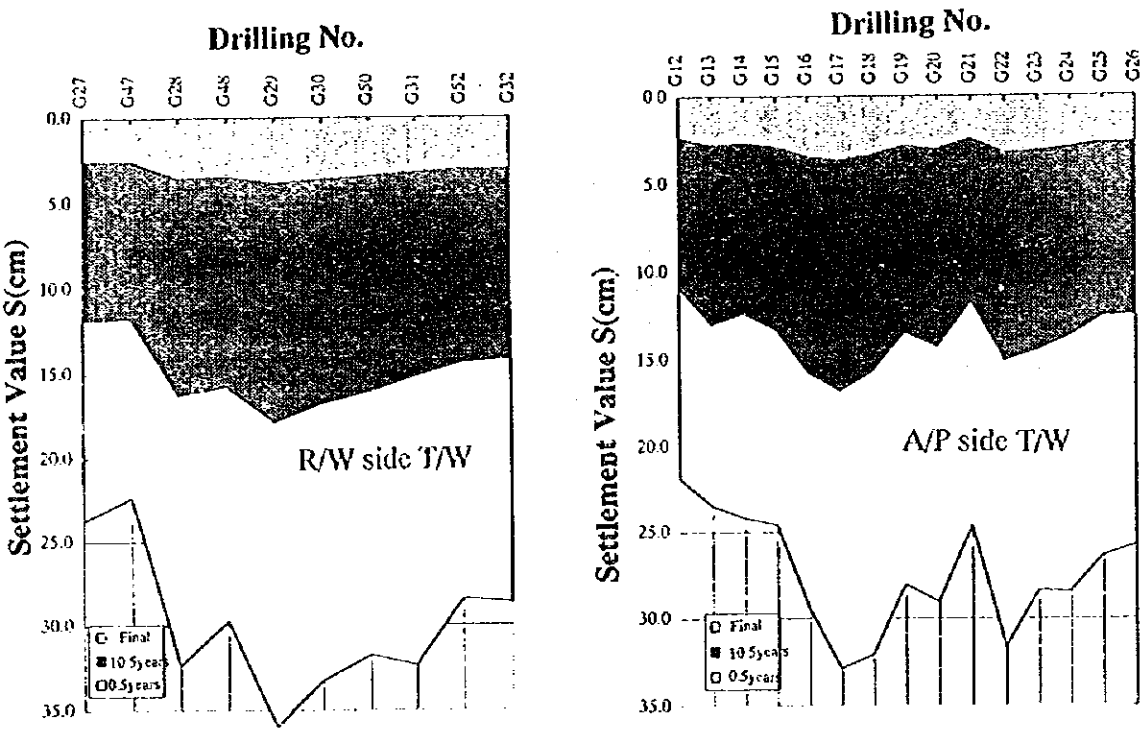
TableII-2.2.1 Settlement Value of Runway, Taxiway and Apron Areas

Area	Final Settlement Value of Each Stratum (cm)			Settlement Value of All Strata (cm)			Settlement Value for ten-year operation(cm)	
	Stratum②	Stratum③,④	Stratum⑤	One year	Eleven year	Final	Absolute	Uneven
Runway	2.1~4.7 (3.3)	8.7~22.6 (16.8)	3.7~8.5 (5.8)	1.8~4.1 (3.1)	8.0~10.2 (13.9)	14.8~34.1 (25.9)	6.2~14.1 (10.8)	0.1~4.3 (1.5)
Taxiway in the R/W side	1.6~7.5 (4.2)	13.5~24.5 (19.0)	4.8~9.7 (6.8)	2.6~3.9 (3.3)	11.8~17.9 (15.0)	22.4~36.1 (29.9)	9.2~14.0 (11.8)	0.1~3.5 (1.0)
Taxiway in the A/P side	2.8~6.4 (4.5)	12.1~22.2 (17.6)	3.6~8.3 (5.3)	2.4~3.7 (3.0)	10.9~16.8 (13.7)	~21.8~33.0 (27.4)	8.5~13.1 (10.7)	0.1~2.6 (1.1)
Apron	1.7~3.7 (2.6)	5.1~14.5 (10.2)	2.1~6.6 (3.9)	1.2~2.6 (3.9)	5.6~12.0 (8.8)	8.9~22.7 (16.7)	4.4~9.4 (6.9)	0.0~4.3 (1.7)

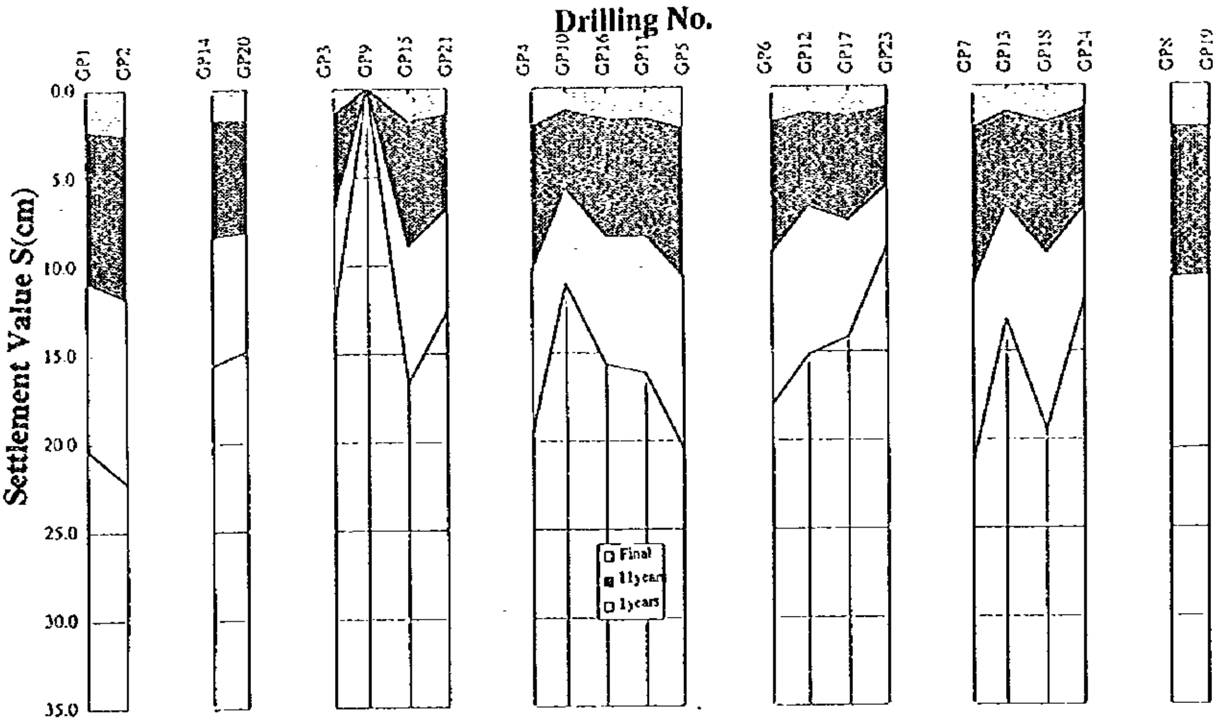
In the Apron area, a dike (about 7.5 m in elevation) is being used as a road now. As this place doesn't settled, a surrounding settlement of 4 ~ 9 cm will appear as uneven settlement.



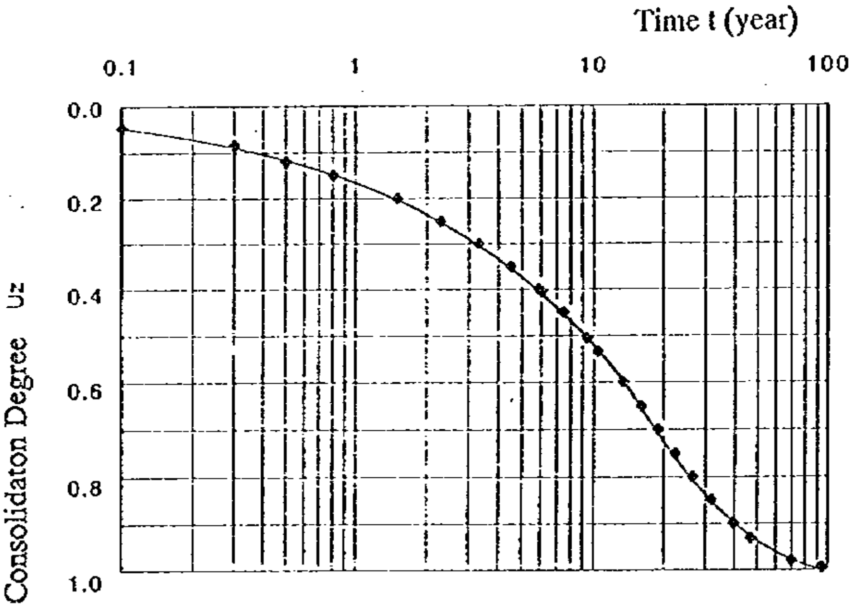
FigureII-2.2.1 Settlement Values of Untreated Ground (Runway)



FigureII-2.2.2 Settlement Values of Untreated Ground (Taxiway)



FigureII1-2.2.3 Settlement Values of Untreated Ground (Apron)



FigureII1-2.2.4 Relation between Time and Consolidation Degree of Untreated Ground (k45)

2.2.3 Allowable Settlement Values and Necessity of Ground Improvement

(1) ICAO Standards and Allowable Settlement Values

No standard values are clearly mentioned as allowable settlement values for designing the airport basic facilities such as runway, taxiway and apron,. Relating standard values are ICAO standards for longitudinal and transverse gradients. The following figures are allowable differential settlement values Sa. They were obtained by setting a runway vertical section curve at gradient changing points. This curve provides the most severe standards among others. The larger the width of settlement is, the higher allowable settlement values can be set up; the lower the width is, the lower allowable settlement values can be. According to examples of analysis of Tokyo International (Haneda) Airport, the average width of differential settlements is around 200 m. And 50 cm is chosen an allowable residual settlement value in the Phase I and II Areas for ten years of common use (differential settlement value is about the half of this).

If the width of differential settlement is around 100 m (half of the above-mentioned value), and the width and value of differential settlement under this level are considered as mere deformation, then the differential settlement value is around Sa = 10 cm. The ground of Pudong Airport is uniform in comparison with that of Haneda Airport. Therefore, a value around Sa = 10 cm may be established as standard.

	Runway		Taxiway	
TL=	39	m	39	m
R=	30,000	m	3,000	m
θ =	0.001300	Rad	0.012999	Rad
SL+R=	30,000.025	m	3,000.253	m
SL=	2.5	cm	25.3	cm
R-Sa/2=	29999.975	m	2999.747	m
Sa=	5.1	cm	50.7	cm
Vertical Gradient =	0.130	%	1.300	%

	Runway		Taxiway	
TL=	55	m	45	m
R=	30,000	m	3,000	m
θ =	0.001833	Rad	0.014999	Rad
SL+R=	30,000.050	m	3,000.337	m
SL=	5.0	cm	33.7	cm
R-Sa/2=	29999.95	m	2999.66256	m
* Sa=	10.1	cm	67.5	cm
Vertical Gradient =	0.183	%	1.500	%

	Runway	
TL=	100	m
R=	30,000	m
θ =	0.003333	Rad
SL+R=	30,000.167	m
SL=	16.7	cm
R-Sa/2=	29999.833	m
Sa=	33.3	cm
Vertical Gradient =	0.333	%

TL : Tangent(≒uneven settlement width/2)

R : Radius of the vertical section curve

θ = arctan(TL/R)

SL+R= R/cos θ

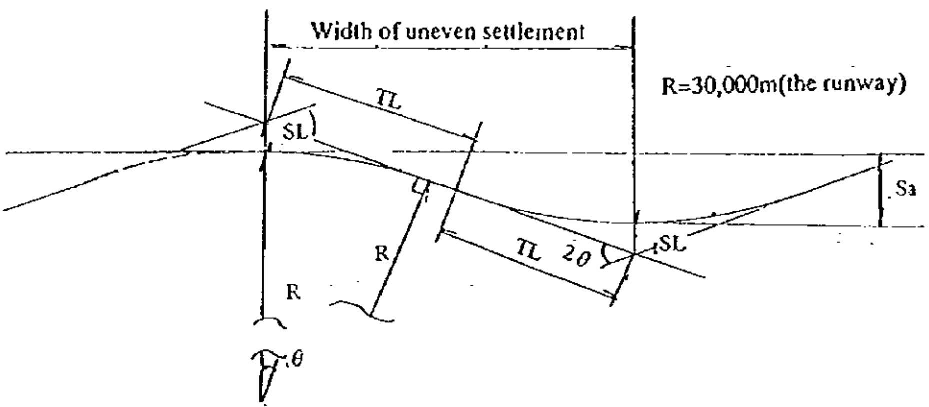
SL= SL+R−R

R-Sa/2= Rcos θ

Sa= (R−(R-Sa/2))x2

Vertical Gradient = tan θ

*See the following figure.



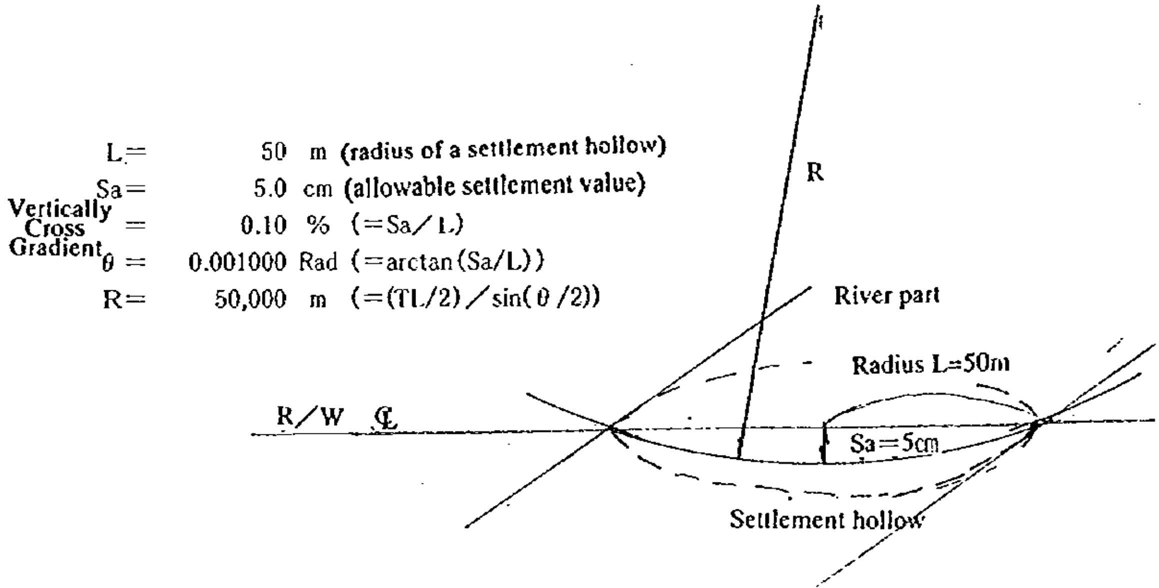
(2) Standards of Chinese Side

On the other hand, the Chinese side has established the following as basic values of allowable settlement values for ten years in the common use period.

Allowable settlement value : 10 cm or less

Allowable differential settlement value : 5 cm or less within a settlement hollow with a 50 m radius (gradient variation of 0.1% or less)

It is considered that these are based on the following the assumptions;



(3) Necessity of Ground Improvement

Settlement values under the runway exceed the Chinese standards, i.e. "10 Cm or less for ten years". This requires ground improvement against settlement. However, the ground of this area is relatively uniform. In addition, differential settlement values are as small as 2 ~ 3 cm. Therefore, it is judged that the ground will not have particular influences on operation of airplanes and structure of pavement. Consequently, it is considered that there is little necessity to actively improve the ground against settlement. However, improvement of sub-base should be made.

2.2.4 Examination of Liquefaction of the Ground

According to the investigation results so far, Stratum ②-4 is a sandy-silt layer and distributed at or above a depth of 7 m. N values obtained from standard penetration tests are 3 ~ 18. The average N value of the areas is about 10. The values of static cone penetration tests of each area range qc = 16 ~ 29 kgf/cm^2. Here, these test values are applied to "Ground Basic Design Standards" of Shanghai City. Then, according to the statement, "Stratum ②-4 may liquefy in the case of earthquake of Intensity Class 7 (equivalent to the 5th intensity on Japanese scale)". However, possibility of liquefaction of Stratum ②-4 is considered to be low. It is because this stratum is a sandy-silt layer with the sand content ratio of generally 20% or less, and its N value is high for its sand content ratio.

In the FL value method, the liquefaction strength ratio R at a certain depth in the ground and the ratio of repetitive shearing stress which occurs at the said depth during earthquake are estimated from:

* the N value of the ground,
* granular diameter, and
* the maximum acceleration on the ground surface.

Then, using these ratios, the safety rate against liquefaction (FL value) is determined as follows:

FL ≤ 1: Possible to liquefy

FL > 1: A few possibility of liquefaction

The results of analysis are shown in TableII1-2.2.2. According to the standards of Shanghai City, the horizontal seismic intensity (kh) is 0.08 in the Shanghai area, but kh = 0.10 and 0.15 have been applied to the analysis taking the importance of the international airport into consideration. According to the results of analysis, FL value is 1.15 in minimum even in case of kh = 0.15. Consequently it is estimated that the possibility of liquefaction is very little.

TableII1-2.2.2 Results of Judgment of Liquefaction of Stratum ②－4

(during FS survey by Japanese side)

Drilling No.			G2-1	G2-2	G2-3	G2-4
Depth(m)			5.6～5.9	5.6～5.9	4.6～4.9	4.6～4.9
N Value			4	14	4	12
σ v (kgf/cm^2)			1.101	1.103	0.904	0.909
σ v' (kgf/cm^2)			0.562	0.830	0.466	0.717
D50 (mm)			0.076	0.036	0.030	0.028
Fine-grain content ratio Fc (%)			45	77	90	91
Dynamic shearing strength ratio R		R1	0.157	0.266	0.163	0.256
		R2	0.149	0.190	0.190	0.190
		R3	0.002	0.148	0.200	0.204
		R	0.308	0.604	0.553	0.650
Decrease factor γ d			0.915	0.915	0.930	0.930
Shearing force ratio during earthquake L	kh=0.1		0.179	0.122	0.181	0.118
	kh=0.15		0.269	0.182	0.271	0.177
FL Value	kh=0.1		1.72	4.97	3.06	5.52
	kh=0.15		1.15	3.32	2.04	3.68
Possibility of liquefaction			No	No	No	No

2.3 Comparison and Study of Ground Improvement Methods

2.3.1 Comparison and Selection of Recommended Ground Improvement Methods

The ground in the airport site area has soft Strata ②-1, 2 and 3 as a subgrade course. Their soil is classified as silty clay ~ clay. Under these strata, there is Stratum ②-4 consisting of relatively good sandy silt. It is followed by Strata ③, ④ and ⑤. Their soil is classified as clay ~ silty clay. These strata show relatively large settlement in this area. Banking height for this site reclamation is low. However, soft Strata ④ and ⑤ are thick, and have the consolidation settlement of 15 ~ 34 cm. Speed of consolidation is mild.

Considering:

* these ground conditions,
* consolidation characteristics, and
* the fact that the structures on the surface will be the basic taking-off and landing facilities such as the runway and taxiways,

the following countermeasure methods are selected for respective purpose:

1) Surface Improvement Plan: subgrade improvement (surface-stratum mixing treatment or replacement method). To improve the sub-base part only (increase of bearing capacity), accepting settlement of and under Strata ②-3 and 4.

2) Shallow Improvement Plan: Ram Drop Compaction Method.

To improve settlement of shallow strata (down to Stratum ②-4) and the subgrade course, accepting settlement of deep strata (Strata ④ and ⑤).

3) Deep Improvement Plan: Preload + Vertical Drain Method (+ improvement of the subgrade).

To counter settlement of deep strata (Strata ④ and ⑤).

TableII-2.3.1 is a list of countermeasure methods for soft ground. It also contains principles of countermeasure methods, purposes, ground to be covered and so on.

2.3.2 Study of Subgrade (Surface) Improvement Method

In the proposed airport construction site, Strata ②-1 and 2 will be used as the in-situ ground or fill material. In other words, they will constitute the subgrade of basic taking off and landing facilities. These strata are loose sediment. According to the results of CBR tests for the original ground, CBR is as low as about 3%, but it will increase up to around 7% after compaction. Therefore, it is desirable to improve them. From the viewpoint of making the

TableII1-2.3.1 List of Countermeasure Methods for Soft Ground

Technique and Main Principles of Countermeasure		General Name of Methods	Summary Description of Methods	Purpose of Improvement	Soil to be Applied	
I. Changes of form of structures		Pressing-down fill-up method	Reduce moment and ensure stability by pressing down at toe of slope of the main body of banking	Prevention of destruction due to sliding	Clayey/organic soil	10
		Load reduction method (styroform etc.)	Designed to ensure stability and reduce settlement by decreasing load.	Reduction of settlement		10
		Reinforced earth method (Terre Armee)	Designed to reduce settlement by placing reinforcing material in soil structures	Prevention of destruction	Sandy/clayey soil	9
		Sheet pile method	Promote stability by restricting movement of ground with sheet plates.	Prevention of destruction due to sliding	Sandy/clayey/organic soil	—
		Various kinds of foundation methods	Move external force to bearing strata through basic structures.	Restraint of deformation/settlement of surrounding ground		—
II. Removal/replacement		Replacement method	Replacement of soft ground with soil of good quality	Prevention of destruction due to sliding, reduction of settlement	Clayey/Sandy/organic soil	10
III. Improvement of characteristics of soft ground	Consolidation /drainage	Loading fill-up method (preloading method, surcharge method)	Designed to promote consolidation of the ground by increasing pore water pressure with fill-up load.	Reduction of residual settlement Increase in ground strength	Clayey/organic soil	1
		Air pressure method (vacuum consolidation method)	Designed to promote consolidation of the ground by reducing pore water pressure with a vacuum.(same as above)			
		Method to reduce underground water level	Designed to promote consolidation of the ground by reducing the underground water level. (same as above)			
		Vertical drain method (sand drain, bagging sand drain, plastic board drain)	Promote consolidation by shortening drain distance with vertical drain. (commonly used with reduction of loaded banking, air pressure, underground water level, etc.)	Promotion of consolidation, reduction of residual settlement Increase in strength of the ground	Clayey/organic soil	2
		Crude-lime pile method	Promote consolidation by means of water absorption of crude lime/pillar-state placing.		Clayey soil	10
		Electric penetration method, semipermeable membrane method	Designed to promote consolidation by collection water with use of difference in electric potential or in concentration of solution. (examples are extremely rare)			—
		Crushed-stone pile method	Promote early dispersion of pore pressure of loose sand during earthquake	Prevention of liquefaction	Sandy soil	6
		Surface-stratum drainage method (trench, bowl drying)	Accelerate dryness by promoting draining of the surface layer. (with the nature of temporary construction)	Increase in strength of the surface ground	Clayey/organic soil	5
	Compaction	Sand compaction pile method	Compact loose sand with forced pressurization and vibration of sand piles.	Prevention of liquefaction Reduction of settlement Increase in strength of the ground	Sandy soil	3
		Rod compaction method	Compact with a vibration rod. (accompanied by resupply of sand)			6
		Vibrofloatation method	- ditto -			10
		Ram-drop compaction method (dynamic consolidation method)	Compact loose sand with impact at the time of heavy ram drop.	Reduction of settlement Prevention of liquefaction	Sandy soil	10
		Explosion method/electric impact method	Compact with impact of explosion or electric discharge. (examples are extremely rare)			—
	Concretion/heat treatment	Surface strata mixing treatment method	Form the surface slab compacted with stabilizing material as cement. (with the nature of temporary construction)	Securing trafficability	Clayey/organic soil	5
		Deep strata mixing treatment method (mechanically stirring method, injection stirring method	Form pillar bodies, blocks etc. reaching the deep layer with stabilizing material such as cement. (e.g.propagation of load to the bearing layer etc.)	Prevention of destruction due to sliding Restraint and reduction of settlement Restraint of shearing deformation Prevention of heaving	Clayey/sandy/organic soil	4
		Medical-fluid injection method	Compact the ground by injecting a liquid solution such as water glass.			7
		Sintering method	Promote drying and compacting of the ground by blowing hot air etc. into the underground. (small number of examples in Japan)			—
		Freezing method	Compact the ground by freezing artificially in a certain period. (with the nature of temporary construction)			8
IV. Reinforcement		Soil covering method (sand mat method)	Promote dispersion of stress by covering the ground surface to some thickness.	Securing trafficability	Clayey/organic soil	5
		Surface-strata covering method (sheets, nets, grids, bamboo nets etc.)	Promote stability of covering/filling soil by placing any of or combination of sheets, nets, ropes etc. on the surface layer of the soft ground.	Prevention of local destruction, reduction of local settlement		5
		Mattress method	Promote dispersion of loading by sinking crossbeam made of wood/steel to the soft ground.	Prevention of destruction due to sliding, dispersion of stress		—
		Route piling, soil nailing	Route piling is designed to promote close adhesion with the natural ground by inserting an iron bar to a drilling hole and the inserting cement with pressure. (soil nailing method is same one) Reinforce The ground by driving in route piles in the manner of combined piles.	Securing a gradient Formation of soil and structures	Sandy/clayey/organic soil	—
		Pile net method, pile cap method	Bear structures on the upper part directly with piles. (combine pile heads)	Prevention of destruction due to sliding Restraint of deformation/settlement of the surrounding ground		10
		Pile slab method				10
		Sand compaction pile method	Reinforce by driving compacted sand/crushed stone piles into the clay ground. (with all of three principles, i.e.consolidation, compaction and reinforcing)	Increase in ground strength, reduction of settlement, promotion of settlement, prevention of destruction due to sliding	Clayey/organic soil	3
		Deep strata mixing treatment method	Form underground structures with strong soil treated for stability	Prevention of destruction due to sliding		4

bearing ground uniform, improvement of the subgrade is also considered desirable. Possible improvement methods include:

1) Replacement with material of good quality

2) Surface mixing treatment method.

As to surface mixing technique, in China, small-sized improvement is carried out with manpower or wide by used machines such as backhoes. However, there are no examples of large-sized execution using stabilizers and so on. Therefore, subgrade improvement by means of replacement shall be planned.

In the case of subgrade improvement by means of replacement, attention should be paid to the height of the present ground, the underwater level, necessary subgrade bearing capacity etc. As a result, a supposed section to be improved by replacement is as follows:

* Subgrade bearing capacity after improvement = 10%

* Improved thickness = 1.0 m

* Modified CBR of replacement material

$$CBRm = \left\{ \frac{h1 \times CBR1^{1/3} + \times h2 \times CBR2^{1/3} + \cdots + hn \times CBRn^{1/3}}{h} \right\}$$

$$CBR1^{1/3} = 2x10^{1/3} - 3^{1/3} = 2.86$$

$$CBR1 = 23.4\%$$

where, CBRm: CBRs at each point (subgrade bearing capacity after improvement = 10%)

CBR1, CBR2, —: CBR of Stratum 1, Stratum 2, — respectively

CBR1: Stratum 1 (stratum to be improved)

CBR2: Stratum 2 (original ground = 3.0%)

h1, h2 —: thickness of Stratum 1, Stratum 2 — respectively

h=h1+h2+ — = thickness of the subgrade (2 m)

h1: thickness of stratum to be improved

2.3.3 Study of Ram Drop Compaction (Shallow Improvement)

In Japan, this method is employed only for refuse disposal sites and filled-up lands at sea because of its influence on the surrounding environment such as noise, vibration etc. Instead, shallow / deep strata mixing treatment technologies advance. On the other hand, in China, this method seems to be used actively instead of the shallow mixing treatment technique.

However, theoretical calculation of this method is not sufficient, and largely depends on results of field tests. This method will be studied in the next section, comparing with the results of field tests.

2.3.4 Study of Preload + Vertical Drain (Deep Strata Improvement) Method

(1) Selection of Deep Improvement Method (Countermeasure for Deep Strata Settlement)

Methods to promote consolidation settlement of deep strata (Strata ④ and ⑤) and to reduce residual settlement values include:

- Preload Method
- Atmospheric Pressure Method, Underground Water Level Reduction Method
- Preload + Vertical Drain (sand drain/plastic board drain) Method
- Other Deep Strata Mixing Methods, and Crude-Lime Pile Method, Electric Penetration Method and Semipermeable Membrane Method.

In this airport site, the preload + vertical drain method is selected because:

1) the layers subject to settlement are deep,

2) the coefficient of consolidation is relatively small, and

3) it is necessary to finish ground improvement work in a short period.

If the underground water level can be reduced, it is possible to employ the underground level reduction method instead of the preload method. Here, discussion is made on the preload + vertical drain method.

(2) Examination of the Preload + Vertical Drain Method

Examination of the vertical drain method was carried out at Drilling No. K.45. Its drilling depth passes through Stratum ⑦-2 in the vicinity of the center of the runway. Assumptions as analysis conditions are as follows:

* The work period to be taken for the soft ground should be one year considering all construction process.

* The preloading period would be four months.

Cases examined and the results of them are shown in TableII-2.3.2. It was assumed that drain material would be plastic boards with a width of 10 cm and a thickness of 3 mm and the distribution would be square. As an analysis method, Barron's solution (Takagi's solution) from the diffused consolidation equation was used. Other analysis conditions were same as those for the present (untreated) ground.

TableII-2.3.2 Results of Examination of Preload + Vertical Drain Method

Height of Preload Preload	Final Settlement Value (cm)	Settlement Value after the Preloading Period of Four Months (Consolidation Degree)		
		Drain Pitch 1.0m	Drain Pitch 1.2m	Drain Pitch 1.5m
5.6m in elevation	26.6	25.5cm(96%)	22.9cm(85%)	18.1cm(68%)
5.8m in elevation	29.5	28.3cm(96%)	25.1cm(85%)	20.1cm(68%)
6.0m in elevation	32.4	31.1cm(96%)	27.5cm(85%)	22.0cm(68%)

cf. : Final settlement value at Drilling No.45 in Strata②～④ : 19.7cm, strata⑤ : 4.7cm, all strata : 24.4cm

As a result of examination described above, the followings are selected as combination exceeding the final settlement value of 24.4 cm at this point:

* Drain Pitch 1.0 m + Preload 5.6 m in elevation (ground elevation 3.9 m in, + height of preload banking 1.7 m)

* Drain Pitch 1.2 m + Preload 5.8 m in elevation (ground elevation 3.9 m, + height of preload banking 1.9 m)

The deepest placing depth of vertical drains which has been realized in China is around 20 m. This means that improvement will be able to be made only down to Stratum ④. However, the settlement value of Stratum ⑤is 10 cm or less. It is consequently possible to limit final residual settlement values to 10 cm or less if strata down to Stratum ④ are improved.

2.4 Summary of Results of Field Tests for Ground Improvement Methods

2.4.1 Purposes and Test Plan

(1) Summary of Field Tests

Among the above-mentioned methods proposed for comparison and selection, the following methods were tested in the site:

1) Ram Drop Compaction Method

2) Preload + Vertical Drain Method

3) Underground Water Level Reduction Method

The purposes are to find whether or not the effect will be obtained and to confirm uneven settlement of the river part, etc. Construction for test was carried out on Seshin Street in the area 300 m west of the center of the runway. The processes are described below.

After then, observations have been made continuously in the site, regarding settlement, pore water pressure, etc.

March 1 - April 19	:	Construction of temporary roads, geological surveys in the test area and removal of sludge and filling-up in a channel
April 11 - May 3	:	Full placing for ram drop compaction and finishing placing
April 23 - June 10	:	Full placing for ram drop compaction and finishing placing
April - May 9	:	Placing of vertical drain, sheet piles and well point to stop surrounding water, construction of deep wells, and start of pumping up

(2) Tested Ground Improvement Methods

Kinds and purposes of ground improvement methods tested are as follows.

1) Ram Drop Compaction Method (Dynamic Consolidation Method)

For the ram drop compaction method, the following were planned to examine the manners, energy, etc. of heavy ram drop and the effect.

Test Area No.	Mat Material etc.	Drop Energy (KN/m)	Interval of Drop Point (m)	Pattern of Drop Point	Drop Stage	Number of Drops in Each Stage
T1-1	Sand mat80cm Drain material D=6m,□2.5m	2,000(point type) 800(side type)	3.5 0	square	2 2	8~9 3~5
T1-2	Surrounding ordinary soil	1,500(point type) 800(side type)	3.0 0	square	2 1	10~12 3~5
T1-3	Slug mat 80cm	2,000(point type) 800(side type)	3.5 0	square	2 1	8~9 3~5
T1-4	Sand and gravel mat 80cm Drain material D=6m,□2.5m	1,500(point type) 800(side type)	3.0 0	square	2 1	8~9 3~5

Filled materials of the channel T1-1 area : Fly ash, T1-3 area : surrounding ordinary soil

2) Preload + Vertical Drain Method

For the preload + vertical drain method, the following types were planned to examine the effects of length and pitch of drains and height of preload.

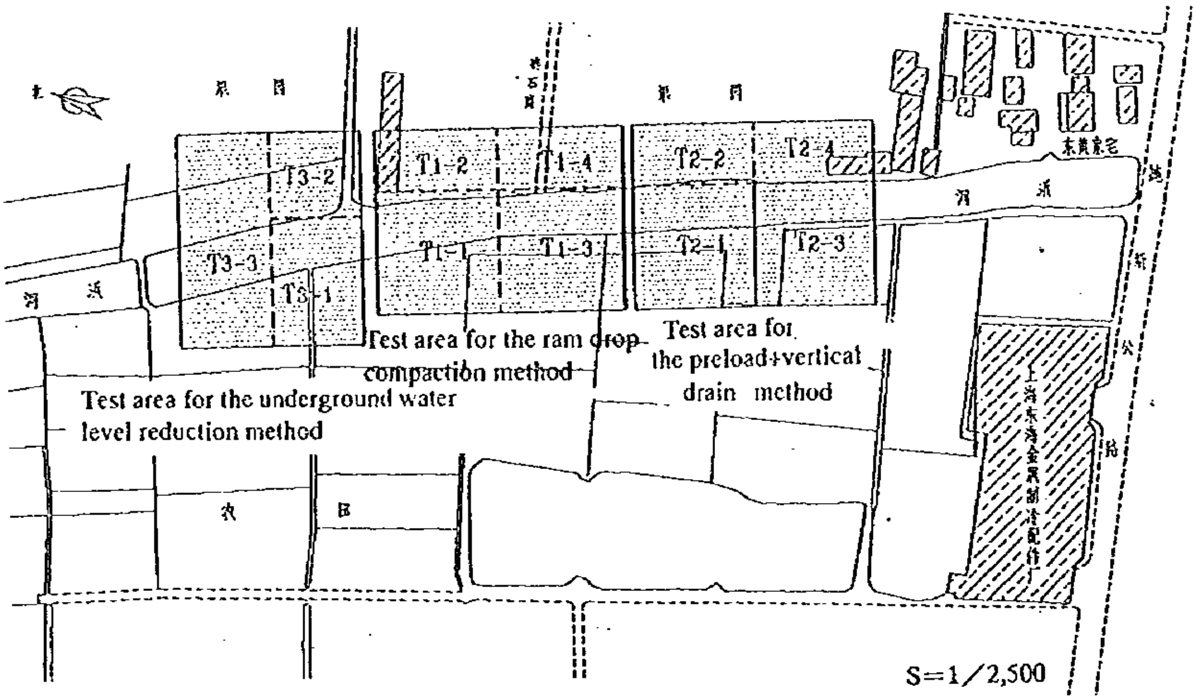
Test Area No.	Surface of Banking in Elevation (m)	Thickness of Banking (m)	Pitch of Placing Drains	Length of Drains
T2-1	5.3	6.723	1.2	21.0
T2-2	5.3	6.723	1.2	21.0
T2-3	4.5	5.829	1.5	6.0
T2-4	4.5	5.829	2.0	6.0

3) Underground Water Level Reduction Method

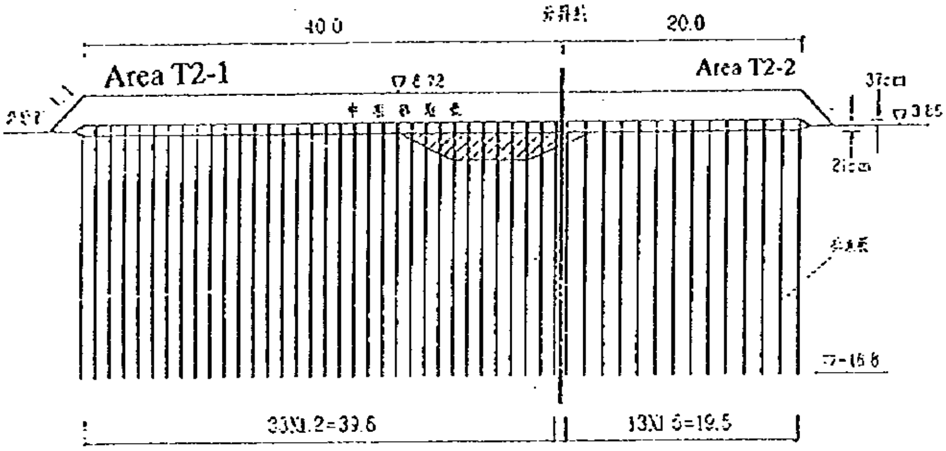
For the underground water level reduction method, the following types were planned as well as those for the preload + vertical drain method.

Test Area No.	Depth of Deep well (m)	Pitch of Placing Drains (m)	Length of Drains (m)
T3-1	14.6	1.5	22.0
T3-2	14.6	1.5	15.0
T3-3	14.6	-	-

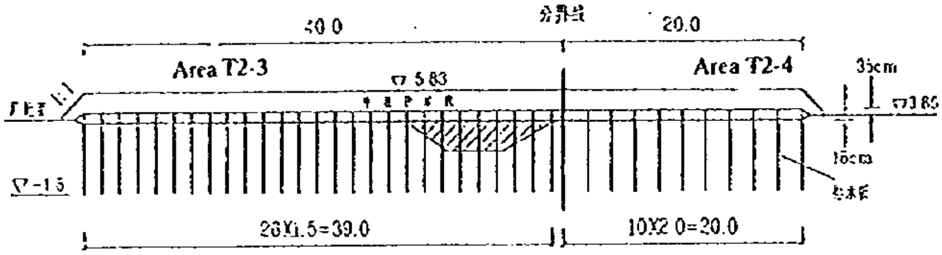
The areas planned to be tested are shown in FigureII1-2.4.1. Their cross sections of them are shown in FiguresII1-2.4.2 ~ 2.4.3. Geological and soil conditions in the test areas are similar to those in the Phase I proposed airport construction site.



FigureII1-2.4.1 Areas of Field Tests for Grand Improvement Methods

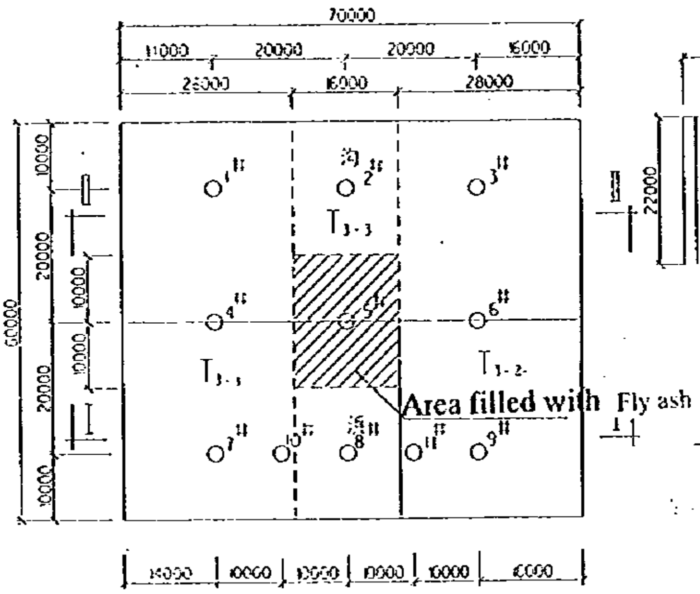


I-I Cross-Section

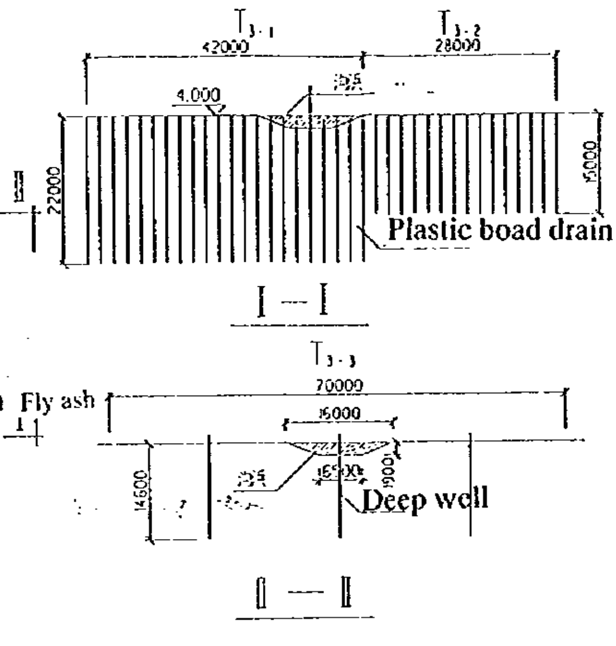


II-II Cross-Section

FigureII1-2.4.2 Cross-Section of Test Area T2 (Preload + Vertical Drain Method)



Distribution of Deep Wells



FigureII1-2.4.3 Plan and Cross Section of Deep Wells in Test Area T3 (Underground Water Level Reduction Method)

2.4.2 Results of Test for Ram Drop Method

(1) Relation between drop energy and average settlement values of the whole area is as follows:

Test Area	Drop Energy	Average settlement value in each area	Remarks
T1-1	2,000kN·m	39.9cm	Sand mat 80cm, placing drain
T1-2	1,500kN·m	34.8cm	Filled with surrounding ordinary soil
T1-3	2,000kN·m	30.9cm	Slug mat 80cm
T1-4	1,500kN·m	29.1cm	Sand and gravel mat 80cm, placing drain

* Weight of a ram 12tf × drop height (13-17m)=153-204tf · m=1,500~2,000kN · m

Relation between number of drops and settlement values is as shown in the figure below. In the T1-1, 3 and 4 areas, 7 times of drops resulted in 85% of the whole settlement value. Therefore, the optimum number of drops is judged to be 8 or 9. In the T1-2 area, there is no mat material and tamping is carried out directly on the surface of the area. Accordingly, in 5 or 6 times of tamping of the first stage, 85% of the whole settlement value was exceeded.

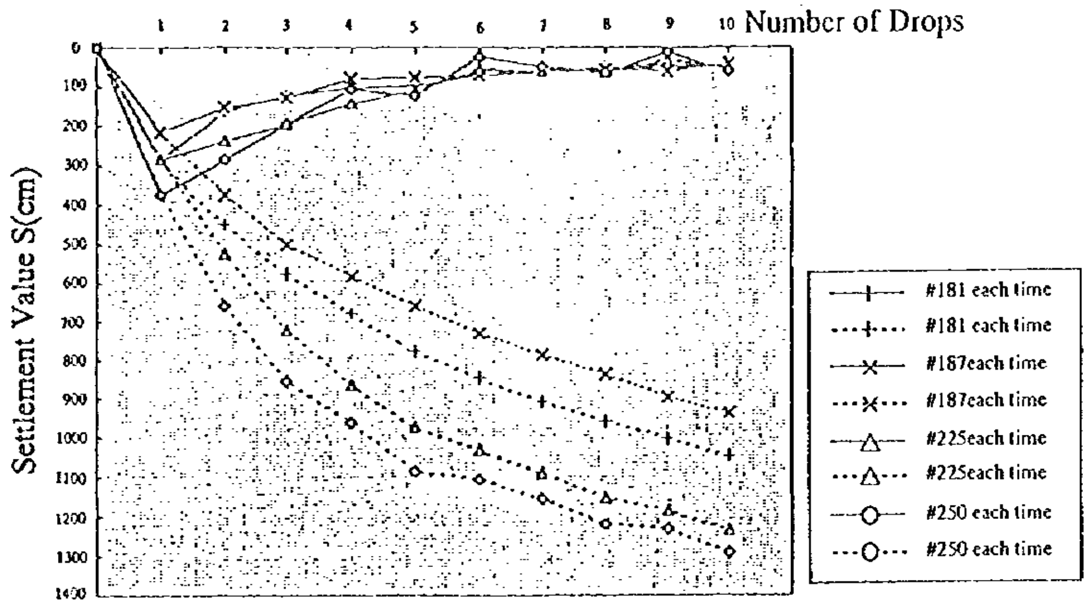
The class of drop energy used in China is generally 1,000 ~ 2,000 kN·m. The largest one was planned in this test. From Japanese empirical formula, the relation between drop energy and depth of improvement is as shown below (same as in China). The test is carried out with a little smaller energy and closer pitches than those shown below.

Depth of improvement $D(m) = \alpha\sqrt{\text{ram weight } W(tf)\cdot \text{drop height } H(m)}$

α: Influence factor (0.5 - 0.6: Silt ground and general viscous soil. If the underground water level is high -- Chinese reference book)

Here, with depth of improvement D=8 m and α=0.5, W·H = (8/0.5)2 = 256tf·m = 2,510kN·m

Pitch of dropping points 1(m) depth of improvement D(m) = 8 m



FigureII1-2.4.4 Number of Drops, Settlement Values and Accumulative Percentage (TI-3: Test Area with Slug Mat 80 cm)

(2) Improved Depth and Improved Strength

Depth distribution of N values before and after improvement is shown in FigureII1-2.4.5. Increase of N values is observed in soil consisting of silty sand ~ sandy silt in the Strata ②-2, ②-3-1 ~ 4 to a depth of about 8 m. Increase of N values is not recognized in the Strata ②1 and 3 consisting of silty clay ~ clayey silt. These strata exist in the upper and middle parts. From both Japanese and Chinese empirical formulas, relation between dropping energy and increase of N values is as follows.

Dropping energy E(tf·m/m^2) = Ev · D

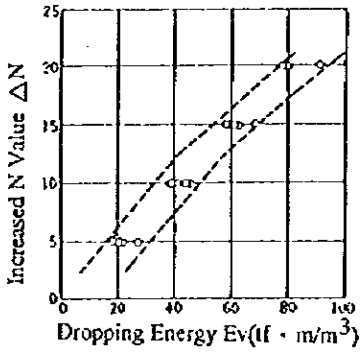
Ev = Dropping energy (tf·m/m^3) per 1 m^3 of soil to be improved

T1-1, 3: E = 8 times x 204tf·m/(3.5 m x 3.5 m) = 133tf·m/m^2

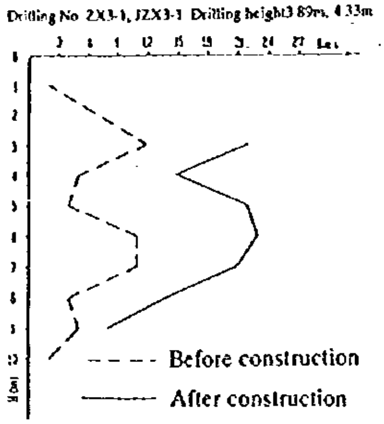
Ev = 133/8 m = 17

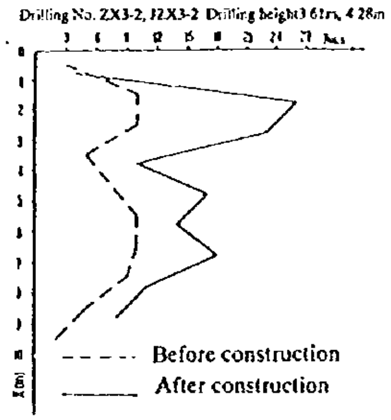
T1-1,3: E = 8 times x 153tf·m/(3.0 m x 3.0 m) = 136tf·m/m^2

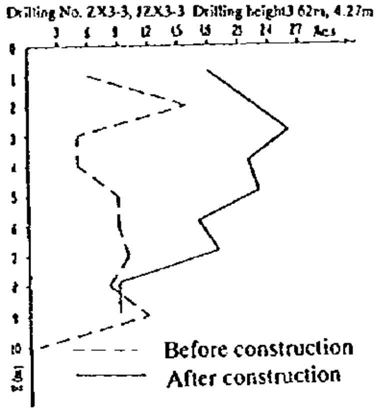
Ev = 136/8 m = 17



Applying these energies to the figure on the right side, the increase of N value is something like "Δn = 3-5". The results of "Δn = 3.10" were obtained in the test. Increases in density etc. are shown TableII1-2.4.1. Effect of placing drain has not been observed.







FigureII1-2.4.5 Distribution of N Values Before and After Ground Improvement (T1-3: Test Area with Slug Mat 80 cm)

(3) Bearing Capacity etc. after Improvement

Ground bearing capacity after improvement is as shown in the table below. Where mat materials are placed on the present ground, the ground bearing capacity shows higher values due to the effect of mat materials.

TableII1-2.4.2 Ground Bearing Capacity after Improvement (Average in each areas)

Test Area	Modulus of elasticity En(Mpa)		Coefficient of Bearing Capacity k75(MN/m3)		CBR in Insite Tests (%)	
	Before improvement	After improvement	Before improvement	After improvement	Before improvement	After improvement
T1-1 (Sand mat 80cm)	19.4	55.0	40.4	50.9	2.8	12.0
T1-2 (Surrounding ordinary soil)	12.4	24.7	33.1	24.2	5.1	5.5
T1-3 (Slug mat 80cm)	21.2	117.6	29.1	141.3	5.9	24.7
T1-4 (Sand and gravel mat 80cm)	12.7	70.7	34.6	62.0	3.5	11.4

* A Test stratum before improvement is Stratum ②-1; The test stratum after improvement is a mat layer or a filled layer in each area.

Settlement values in case of banking of 2.0 m height on the ground with aforementioned work completed were as follows. As to Strata ②-1~3, values of settlement were 1 ~ 2 cm of immediate settlement only under the work. Those of Strata ③ and ④ have been 2 ~ 7 cm at a time of about two months later.

TableII1-2.4.1 List of Results of Soil Tests at Each Stratum in the Ram Drop Compaction Method

Stratum No.	Name	Water Content Wn(%)		Dry Density γ d(tf/m3)		Prosity e		Standard Penetration Factor M v (E-2cm2/kgf)		Standard Penetration N値		Static Cone Penetration E s (kPa)		Static Cone Penetration Q c (kPa)		Calculated Strength of the Ground (kPa)	
		Before	After	Before	After	Before	After	Before	After	Before	After	Before	After	Before	After	Before	After
②-1-1	Yellow silty clay ~clay	33.1	27.0	1.434	-	0.94	-	2.32	-	2.0	2.0	18.0	-	0.70	-	110	125
		31.0	25.2	1.434	1.582	0.94	0.76	2.32	2.19	2.0	-	18.0	22.5	0.70	1.35	110	130
		31.0	43.1	1.439	1.277	0.94	1.18	2.32	2.97	2.0	4.0	18.0	-	0.70	-	110	125
		33.1	29.7	1.434	1.448	0.94	0.99	2.32	2.44	2.0	15.0	18.0	-	0.70	-	110	130
②-1-2	Yellow silty clay	29.0	19.4	1.455	1.607	0.88	0.74	0.68	0.55	10.0	16.0	35.9	72.0	4.93	10.50	130	300
		29.0	22.3	1.455	1.610	0.88	0.72	0.68	-	10.0	14.6	35.9	67.5	4.93	11.00	130	300
		29.0	-	1.455	-	0.88	-	0.68	-	10.0	-	35.9	30.0	4.93	3.50	130	-
		29.0	24.1	1.455	1.521	0.88	0.80	0.68	-	10.0	24.5	35.9	80.0	4.93	9.15	130	300
②-3-1	Gray silty sand	28.1	24.9	1.505	1.536	0.82	0.80	0.64	0.51	8.8	20.3	35.3	73.3	4.34	8.30	130	300
		28.1	27.0	1.505	1.518	0.82	0.82	0.64	0.52	8.8	16.4	35.3	75.0	4.34	9.25	130	300
		28.1	27.7	1.505	1.502	0.82	0.82	0.64	0.56	8.8	20.0	35.3	85.0	4.34	8.50	130	300
		28.1	26.4	1.454	1.509	0.82	0.84	0.64	0.78	8.8	12.2	35.3	80.0	4.34	9.00	130	300
②-3-2	Gray sandy ~clayey silt	31.0	28.7	1.472	1.491	0.87	0.86	0.82	0.66	8.3	18.7	33.0	59.3	3.55	6.70	125	200
		31.0	28.4	1.472	1.518	0.87	0.83	0.82	0.61	8.3	19.3	33.0	70.0	3.55	6.70	125	200
		31.0	30.5	1.472	1.494	0.87	0.83	0.82	-	-	-	10.2	27.5	0.49	0.80	65	100
		31.0	29.8	1.472	1.478	0.87	0.89	0.82	0.66	-	-	10.2	40.0	0.49	2.00	65	100
②-3-3	Gray silty sand	27.4	25.0	1.538	1.584	0.79	0.74	0.60	0.59	10.4	18.5	57.9	81.0	6.57	8.00	150	300
		27.4	27.2	1.538	1.524	0.79	0.82	0.60	0.63	10.4	13.8	57.9	85.0	6.57	10.50	150	300
		27.4	25.8	1.538	1.549	0.79	0.79	0.60	0.51	8.3	19.8	33.0	65.0	3.50	6.30	125	200
		27.4	26.8	1.538	1.529	0.79	0.81	0.60	0.61	8.3	20.0	33.0	65.0	3.50	9.00	125	250
②-3-4	Gray sandy ~clayey silt	32.9	29.4	1.420	1.466	0.94	0.89	0.94	0.92	7.3	10.1	35.8	35.0	2.33	3.20	115	140
		32.9	31.3	1.420	1.446	0.94	0.93	0.94	0.86	7.3	9.3	35.8	42.5	2.33	3.10	115	150
		32.9	31.2	1.420	1.439	0.94	0.94	0.94	0.88	10.4	19.7	57.9	75.0	6.57	8.00	150	300
		32.9	29.2	1.420	1.477	0.94	0.89	0.94	1.06	10.4	13.1	57.9	92.5	6.57	9.50	150	300
③	Gray silty clay	42.9	39.6	1.257	1.294	1.21	1.15	3.66	2.68	2.0	4.0	12.8	12.5	0.55	0.70	70	80
		42.9	44.8	1.257	1.205	1.21	1.37	3.66	4.83	2.0	3.0	12.8	12.0	0.55	0.50	70	75
		42.9	41.6	1.257	1.268	1.21	1.20	3.66	3.92	7.3	11.3	35.8	43.3	2.33	2.90	115	150
		42.9	40.5	1.257	1.286	1.21	1.16	3.66	3.17	7.3	10.0	35.8	45.0	2.33	3.00	115	130
④	Gray silty clay	48.6	48.2	1.181	1.178	1.36	1.38	3.84	4.07	-	-	-	-	-	-	-	-
		48.6	47.7	1.181	1.209	1.36	1.34	3.84	4.22	-	-	-	-	-	-	-	-
		48.6	46.6	1.181	1.197	1.36	1.34	3.84	4.24	2.0	-	12.8	-	0.55	-	70	80
		48.6	48.5	1.181	1.189	1.36	1.37	3.84	-	2.0	-	12.8	20.0	0.55	0.88	70	70
Fill back in the river part	Ordinary soil	25.2	19.0	1.476	1.588	0.89	0.77	0.71	0.67	11.0	24.4	30.0	80.0	4.40	11.00	140	230
	Fly ash	42.2	37.3	1.048	1.093	1.07	0.70	0.69	0.67	8.0	20.0	35.0	120.0	3.5	12.0	130	200

Uppermostu: T1-1 (Sand most 80cm, drain material)
Middle-lower column: T1-3 (Slug mat 80cm)
Upper-middle column: T1-1(Surrounding ordinary soil)
Lower most column: T1-1 (Sand and gravel mat 80cm, drain material)

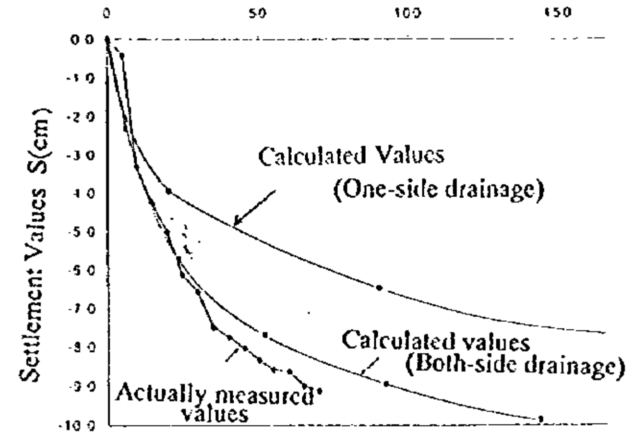
2.4.3 Results of Tests for Preload + Vertical Drain Method

The results of tests for the preload + vertical drain method (in T2) are shown in the following table in comparison with the theoretically calculated values. In the T2-1 ~ 2-2 areas, drains were placed to a depth of 21 m. Consolidation speed was higher than that in the T2-3 ~ 2-4 areas. It was observed that the results are almost same as those of theoretical calculation. Therefore, the effect of this method could be confirmed. Comparison between theoretical calculation and observation results in the T2-3 area is shown in FigureII1-2.4.6. Making stratum ⑦-2 a drainage layer (both-side drainage) results in values close to field observation results. The field observation results showed higher consolidation speed because of some narrow strata.

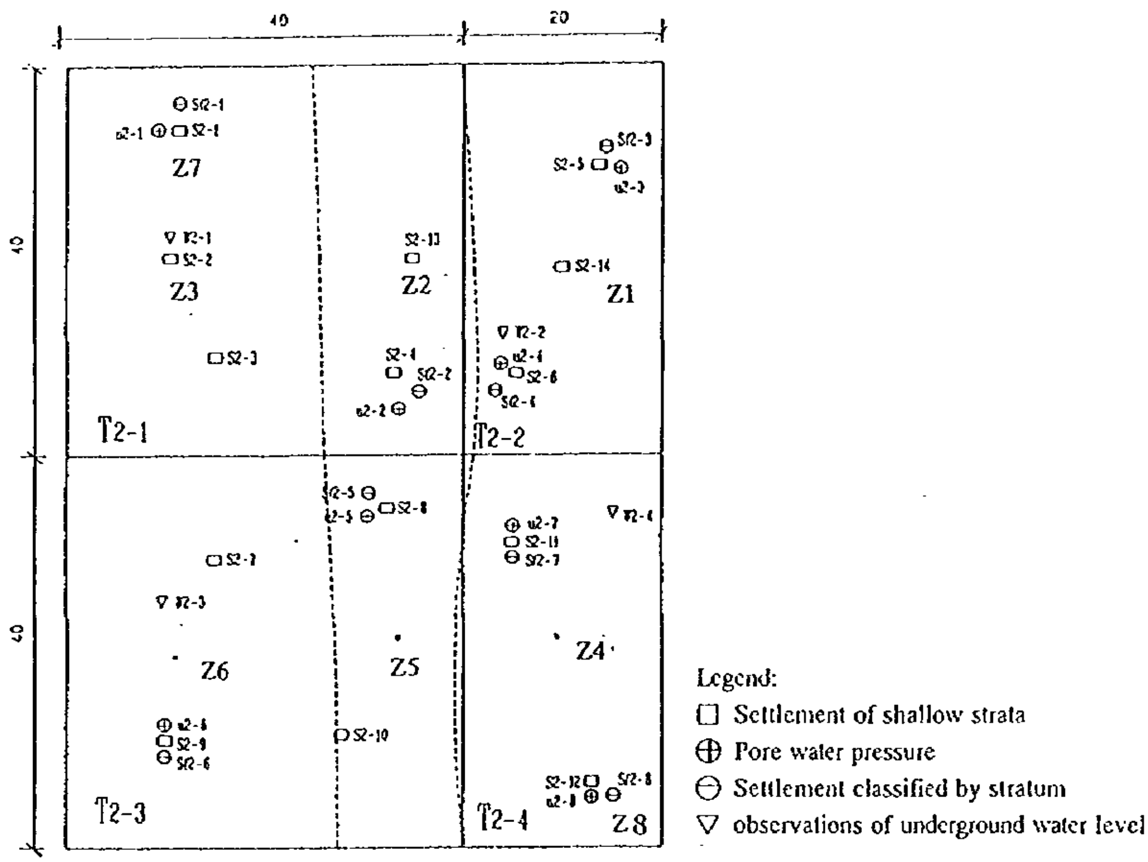
As to settlement values of the channel part, theoretically calculated values are greater. The possible reason is that the coefficient of volume compression has been "Mv = fixed" in theoretical calculation. The channels were digged for waterways, and this portion has already been in the state of excess- consolidation. Therefore, it is interpreted that settlement will rarely occur due to filling up of the channels. Observation positions and the results of surface settlement (settlement of all strata) are shown in FiguresII1-2.4.7 ~ 2.4.8.

TableII1-2.4.3 Results of Observed and Calculated Settlements of T2 Area

Test Area No.	Embankment Height (m)	Interval of Drain (m)	Length of Drain (m)	Settlement under Construction (m)	Settlement during Observation Period (cm)	Theoretically Calculated Settlement (cm)	
T2-1	2.9	1.2	21.0	7.5 ~ 11.2	41.8 ~ 48.4	43.5 ~ 46.7	(63.6)
T2-2	2.9	1.51	21.0	7.3 ~ 8.5	33.6 ~ 36.7	43.5 ~ 46.7	
T2-3	2.0	1.5	6.0	2.1 ~ 3.1	8.3 ~20.3	23.7 ~ 37.7	(49.2)
T2-4	2.0	6.0	6.0	2.8 ~ 12.8	2.8 ~ 12.8	23.7 ~ 37.7	



FigureII1-2.4.6 Comparison between Results of Settlement Values Obtained from Field Observation and Values of Theoretical Calculation in T2-3 Area

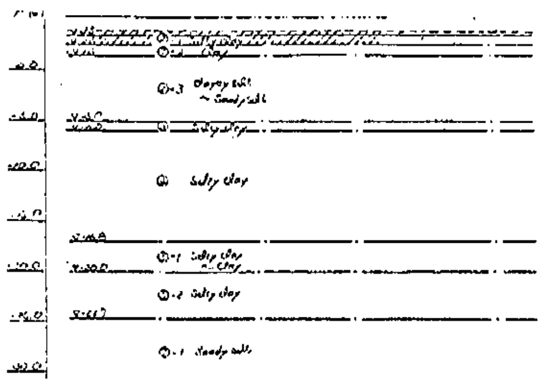
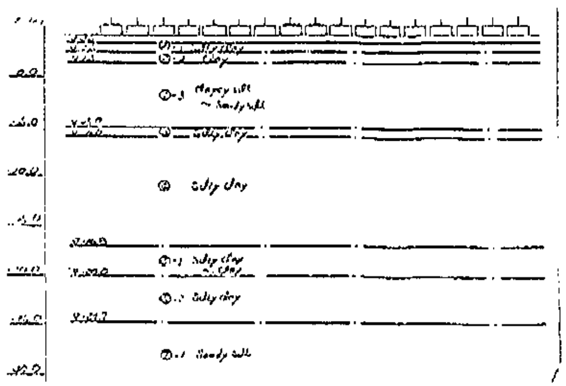
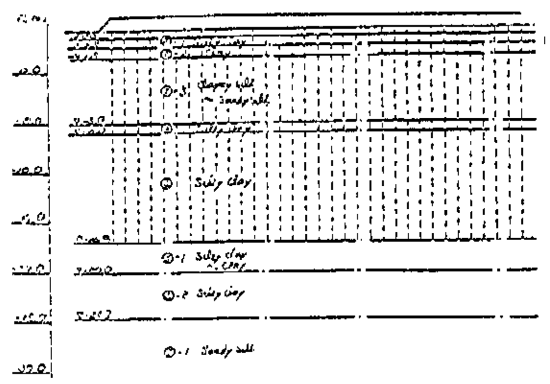


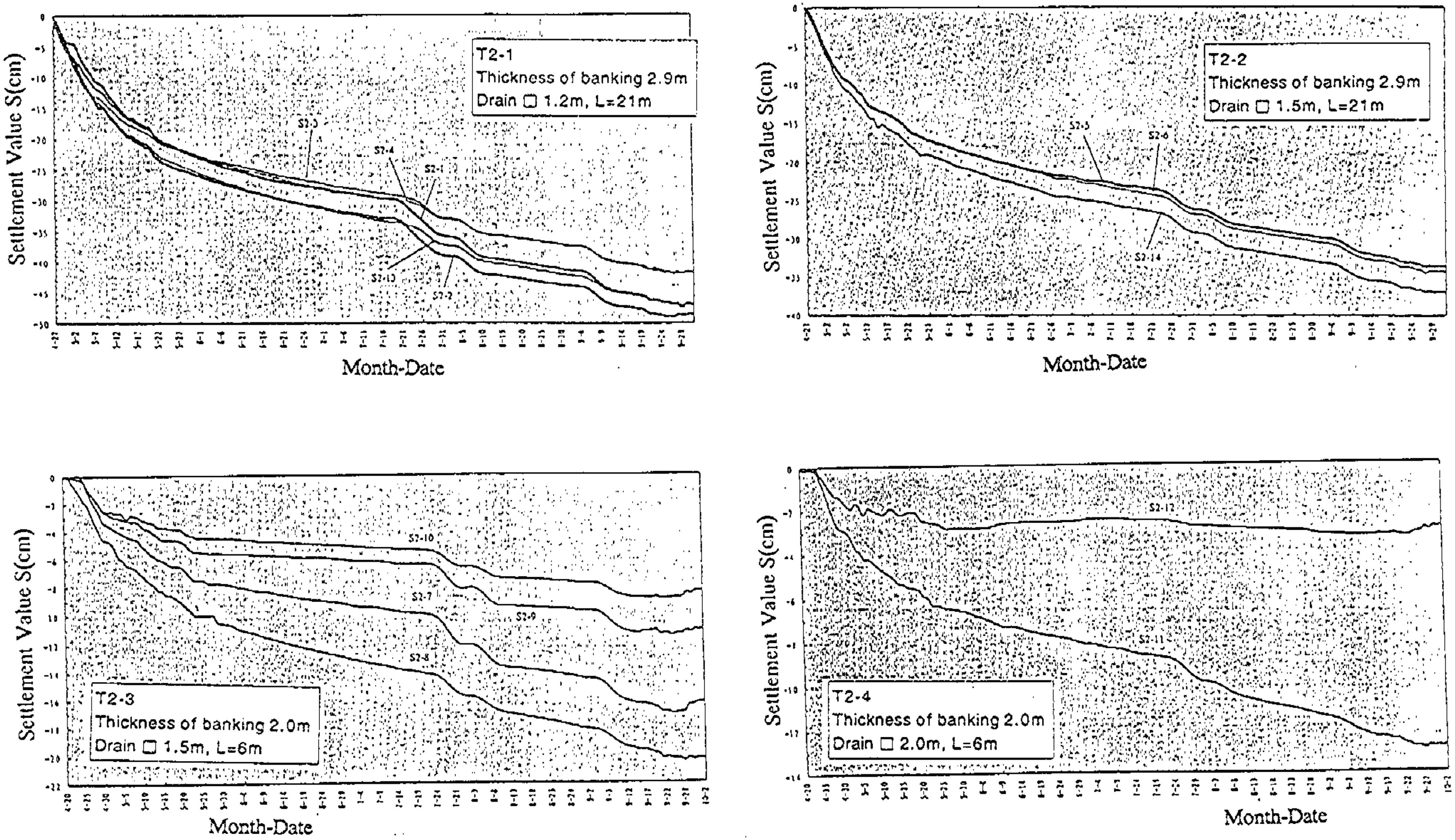
FigureII1-2.4.7 Positions of Various Buried Instruments

2.4.4 Results of Tests of Underground Water Level Reduction Method

According to the test results, it took relatively long time to reduce the underground water level partly because coefficient of permeability is low. The results also showed no sufficient values of settlement and the effectiveness of this method was not recognized.

TableIII-2.5.1 List of Comparison and Examination of Ground Improvement Methods

1) Subgrade Improvement (Shallow Mixing Treatment of Replacement Method)	2) Ram Drop Compaction Method	3) Preload+Vertical Drain Method (+ Subgrade Improvement)
		
Designed to improve strata②-1 and 2. Improvement of the only is to be implemented by means of replacement or with lime or cement mixing method. Final residual settlement values: 9-34cm Absolute settlement values ten years after the airport opening: 4-14cm Uneven settlement values ten years after the airport opening: 0-4cm	Improvement of strata down to Strata②-3,4 (at depth of about 8.0m). Top soil will be removed, mat material (subbed material) placed, and compaction applied with a heavy ram. This will reduce settlement of the above-mentioned strata and increase strength at the same time. Strata②-3,4(sandy silt): Increase in N value: about 9 about11 Strata②-1.2(silty clay): CBR increase: about 3.5% about 4.0% Final residual settlement values: 12-35cm Absolute settlement values 10 years after airport opening: 5-14cm Uneven settlement values 10 years after airport opening: 0-4cm	Improvement of soil down to Stratum④ (at depth of about 20m). Drains are driven down to this stratum. At the same time, preload is applied to increase drainage speed and to promote settlement. Final residual settlement values: a few cm (only settlement due to secondary consolidation) Absolute settlement values ten years after the airport opening: almost 0 Uneven settlement values the years after the airport opening: almost 0
Replacement of ②-1.2 (t=100cm) Drain work Additional filling-up of (t=20cm) Total	Placement of mat material (subbed material) (t= 80cm) Ram drop work (1500-2000KNm, 6.0m square pattern) Additional filling up of (t=about 50cm) Total	Placement of sand mat (t=50) Placement of vertical drains (length=21m, 1.2m square pattern) Preloading(t=2.2m) Improvement, filling up of sub-base (t=about 60cm) Total
* There is little experience with shallow strata mixing treatment methods in China. So it is difficult to obtain machines for construction. Accordingly, there is a strong possibility of employing the replacement method. * The replacement method is same as Plan 2) but requires drain work technique. * The construction period allowed for Phase Ⅰ Area is about one year. It is relatively short.	* Frequently used in China, with many construction examples. Relatively short construction period of about half a year in Phase Ⅰ Area.	* Requires the construction period of nearly 2 years to place vertical drains and to preload.
* This is the most reliable method from viewpoint of improvement. It can realize the uniform ground of good quality. * Formation of the uniform ground of good quality will promote dispersion of load to the lower strata. It will also contribute to reduction of uneven settlement. * Residual settlement value 10 years after airport opening does not meet Chinese installation requirements. But uneven settlement values meet them.	* Strata②-1～4 can be made more uniform. * Increase in strength of Stratum②-3 will bring about effect of dispersion of load to the lower layers. It will contribute to reduction of uneven settlement. * Applying more compaction than necessary will increase settlement during construction. It will lead to more work volume. * There will be little improvement effect on silty clay Strata ②-1 and 2. It will result in necessity of thicker mat layer. * Residual settlement values 10 years after airport opening does not meet Chinese installation requirements. But uneven settlement values meet them.	* In terms of reducing settlement, this method has many examples and is reliable. * Settlement values in this site are low. Doubts remain about its investment effect(settlement can be remedied by maintenance and repair of pavement). * Measures such as reduction of drain pitch, increase of preload etc. will be required to shorten the construction period. These measures may disturb the ground, and may cause large settlement. * There are difficulties in securing the required amount of preload material. (For securing materials dredged from the Yangzhou, it takes time for such works as installation of dredgers and sand-discharge pipes.)
△	○	△



FigureII1-2.4.8 Times and Settlement-Value in Each Area (Results of field observation)

2.5 Soft Ground Improvement Plan

2.5.1 Planning and Designing Conditions

(1) Characteristics of the Ground of this Region

As mentioned so far, characteristics of the ground in this region are summarized as follows:

1) Silty clay ~ clayey silt of Strata②-1 ~ 3 are distributed in the sub-base course (at a depth of about 2 m). They are extremely soft and uneven.

2) In the lower part of the above-mentioned strata, there is about 5-8 m-thick sandy silt of ②-4. The Standards of Shanghai City state that the silt may liquefy. However, from the results of examination by the Japanese side, it has been judged that there is no possibility of liquefaction.

3) Strata ③, ④ and ⑤-12 are under the strata mentioned above. They consist of silty clay ~ clay. All of them contain almost no sand. These layers are thick and show typical consolidation settlement. The final settlement values are 15 - 35 cm, including those of the strata mentioned above. The period required for 90% consolidation degree is estimated to be around 40 years.

(2) Planning and Designing Conditions

On the other hand, technical demands by the Chinese side regarding deformation and strength of the ground are as follows:

1) For Ground Settlement

• Residual settlement values in the common-use period of ten years after airport opening < 10 cm

• Uneven settlement values < 5 cm (radius < 50 m, gradient change 0.1%)

The following three restriction values are to be set up in practice:

• Average consolidation degree of the basic ground > 85%

• Ground settlement speed < 0.1 mm/day

• Residual settlement values expected from the data of field observations < 10 cm

2) For Bearing Capacity of Subgrade Course

• Coefficient of bearing capacity k75>60 mMN/m3(6kgf/cm3), CBR>10%

Judgment standards regarding the soil tests are as follows:

1) Consolidation Degree of the ground (the part other than channel)

Type of Earthwork	Depth(m)	Degree of Consolidation
Banking	0～1	98%
	1～4	95%
	4 or more	93%
Cutting, filling back	0～0.4	98%

2) Ground Strength

• Average N value of standard penetration tests > 8
• Resistance value of static cone penetration tests, qc, > 4Mpa (5 m or less above the ground surface)
• Average degree of consolidation after completion of pre-consolidation > 85% (Surface settlement, settlement classified by stratum and pore water pressure should be observed continuously).
• Variant index of uneven settlement reduction and of physical/mechanical parameters of treated ground, 8 < 3

2.5.2 Comparison and Selection of Ground Improvement Methods

Considering these conditions of strata and technical demands for designing/planning, one of the following methods should be selected for respective purposes:

1) Surface Improvement Plan: Subgrade Improvement (Surface Mixing Treatment or Replacement method):

Settlement of Strata ②-3 and 4 and under them will be accepted. Improvement of the sub-base part only will be aimed at.

2) Shallow Improvement Plan: Ram Drop Compaction Method:

Settlement of deep strata (Strata ④ and ⑤) will be accepted. Improvement of settlement of shallow strata (those down to Strata ②-4) and sub-base improvement will be aimed at.

3) Deep Improvement Plan: Preload + Vertical Drain Method (+ Subgrade Improvement):

Countermeasure against settlement of deep strata (Strata ④ and ⑤) will be aimed at.

Comparison and study of the methods based on the respective purposes and the results of the field tests are as described in Sections 2.3 and 2.4. The results of comparison and study of the above-mentioned three methods are summarized in TableII1-2.5.1.

Main reasons for comparison and selection are as follows:

• Settlement in the site is relatively small and consolidation speed is also small. In addition, the structures on the ground are pavements for which a certain degree of settlement is allowable. Judging from these conditions, it is not necessary to actively employ Plan 3) because this method is costly and also requires long construction period.

• Plan 2) has relatively many construction examples in China and is inexpensive. In addition, the construction period is so short that this method is suitable for construction of this airport in accordance with Chinese demand (China is recently in a hurry for construction of the infrastructure because of the social and economic circumstances).

• One of the standard value determined by the Chinese side is "Residual settlement values in common-use period of ten years < 10 cm". In the case of Plan 2, they are exceeded. However, the settlement is uneven settlement which influences the pavement structure and operation of airplanes. The values of uneven settlement in the site are small and the allowable value of uneven settlement is satisfied.

• Among surface strata treatment methods, in case of mixing treatment method, there is no example of large-sized use of mixing treatment machines for exclusive use in China (possibility of preparing mixing treatment machines for exclusive use is unknown).

• In the replacement method, it can be considered to use sand dredged from the Yanzhou river. However, it requires some construction period to make up dredging facilities. In addition, construction cost will be higher.

• If slag, debris etc. is used in the replacement method of Plan 1), in comparison with the ram drop method of Plan 2), the only difference is whether compaction is done with a rolling compacting machine or with ram drops. In this site with high underground level, replacement will require drain work. Therefore, the ram drop method will cost less.

• In the ram drop method, compaction of strata ②-1 ~ 4 can be expected. So it has an element enabling improvement of unevenness of these strata as well.

As a result, Plan 2) Ram Drop Method should be selected for the ground improvement in this construction.

2.5.3 Area of Improvement by Ram Drop Compaction Method and Specifications of Work

(1) Bases

Examination and the results of field tests of the ram drop compaction method are as described in Section 2.3 and 2.4. The followings were officially decided by Chinese side from the aforementioned results of comparison and selection:

1) Minutes of the Air - Side Ground Improvement Meeting (August 9, 1996)
2) Intermediate data on ground improvement tests for Shanghai Pudong International Airport (July 1996)
3) The report of geological surveys in the Pre - F / S. (Feasible study stage in the paved area of Shanghai Pudong International Airport (February 1996); the report of detailed geological survey in the runway area (August 1996)
4) Related design literature and standards
5) Guidelines by the ground improvement meeting within the Department of Direction for Shanghai Pudong International Airport (August 24 and 26, 1996)

(2) Area of Improvement and Specifications of Work

This method is mainly applied to subgrade improvement as well as to all of soil under pavement. The degree of importance is different between the main body of pavement and the shoulder part (including the over-run part). Therefore, the former is named Area A and the latter is named Area B, then specifications of construction regarding ground improvement (ram drop compaction) work have been decided respectively as follows:

1) Height of subgrade: refer to the figure of height for pavement plans.
2) Inspection indices for mat layers in the ram drop compaction method (management standards)

- Ground bearing capacity of the sub-base (the surface of the mat layer), k75, and CBR values should be as follows:

  Area A : k75 = 80MN/m^3 (8.2kgf/cm^3), allowable range 60 - 120 Mn/m^3 (6.1 - 12.2kgf/cm)

  CBR = 10%, allowable range 8 - 13%

  Area B : k75 = 60MN/m^3 (6.1kgf/cm^3), allowable range 40 - 80 MN/m3) (4.1 - 8.2kgf/cm^3)

- Dry density of compaction of the mat layer > 96% of maximum dry density

3) Inspection indexes of the ground (management standards)

- N values (converted values) of standard penetration tests to the ground located 5 m or more under the bottom of the mat layer and Ps values (converted values) obtained from cone penetration test values:

Area A : $10 \leq$ N value ≤ 25,

$4Mpa \leq Ps \leq 9Mps$ ($41kgf/cm \leq Ps \leq 92kgf/cm2$)

Area B : $8 \leq$ N value ≤ 23,

$3Mpa \leq Ps \leq 8Mps$ ($31kgf/cm \leq Ps \leq 82kgf/cm2$)

- Void ratio (converted values) of the ground located 5 m or more under the bottom of the mat layer

  Area A: $e \leq 0.9$, Area B: $e \leq 0.95$

- Deviation of each test item $8 < 0.3$

- Calculation formula for converted values

$$A = \sum_{i=1}^{n} \frac{Ai \cdot Hi}{H}$$

where,

A: value converted from the measured value

Ai: measured value of the "i"th stratum

H: total thickness of strata converted

Hi: thickness of the "i"th stratum

Note:

Area A: The main body parts of the runway and the parallel/high-speed escape/approach taxiways. Slag will be used as mat material.

Area B: The above-mentioned shoulder and over-run parts. Debris will be used as mat material.

2.5.4 Settlement Values and Sub-Base Bearing Capacity after Improvement

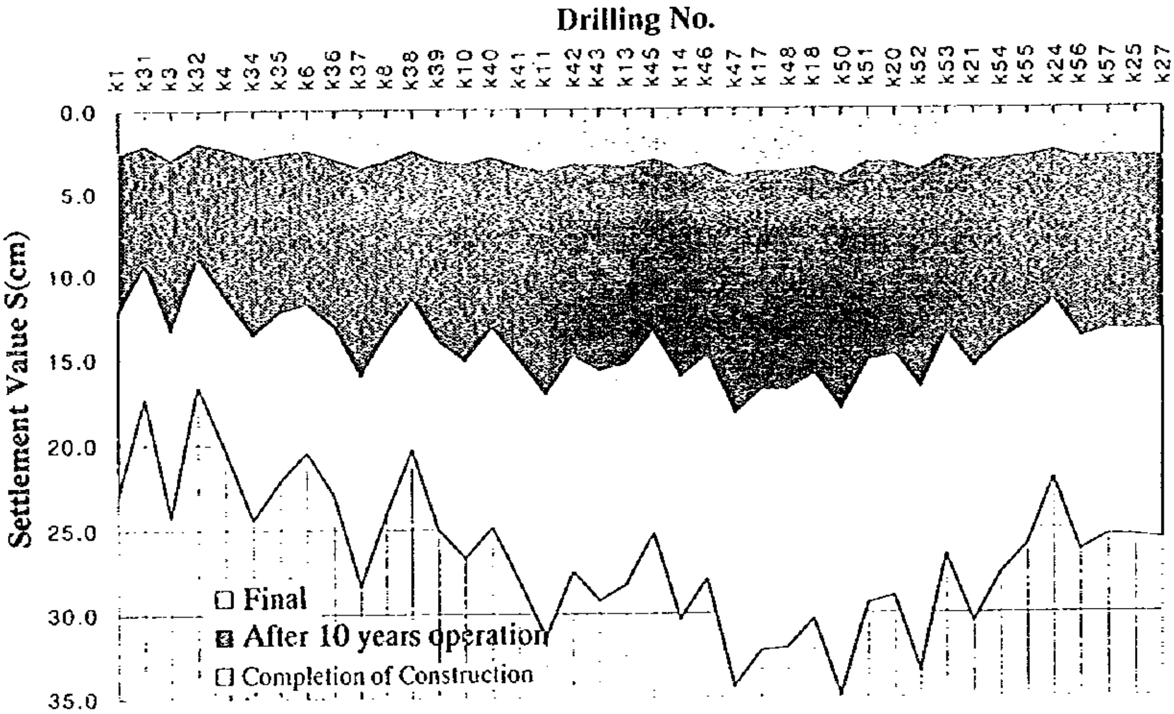
(1) Settlement Values After Improvement

Settlement values in the runway, taxiways and apron after subgrade improvement by the ram drop method are as shown in FiguresII1-2.5.1 ~ 2.5.3. In comparison with the untreated ground, no settlement values of Strata ②-1 ~ 4 will disappear as a result of ram drop compaction. However, the ground will subside by 30 cm due to this compaction. Therefore, it is necessary to apply additional filling-up of 30 cm. In addition, this increase in load will make settlement of Strata ④ and ⑤ larger. Then, the total settlement value will be larger, although slightly, than that of the untreated ground.

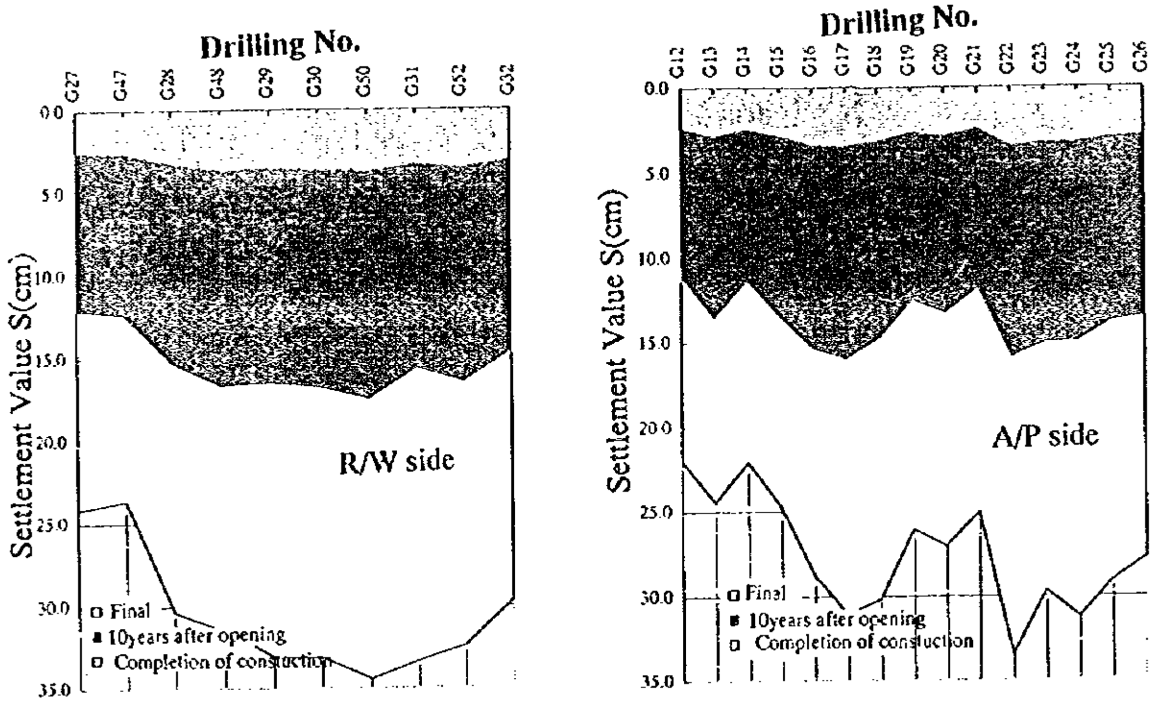
The final settlement values in the runway, the taxiways and apron are 17 ~ 35 cm. Relation between time and settlement values is same as that of the untreated ground. The settlement value of 50% plus will occur 10 years after airport opening. Settlement values (residual values) during this period will be 7 ~ 14 cm. Uneven settlement values among each drilling position (100 - 200 m) will be as low as 0-4 cm. For reference, a reduced amount of approximately 85% will occur 30 years after airport opening. Settlement values (residual settlement values) during this period will be 12-25 cm. Uneven settlement values among each drilling position (100-200 m) is expected to be 0-7 cm.

TableII1-2.5.2 Settlement Value after Ground Improvement with Ram Drop Compaction Method

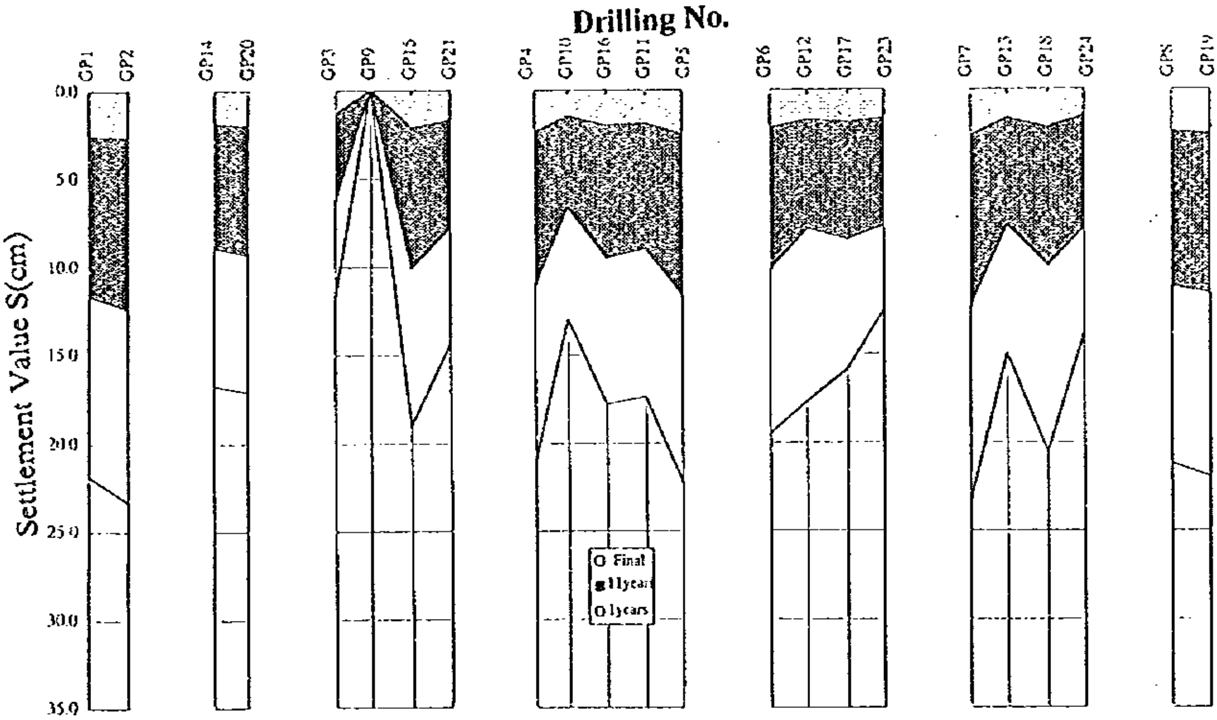
Area	Settlement Value of all Strata(③ ~ ⑤) (cm)				Settlement Value for 10 Years Operation(cm)		Settlement Value for 30 Years Operation(cm)	
	One year	Eleven years	31 years	Final	Absolute	Uneven	Absolute	Uneven
Runway	2.0~4.1 (3.2)	9.0~18.4 (14.1)	13.9~28.7 (22.3)	16.6~34.9 (26.4)	7.0~14.2 (10.9)	0.0~3.4 (1.4)	11.9~24.6 (11.9)	0.0~5.8 (2.5)
Taxiway on the R/W side	2.6~3.8 (3.4)	12.1~17.5 (15.4)	20.1~29.3 (26.0)	23.7~34.5 (30.6)	9.5~13.7 (12.0)	0.2 ~2.1 (0.9)	17.4~25.5 (22.7)	0.1~5.0 (1.4)
Taxiway on the A/P side	2.4~3.5 (3.0)	11.0~15.9 (13.8)	18.6~28.5 (23.4)	21.9~33.5 (27.6)	8.5~12.4 (10.8)	0.1~3.1 (1.2)	16.2~25.0 (20.4)	0.5~6.2 (2.1)
Apron	1.5~2.8 (2.1)	6.7~12.7 (9.6)	10.4~20.5 (15.5)	12.2~24.1 (18.1)	5.2~10.0 (7.5)	0.3~4.2 (1.7)	8.7~17.7 (13.4)	0.2~7.0 (3.0)



FigureII1-2.5.1 Settlement Values after Ground Improvement with Ram Drop Impaction Method (in the runway part)



FigureII1-2.5.2 Settlement Values after Ground Improvement with Ram Drop Compaction (in the Taxiway Part)



FigureII1-2.5.3 Settlement Values after Ground Improvement with Ram Drop Compaction (in the Apron Part)

(2) Bearing capacity of subgrade after improvement

Subgrade bearing capacity after improvement is as described in Section 2.4.2. If sludge / debris is used as mat material, it is considered that the following values aimed at will be fully ensured:

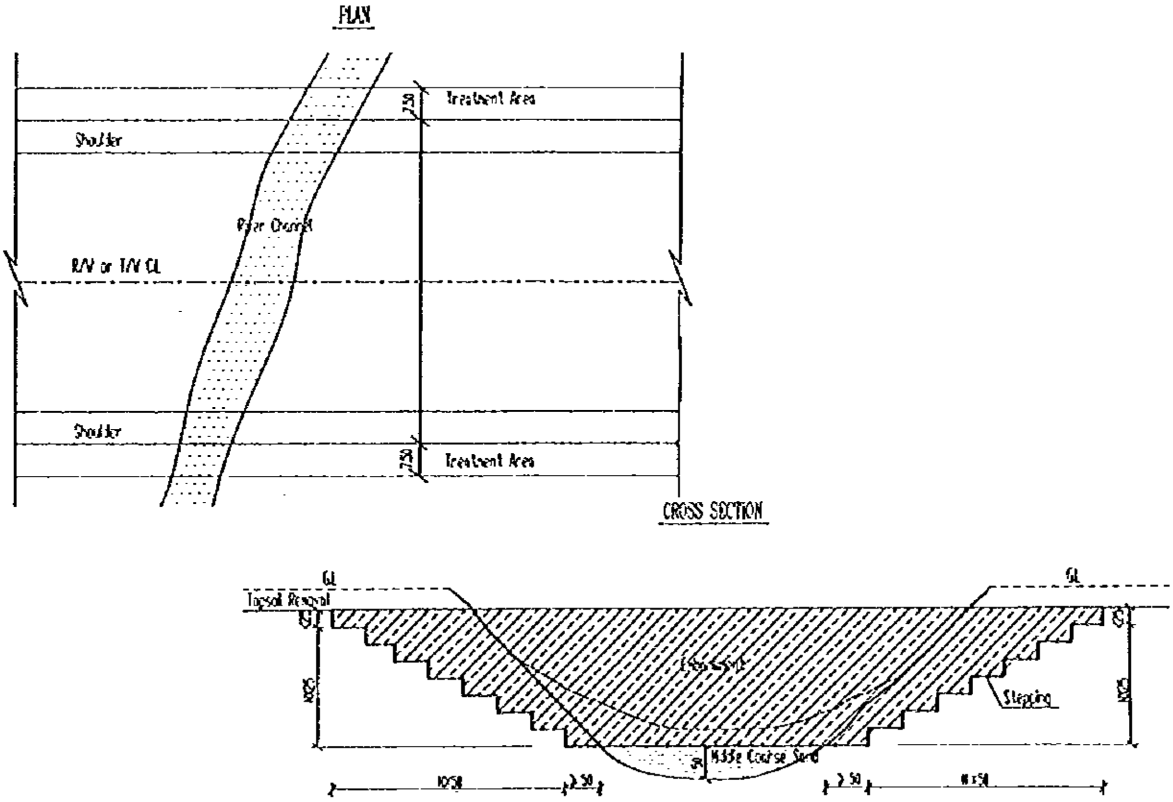
Area A: K_{75} = 80MN/m^3 (8.2kgf/cm^3), allowable range 60 - 120MN/m^3 (6.1 - 12.2kgf/cm^3)
CBR = 10%, allowable range 8 - 13%

Area B: K75 = 60MN/m3 (6.1kgf/cm3), allowable range 40 - 80MN/m3 (4.1 - 8.2kgf/cm3)
CBR = 10%, allowable range 8 - 13%

However, they should be considered as management standards. It is necessary to exercise sufficient management.

2.5.5 Plans for Filling-Up of Channels and Others

As to filling-up of channels (rivers), it is judged that a large amount of settlement of strata ③ ~ ⑤ due to filling-up will not take place. However, sludge accumulating on the river bottom shows a large amount of settlement, consequently uneven settlement between the part of a channel and another part might occur. Therefore, it is necessary to remove the sludge under the pavement of the basic taking-off and landing facilities, in a manner shown below:



FigureII-2.5.4 Treatment of Sludge in the Channel (River) Part

CHAPTER 3 DRAINAGE PLAN

3.1 Basic Policy

The Pudong area is situated in the Yangzhou River delta region and is subject to flooding of the Yangzhou River, seasonal heavy rains, typhoons, and high tides. The highest recorded water level has reached +5.69 m.

The canals around the airport drain into the Chuan Yang river and the Tai Yi river. They are controlled by floodgates built on the confluence with the Yangzhou River.

Normally, the water level in the rivers is regulated to be between 2.5 m ~ 2.8 m. The characteristics of the tide levels beyond the dikes around the airport area are shown in TableII1-3.1.1 .

TableII1-3.1.1 Tide Levels around the Airport Area

(unit : m)

	High Water Level	Low Water Level	Difference
Maximum	+5.69	+2.44	4.65
Minimum	+1.08	-0.52	0.02
Average	+3.24	+0.57	2.67

The airport will encompass a large area and be sited in a flat terrain. Furthermore, factors such as runway pavement and terminal buildings increase the runoff volume. Accordingly the design of the drainage system is an important item.

Proposed designs for the drainage systems in the area including the airport are the Two-Step Pump Station Plan by the Chinese Feasibility Study , the One-Step Pump Station Plan by the Japanese Feasibility Study and an Intermediate Plan. A comparison between the three plans is shown in TableII1-3.1.2 . The two-Step Pump Station Plan is concluded to be the best plan and will be incorporated into the present design.

This plan places the first step pump station within the flight area. This station will be operated by the airport authorities. The second step pump station is placed outside the flight area and will be operated by the regional administrative authorities.

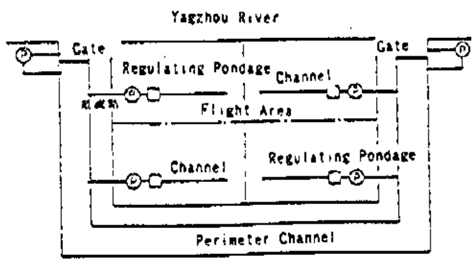
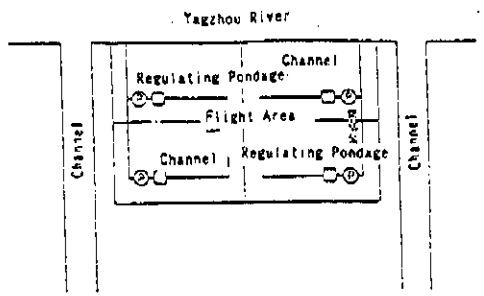
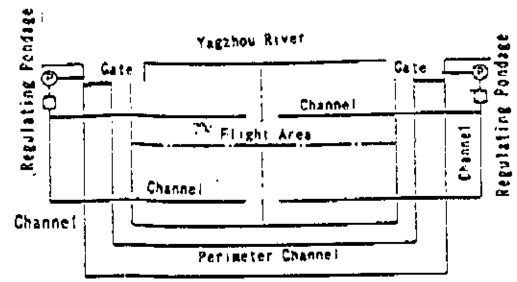
According to the Chinese F/S study, the control water level outside the airport area is set at 2.35 m in relation to the airport site preparation plan and is required not to affect the surrounding agricultural enterprises or the riverine traffic.

The basic policy for the drainage system in the airport area is to construct the first step pump station consisting of an automatic regulatory floodgate, a large pump station, and a regulating pond at the outlet of the flight area. The control level of the regulating pond will be lowered to about +1.0 m to allow gravity outflow of the rainwater from the flight area and permit rapid discharge out of the site .

The position of the outlet facilities should be placed as close as possible to the flight area in order to secure hydraulic gradient sufficient for gravity discharge. Rainwater from areas outside the flight area such as the Terminal Building area and the Related Facilities area will be collected by a separate drainage system which will discharge into the channels surrounding the airport.

The design parameters for discharge facilities in the flight area will be to meet the rainfall intensity with a return period of 5 years and also not to prevent disruption of aircraft operations by standing water with a return period of 50 years.

TableII1-3.1.2 Comparison of Drainage Systems

Item	No. 1 Chinese Feasibility Study	No. 2 Japanese Feasibility Study	No. 3
Cpmceptual Drawing			
Drainage System	• Construct regulating poundage and pump stations dedicated for the flight area. Discharge to the perimeter canal. constrict additional pump stations for the perimeter canal.	• Perimeter canal is not constructed and Sewage is discharged directly to the Yangzhou River from drainage facilities constructed in the airport area.	• Pump station for the airport area are constructed outside the airport area and are combined with pump stations for outlying area.
Merits	• The operation of the pump station reserved for the flight area is relatively easy. • Selection of pumps (capacity, numbers) is easy. • The drains and discharge channels are short and are easy to maintain. Pump head and head loss are low, requiring less power • Number of pump units is large, dispersing damage liability due to pump facilities. • The excavated soil from the perimeter canal can be used for embankment of the airport area.	• This is the most simple system. • Maintenance and operation are easy as the drainage channels are short.	• Pump stations are concentrated in two sites, making maintenance and operation most simple. • Operation costs are cheapest among three alternatives. • Permeter canal excavation soil can be used for airport embankment works.
Demerits	• Maintenance and operation of pump stations are less manageable because there are 6 pump stations.	• Measures are required to prevent outside rainwater from invading the airport premises. • Discharge channels are long, requiring greater power for pumping and maintenance and operation are less easy. • Airport embankment soil cannot be acquired from the surrounding area. Disadvantage for landscaping works.	• The drainage channels are the longest, causing following problems. 1, Syphons are likely to be required for the crossing of the existing canals, maintenance and operation will be a problem. 2, Gravity flow up to the pump stations requires the inlet pipe to be very deep, thus necessitating larger civil structures and also more power for the pumps. • Combination with external areas requires that these areas shall also be designed for a 50-year return period.
Evaluation	Adopted	-	-

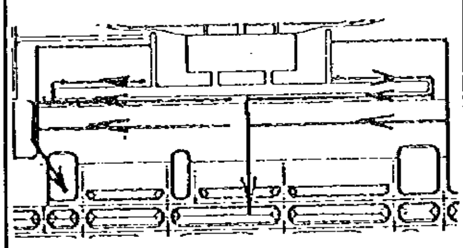
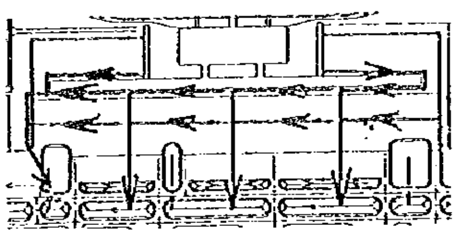
3.2 Design of the Drainage Network

As the airport area is extremely large, the drainage network in the flight area will be divided into two north-south networks at the center of the runway. Drainage channels will be provided in the compound and will conduct the discharge to the regulating ponds at the outlets.

The drainage networks will have main drainage channels set parallel to the runway and parallel taxiway, running north and south. An alternative system concentrating the main drainage channels across the runway and parallel taxiways is possible, but due to the grave situation when the crossing structures are damaged, the economic unviability due to the large dimensions of the main drains, and the difficulties in designing the hydraulic gradient, it has been concluded that the alternative system is not advantageous.

In the design of the apron area, there are two possibilities of conducting the drainage channels from the apron, one which concentrates the crossings, in one spot and the other which disperses the crossings to three points. The designs are compared in FigureII1-3.2.1. The drainage structures under the apron should be kept as small as possible in view of the differential subsidence and because dispersing the structures makes maintenance and inspection easier. Therefore, the dispersed system has been concluded to be appropriate. Furthermore, the routes for the dispersed system channels has been chosen to avoid static load of the parked aircraft as much as possible.

FigureII1-3.2.1 Comparison of Apron Drainage Networks

		Concentrated System	Dispersed System
Basic Configuration			
Length of Drainage Channels	Open Ditch	3943 m	3943 m
	Culvert	572 m	1242 m
	Total	4515 m	5185 m
Section Dimensions	Open Ditch	0.9 X 0.9 ~ 1.7 X 1.7	0.8 X 0.8 ~ 1.4 X 1.4
	Culvert	1.3 X 1.3 ~ 1.9 X 1.9	0.7 X 0.7 ~ 1.6 X 1.6

The catchment area for Phase One area comprises the landscaped area for the runway and its appurtenant facilities. The unlandscaped area to the west of the runway will in the future landscaping plans, drain into the Phase One drains. However, this will be countered by widening the open drains at that time and the area will not be included in the present calculations. Notwithstanding the above, the southern sector drain will receive the rainwater discharge from the future apron area and discharge it into its main drains. This is a covered conduit which will require shutdown of the taxiway for widening, so the future discharge will be included in the calculations of this covered conduit. The future apron has been assumed to have the same dimensions as the Phase One apron in this calculation.

3.3 Drainage Channel Facilities

3.3.1 Design Parameters

Chinese standards were followed in calculating the discharge volume and drainage capacity.

(1) Discharge Volume Calculation

1) Frequency of Occurrence, Rainfall Intensity : I

The frequency of occurrence is set as follows :

- Flight area 5 years
- Other areas 3 years

The rainfall intensity is determined by the following formula :

$$I = \frac{(9.45 + 6.732 \log Te) \times 60}{(t + 5.54)^{0.6514}}$$

where Te : frequency of occurrence (year)

t : duration of rainfall (min)

I : rainfall intensity (mm / hr)

2) Coefficient of Discharge : C

Asphalt pavement	: 0.95
Concrete pavement	: 0.90
Planting	: 0.30
Buildings	: 0.90

3) Discharge Volume (rational method)

$$Q = 1/360 \cdot C \cdot I \cdot A$$

where C : coefficient of discharge

I : rainfall intensity mm/hr)

A : drainage area (ha)

4) Duration of Rainfall : t

$$t = t_1 + t_2$$

$$t_1 = \frac{3.261(1.1 - C)\sqrt{D}}{\sqrt[3]{S}}$$

$$t_2 = L / 60V$$

where t_1 = inflow time (min)

t_2 = time of flow (min)

D : inflow distance (m)

S : gradient (%)

C : coefficient of discharge

L : horizontal length of channel (m)

V : average velocity (m/ s)

(2) Drainage Capacity Calculation

1) Velocity : V (Manning formula)

$$V = \frac{1}{n} \cdot R^{\frac{2}{3}} \cdot i^{\frac{1}{3}}$$

where n : coefficient of roughness

R : hydraulic radius (m)

i : gradient

2) Coefficient of Roughness : n

Concrete channel	: 0.014
Covered conduits (including precast channels)	: 0.014
Masonry channel	: 0.017

3) Drainage Capacity : Q

$$Q = A \cdot V$$

where A : cross section (m^2) ,

V : velocity (m/s)

* open channels and covered conduits are both calculated on a full depth basis; open channels will be provided with a 15 cm freeboard, and masonry channels a 20 cm freeboard.

3.3.2 Calculation of Drainage Sections

The drainage channels in the Flight Area will be U-type open channels, U-type channels with concrete covers (in the runway strip and on routes passing near pavement areas), trapezoidal section channels (outside the runway strip), U-type channels with steel grating (apron area), covered conduits of box section, etc. The rational method will be used to determine discharge volume for each area and the Manning formula will be used to calculate the drainage sections of the drainage channels for the respective discharge volumes. The position of the drainage channel to the west of the runway will be 155m from the centerline of the runway, avoiding the critical area of the glide path.

In this design, the discharge from roofs in the Loading, Cargo and Maintenance Areas and the GSE passage will not be collected by the Flight Area drainage channels. It will be dealt with by a separate drainage network and these areas will not be included in the catchment areas of the drainage system.

The results of calculation of discharge volumes and drainage sections for each area are shown in TablesII1-3.3.1 ~ 3.3.4.

TableII1-3.3.1 Calculation of Discharge Volumes and Drainage Sections (1)

District	Drainage Area (ha)						Composite	CA	Σ C・A	Inflow	Time of Flow		Time of	Rainfall	Runoff	Channel Design					
	Concrete	Asphalt	Planting	Others	Total	Cumulative	Runoff			Time	Length	Time of Flow	Concentration	Intensity	Volume	Channel Shape			Gradient	Velocity	Permissible Volume
NO.	C=0.90	C=0.95	C=0.30		Area(ha)	Area(ha)	Coefficient			t1(min)	(m)	t2(min)	t(min)	(mm/hr)	(m3/s)	Type	B	H	i(%)	(m/s)	(m3/s)
A1	7.73	1.36	38.25		47.34	47.34	0.42	19.72	19.72	30.71	2159	29.69	60.4	55.64	3.05	D3	2.00	1.00	0.08	1.212	3.636
A2-1	1.88	0.02	0.33		2.23	2.23	0.81	1.81	1.81	15.17	135	2.26	17.43	110.58	0.56	B1	1.00	1.00	0.08	1.00	0.97
A2-2	2.80	0.13	3.28		6.21	8.44	0.64	3.63	5.44	17.43	195	2.90	20.33	102.34	1.55	U2	1.20	1.20	0.08	1.12	1.59
A2-3	0.00	0.00	0.00		0.00	8.44	0.64	0.00	5.44	20.33	205	3.05	23.38	95.18	1.44	B2	1.20	1.20	0.08	1.12	1.59
A2-4	4.06	0.13	3.84		8.03	16.47	0.63	4.93	10.37	23.38	195	2.40	25.78	90.36	2.60	U4	1.60	1.60	0.08	1.35	3.44
A2-5	0.00	0.00	0.00		0.00	16.47	0.63	0.00	10.37	25.78	205	2.52	28.30	85.92	2.47	B4	1.60	1.60	0.08	1.35	3.44
A2-6	3.79	0.11	3.92		7.82	24.29	0.62	4.69	15.06	28.30	193	2.20	30.50	82.46	3.45	U5	1.80	1.80	0.08	1.46	4.71
A2-7	0.00	0.00	0.00		0.00	24.29	0.62	0.00	15.06	30.50	161	1.83	32.33	79.85	3.34	B5	1.80	1.80	0.08	1.46	4.71
A2-8	4.11	0.21	6.89		11.21	35.50	0.59	5.97	21.03	32.33	405	4.61	36.94	74.09	4.33	U5	1.80	1.80	0.08	1.46	4.71
A2-9	0.00	0.00	0.00		0.00	35.50	0.59	0.00	21.03	36.94	161	1.83	38.77	72.08	4.21	B5	1.80	1.80	0.08	1.46	4.71
A2-10	2.16	0.07	2.18		4.41	39.91	0.59	2.66	23.69	38.77	59	0.67	39.44	71.38	4.70	U5	1.80	1.80	0.08	1.46	4.71
A2-11	0.00	0.00	0.00		0.00	39.91	0.59	0.00	23.69	39.44	166	1.89	41.33	69.49	4.57	B5	1.80	1.80	0.08	1.46	4.71
A5-1-1	6.41	0.00	0.00		6.41	6.41	0.90	5.77	5.77	9.67	315	4.62	14.29	121.70	1.95	G1	1.40	1.70	0.08	1.14	2.45
A5-1-2	0.00	0.00	0.00		0.00	6.41	0.90	0.00	5.77	14.29	215	2.89	17.18	111.38	1.79	B3	1.40	1.40	0.08	1.24	2.41
A5-1-3	4.87	0.00	1.17		6.04	12.45	0.84	4.73	10.50	17.18	100	1.23	18.41	107.62	3.14	B4	1.60	1.60	0.08	1.35	3.44
A5-2-1	7.37	0.00	0.00		7.37	7.37	0.90	6.63	6.63	7.80	390	5.72	13.52	124.88	2.30	G1	1.40	1.70	0.08	1.14	2.45
A5-2-1	0.00	0.00	0.00		0.00	7.37	0.90	0.00	6.63	13.52	215	2.89	16.41	113.91	2.10	B3	1.40	1.40	0.08	1.24	2.41
A5-2-3	3.94	0.00	0.87		4.81	12.18	0.86	3.81	10.44	16.41	100	1.23	17.64	109.93	3.19	B4	1.60	1.60	0.08	1.35	3.44
A5-3-1	5.04	0.00	0.00		5.04	5.04	0.90	4.54	4.54	7.80	255	3.74	11.54	134.13	1.69	G1	1.40	1.70	0.08	1.14	2.45
A5-4	3.19	0.00	0.00		3.19	3.19	0.90	2.87	2.87	7.80	151	2.22	10.02	142.52	1.14	G1	1.40	1.70	0.08	1.14	2.45
A5-3-2	0.00	0.00	0.00		0.00	8.23	0.90	0.00	7.41	11.54	196	2.41	13.95	123.08	2.53	B4	1.60	1.60	0.08	1.35	3.44
A5-3-3	0.94	0.00	0.64		1.58	9.81	0.86	1.04	8.45	13.95	115	1.42	15.37	117.56	2.76	B4	1.60	1.60	0.08	1.35	3.44
A5-5	7.53	0.12	0.61		8.26	8.26	0.86	7.07	7.07	7.80	385	4.03	11.83	132.66	2.61	G2	1.40	1.70	0.12	1.59	3.00
A5-6	3.90	0.00	0.87		4.77	4.77	0.79	3.77	3.77	7.80	100	1.49	9.29	147.05	1.54	B2	1.20	1.20	0.08	1.12	1.59
discharge from A5-1-3																					
A3-1	2.13	0.00	1.73		3.86	16.31	0.79	2.44	12.94	18.41	235	2.68	21.09	100.43	3.61	U5	1.80	1.80	0.08	1.46	4.71
A3-2	0.00	0.00	0.00		0.00	16.31	0.79	0.00	12.94	21.09	85	0.97	22.06	98.12	3.53	B5	1.80	1.80	0.08	1.46	4.71
A3-3	0.00	0.00	0.00		0.00	16.31	0.79	0.00	12.94	22.06	71	0.81	22.87	96.29	3.46	U5	1.80	1.80	0.08	1.46	4.71
discharge from A5-2-3																					
A3-4	2.10	0.00	1.83		3.93	32.42	0.80	2.44	25.82	22.87	255	2.19	25.06	91.74	6.58	U6	3.80	1.80	0.08	1.94	13.25
discharge from A5-3-3																					
A3-5	0.00	0.00	0.00		0.00	42.23	0.81	0.00	34.27	25.06	7	0.06	25.12	91.62	8.72	U6	3.80	1.80	0.08	1.94	13.25
A3-6	0.00	0.00	0.00		0.00	42.23	0.81	0.00	34.27	25.12	85	0.97	26.09	89.78	8.55	B10	3.60	1.80	0.08	1.46	9.42
A3-7	0.00	0.00	0.00		0.00	42.23	0.81	0.00	34.27	26.09	59	0.51	26.60	88.85	8.46	U6	3.80	1.80	0.08	1.94	13.25
discharge from A5-6																					
A3-8	1.76	0.00	1.17		2.93	49.93	0.80	1.94	39.98	26.60	172	1.47	28.07	86.30	9.58	U6	3.80	1.80	0.08	1.94	13.25
A3-9	0.00	0.00	0.00		0.00	49.93	0.80	0.00	39.98	28.07	85	0.97	29.04	84.72	9.41	B10	3.60	1.80	0.08	1.46	9.42
A3-10	0.00	0.00	0.00		0.00	49.93	0.80	0.00	39.98	29.04	29	0.25	29.29	84.32	9.36	U6	3.80	1.80	0.08	1.94	13.25

TableIII-3.3.2 Calculations of Discharge Volumes and Drainage Sections (2)

District NO.	Drainage Area (ha) Concrete C=0.90	Asphalt C=0.95	Planting C=0.30	Others	Total Area(ha)	Cumulative Area(ha)	Composite Runoff Coefficient	CA	Σ C · A	Inflow Time t1(min)	Time of Flow Length (m)	Time of Flow t2(min)	Time of Concentration t(min)	Rainfall Intensity (mm/hr)	Runoff Volume (m3/s)	Channel Design: Channel Shape Type	B	H	Gradient i(%)	Velocity (m/s)	Permissible Volume (m3/s)
A6-1	5.01	0.00	1.02		6.03	6.03	0.8	4.82	4.82	9.11	727	10.8	19.91	103.44	1.38	U2	1.2	1.2	0.08	1.122	1.591
discharge from A5-5																					
A6-2	0.00	0.00	0.00		0.00	14.29	0.83	0.00	11.89	19.91	85	1.05	20.96	100.75	3.33	U4	1.60	1.60	0.08	1.35	3.44
A6-3	0.00	0.00	0.00		0.00	14.29	0.83	0.00	11.89	20.96	114	1.40	22.36	97.43	3.22	B4	1.60	1.60	0.08	1.35	3.44
A6-4	1.04	0.00	1.32		2.36	16.65	0.79	1.33	13.22	22.36	171	1.95	24.31	93.23	3.42	B5	1.80	1.80	0.08	1.46	4.71
A7-1	5.34	0.00	0.00		5.34	5.34	0.90	4.81	4.81	7.80	366	5.37	13.17	126.39	1.69	G1	1.40	1.70	0.08	1.14	2.45
A7-2	0.00	0.00	0.00		0.00	5.34	0.90	0.00	4.81	13.17	100	1.34	14.51	120.83	1.61	B3	1.40	1.40	0.08	1.24	2.41
A7-3	4.01	0.13	0.78		4.92	4.92	0.81	3.97	3.97	7.80	318	4.67	12.47	129.57	1.43	G1	1.40	1.70	0.08	1.14	2.45
A7-4	0.00	0.00	0.00		0.00	4.92	0.81	0.00	3.97	12.47	139	1.87	14.34	121.50	1.34	U3	1.40	1.40	0.08	1.24	2.41
discharge from A6-4																					
A3-11	0.89	0.00	0.32		1.21	67.79	0.80	0.90	54.10	29.04	29	0.25	29.29	84.32	12.67	U6	3.80	1.80	0.08	1.94	13.25
A3-12	0.00	0.00	0.00		0.00	67.79	0.80	0.00	54.10	29.29	85	1.03	30.32	82.73	12.43	B9	4.80	2.00	0.08	1.38	13.08
A3-13	1.80	0.00	1.85		3.65	71.44	0.79	2.18	56.28	30.32	338	2.70	33.02	78.91	12.34	U7	4.25	2.00	0.08	2.09	17.69
discharge from A7-2																					
A3-14	0.00	0.00	0.00		0.00	76.78	0.80	0.00	61.09	33.02	85	1.03	34.05	77.57	13.08	B9	4.80	2.00	0.08	1.38	13.08
A3-15	1.01	0.00	0.74		1.75	78.53	0.79	1.13	62.22	34.05	135	1.08	35.13	76.22	13.17	U7	4.25	2.00	0.08	2.09	17.69
A3-16	0.00	0.00	0.00		0.00	78.53	0.79	0.00	62.22	35.13	96	1.16	36.29	74.84	12.93	B9	4.80	2.00	0.08	1.38	13.08
discharge from A3-16																					
A2-12	0.00	0.00	0.00		0.00	118.44	0.73	0.00	85.91	36.29	11	0.10	41.33	69.49	16.58	D7	3.00	2.00	0.08	1.83	18.32
discharge from A7-4																					
A2-13	0.75	0.34	8.53		9.62	132.98	0.70	3.56	93.44	41.88	350	3.19	44.52	66.57	17.28	D7	3.00	2.00	0.08	1.83	18.32
A8	0.00	0.24	16.33		16.57	16.57	0.31	5.13	5.13	0.00	910	14.06	14.06	122.63	1.75	D1	1.00	1.00	0.08	1.08	2.16
A9	0.00	0.27	18.72		18.99	18.99	0.31	5.87	5.87	0.00	910	14.06	14.06	122.63	2.00	D1	1.00	1.00	0.08	1.08	2.16
discharge from A8																					
A2-14	0.00	0.00	0.00		0.00	149.55	0.66	0.00	98.57	44.52	457	4.16	48.68	63.20	17.30	D7	3.00	2.00	0.08	1.83	18.32
discharge from A9																					
A2-15	0.00	0.28	6.50		6.78	175.32	0.61	2.22	106.66	48.68	275	2.50	51.18	61.37	18.18	D7	3.00	2.00	0.08	1.83	18.32

TableII-3.3.3 Calculations of Discharge Volumes and Drainage Sections (3)

District	Drainage Area (ha)						Composite	CA	Σ C・A	Inflow	Time of Flow		Time of	Rainfall	Runoff	Channel Design					
	Concrete	Asphalt	Planting	Others	Total	Cumulative	Runoff			Time	Length	Time of Flow	Concentration	Intensity	Volume	Channel Shape			Gradient	Velocity	Permissible Volume
NO.	C=0.90	C=0.95	C=0.30		Area(ha)	Area(ha)	Coefficient			t1(min)	(m)	t2(min)	t(min)	(mm/hr)	(m3/s)	Type	B	H	i(%)	(m/s)	(m3/s)
B1	7.73	1.60	55.32		64.65	64.65	0.39	25.07	25.07	30.71	2855	39.26	69.97	50.94	3.55	D3	2.00	1.00	0.08	1.21	3.64
B2-1	1.21	0.00	0.09		1.30	1.30	0.86	1.12	1.12	13.33	137	2.29	15.62	116.66	0.36	B1	1.00	1.00	0.08	1.00	0.97
B2-2	3.09	0.17	3.23		6.49	7.79	0.65	3.91	5.03	15.62	195	2.90	18.52	107.29	1.50	U2	1.20	1.20	0.08	1.12	1.59
B2-3	0.00	0.00	0.00		0.00	7.79	0.65	0.00	5.03	18.52	205	3.05	21.57	99.27	1.39	B2	1.20	1.20	0.08	1.12	1.59
B2-4	3.75	0.13	3.84		7.72	15.51	0.62	4.65	9.68	21.57	195	2.40	23.97	93.93	2.53	U4	1.60	1.60	0.08	1.35	3.44
B2-5	0.00	0.00	0.00		0.00	15.51	0.62	0.00	9.68	23.97	205	2.52	26.49	89.05	2.39	B4	1.60	1.60	0.08	1.35	3.44
B2-6	4.01	0.11	3.92		8.04	23.55	0.62	4.89	14.57	26.49	193	2.20	28.69	85.28	3.45	U5	1.80	1.80	0.08	1.46	4.71
B2-7	0.00	0.00	0.00		0.00	23.55	0.62	0.00	14.57	28.69	161	1.83	30.52	82.43	3.34	B5	1.80	1.80	0.08	1.46	4.71
B2-8	4.21	0.21	6.85		11.27	34.82	0.59	6.04	20.61	30.52	405	4.61	35.13	76.22	4.36	U5	1.80	1.80	0.08	1.46	4.71
B2-9	0.00	0.00	0.00		0.00	34.82	0.59	0.00	20.61	35.13	161	1.83	36.96	74.07	4.24	B5	1.80	1.80	0.08	1.46	4.71
B2-10	2.07	0.07	2.05		4.19	39.01	0.59	2.54	23.15	36.96	59	0.67	37.63	73.32	4.71	U5	1.80	1.80	0.08	1.46	4.71
B2-11	0.00	0.00	0.00		0.00	39.01	0.59	0.00	23.15	37.63	223	2.54	40.17	70.64	4.54	B5	1.80	1.80	0.08	1.46	4.71
B5-1	7.31	0.08	3.49		10.88	10.88	0.71	7.70	7.70	9.11	950	12.76	21.87	98.56	2.11	U3	1.40	1.40	0.08	1.24	2.41
B5-2	0.00	0.00	0.00		0.00	10.88	0.71	0.00	7.70	21.87	96	1.29	23.16	95.65	2.05	B3	1.40	1.40	0.08	1.24	2.41
B4-1	9.86	1.38	11.00		22.24	22.24	0.61	13.49	13.49	15.00	96	1.09	16.09	115.00	4.31	B5	1.80	1.80	0.08	1.46	4.71
B4-2	9.45	0.00	1.18		10.63	10.63	0.83	8.86	8.86	15.00	96	1.09	16.09	115.00	2.83	B5	1.80	1.80	0.08	1.46	4.71
B4-3	11.57	0.00	1.64		13.21	13.21	0.83	10.91	10.91	15.00	96	1.09	16.09	115.00	3.49	B5	1.80	1.80	0.08	1.46	4.71
B4-4	11.64	0.00	1.56		13.20	13.20	0.83	10.94	10.94	15.00	96	1.09	16.09	115.00	3.49	B5	1.80	1.80	0.08	1.46	4.71
B4-5	15.62	0.39	10.39		26.40	26.40	0.66	17.55	17.55	15.00	144	1.74	16.74	112.80	5.50	B7	2.40	2.00	0.08	1.38	6.54

TableII1-3.3.4 Calculation for discharge volumes and drainage sections (4)

District	Drainage Area (ha)						Composite			Inflow	Time of Flow		Time of	Rainfall	Runoff	Channel Design					
	Concrete	Asphalt	Planting	Others	Total	Cumulative	Runoff	CA	Σ C · A	Time	Length	Time of flow	Concentration	Intensity	Volume	Channel Shape			Gradient	Velocity	Permissible Volume
NO.	C=0.90	C=0.95	C=0.30		Area(ha)	Area(ha)	Coefficient			t1(min)	(m)	t2(min)	t(min)	(mm/hr)	(m3/s)	Type	B	H	i(%)	(m/s)	(m3/s)
B3-1	1.15	0.00	2.02		3.17	3.17	0.52	1.64	1.64	22.13	153	2.56	24.69	92.47	0.42	B1	1.00	1.00	0.08	1.00	0.97
B3-2	1.35	0.00	0.76		2.11	5.28	0.58	1.44	3.08	24.69	56	0.94	25.63	90.64	0.78	U1	1.00	1.00	0.08	1.00	0.97
B3-3	0.00	0.00	0.00		0.00	5.28	0.58	0.00	3.08	25.63	85	1.42	27.05	88.05	0.75	B1	1.00	1.00	0.08	1.00	0.97
B3-4	0.00	0.00	0.00		0.00	5.28	0.58	0.00	3.08	27.05	9	0.15	27.20	87.79	0.75	U1	1.00	1.00	0.08	1.00	0.97
discharge from B5-2																					
B3-5	2.16	0.02	1.62		3.80	19.96	0.66	2.45	13.23	27.20	280	3.45	30.65	82.24	3.02	U4	1.60	1.60	0.08	1.35	3.44
discharge from B4-1																					
B3-6	0.00	0.00	0.00		0.00	42.20	0.63	0.00	26.72	30.65	11	0.09	30.74	82.11	6.09	U6	3.80	1.80	0.08	1.94	13.25
B3-7	0.00	0.00	0.00		0.00	42.20	0.63	0.00	26.72	30.74	85	0.30	31.04	81.67	6.06	B10	3.60	1.80	0.08	4.71	9.42
B3-8	1.99	0.02	1.54		3.55	45.75	0.63	2.27	28.99	31.04	254	2.18	33.22	78.65	6.33	U6	3.80	1.80	0.08	1.94	13.25
discharge from B4-2																					
B3-9	0.00	0.00	0.00		0.00	56.38	0.67	0.00	37.85	33.22	15	0.13	33.35	78.48	8.25	U6	3.80	1.80	0.08	1.94	13.25
B3-10	0.00	0.00	0.00		0.00	56.38	0.67	0.00	37.85	33.35	85	0.30	33.65	78.08	8.21	B10	3.60	1.80	0.08	4.71	9.42
B3-11	1.88	2.02	1.41		5.31	61.69	0.68	4.03	41.88	33.65	248	2.13	35.78	75.44	8.78	U6	3.80	1.80	0.08	1.94	13.25
B3-12	0.00	0.00	0.00		0.00	61.69	0.68	0.00	41.88	35.78	85	0.30	36.08	75.08	8.73	B10	3.60	1.80	0.08	4.71	9.42
B3-13	0.00	0.00	0.00		0.00	61.69	0.68	0.00	41.88	36.08	18	0.15	36.23	74.91	8.71	U6	3.80	1.80	0.08	1.94	13.25
discharge from B4-3																					
B3-14	3.36	0.03	2.52		5.91	80.81	0.70	3.81	56.60	36.23	400	3.43	39.66	71.15	11.19	U6	3.80	1.80	0.08	1.94	13.25
discharge from B4-4																					
B3-15	0.00	0.00	0.00		0.00	94.01	0.72	0.00	67.54	39.66	63	0.54	40.20	70.61	13.25	U6	3.80	1.80	0.08	1.94	13.25
B3-16	0.00	0.00	0.00		0.00	94.01	0.72	0.00	67.54	40.20	85	1.03	41.23	69.59	13.06	B9	4.80	2.00	0.08	1.38	13.08
B3-17	1.44	0.00	0.74		2.18	96.19	0.72	1.52	69.06	41.23	135	1.08	42.31	68.56	13.15	U7	4.25	2.00	0.08	2.09	17.69
B3-18	0.00	0.00	0.00		0.00	96.19	0.72	0.00	69.06	42.31	162	1.96	44.27	66.79	12.81	B9	4.80	2.00	0.08	1.38	13.08
B6-1-1	9.72	0.24	1.01		10.97	10.97	0.85	9.28	9.28	11.39	312	3.46	14.85	119.51	3.08	G2	1.40	1.70	0.14	1.50	3.24
B6-1-2	0.00	0.00	0.00		0.00	10.97	0.85	0.00	9.28	14.85	242	2.98	17.83	109.35	2.82	B4	1.60	1.60	0.08	1.35	3.44
B6-2-1	10.35	0.00	0.00		10.35	10.35	0.90	9.32	9.32	11.39	300	3.33	14.72	120.01	3.11	G2	1.40	1.70	0.14	1.50	3.24
B6-2-2	0.00	0.00	0.00		0.00	10.35	0.90	0.00	9.32	14.72	242	2.98	17.70	109.75	2.84	B4	1.60	1.60	0.08	1.35	3.44
B6-3-1	10.42	0.19	0.93		11.54	11.54	0.85	9.84	9.84	11.39	333	3.69	15.08	118.64	3.24	G2	1.40	1.70	0.14	1.50	3.24
B6-3-2	0.00	0.00	0.00		0.00	11.54	0.85	0.00	9.84	15.08	242	2.98	18.06	108.65	2.97	B4	1.60	1.60	0.08	1.35	3.44
discharge from B3-2																					
B7	0.66	0.12	10.92		11.70	23.24	0.59	3.98	13.82	18.06	288	3.24	21.30	99.92	3.84	D4	2.00	1.50	0.08	1.48	7.78
discharge from B3-18 & B2-11																					
B2-12	0.00	0.00	0.00		0.00	135.42	0.68	0.00	92.33	44.27	20	0.18	44.45	66.64	17.09	D7	3.00	2.00	0.08	1.83	18.32
discharge from B4-5																					
B2-13	1.95	0.12	12.33		14.40	176.22	0.66	5.57	115.45	44.45	350	3.10	47.55	64.07	20.55	D8	3.50	2.00	0.08	1.88	20.68
discharge from B6-1-2																					
B2-14	0.66	0.12	8.22		9.00	196.19	0.65	3.17	127.90	47.55	300	2.60	50.15	62.11	22.07	D9	4.00	2.00	0.08	1.92	23.08
discharge from B6-2-2																					
B2-15	0.10	0.00	1.25		1.35	207.89	0.66	0.47	137.69	50.15	45	0.38	50.53	61.83	23.65	D10	4.50	2.00	0.08	1.96	25.49
discharge from B7																					
B2-16	0.00	0.00	0.00		0.00	231.13	0.66	0.00	151.51	50.53	460	3.91	54.44	59.18	24.91	D10	4.50	2.00	0.08	1.96	25.49

3.4 Regulating Reservoir and Pumping Facilities

3.4.1 Basic Policy

The pumping capacity of the drainage pumps and the dimensions of the regulating reservoir will be determined according to the following basic policy.

•The regulating reservoir shall have sufficient capacity to temporarily hold, safely and without standing water in the airport area, a part of the discharge volume from rainfall with a 5-year return period. The duration of rainfall shall be 24 hours with a latterly concentrated pattern.

• Due to the freeboard of the drainage facilities, the inflow volume may exceed the design volume. The volume of the regulating reservoir shall be sufficient to insure safety in case of rainfall with a return period of 50-years (standing water will be allowed).

• The drainage pumps will have sufficient capacity to discharge within 24 hours the standing water from a 50-year-frequency rainfall .

• The standing water volume shall be the total of holding capacity in drainage channels + permissible standing water volume of low lying areas in the runway area.

• Higher values of pump capacity and regulating reservoir volume obtained from calculations based on 5-year and 50-year-frequency rainfall shall be utilized to be on the safe side.

3.4.2 Design Conditions

(1) Design Drainage Basin A and Coefficient of Discharge C

The northern drainage basin is designated as Area A and the southern drainage basin as Area B. The design drainage basins and coefficient of discharge for the drainage networks are shown in TableII1-3.4.1. The is used weighted average coefficient of discharge for 5-year - frequency rainfall is used. However, the value of 0.90 will be used in the 50-year return frequency design.

TableII-3.4.1 Drainage Basin and Coefficient of Discharge

		Area A	Area B
Drainage Basin A		223 ha	222 ha
Coefficient of Discharge	5-year return	0.57	0.58
	50-year return	0.90	0.90

(2) Rainfall Intensity I

• Rainfall Intensity for 5-year-frequency Rainfall

$$I = \frac{(9.45 + 6.7932 \log Te) \times 60}{(t + 5.54)^{0.6514}}$$

where Te : return frequency of rainfall (year)

t : duration of rainfall (min)

I : rainfall intensity (mm/hr)

• Rainfall Intensity for 50-year-frequency Rainfall

$$I = \frac{107.4}{T^{0.724}}$$

where T : duration of rainfall (hr)

I : rainfall intensity (mm/hr)

(3) Discharge Volume Q

$$Q = \frac{1}{360} C \cdot I \cdot A$$

where C : coefficient of discharge

I : rainfall intensity (mm/hr)

A : drainage basin (ha)

3.4.3 Drainage Pump Capacity and Regulating Reservoir Volume

The Drainage Pump Capacity and the Regulating Reservoir Volume are mutually balancing. The volume for the Regulating Reservoir required for each drainage pump capacity design shall be calculated and the best balance for the two facilities will be determined. The formula used in the calculations is as follows:

(1) for 5-year-frequency Rainfall Intensity

$V_1 = \sum V - (Q \times 60 \times \Delta t + V_2)$

where V1 : volume of regulating reservoir (m^3)

Q : total drainage pump capacity (m^3/s)

$\sum V$: total rainfall at the time the drainage capacity of the drainage pumps is exceeded.

Δ^t : duration of rainfall after the drainage capacity of the pumps has been exceeded.

V_2 : drainage channel capacity (m^3)

(2) for 50-Year-Frequency Rainfall Intensity

$V1 = \sum V - (Q \times 60 \times \Delta t + V4)$

where V_4 : standing water volume (m^3)

[$V_4 = V_2$ (drainage channel capacity) $+V_3$ (runway strip capacity)]

V_3 : Area A 195,000cubic meters

Area B 201,000 cubic meters

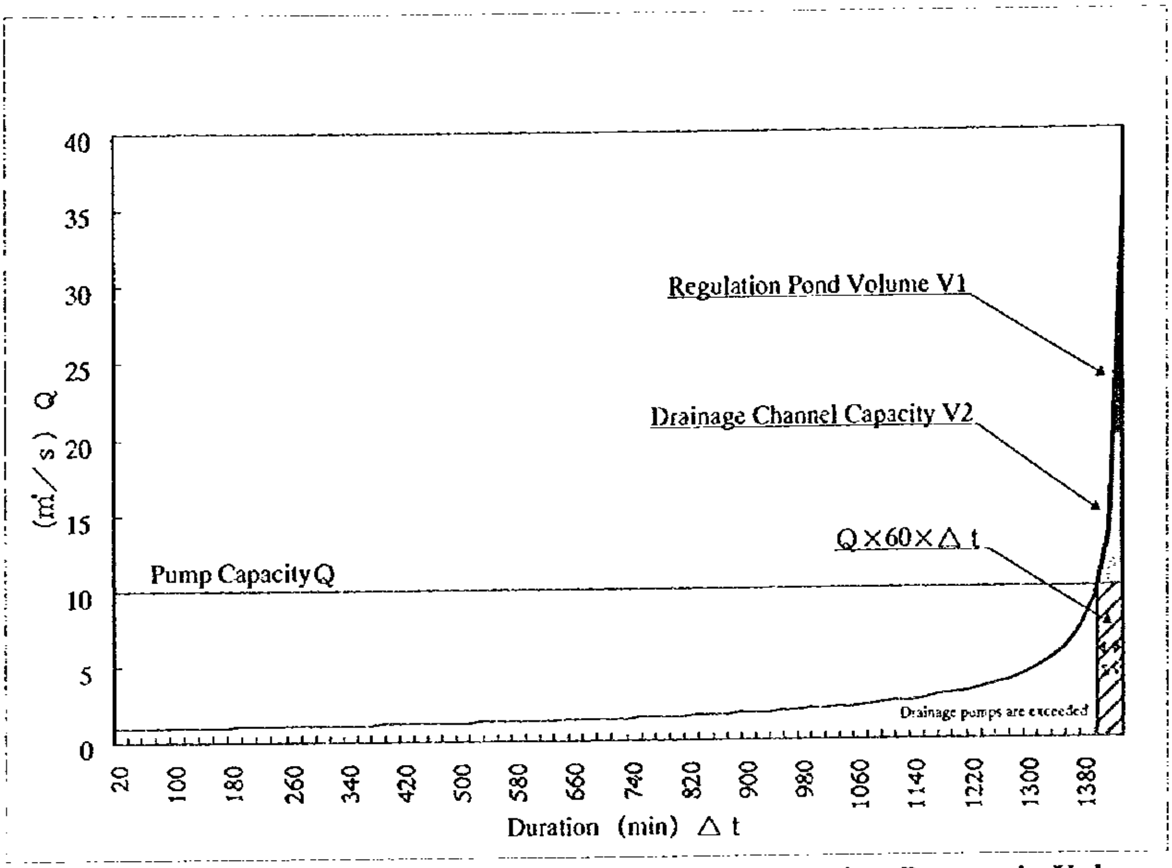
(3) Determination of Capacity

From the above, the relation between pump capacity and regulating reservoir volume can be calculated. The results are shown in FiguresII1-3.4.1 ~ 3.4.4. In this study, the cost for the drainage pumps is higher than the construction cost for the regulating reservoirs. Therefore, enlarging the regulating reservoir will improve the economic aspect. However, the groundwater level in the airport area is high. It is considered likely that groundwater will constantly leak into the regulating reservoir. Accordingly, the controlled water level has been set at about 2.0 m.

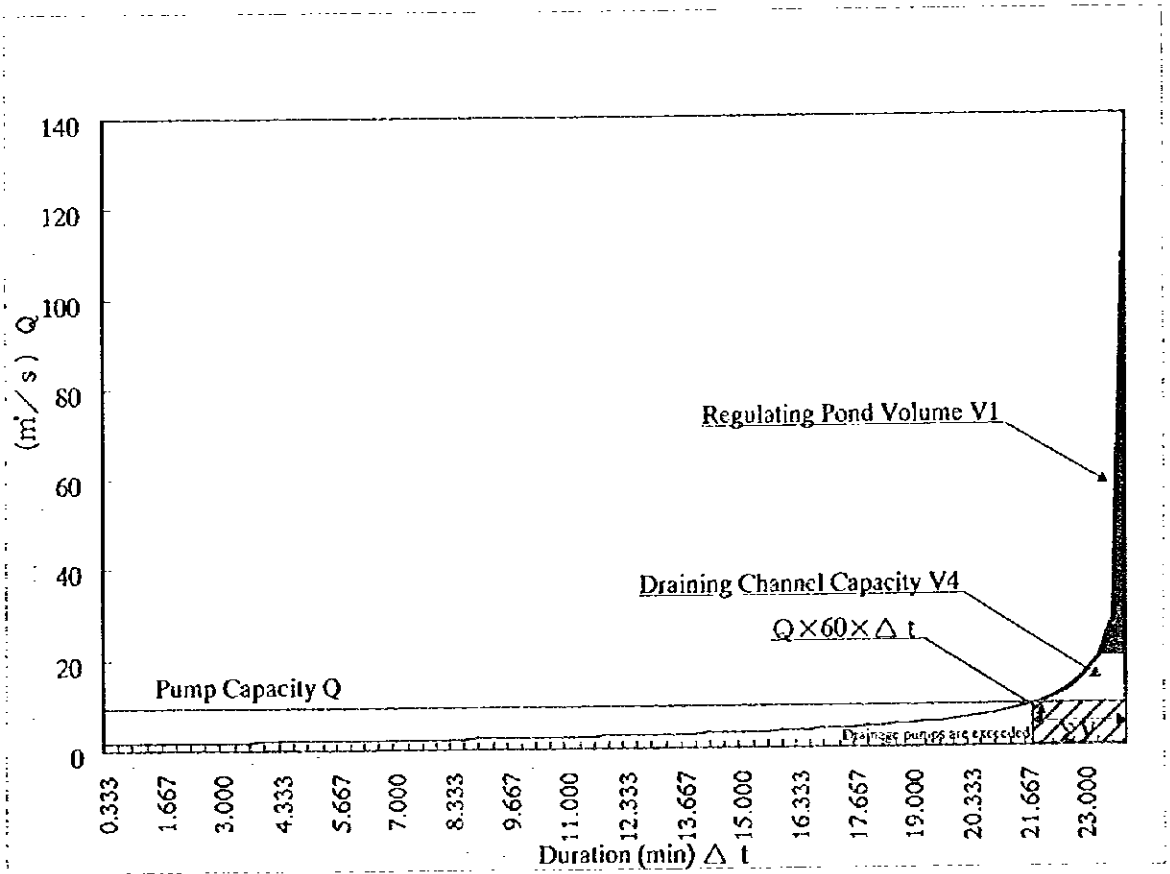
It is a common Chinese practice to allow about 20~ 30 % extra for the drainage pump capacity. In the present study, holding capacity in the drainage channels has not been included in the calculations. They are regarded as extra capacity.

From the above study, if the design depth of the regulating reservoir are set at about 2.0m, the drainage pump capacity of 10 m^3/s is found to be appropriate as shown in TablesII1-3.4.2 and 3.4.3. The capacity for the regulating reservoirs and drainage pumps can be set as follows.

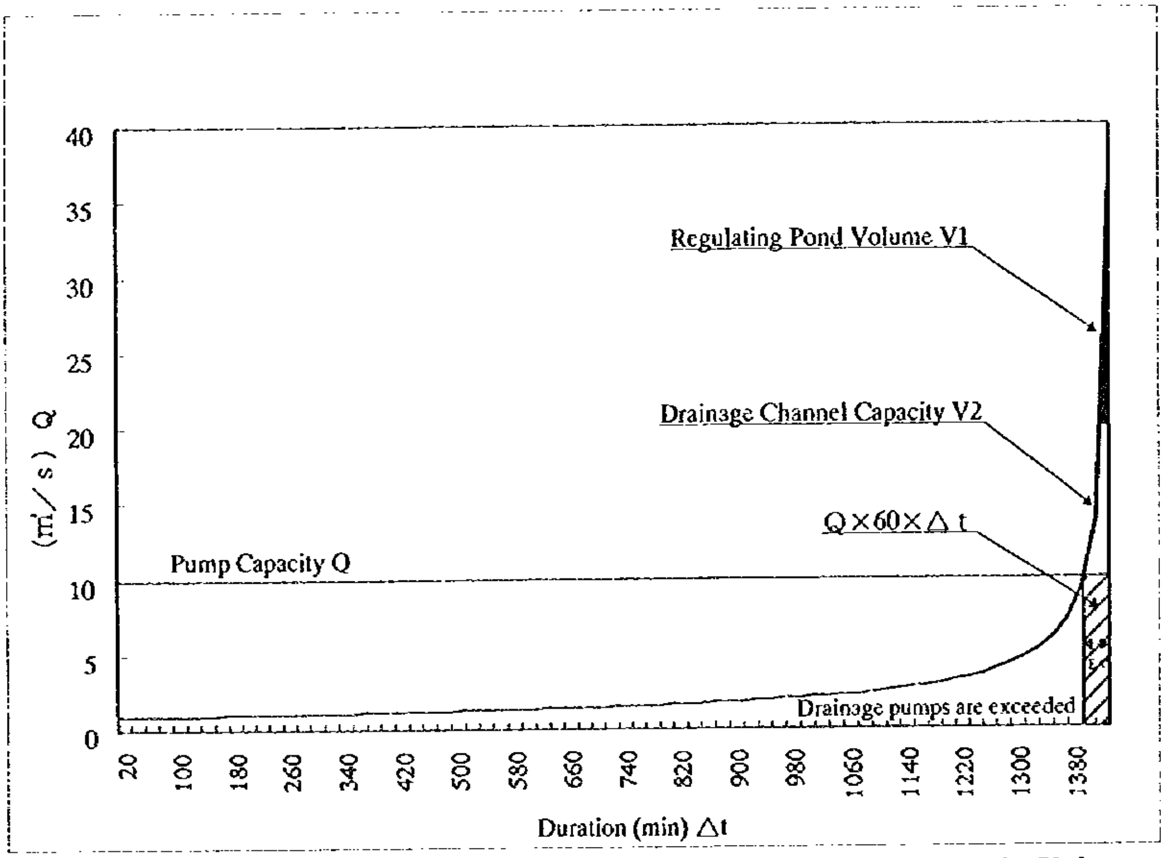
- Area A: drainage pump capacity 10 m^3/s; regulating reservoir capacity : 36,500 m^3
- Area B: drainage pump capacity 10 m^3/s; regulating reservoir capacity : 37,500 m^3



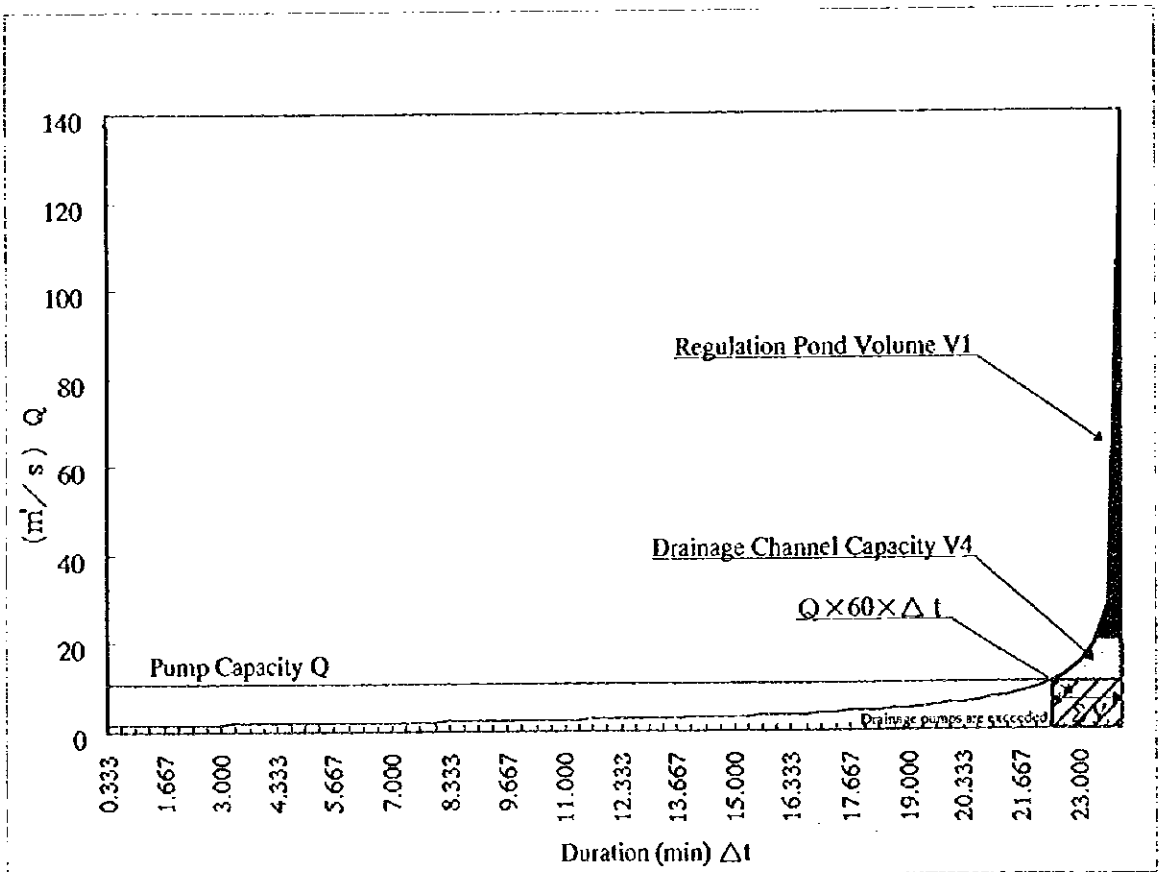
FigureII-3.4.1 Drainage Pump Capacity and Regulating Reservoir Volume (Regulating Reservoir A,5-year Frequency)



FigureII-3.4.2 Drainage Pump Capacity and Regulating Reservoir Volume (Regulating Reservoir A,50-year Frequency)



FigureII-3.4.3 Drainage Pump Capacity and Regulating Reservoir Volume (Regulating Reservoir B,5-year Frequency)



FigureII-3.4.4 Drainage Pump Capacity and Regulating Reservoir Volume (Regulating Reservoir B,50-year Frequency)

TableII-3.4.2 Drainage Pump Capacity and Construction Cost (Regulating Reservoir A)

Drainage Capacity Q (m³/s)	12	11	10	9	8
Regulating Reservoir Volume V (m³)	31,700	34,000	36,500	39,100	42,700
Cost of Drainage Pumps (100 million yen)	10.4	9.2	8.0	7.6	7.0
Construction Cost of Regulating Reservoir (100 million yen)	1.3	1.3	1.4	1.4	1.5
Total	11.7	10.5	9.4	9.0	8.5
Return Period of Rainfall Intensity	5 years	5 years	5 years	5 years	5 years
Depth of Regulating Reservoir (m)	1.7	1.85	2.0	2.15	2.35

TableII-3.4.3 Drainage Pump Capacity and Construction Cost (Regulating Reservoir B)

Drainage Capacity Q (m³/s)	12	11	10	9	8
Regulating Reservoir Volume V (m³)	32,700	35,000	37,500	40,100	43,900
Cost of Drainage Pumps (100 million yen)	10.4	9.2	8.0	7.6	7.0
Construction Cost of Regulating Reservoir(100 million yen)	1.6	1.7	1.7	1.8	1.9
Total	12.0	10.9	9.7	9.4	8.9
Return Period for Rainfall Intensity	5 years	5 years	5 years	5 years	5 years
Depth of Regulating Reservoir (m)	1.78m²	1.95	2.0	2.15	2.50

3.4.4 Number of Drainage Pumps and Pump Diameter

The drainage requirements of pumps fluctuate with the actual rainfall. The pump capacity should be designed to incorporate more than two numbers each of large capacity pumps and small capacity pumps. One large capacity pump will be added as a stand - by unit. The pump diameter is calculated from the following formula for discharge volume and inlet diameter of the pump.

$$D = 146\sqrt{\frac{Q}{V}}$$

where D : inlet diameter of pump (mm)

Q : discharge volume of pump(m³/min.)

V : velocity at inlet (1.5~3.0 m/s)

The number of pumps should be decided for the most appropriate balance of construction cost and maintenance cost. Usually, the construction cost increases in proportion to the number of pumps. The number of pumps has been set here at three large volume unit and three small volume units, including the reserve pump. The design is made based on the following specifications.

- Area A ϕ 1200 x 172 m^3/min x 3.4 m x 160 kW x 3 nos. (1 reserve no.)

  ϕ 900 x 86 m^3/min x 3.4 m x 75 kW x 3 nos.

- Area B ϕ 1200 x 172 m^3/min x 4.0 m x 200 kW x 3 nos. (1 reserve no.)

  ϕ 900 x 86 m^3/min x 4.0 m x 90 kW x 3 nos.

3.4.5 Pump Type

Generally, pumps are classified into three types: vertical axis type, horizontal axis type, and submersible type. The three types are compared in TableII1-3.4.4. The submersible type is used in the present design for the following reasons:

- It has the fewest restrictions on installation space. All necessary equipment is installed under water and no superstructure is required.
- It has the fastest start-up time and quickly responds to rainfall.
- The electric motor is immersed in water. However, the discharge volume is small and actual operation is only a few times per year, allowing ease of inspection.
- There are many performance data for small pumps. Pumps with good corrosion resistance are available.

TableII1-3.4.4 Comparison of Pump Types

Item	Vertical Axis Type	Horizontal Axis Type	Submersible Type
Applicable Motor	○	◎	△
Installation Space	◎	○	◎
Superstructure	○	◎	◎
Ease of Operation	○	○	○
Attachments	○	○	◎
	○	○	○
Pumping Capability	◎	○	◎
Corrosion Resistance	△	◎	△
Ease of Installation	○	◎	◎
Ease of Dismantling and Inspection	○	◎	○
Emergency Power	○	◎	○
Ease of Repair	○	○	△
Economy	△	○	◎

3.4.6 Control Method of Pumps

Generally, in order to start pump operation, it is necessary to perform a series of steps including the sealing of pumps, cooling and lubrication, starting and stopping of motors, and opening and closing of discharge valves . To do these operations one by one is complicated and the order of operations differ with the pump type, which can be the cause of improper handling. In order to reduce the risk of mistakes, it is usual to automate or interlock operations.

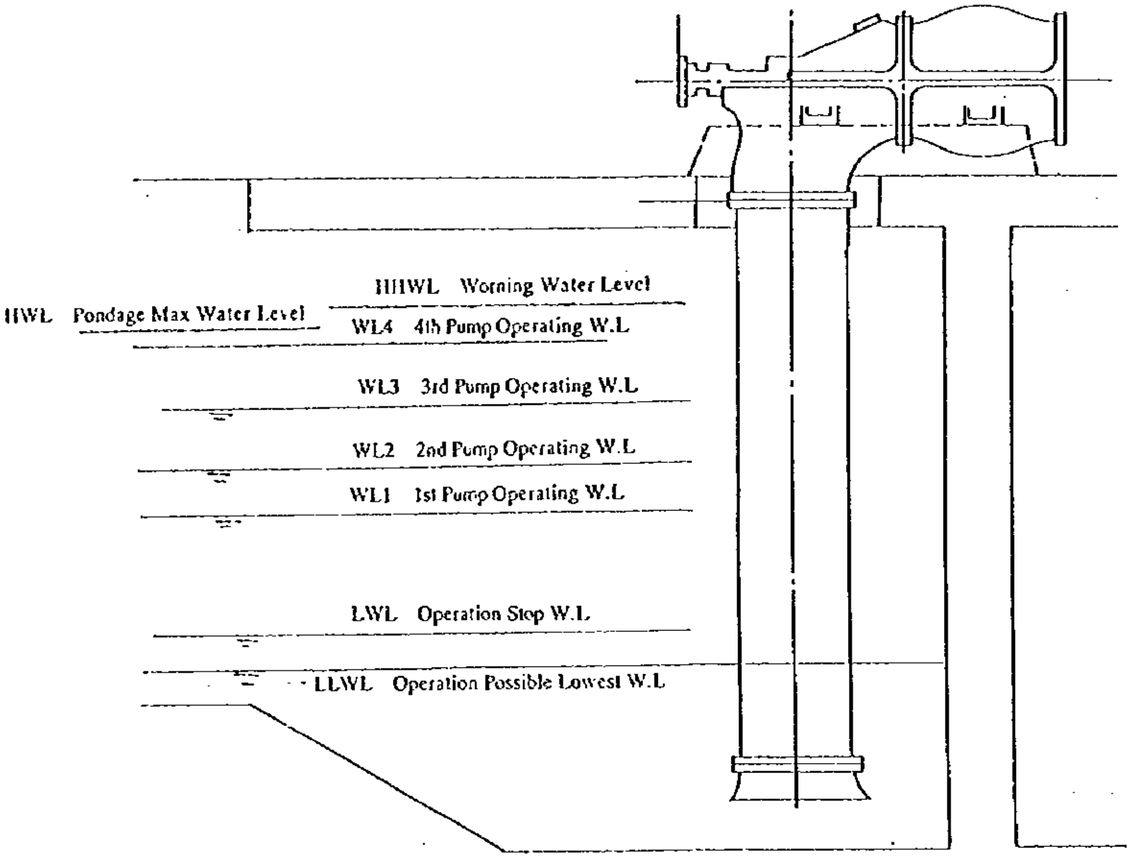
There are two types of pump operations. Single-man control method is a type where a single start switch will commence a series of programmed and interlocked operations for starting or stopping the pumps. Automated control method is a type where the system monitors the control data for operations and automatically adjusts the pump operations.

The selection of operation type is done following a comprehensive analysis of fluctuation of inflow, pump type and operation frequency, construction cost, maintenance and repair costs, number and technical level of operational personnel, and the initial measures at the beginning of operations. The recent trend is to adopt automated operation due to improvement in control technology, reduction of personnel, and energy conservation needs.

However, in any case, it is necessary to allow independent operation at site for test runs and adjustments. Furthermore, in case of automated operation, it is necessary to provide back-up capability for single-man operations .

Automated operation of pumps can be a simple system in which only the water level is monitored and starting and stopping of the pumps only are controlled or it can combine selection of the number of pumps to operate and discharge volume control. Many other variations are possible.

In this project, regulating reservoirs are planned before the pump stations. The capacity of the reservoirs are very large and the rise of water level in the reservoirs is slow. Therefore, emergency operation of pumps to cope with rainfall is not required. From economical and ease of operation monitoring considerations, it is concluded that the discharge volume control method with water level control and pump number control is the most appropriate.



FigureIII-3.4.5 Water Level and Pump Number Control

3.4.7 Comprehensive Control Method

(1) Summary

In the present airport project area there are several storm water pump stations and sewage pump stations. In recent years, it has become important to establish a comprehensive control system to realize optimal operations based on accurate and quick management of a large number of data by grasping the status of the various facilities at one site. A brief description of the system is given below.

(2) Management System

In the case when wide-area management of airport facilities for comprehensive operations is undertaken, the following points have been iterated as the basic operational objectives of the comprehensive control system.

• Management of discharge water quality

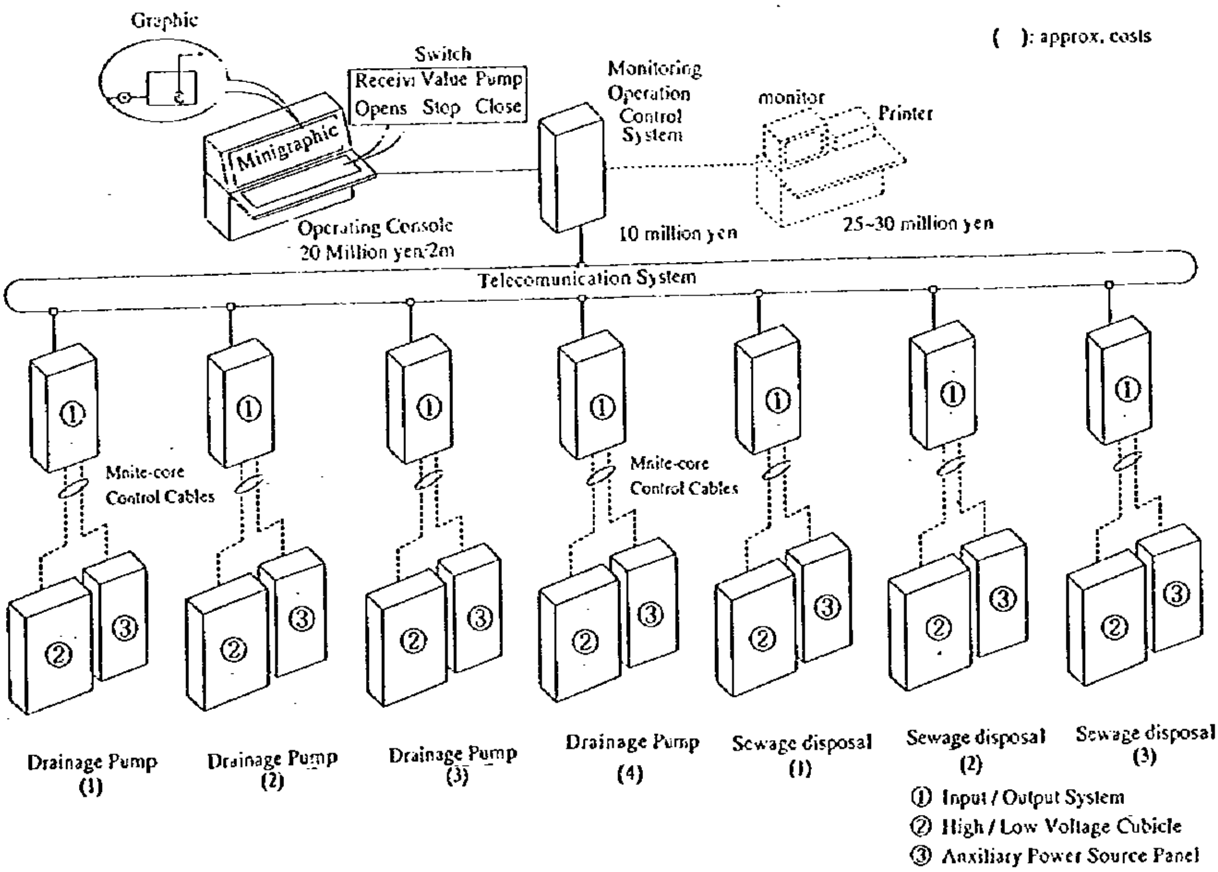
• Securing stability of stormwater drainage

• Efficient management of equipment maintenance and operation control

The central issue in recent cases has been to respond to the increasing complication and improvement of the facilities while at the same time, establishing a comprehensive management system based on wide-area networks caused by enlarged system boundaries.

(3) Summary of Process Control

Process control can be roughly divided into volume control and quality control. Volume control is represented by pump control. This has as its objectives, the dependable discharge of stormwater to prevent flooding, the optimal management in response to the fluctuations in inflow, the control of pumping volume to realize appropriate water levels in conduits, etc. Quality control has as its main objectives, the control of aeration volume, the control of reticulated sludge, and the management of discharge water quality.



FigureII-3.4.6 Comprehensive Sewage Control Diagram (Proposed)

3.4.8 Screening System

The screening facilities will be set at the inflow channel of the drainage pump facilities. It will prevent the infiltration of debris carried by inflowing stormwater. Removal of foreign matters will reduce damage and clogging of the pumps and enable smooth operation. Since the operation of the drainage pumps will be mainly during heavy rains, an automatic raking machine will be provided to prevent foreign matters from accumulating on the screen and hampering the smooth inflow of stormwater. A conveyor and hopper will also be provided to convey and hold the raked up foreign matters.

(1) Type of Automatic Raking Machines

The common types of raking machines are:

A) Intermittent Operation Type

- Fixed Pin-rack Type
- Movable Pin-rack Type
- Revolving Type

B) Continuous Operation Type

- Belt Type
- Uragaki Type
- Conveyor Type

(2) Selection of Raking Machine Type

In the proposed drainage pump facilities, stormwater under gravity flow without passing other pumps will be the main inflow and pump operations are foreseen to be performed only a few times per year. Continuous operation type rakes which have revolving elements under water, will need frequent inspection for corrosion. Therefore the intermittent operation types which do not have revolving elements under water when not operating are concluded to be the most appropriate for this project. Among the intermittent operation types, the revolving type is used for facilities with a small inflow volume and is designed for channels of less than 2 meters wide. The channels in this project are either 3.6 meters or 2.7 meters depending on the pump orifice diameter making pin-rack types suitable. Among the pin-rack types, the movable type requires experienced operators. In conclusion, the Fixed Pin-rack Type with fully automated operation capabilities and ease of inspection is recommended for this project.

3.4.9 Pump Superstructure

A summary of the superstructure of the drainage pump facilities is given below.

Name of Building	: Drainage Pump Control Room
Structure	: Steel reinforced concrete moment resisting frame construction, single story, brick masonry walls with paint finished mortar.
Foundation	: Steel reinforced concrete continuous foundation
Building Area	: 150 m^2
Floor Area	: 150 m^2
Exterior Finish	: Walls: mortar with paint finish Roof: exposed asphalt roofing Aluminum sashes and steel flush doors with paint finish
Interior Finish	: Floor: mortar with floor coating Walls: mortar with paint finish Ceilings: unfinished concrete
Electrical Equipment	: Telephone facilities Automatic fire alarm facilities (signal to be relayed to central control room) Lighting Power Room, Rest Room, Storage: 300 lux Control Room : 500 lux
Ventilation Equipment	: Power Room, Control Room : Thermostat activated Rest Room, Storage : Lighting interlocked
Fire Extinguishing Equip.	: Large fire extinguisher for electrical fires

3.4.10 Oil Separator Facilities

Eventual impact of the project airport on regional society shall be given due consideration and appropriate measures be taken to protect the environment. The oil separator facilities will be placed at the outlet of the drainage channels for this purpose.

(1) Treatment Method

There are several treatment methods for oil separators. The oil outflow from an airport comes mainly from the apron area and, based on data of other airports, the concentration is not high. The most simple API (American Petroleum Institute) method will be adopted for this project from economical and ease of construction view points.

(2) Design Conditions (Original criteria are taken from Japanese data)

Design Droplet Size	: 150 μ
Drainage Area	: 223 ha
Discharge Load	: 0.072 kg /day・ha
Discharge Volume	: 223 ha x 0.072 = 16.1 kg/day
Discharge Water Quality	: 5 ppm
Inflow Time	: approximately 160 minutes
Ascending Rate of Droplet	: Vt = 9 cm/min
Horizontal Velocity	: VH = 90 cm/min
Design Water Volume	: Qm = 21 m^2/min (16.1 ÷ 5 ÷ 160)
Safety Coefficient	: F = 1.5

(3) Required Capacity

• Minimum Surface Area

$$A_H = F\frac{Qm}{V_t} = 1.5 \text{ x } \frac{21}{0.09} = 350 \text{ m}^2$$

• Required Length

$$L = \text{F}\frac{V_H}{V_t}\, d = 1.5 \text{ x } \frac{0.9}{0.09} \text{x2} = 30 \text{ m } (\, d = \text{depth of water } 2.0 \text{ m})$$

• Required Width

$$\text{B} = 350 \text{ m}^2 \div 30 \text{ m} = 11.7 \text{ m} \rightarrow 12 \text{ m}$$