THE SULTANATE OF OMAN

THE DETAILED DESIGN REPORT
ON
THE WADI JIZZI
AGRICULTURAL DEVELOPMENT PROJECT

ANNEX D. DESIGN OF PROJECT FACILITIES

ANNEX E. COST ESTIMATION

ANNEX F. ECONOMIC EVALUATION

JUNE 1986

JAPAN INTERNATIONAL COOPERATION AGENCY

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D-1. Detention Dam

1. Dam Site and Dam Axis

1.1. Dam Site

The detention dam site finding study has been made along the Wadi Jizzi during the feasibility stage in order to determined the best dam site from both the economic and technical points of view. Of these, the site situated approximately about 3.5 km in the south-east from the Sihlat village was districtly more favorable than the other taking into account the following basic considerations.

- The dam site should be located in the downstream reaches of the river system in order to command the river basin as large as possible, in consideration of the full utilization of the water resources.

- The dam site shall provide enough infiltration capacity into aquifers along the wadi course on the adjacent downstream of the detention dam site.

- The high embankment should be avoided for the safety of the dam body against sliding and piping failures.

- The site with less embankment volume should be selected in order to diminish the construction cost of the dam body.

- The embankment materials to be used for the dam body shall be easily obtained around the dam site in order to execute the economical construction.

From the above considerations, the proposed dam site is finally selected as the most suitable one near the Sihlat village which is located about 15 km by road south-west of Qabail.

The catchment area of the propsed dam site is 812 sq.km and the reservoir will stretch east to west along the wadi valley with an area of 1.25 sq.km at the full water level when the dam is constructed.

1.2. Dam Axis

Two dam axes have been examined around the proposed dam site and the combined axis was selected based on the results of comparative study during the feasibility study stage. After obtaining the data of topographical survey in the detailed design stage, the dam axis is slightly rearranged taking into account the increase in dam height and spillway discharge, location of emergency spillway at the left bank and topographical condition. In conclusion, the combined axis having 130° intersectional angle is finally selected.

2. Dam Site Geology

Geology of the detention dam site consists of recent river deposits, middle terrace deposits, upper terrace deposits and bed rocks as shown in Fig. D-1. The main geological features of them are as follows.

Bedrock consists of serpentine, and is overlain by upper terrace deposits. It is generally hard except some parts, and impermeable layer. However, it does not come into contact with the dam embankment.

Upper terrace deposits consist of well consolidated sands and gravels, from which the deposits are regarded as soft rock. They are overlain by middle terrace deposits, and outcrop at the both abutments that come into contact directly with the dam embankment. They are weathered and weakly cemented in the surface. Therefore the permeability is high at the abutments.

Middle terrace deposits consist of consolidated sands and gravels with moderately soft, but are regarded as soft rock. They are overlain by recent river deposits. Because they have abundant opened joints and vesicles in lower part, the permeability is high.

Recent river deposits distribute in the Wadi channel, and consist of very loose sands and gravels. The average thickness is 3.0 m.

Regarding the permeability of the foundation, it can be divided into 5 zones from lugeon value as described belows.

Zone 1 consists of very loose recent river deposits. Average coefficient of permeability obtained from the field tests is 9.77×10^{-3} cm/sec in falling head method.

Zone 2 corresponds to subsurface layer of terrace deposits. Average coefficient of permeability is 1.8 x 10^{-3} cm/sec. This zone is divided into upper part of middle terrace deposits in the river bed and upper terrace deposits in the both abutments, and these terrace deposits consist of weakly consolidated and weathered sand and gravel for the former.

On the other hand, the latter consists of medium sand and is moderately hard, nevertheless it has sparsely opened joints.

Zone 3 consists of lower part of middle terrace deposits. Average coefficient of permeability is 5.12 x 10^{-4} cm/sec. This zone is weakly consolidated and generally vesicular.

Zone 4 distributes narrowly around above mentioned area. Average coefficient of permeability is 1.30 x 10^{-4} cm/sec. The zone is moderately hard and joints are rather sparse.

Zone 5 consists of upper terrace deposits and bed rock. Average coefficient of permeability is 1.8 x 10^{-5} cm/sec. This zone is impermeable layer.

As for the bearing capacity of foundation, it seems that terrace deposits in the foundation have sufficient bearing capacity for the embanked load, because, N-value (cone penetration resistance) in the terrace deposits is more than 100. However, for the recent river deposits and upper layer of terrace deposits, it is necessary to remove wholly from the trench base of detention dam so that the dam foundation be free from the liquefaction, bearing capacity and seepage problems.

No faults and shear-zones have been revealed by the results of field investigations, interpretation of aerial photographs and core drillings.

The result of permeability tests in lugeon value is shown in Fig. D-2.

Fig. D-1 GEOLOGICAL PROFILE OF DETENTION DAM

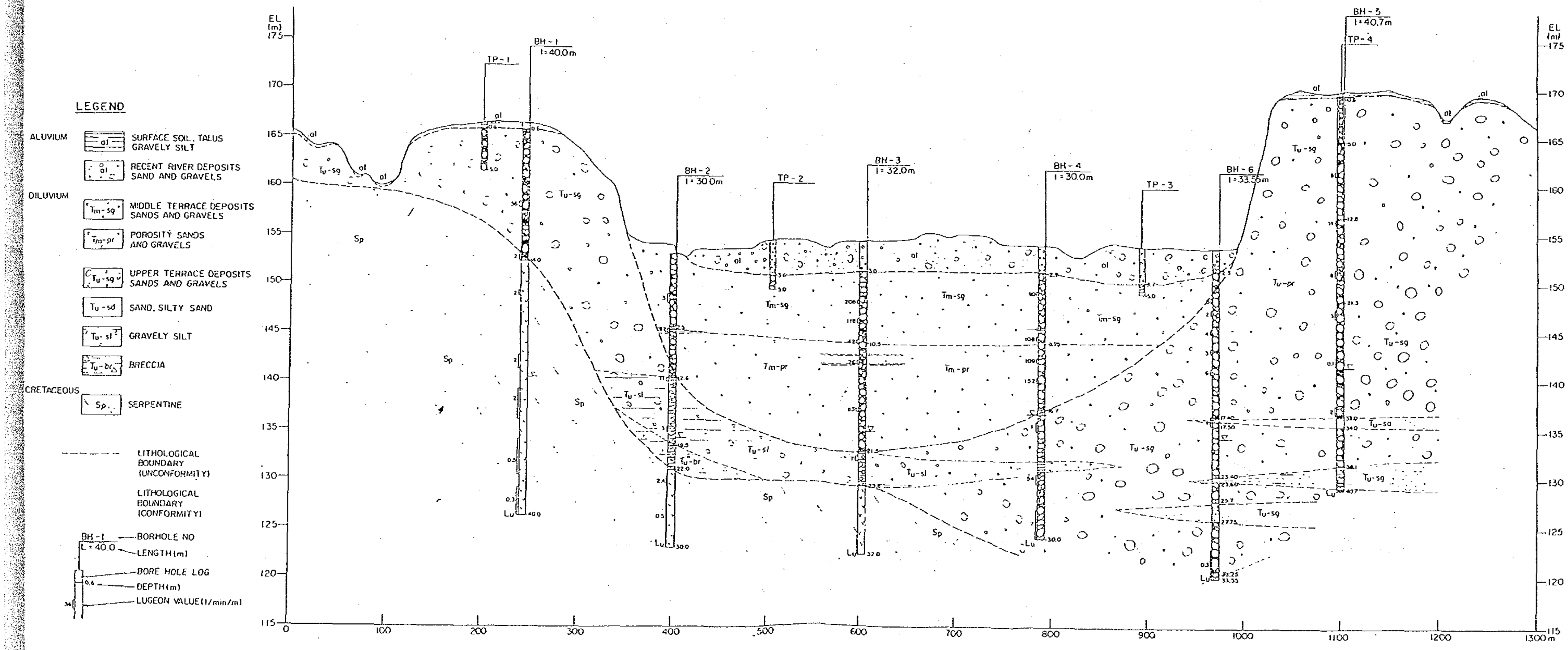
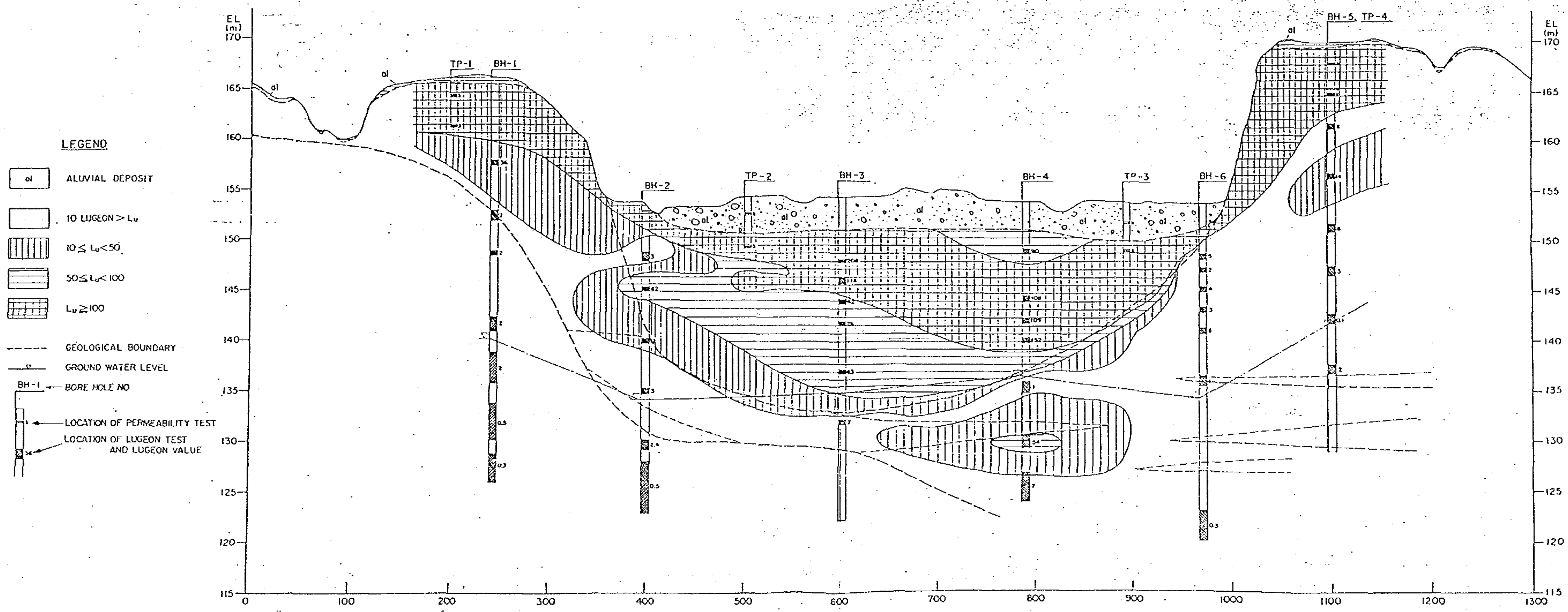


Fig. D-2 LUGEON MAP OF DAM FOUNDATION



3. Dam Body and Foundation

3.1. Typical Section

1) General

In designing dams, the considerations should be commonly given to minimizing construction cost and to securing safety against the sliding and seepage failures. In order to minimize the cost, the most economical materials available including the excavated materials from the dam foundation and appurtenant structures should be maximally utilized.

From the viewpoints of quantities, qualities and costs, the most suitable and economical materials available around the dam site are terrace and river deposits of sand and gravel materials, so that these materials should be used for a large portion of the dam body.

2) Embankment materials

The embankment materials for each zone at the detention dam are planed to obtain from the borrow areas as mentioned below:

Sand and gravel materials ... Materials excavated at both spillway sites (Terrace deposits)

Filter materials .... Materials excavated at the wadi course (River deposits)

Riprap materials .... Boulders distributed at the wadi course

From the results of materials investigations, the brief descriptions in each borrow area are as follows:

- Since the terrace deposits are widely distributed all over the dam site, the service and emergency spillway sites located at right and left abutments respectively are selected as the borrow area for the sand and gravel materials through the grizzly plant for the separation of cobbles and boulders taking into account the excavation volume at the spillway sites, physical properties of these materials and the borrowing condition.

  The materials excavated from both spillway sites consist of sand and gravel with a little quantity of clay belonging to the well consolidated and can be classified into GW-GC under the Unified Soil Classification System.

- The materials to be used for the filter zone are derived from recent river deposits on the wadi course by using the screening plant for the arrangement of grading, and these materials can be easily borrowed from any places along the wadi course.

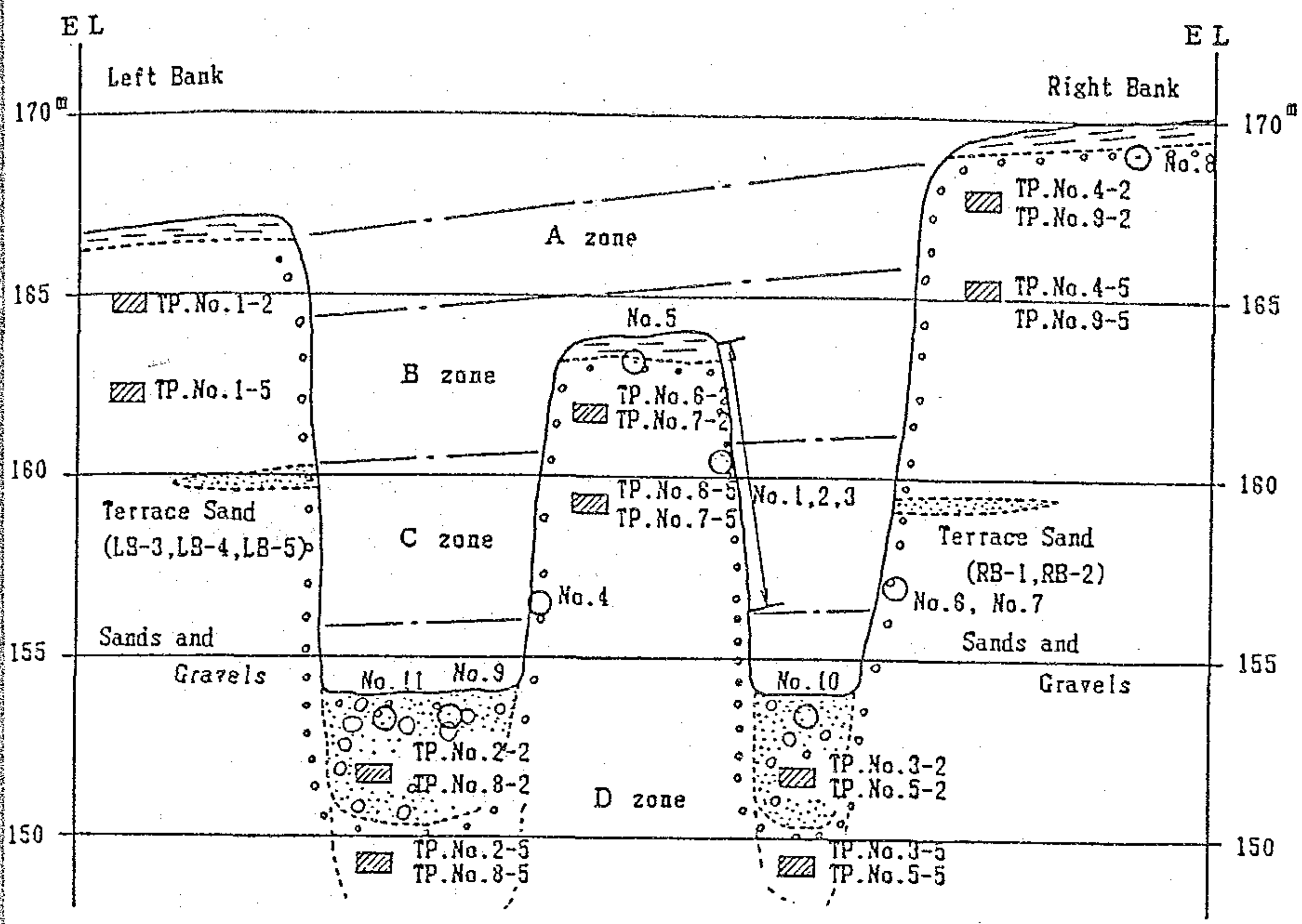
- Boulders scatteringly distributed along the wadi course at the dam site will be mainly used for the riprap and rock materials under gathering by man-power. And also, the talus deposits located about 2.0 km away from the dam site can be used for the riprap and rock materials, however, it seems inevitable to employ a small scale blasting on the quarrying works.

  The typical profile and sampling points at the proposed borrow areas is shown in Fig. D-3.

3) Crest elevation

Freeboard is the difference between the crest elevation of dam body and the maximum water surface level in the reservoir and was obtained by 0.8 m based on the height of wave due to wind (refer to Appendix I-1, Feasibility Report and Definitive Plan Report).

Fig. D-3 TYPICAL PROFILE AND SAMPLING POINTS AT BORROW AREA
(NOT TO SCALE)



Sampling ▨ (1985) TP.No.1~TP.No.9

⊙ (1983) No.1 ~No.11

A zone ; Terrace Deposits(Upper part). Consolidated but weathered.
Sample No. TP.No.1-2, TP.No.4-2, TP.No.9-2

B zone ; Terrace Deposits.(Middle part). Consolidated.little weathered
Sample No. TP.No.1-5, TP.No.4-5, TP.No.6-2, TP.No.7-2, TP.No.9-5

C zone ; Terrace Deposits.(Lower part). Well consolidated
Sample No. TP.No.6-5, TP.No.7-5

D zone ; Middle Terrace Deposits. Well consolidated. Fresh.
Sample No. TP.No.2-5, TP.No.3-5, TP.No.5-5, TP.No.8-5

River Deposits ; (F)
Sample No. TP.No.2-2, TP.No.3-2, TP.No.5-2, TP.No.8-2

According to the hydraulic study of spillway during the detailed design stage, a rising height of water surface from the full storage level due to release of the design flood discharge at the spillways was designed at 5.3 m and its corresponding water surface elevation is fixed at 169.2 m. Therefore, the dam crest elevation without extra bank can be obtained by adding the freeboard to the maximum water surface, as follows:

Dam Crest Elevation = EL.169.2 + 0.8 = EL.170.0 m

4) Typical section

As for the type of detention dam, the zone type of fill dam with upstream rock zone was selected in preference over the other type during the feasibility stage in consideration of the purpose of detention dam, dam height, available materials for the dam body, geological condition and economic construction.

However, in detailed design stage, the upstream rock zone is replaced by sand and gravel fill taking into account the quality, quantity and construction cost of both the materials. Following items are supplementary considered in final determination of the typical section of the detention dam.

- The upstream slope of the dam body is protected by 0.45 m thick riprap against wave action.
- Bedding filter in 0.25 m thick should be provided under the riprap fill at the upstream slope in order to prevent the fine particles in the sand and gravel zone from movement and suction by seepage flow in drawdown condition of the reservoir.
- Poor gradation materials should be embanked at downstream places of the intercepter to mitigate the seepage problem.

The typical cross section of detention dam is shown in Fig. D-4.

3.2. Stability Analysis

For fill type dam, stability of the dam body means that a soil mass with skelton stress can keep it equilibrium state against the external forces. Therefore, taking into account the above mentioned condition and the method of testing, the stability analysis is made by total stress method.

1) Design values

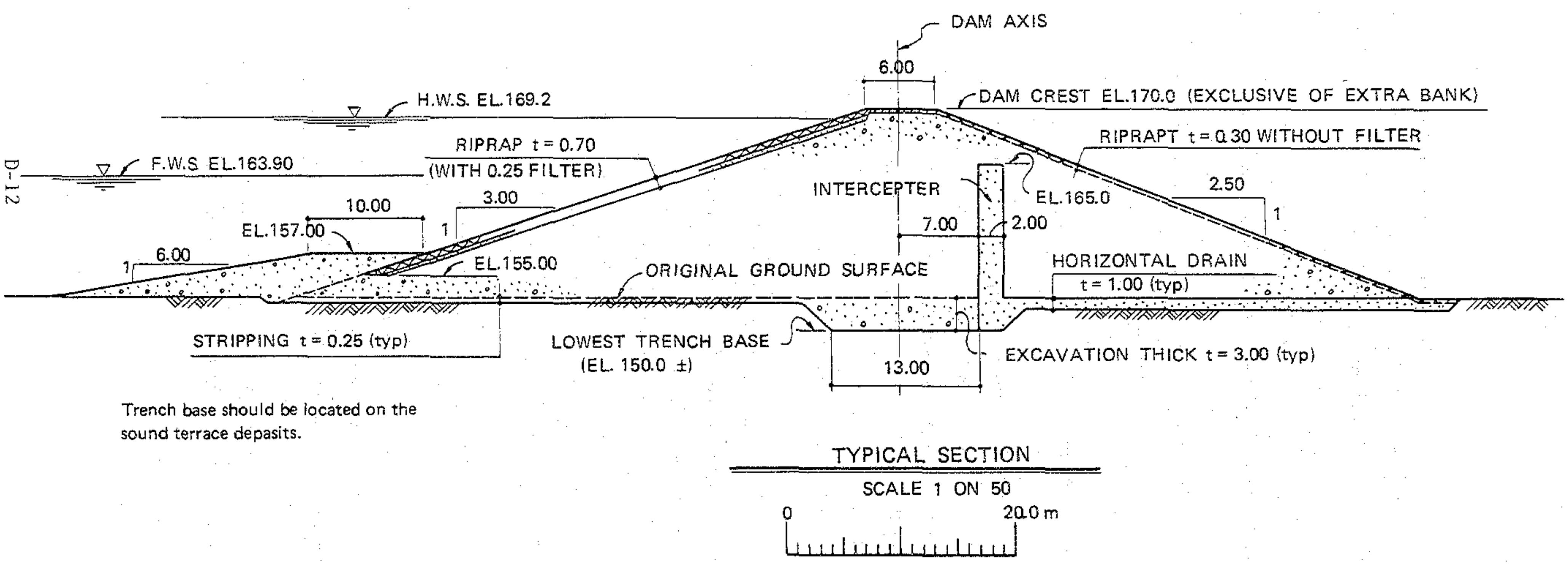
Design values of density and shearing strength to be used for the stability analysis of dam body are considerably influenced by water content, degree and method of compaction and working conditions. In general, the embankment of dam should be executed more than 95 percent of the maximum dry density on wet side by controlling the water content.

It is desirable that the design values of embankment materials for the detention dam should be decided according to the results of soil tests, however, some of them were assumed by the past data obtained in the similar materials in Japan due to lack of mechanical test in quality and quantity. Design values of sand and gravel, filter and foundation materials concerning the stability analysis are discussed below.

a) Sand and gravel materials (Semi-pervious zone)

Since the sand and gravel zone is mainly embanked with terrace deposit materials excavated from both the service and emergency spillway sites through the grizzly plant for the separation of cobbles and boulders various materials ranging from silt to gravel will be mixed, which mainly classified into GW and GC. Considering such conditions, the soil tests for the sand and

Fig. D-4. TYPICAL SECTION OF DETENTION DAM



gravel materials were carried out by using the large scale testing equipment and the results are shown in Table D-1.

Because the embankment of sand and gravel materials should be executed around the optimum water content by splinkling water, the design values of density are estimated at 98 percent of the maximum dry density of compaction tests which were conducted at the optimum water content condition, and are shown in the following table.

As for the shearing strength, the shear box tests with consolidated-undrained condition were carried out at the optimum water content condition, but the triaxial compression tests have not been conducted for the sand and gravel materials. Therefore, the value of shearing strength is decided at 90 percent strength of shear box test at the optimum water content as follows.

			Density			Shearing Strength		
Gs 1/	W 2/	e 3/	γd 4/	γt 5/	γsat 6/	ϕ 7/	C 8/	K 9/
	(%)		(t/m^3)	(t/m^3)	(t/m^3)	(deg.)	(t/m^2)	(cm/sec)
2.72	7.1	0.42	1.92	2.06	2.21	35°00'	0.0	1.9×10^{-4}

1/ specific gravity, obtained by the results of soil tests
2/ water content, obtained by the results of soil tests
3/ void ratio, $e = Gs/\gamma d - 1$
4/ dry density, obtained by the results of soil tests
5/ wet density, $\gamma t = \gamma d\ (1 + W/100)$
6/ saturated density, $\gamma sat = (Gs + e)/(1 + e)$
7/ angle of internal friction, obtained by the results of soil tests
8/ cohesion, obtained by the results of soil tests
9/ coefficient of permeability, obtained by the results of soil tests

b) Filter material (Intercepter)

Since the filter zone is embanked by excavated materials from the river deposits with arrangement in gradation distribution, the soil mechanic properties of these materials are almost equal to those of the sand and gravel. And also, the embankment of filter materials should be executed around the optimum water content by controlling the moisture.

For the design values of density, estimation is made by using the same procedure on the sand and gravel materials, and the results are shown in the following table. The values of shearing strength for the filter materials are decided by the same procedure on the sand and gravel materials as shown in the following table.

			Density			Shearing Strength		
Gs	W	e	γd	γt	γsat	ϕ 1/	C	K
	(%)		(t/m^3)	(t/m^3)	(t/m^3)	(deg.)	(t/m^2)	(cm/sec)
2.87	4.2	0.39	2.07	2.16	2.35	35°00'	0.0	5.6×10^{-2}

1/ angle of internal friction, assumed by the data of past soil tests in Japan and the results of soil tests (obtained value 36°00')

The sings in the above table are same in the paragraph of sand and gravel materials except for the explained herein.

As for the permeability of filter zone (intercepter and horizontal drain), there is no effective tests for the materials with arranged gradation, the following Hazen formula is adopted to presume the coefficient of permeability.

$$K = C \cdot D_{10}^{2}$$

Where, K: coefficient of permeability in cm/sec
C: constant and adopted by 100, usually
D_{10}: grain size of materials finer than 10 percent by weight of total volume of the materials and adopted by 0.075 cm from Fig. D-15.

The permeability coefficient of the filter zone is computed to be about 5.6×10^{-1} cm/sec from the above formula, and to this value 10 times of the safety factor is adopted in considering the mixture of fines and segregation of materials during the construction and unexpected accidents after completion, therefore, the permeability coefficient of filter zone is obtained by 5.6×10^{-2} cm/sec.

Table D-1 SUMMARY OF SOIL AND ROCK TEST (Executed during Detailed Design Stage)

Soil Tests

Sampling Numbers / Testing Items	Terrace Deposits									
	A Zone			B Zone					C Zone	
	TP No.1-2	TP No.4-2	TP No.9-2	TP No.1-5	TP No.4-5	TP No.6-2	TP No.7-2	TP No.9-5	TP No.6-5	TP No.7-5
Unified Soil Classification System	GW-GC	GW-GC	GW-GC	GW-GC	GW-GC	GW-GC	GW-GC	GW-GC	GW-GC	GW-GC
Grain Size Analysis: Summation % < 0.074 mm	6.68	7.63	7.76	7.77	7.12	7.33	7.12	7.28	6.88	7.30
" < 476 mm	35.33	41.74	41.54	30.18	35.75	47.85	43.75	51.08	30.43	34.83
" < 50.8 mm	75.22	88.81	85.73	84.92	92.67	79.44	87.31	92.04	80.73	80.24
Uniformity Coefficient Uc	54.76	52.50	46.43	47.00	32.50	38.52	26.92	26.67	44.74	37.70
Curvature Coefficient Uc'	1.20	2.74	1.65	1.80	2.29	0.44	1.35	1.07	0.84	0.73
Moisture Content W (%)	1.28	3.77	2.69	1.60	5.37	1.55	2.63	2.92	1.91	1.91
Bulk Density (t/m^3)	2.16	2.24	2.11	2.19	2.14	2.13	2.05	2.27	2.29	2.21
Specific Gravity Gs	2.77	2.67	2.59	2.74	2.77	2.81	2.75	2.63	2.82	2.72
Water Absorption (%)	1.18	2.78	2.13	1.10	1.08	1.69	1.41	3.01	2.49	1.10
Soundness (%)	-	-	-	-	-	-	-	-	-	-
Compaction Tests: Optimum Water Content Wopt (%)	-	7.50	-	-	9.90	-	4.00	-	-	-
Compaction Tests: Max. Dry Density γd max. (t/m^3)	-	1.81	-	-	1.88	-	2.07	-	-	-
Permeability Coefficient K (cm/sec)	-	$3.5x10^{-5}$	-	-	$1.3x10^{-4}$	-	$3.9x10^{-4}$	-	-	-
Shear Box Tests: Cohesion C (t/m^2)	0.0	0.0	-	-	0.0	-	-	-	-	-
Shear Box Tests: Internal Friction Angle φ (deg.)	40°00'	38°00'	-	-	39°00'	-	-	-	-	-

Sampling Numbers / Testing Items	Terrace Deposits								
	D Zone				Terrace Sand				
	TP.No.2-5	TP No.3-5	TP No.5-5	TP No.3-3	RB-1	RB-2	LB-3	LB-4	LB-5
Unified Soil Classification System	GW-GC	GW-GC	GW-GC	GW-GC	SW-SP	SW-SP	SP	SP	SP
Grain Size Analysis: Summation % < 0.074 mm	7.28	7.09	6.88	6.39	-	-	-	-	-
" < 476 mm	41.61	34.07	38.75	31.64	-	-	-	-	-
" < 50.8 mm	93.28	68.78	84.75	89.38	-	-	-	-	-
Uniformity Coefficient Uc	40.38	38.71	65.33	40.00	-	-	-	-	-
Curvature Coefficient Uc'	1.12	0.78	1.30	2.56	-	-	-	-	-
Moisture Content W (%)	1.33	10.38	4.61	1.32	-	-	-	-	-
Bulk Density (t/m^3)	2.07	1.96	2.19	2.07	2.035	1.948	2.120	2.200	2.176
Specific Gravity Gs	2.75	2.75	2.73	2.78	Sa;2.57 Si;2.64	Sa;2.58 Si;2.66	Sa;2.62 Si;2.66	Sa;2.55 Si;2.61	Sa;2.58 Si;2.65
Water Absorption (%)	2.88	1.91	1.52	1.22	-	-	-	-	-
Soundness (%)	-	-	-	-	-	-	-	-	-
Compaction Tests: Optimum Water Content Wopt (%)	-	-	-	-	-	-	-	-	-
Compaction Tests: Max. Dry Density γd max. (t/m^3)	-	-	-	-	-	-	-	-	-
Permeability Coefficient K (cm/sec)	-	-	-	-	-	-	-	-	-
Shear Box Tests: Cohesion C (t/m^2)	-	-	-	-	-	-	-	-	-
Shear Box Tests: Internal Friction Angle φ (deg.)	-	-	-	-	-	-	-	-	-

Sampling Numbers / Testing Items	River Deposits							
	Dam Site				Upstream Point	Dam Site Point	Middle-Stream Point	Down-Stream Point
	TP No.2-2	TP No.3-2	TP No.5-2	TP No.3-2				
Unified Soil Classification System	GW	SW	GW	GW	GW	GW	GP	SW
Grain Size Analysis: Summation % < 0.074 mm	0.67	1.47	1.11	0.90	0.46	0.79	1.56	1.21
" < 476 mm	37.93	65.13	35.70	28.93	19.19	33.16	22.48	57.50
" < 50.8 mm	88.30	96.15	82.04	75.66	52.88	81.98	80.55	0.0
Uniformity Coefficient Uc	64.00	11.54	37.50	44.35	61.90	36.89	44.29	15.59
Curvature Coefficient Uc'	1.63	0.37	1.24	1.69	3.52	0.89	4.61	0.35
Moisture Content W (%)	7.48	4.49	3.53	13.43	-	-	-	-
Bulk Density (t/m^3)	1.89	1.511	1.76	1.84	-	-	-	-
Specific Gravity Gs	2.97	2.83	2.82	2.87	-	-	-	-
Water Absorption (%)	0.925	0.57	0.81	0.49	-	-	-	-
Soundness (%)	0.10	0.09	-	0.06	-	-	-	-
Compaction Tests: Optimum Water Content Wopt (%)	3.50	5.00	-	-	-	-	-	-
Compaction Tests: Max. Dry Density γd max. (t/m^3)	2.22	2.01	-	-	-	-	-	-
Permeability Coefficient K (cm/sec)	$1.3x10^{-3}$	$2.3x10^{-4}$	-	-	-	-	-	-
Shear Box Tests: Cohesion C (t/m^2)	0.00	0.0	-	-	-	-	-	-
Shear Box Tests: Internal Friction Angle φ (deg.)	38°00'	42°00'	-	-	-	-	-	-

Rock Tests

Sampling No. / Testing Items	Quarry Rocks					
	R-1	R-2	R-3	R-4	R-5	R-6
Moisture Content (%)	0.11	0.05	0.25	0.16	0.21	-
Bulk Density (t/m^3)	2.68	2.70	2.71	2.71	2.72	-
Specific Gravity Gs	2.70	2.73	2.75	2.70	2.73	2.77
Water Absorption (%)	0.16	0.06	0.09	0.04	0.07	0.14
Soundness (%)	0.25	0.55	0.32	0.36	0.25	0.44
Compressive Strength (kg/cm^2)	240	816	765	457	487	-

Notes:

1. GW: Well-grated gravles, gravel-sand mixtures, little or no fines.
2. GC: Clayey gravels, poorly graded gravel-sand-cley mixtures.
3. GP: Poorly graded gravels, gravel-sand mixtures, little or no fines.
4. SW: Well-graded sands, gravelly sands, little or no fines.
5. SP: Poorly graded sands, gravelly sands, little or no fines.
6. GW-GC : Mixed with GW and GC materials.
7. Sa, Si: Specific gravity of sand and silt materials, respectively.
8. Soundness shows the durability magnesium salfate test.
9. Compaction test is executed under Ec=5.6 kg.cm/cm^3 condition.
10. Specimen condition on permeability test is same in compaction test.
11. Shear box test is executed under quick-undrained condition.

c) Foundation material

Materials composing the dam foundation in the river course are mainly classified into two layers according to the distribution of permeability coefficient, bulk density and corn penetration resistance (N-value); an upper layer developes about 3 m deep from existing ground surface with N-value of about 40 (assumed value from the deposit condition and bulk density) and the other is developes deeper than 3 m with N-values of more than 100.

The upper layer should be excavated at least 3 m deep from the surface of river bed so that the trench foundation of detention dam be free from the liquefaction, bearing capacity and seepage problems.

Since sufficient soil test of dam foundation materials on the river course has not been conducted except a series of physical tests, the design values of density and shearing strength for those materials were assumed from the results of soil tests and N-values.

<u>Upper layer</u> (River deposit)

As for the design values of density, estimation is made by using the results of soil tests (refer to Table D-1, No.2-2, No.3-2, No.5-2 and No.8-2) and the results are shown in the following table. The value of internal friction angle for the upper layer materials is obtained by the following Dunham's formula assuming that N-value is 25[1/].

$$\phi = \sqrt{12\ N} + 15 = 32°19' \fallingdotseq 32°00'$$

<u>Lower layer</u> (Terrace deposit)

The design values of density are determined based on the results of physical tests (TP No.2-5, No.3-5, No.5-5 and No.8-5) and

---

1/ Taking into consideration the intercalation of lenticular and thin silt or fine sand, bulk density and existence of gravel or cobble in the layer, the relevant N-value is presumed by 25.

the results are shown in the following table. For the design value of shearing strength, the same value of sand and gravel materials can be used for the lower layer materials considering the N-value (more than 100), gradation distribution and bulk density.

Layer	Gs	W	e	Density γd	Density γt	Density γsat	Shearing Strength ϕ	Shearing Strength C
		(%)		(t/m^3)	(t/m^3)	(t/m^3)	(deg.)	(t/m^2)
Upper	2.87	7.2	0.76	1.63	1.75	2.06	32°00'	0.0
Lower	2.75	4.5	0.39	1.98	2.07	2.26	35°00'	0.0

The signs in the above table are same in the paragraph of sand and gravel materials.

d) Riprap material

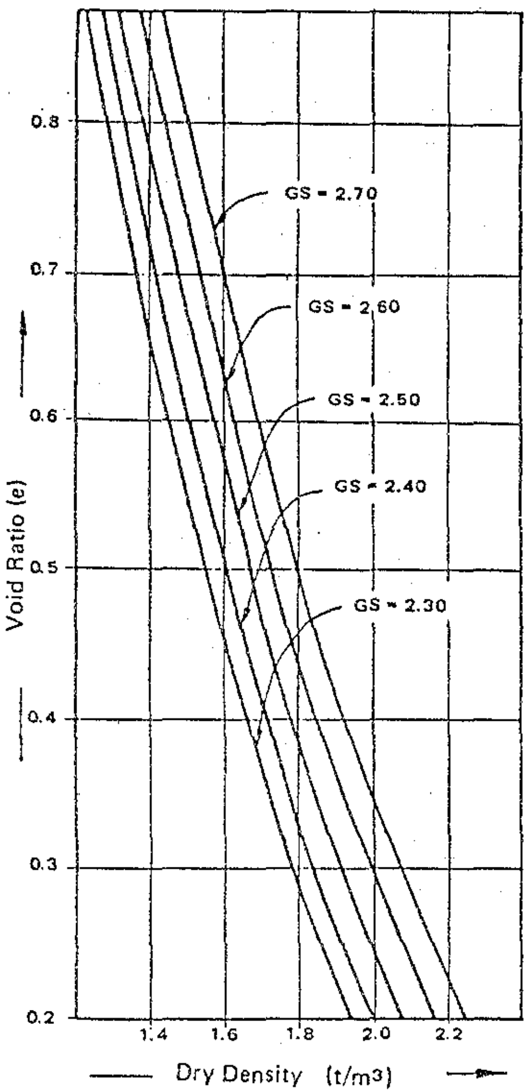
In order to check the surface slope failure of the dam body, the design values of riprap materials are assumed based on the data obtained from the past soil tests in Japan taking the results of rock tests into consideration.

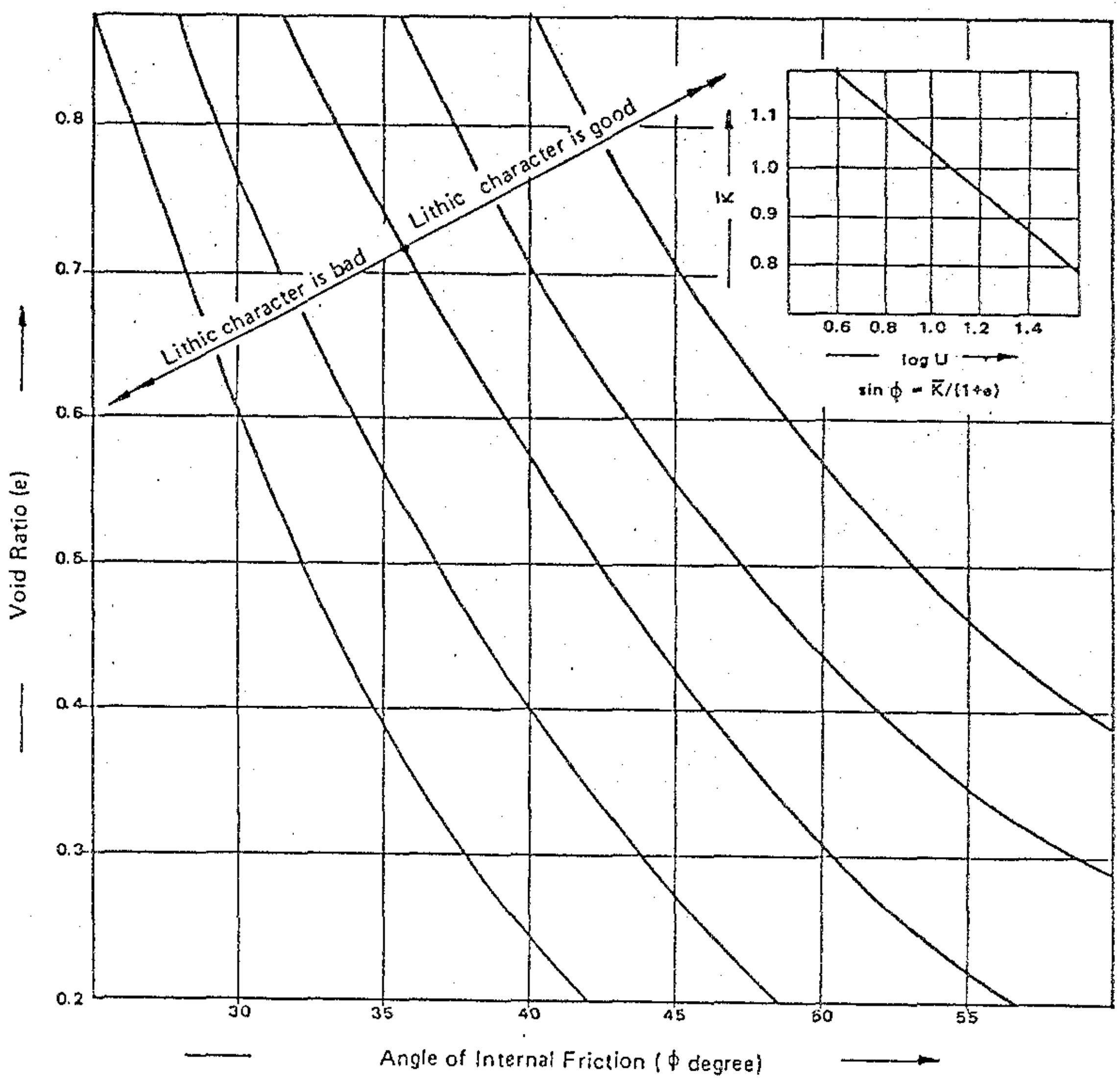
The results of rock tests for the limestone (quarry site) and diabase (river-bed boulders) rocks executed during the feasibility and detailed design stages are summarized in the following table.

Item	Quarry Rock (Limestone) Feasibility	Quarry Rock (Limestone) Detailed Design	River-bed Boulder (Diabase)	Terrace Boulder (Diabase)
Specific Gravity	2.705	2.73	3.047	2.960
Absorption (%)	0.38	0.32	0.40	0.27
Soundness (%)	1.14	0.36	0.31	0.26
Compression Strength (kg/cm^2)	895	553	809	1,346

As for the shearing strength of riprap materials, Fig. D-5 shows relationship between void ratio and angle of internal friction of the blasted rock materials which was derived from the theory of granular materials by Dr. T. Mogami in Japan.

Fig. D–5 RELATION CURVE BETWEEN DRY DENSITY AND ANGLE OF INTERNAL FRICTION FOR ROCK MATERIAL





Consequently, the design values for riprap materials are obtained as follows.

$Gs^{1/}$	$W^{2/}$	$e^{3/}$	Density $\gamma d^{4/}$	Density $\gamma t^{5/}$	Density $\gamma sat^{6/}$	Shearing Strength $\phi^{7/}$	Shearing Strength $C^{8/}$
	(%)		(t/m^3)	(t/m^3)	(t/m^3)	(deg.)	(t/m^2)
2.78	0.16	0.40	1.98	1.99	2.27	38°00'	0.0

1/ specific gravity, obtained by the results of rock tests
2/ water content, obtained by the results of rock tests
3/ void ratio, assumed by the past data
4/ dry density, $\gamma d = Gs/(1 + e)$
5/ angle of internal friction, assumed by Fig. D-5
6/ cohesion

Design values of the above-mentioned embankment and dam foundation materials for the detention dam are summarized in Table D-2.

Table D-2. SUMMARY OF DESIGN VALUES FOR DETENTION DAM

Material	Density $\gamma d^{1/}$	Density $\gamma t^{2/}$	Density $\gamma sat^{3/}$	Shearing Strength $\phi^{4/}$	Shearing Strength $C^{5/}$	$K^{6/}$
	(t/m^3)	(t/m^3)	(t/m^3)	(deg.)	(t/m^2)	(cm/sec)
Sand and Gravel	1.92	2.06	2.21	35°00'	0.0	1.9x10^{-}[illegible]
Filter	2.07	2.16	2.35	35°00'	0.0	5.6x10^{-}[illegible]
Foundation Layer						
Upper (River Deposit)	1.63	1.75	2.06	32°00'	0.0	-
Lower (Terrace Deposit)	1.98	2.07	2.26	35°00'	0.0	-
Riprap	1.98	1.99	2.27	38°00'	0.0	-

1/ dry density, 2/ wet density, 3/ saturated density
4/ angle of internal friction, 5/ cohesion
6/ coefficient of permeability

In the stability analysis, the wet density (γt) is used for the zones upper than the phreatic line and saturated density (γsat) lower than the phreatic line, respectively.

2) Phreatic line

Since actual phreatic line for the fill type dam is rather difficult to obtain theoretically, an approximate formula which has been derived by Cassagrande is usually adopted for the calculation under consideration of anisotropic medium.

a) Permeability in vertical and horizontal directions

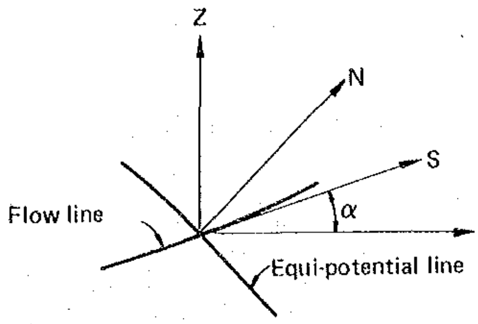
The permeability in vertical and horizontal directions of embanked materials is quite different. Those materials are considered to be an anisotropic medium with the permeability depending on the direction of flow. The ratio of vertical coefficient of permeability (Kv) to horizontal one (Kh) at the compacted embankment materials differs depending on the method of compaction. Since the permeability in vertical and horizontal directions of embanked materials can not be obtained through theoretical analysis, seepage model test and in-situ tests, the degree of anisotropy is presumably obtained through comparison of pore pressure distribution based on the records by piezometers embedded in the existing dams with the distributions by calculation under anisotropic ratio varried. Generally, in case that the compaction is made by flat type vibrating roller or tire roller, Kh may nearly be equal to 25 times of Kv.

b) Convert ratio

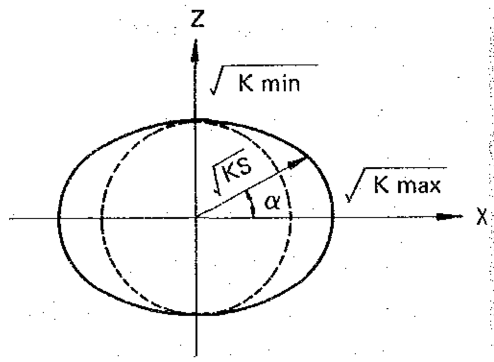
An anisotropy in permeability can be treated as isotropic by transforming the section. Transformed section is obtained by reducing the horizontal dimension of coordinate by the ratio of $\sqrt{Kv / Kh}$.

This transform ratio can be derived theoretically by the following procedure.

In the right figure, S and n shows tangential direction for the flow line and perpendicular direction against the equi-potential line, respectively.



On the isotropic medium, the direction of S closely coincides with that of n due to the flow line and equi-potential line is intersected at right angle. Resultant velocity along the flow line in the figure is shown as follows:



$$Vs = -Kv\frac{\partial h}{\partial s} \qquad \text{(1)}$$

Therefore, the component of velocity corresponding to the direction x and Z can be expressed in the following equations.

$$Vx = -Kx\frac{\partial h}{\partial x} = Vs\ \mathrm{Cos}\ \alpha \qquad \text{(2)}$$

$$Vz = -Kz\frac{\partial h}{\partial z} = Vs\ \mathrm{Sin}\ \alpha \qquad \text{(3)}$$

While, $$\frac{\partial h}{\partial s} = \frac{\partial h}{\partial x}\cdot\frac{\partial x}{\partial s} + \frac{\partial h}{\partial z}\cdot\frac{\partial z}{\partial s} \qquad \text{(4)}$$

Substituting (4) in equations (2) and (3), the following equations are available.

$$\frac{1}{Kx} = \frac{\mathrm{Cos}^2\alpha}{Kx} + \frac{\mathrm{Sin}^2\alpha}{Kz} \qquad \text{(5)}$$

$$\frac{1}{Ks} = \frac{Kx\cdot Kz}{Kx\cdot \mathrm{Sin}^2\alpha + Kz\cdot \mathrm{Cos}^2\alpha} \qquad \text{(6)}$$

Hence, $x = \gamma\cdot\cos\alpha$ and $Z = \gamma\cdot\mathrm{Sin}\alpha$ substitutes in the above equations, the following equation is obtained.

$$\frac{\gamma^2}{Ks} = \frac{X^2}{Kx} + \frac{Z^2}{Kz} \qquad \text{(7)}$$

This is the equation of an ellipse and a half of its major and minor axes can be expressed by $\sqrt{Kx}$ and $\sqrt{Kz}$, respectively. When taking permeability coefficients in the directions of across and parallel to embanked plane surface as Kx = K max and Kz = K min, the permeability coefficients in any direction which gives angle of α to the x axis can be easily obtained by graphical solution as shown in the above figure. According to the equation (7) and the above figure, when the scale in the direction of x is transformed into $X = x \cdot \sqrt{Kz/Kx}$ an ellipse will become a circle, and the permeability will be an isotropic medium that is not changed by direction.

In other words, the effect of an anisotropy in the permeability can be replaced by an equivalent shrinking of the coordinates. Namely, transformed section which is obtained by shrinkage of horizontal direction of coordinate by the ratio of $\sqrt{Kv/Kh}$ (Kv and Kh is permeability coefficient of vertical and horizontal directions, respectively).

c) Seepage of free surface (Phreatic line)

In the transformed section made by using the above ratio, the seepage of free surface (phreatic line) in the sand and gravel zone can be obtained by an approximate formula (refer to Fig. D-6) which has been derived by Cassagrande with Kozeny's parabola at several elevations of reservoir water surface level and the results in transformed and original sections at the detention dam are shown in Fig. D-6.

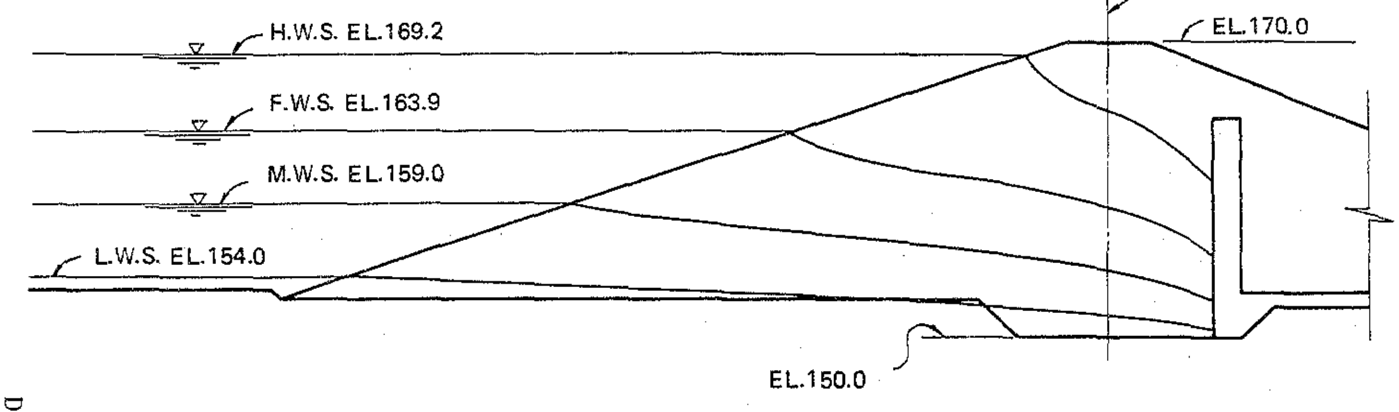
3) Sliding failure analysis

a) Analysis method

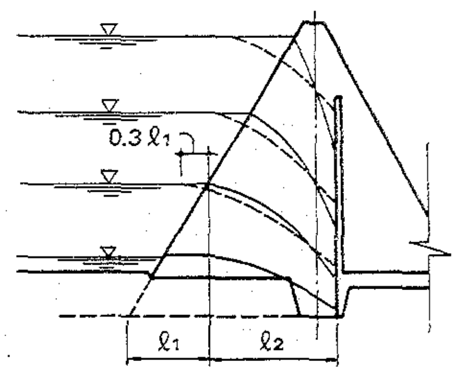
There are a number of methods available for slope stability analysis of the dam body, these are Slip Circle, Fellenius, Bishop, Wedge and Morgenstern methods: however, the Slip Circle method is

Fig. D–6. PHREATIC LINE (SEE PAGE OF FREE SURFACE)

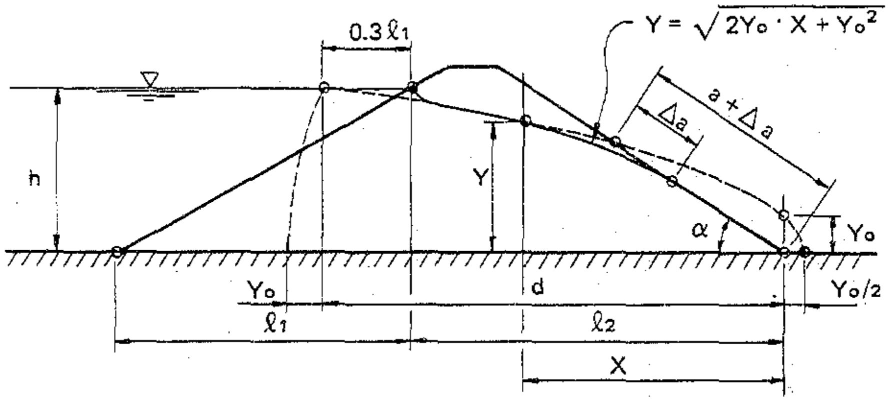
$Kh = 25\ KV$
Shrinkage ratio of coordinatation $\sqrt{Kh/KV} = 5.00$



ORIGINAL SECTION



TRANSFORMED SECTION



	h	ℓ_1	ℓ_2	d	Y_o	$Y=\sqrt{2Y_oX+Y_o^2}$	Δa
HWS EL169.2	19.2	11.52	2.48	5.94	14.16	$Y=\sqrt{28.32X+200.51}$	3.54
FWS EL163.9	13.9	8.34	5.66	8.16	7.96	$Y=\sqrt{15.92X+63.36}$	1.99
MWS EL159.0	9.0	5.40	8.60	10.22	3.40	$Y=\sqrt{6.80x+11.56}$	0.85
LWS EL154.0	4.0	2.40	11.60	12.32	0.63	$Y=\sqrt{1.26x+0.40}$	0.16

$d = 0.3\,\ell_1 + \ell_2,\ Y_o = \sqrt{h^2 + d^2} - d,\ a + \Delta a = Y_o / (1 - \cos\alpha)$

$\alpha = 90^\circ,\ \Delta a = c\,(a + \Delta a),\ c = 0.25\ (\alpha = 90^\circ)$

exclusively employed taking into account that this method enables to obtain the lowest factor of safety in general as shown in the following table[1/] and has been most commonly used from the old time with many actual results as well as to simplify the calculation which is advantaged to repeat for different slip circles until obtained in the smallest value of safety factor with trial and error procedure.

Analysis Method	Shape of Failure Surface	Earth-Pressure on Both Sides of Slice	Factor of Safety
Slip Circle	Circular	Neglect	1.75
Slice	Non-circular	"	1.97
Modified Fellenius	Non-circular	Account	2.15
Simplified Fellenius	"	"	2.48
Wedge	Straight Lines	"	2.08

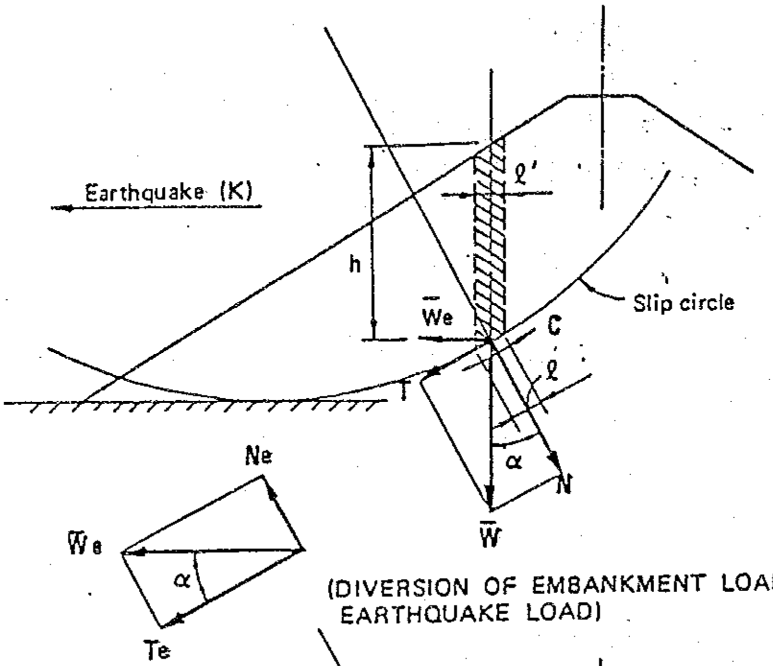
Calculation for the stability analysis is carried out by the slip circle method as shown in Fig. D-7 and the factor of safety is obtained by the following equation.

$$F.S. = \frac{\Sigma[(N-Ne-U)\cdot\tan\phi + C\cdot\ell]}{\Sigma\ (T + Te)}$$

Where, F.S.: factor of safety
N: normal force acting on slip circle of each slice
Ne: normal force of earthquake load acting on slip circle of each slice
U: hydrostatic pressure acting on slip circle of each slice
ϕ: friction angle of materials on slip circle of each slice
C: cohesion of materials on slip circle of each slice
: arc length of slip circle of each slice
T: tangential force acting on slip circle of each slice
Te: tangential force of earthquake load acting on slip circle of each slice

1/ Data source is "Rock-fill dam in Japan" by Dr. Mikuni

Fig. D–7 STABILITY ANALYSIS WITH SLIP CIRCLE METHOD



(DIVERSION OF EMBANKMENT LOAD AND EARTHQUAKE LOAD)

$\bar{W} = h \times \ell' \times \gamma$

ℓ': width of slice

γ : unit weight

$N = \bar{W} \times \cos\alpha$

$T = \bar{W} \times \sin\alpha$

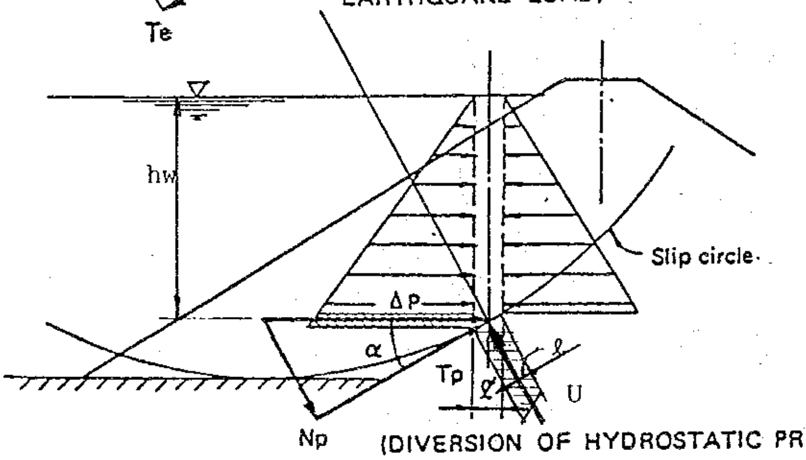
$\ell = \ell'/\cos\alpha$

$\bar{W}e = \bar{W} \times K$

K: seismic coefficie

$Ne = \bar{W}e \times \sin\alpha$

$Te = \bar{W}e \times \cos\alpha$



(DIVERSION OF HYDROSTATIC PRESSURE)

$Np = \Delta P \times \sin\alpha$

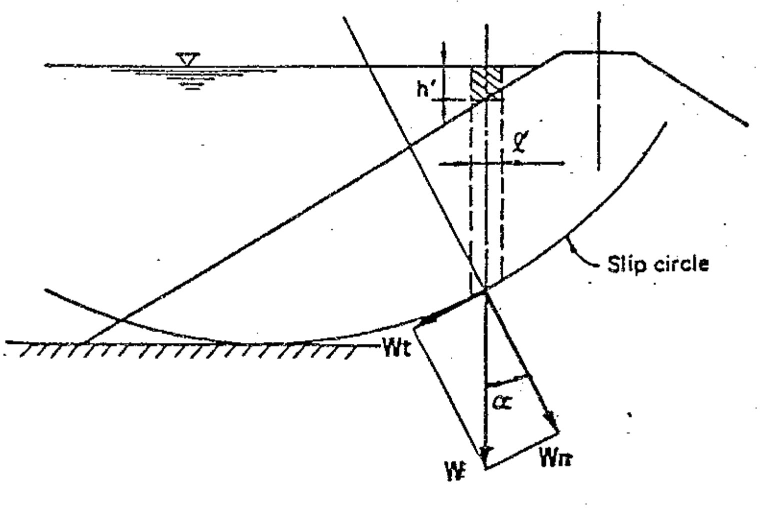
$Tp = \Delta P \times \cos\alpha$

ΔP: difference of hydr
static pressure be
tween both side of
slice

$U = hw \times \ell \times \gamma'$

ℓ : width of slice

γ': unit weight of storage water



(DIVERSION OF SURCHARGE WATER)

$W = h' \times \ell' \times \gamma'$

ℓ': width of slice

γ': unit weight of storage water

$Wn = W \times \cos\alpha$

$Wt = W \times \sin\alpha$

b) Factor of safety

There are various evaluation criteria for the factor of safety on the stability analysis taking into account the conditions of material tests, analysis method, control of embankment and earthquake. However, in this case, a concept of the minimum safety factor which described in the "Design Criteria for Dams" established by Japanese National Committee on Large Dams is introduced as a criteria for the detention dam.

The factor of safety shall be 1.1 with some allowance. The allowance shall be determined according to the nature of materials, the estimated design values, quality control of materials, construction method and calculation method. Generally, 0.1 is accepted as the standard value for allowance.

Consequently, the factor of safety must be not less than 1.2 in any case as shown in the following table.

Reservoir Condition	Seismic Coefficient	Min. Factor of Safety[1/]	Remarks
Immediately After Completion Empty	0.05	1.20	1.2[2/,3/] without earthquake 1.0[3/] with earthquake
After Completion with Full Water Level	0.10	1.20	1.5[2/,3/] without earthquake 1.0[2/], 1.1[3/] with earthquake
After Completion with Middle & Low Water Level	0.10	1.20	1.5[2/,3/] without earthquake 1.0[2/], 1.1[3/] with earthquake
After Completion with High Water Level	Nil	1.20	1.4[2/] without earthquake
After Completion under Drawdown (F.W.S. to L.W.S.)	0.05	1.20	1.2[2/,3/] without earthquake

1/ Proposed criteria for the Wadi Jizzi detention dam
2/ Criteria employed by U.S.Army Corps of Engineers
3/ Criteria employed by State of California Department of Water Resources

c) Case of analysis

The stability analysis against sliding failure for the dam body is carried out for the several cases as shown in Table D-3 taking into account the reservoir conditions and earthquake.

Table D-3. CALCULATION CASES OF STABILITY ANALYSIS

Case	Reservoir Condition	Slope	Seismic Coefficient
1	After Completion F.W.S.1/	Upstream	K = 0.10
2	- do -	Downstream	K = 0.10
3	After Completion M.W.S.2/	Upstream	K = 0.10
4	- do - L.W.S.3/	- do -	K = 0.10
5	- do - H.W.S.4/	- do -	Nil
6	- do -	Downstream	Nil
7	Immediately After Completion, Empty	Upstream	K = 0.05
8	- do -	Downstream	K = 0.05
9	Rapid Drawdown F.W.S. to L.W.S.	Upstream	K = 0.05

1/ full water surface level and EL.163.90 m is adopted
2/ middle water surface level and EL.159.0 m is adopted
3/ low water surface level and EL.154.0 m is adopted
4/ high water surface level and EL.169.20 m is adopted

In case 7 and 8, there is no need to consider the hydrostatic pressure due to the empty of reservoir.

d) Seismic coefficient

From the observed data of earthquake around the Arabian peninsula, there were no epicenters in Oman. However, in the southern part of neighbouring Iran about 250 km away from Sohar, a number of big earthquakes have occurred in the past. From the above fact, it seems to be reasonable that the intensity of 0.10 to be adopted as horizontal seismicity to design the dam body owing to the attenuation of the shock wave over those distance from the epicenters. For high dam, if the natural period of oscillation is long and is almost equal to the predominant period of the ground, the seismic coefficient of dam body may exceed remarkably that of the ground. However, on the fill type dam, the seismic coefficient is generally taken to be equal to the ground seismicity taking into account that the earthquake vibration usually lasts less than one minute. As for the special cases such as immediately after completion of embankment and rapid drawdown, the seismic coefficient of dam body should be employed at one-half of that for normal cases because there is rare probability of the simultaneous occurrence of design earthquake and special cases. And also, in case of high water level corresponding to reservoir water surface at the Probable Maximum Flood with duration less than few hours, the earthquake force is neglected in the stability analysis.

e) Result of analysis

The calculation should be repeated for different slip circles until the smallest value of safety factor is obtained. This procedure involves trial and error, and the electronic computer is advantageously used. The procedure is schematically illustrated in the flow chart as shown in Fig. D-8.

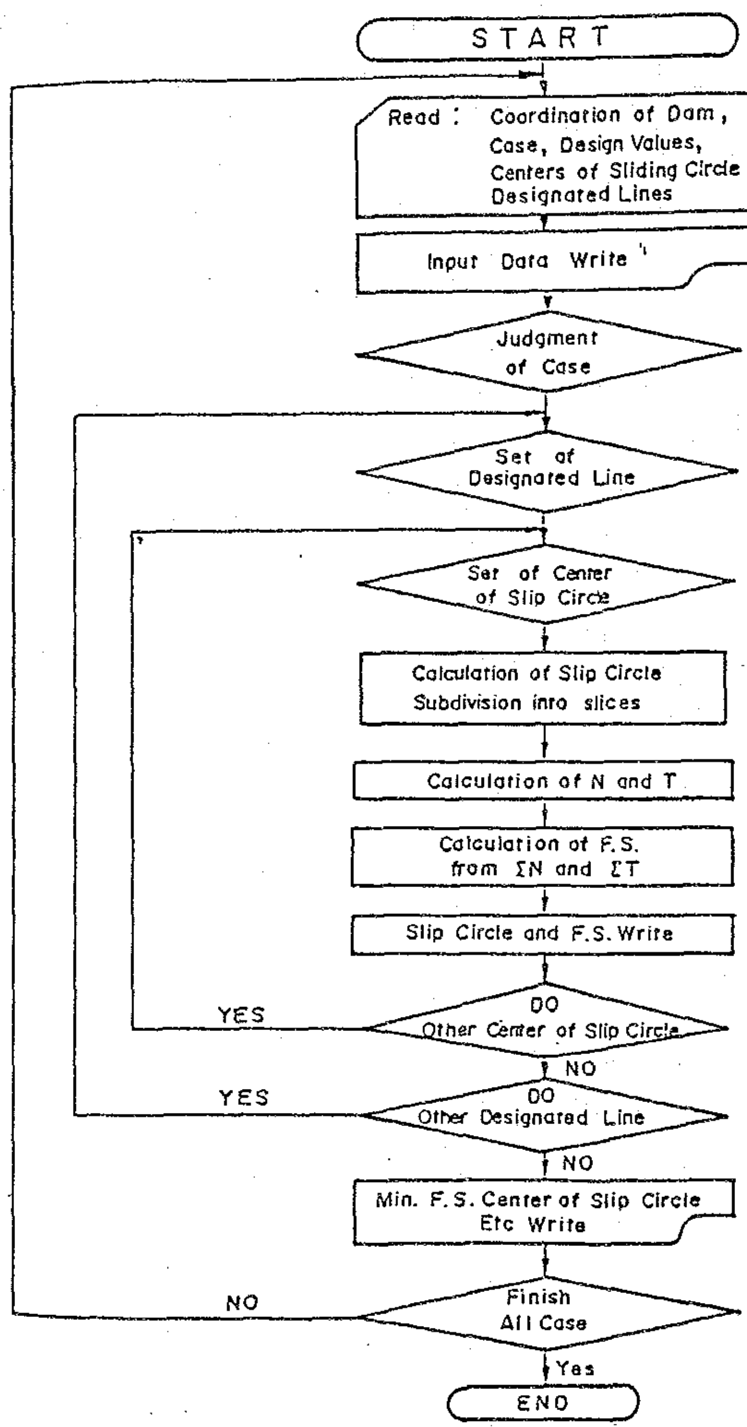
The calculation results for the above cases and reservoir conditions at the detention dam are shown in Fig.D-9, D-10 and are summarized in Table D-4.

Table D-4. RESULTS OF STABILITY ANALYSIS ON DETENTION DAM

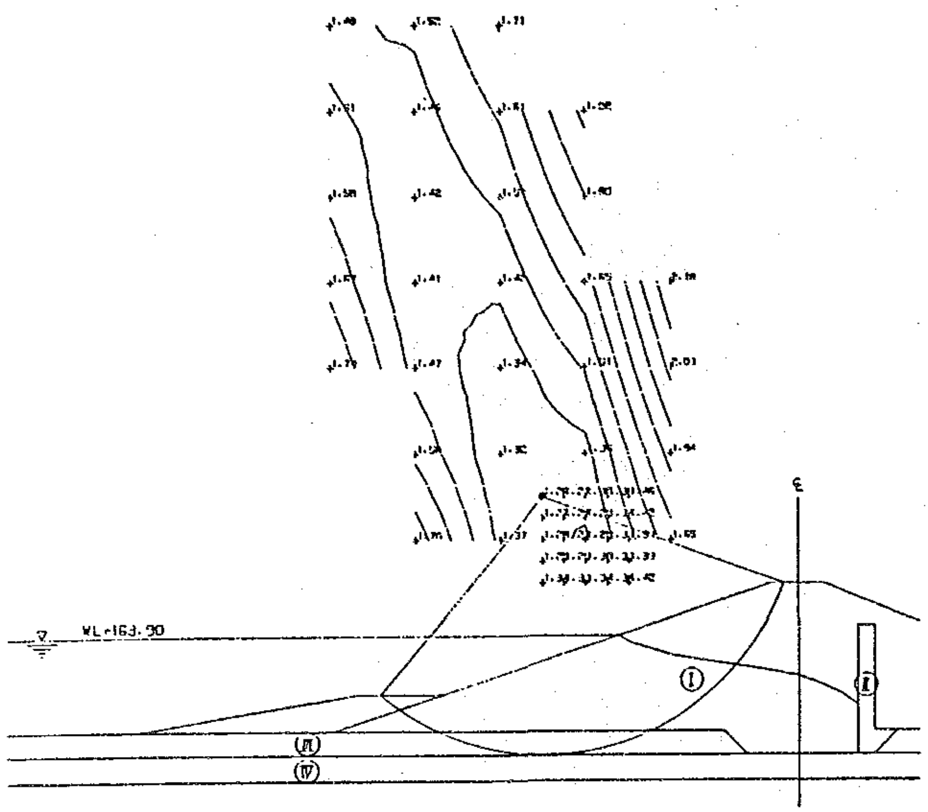
Case	Reservoir Condition	K 1/	Slope	Factor of Safety
1	After Completion F.W.S. 2/	0.10	Upstream	F.S.= 1.265
2	- do -	0.10	Downstream	F.S.= 1.239
3	After Completion M.W.S. 3/	0.10	Upstream	F.S.= 1.327
4	- do - L.W.S. 4/	0.10	- do -	F.S.= 1.551
5	- do - H.W.S. 5/	Nil	- do -	F.S.= 2.326
6	- do -	Nil	Downstream	F.S.= 1.820
7	Immediately After Completion, Empty	0.05	Upstream	F.S.= 2.090
8	- do -	0.05	Downstream	F.S.= 1.569
9	Rapid Drawdown F.W.S. to L.W.S.	0.05	Upstream	F.S.= 1.241

1/ seismic coefficient, K = 0.10 in normal case and K = 0.05 or zero in special case
2/ full water surface level and EL.163.90 m is adopted
3/ middle water surface level and EL.159.0 m is adopted
4/ low water surface level and EL.154.0 m is adopted
5/ high water surface level and EL.169.20 m is adopted

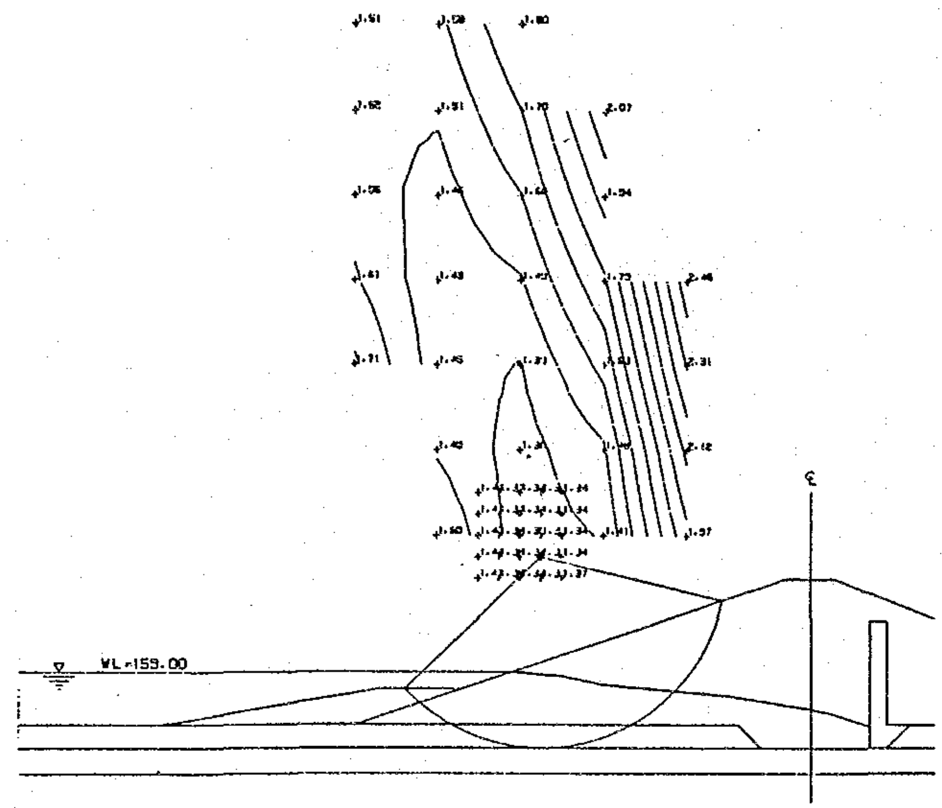
Fig. D-8 FLOW CHART OF STABILITY ANALYSIS BY COMPUTER



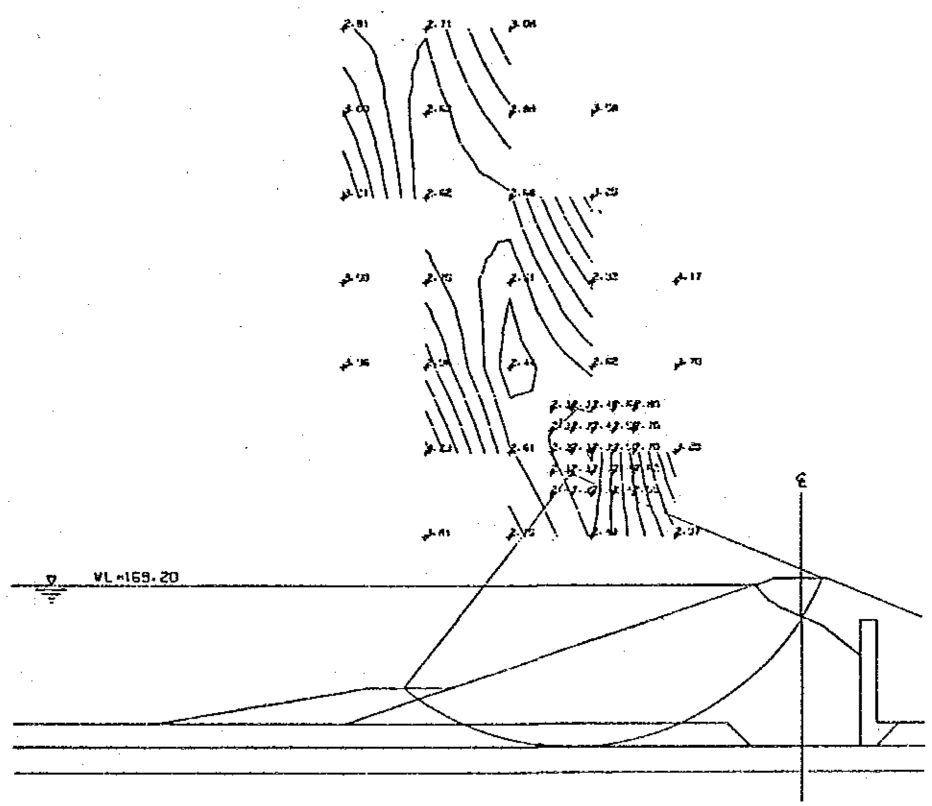
Case 1. After Completion with F.W.S.
Upstream Slope

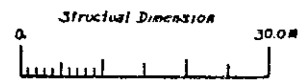


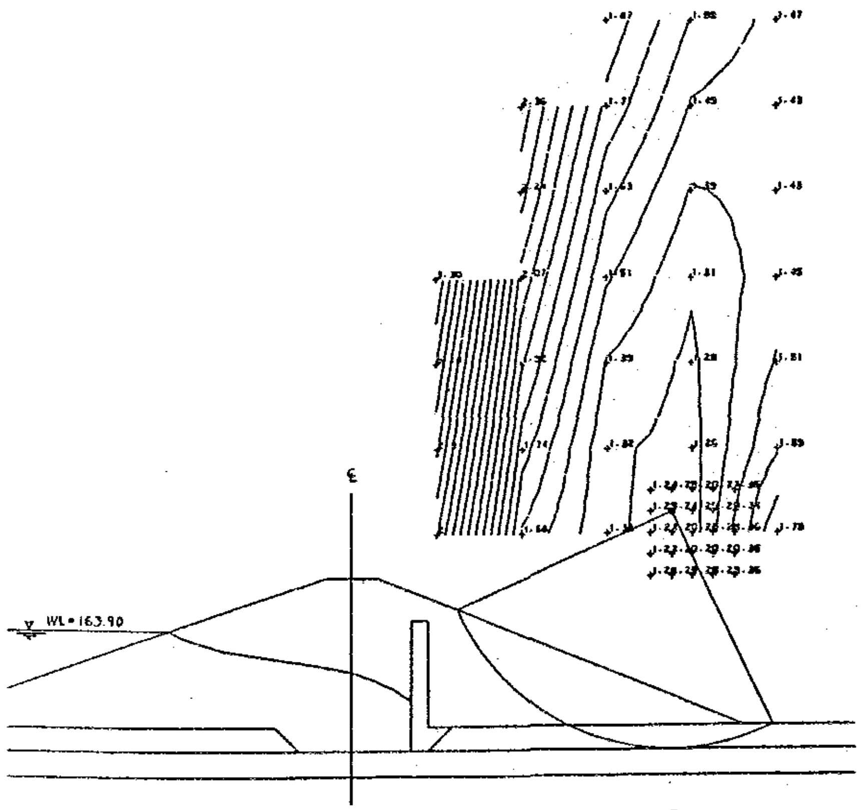
Case 3. After Completion with M.W.S.
Upstream Slope



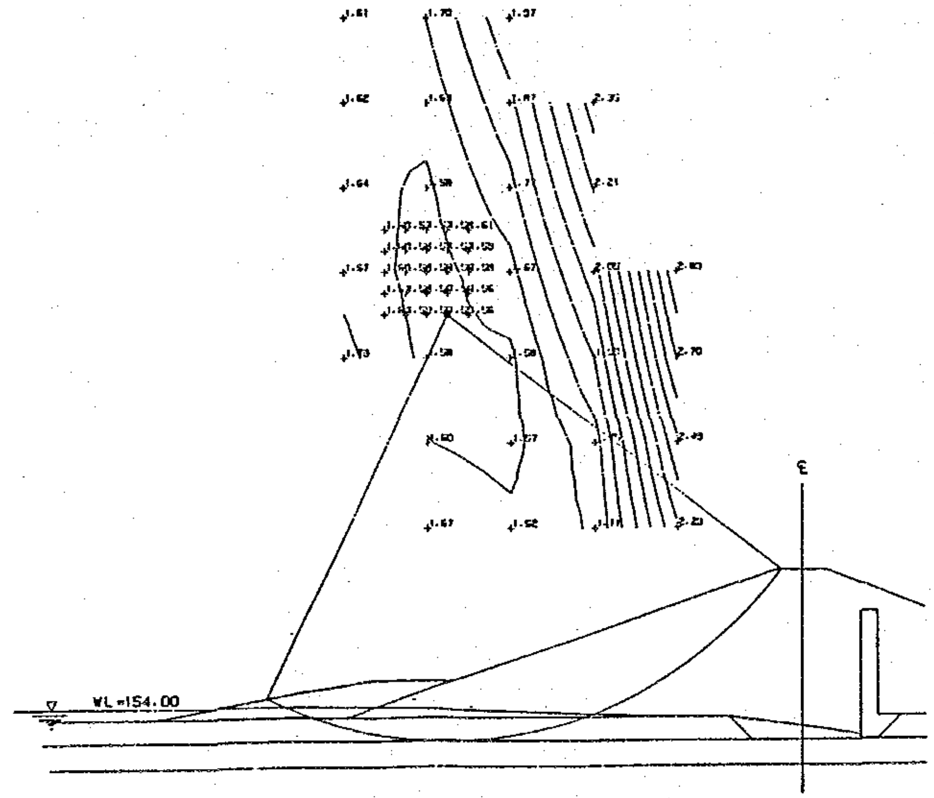
Case 5. After Completion with H.W.S.
Upstream Slope







Case 2. After Completion with F.W.S.
Downstream Slope



Case 4. After Completion with L.W.S.
Upstream Slope

Design Values:

Zone		Density γt (t/m³)	Density γsat (t/m³)	Shearing Strength ϕ (deg.)	Shearing Strength C (t/m²)	K (cm/sec)
I	Sand & Gravel	2.06	2.21	35°00'	0.0	1.9×10^{-4}
II	Filter (Intercepter)	2.16	2.35	35°00'	0.0	5.6×10^{-2}
III	Upper Layer of Foundation	1.63	2.06	32°00'	0.0	-
IV	Lower Layer of Foundation	1.98	2.26	35°00'	0.0	-

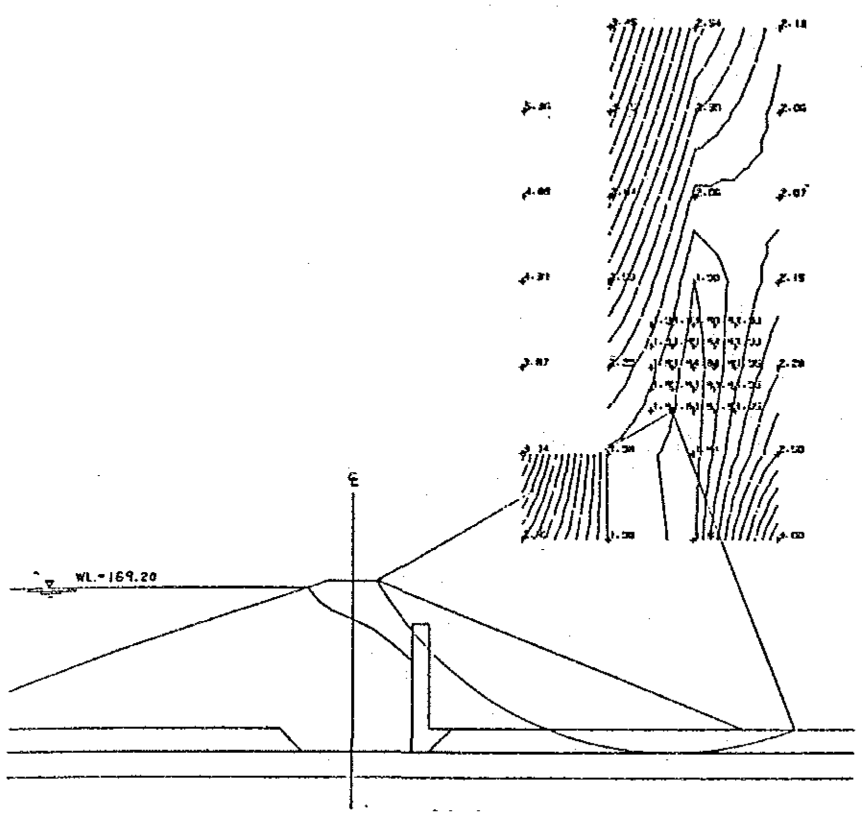
γt: wet density, γsat: saturated density, ϕ: angle of internal friction,
C: cohesion, K: coefficient of permeability

Results of Stability Analysis:

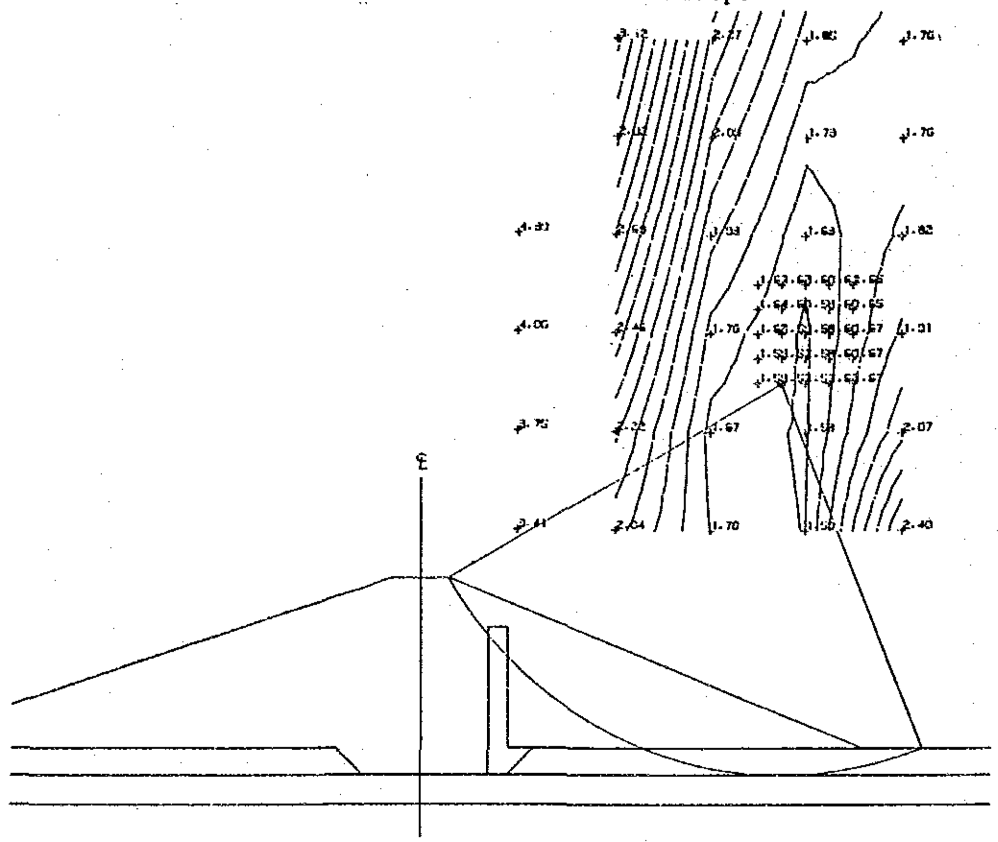
Reservoir Conditions	K	Factor of Safety Upstream Slope	Factor of Safety Downstream Slope
After Completion with F.W.S.	0.10	1.265	1.239
After Completion with M.W.S.	0.10	1.327	-
After Completion with L.W.S.	0.10	1.551	-
After Completion with H.W.S.	Nil	2.326	1.820
Immediately after Completion, Empty	0.05	2.090	1.569
Rapid Drawdown, F.W.S. to L.W.S.	0.05	1.241	-

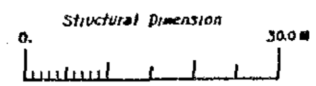
Fig. D-9. RESULTS OF STABILITY ANALYSIS FOR DETENTION DAM (PART 1)

Case 6. After Completion with H.W.S.
Downstream Slope



Case 8. Immediately After Completion
with Empty, Downstream Slope





Design Values:

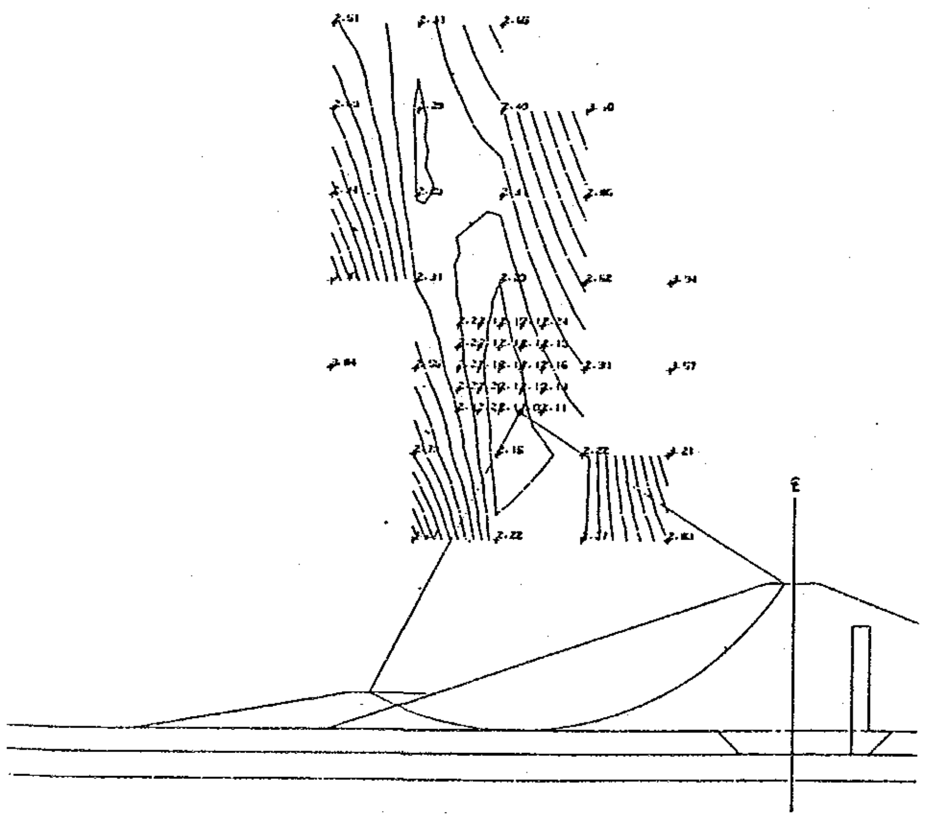
	Zone	Density γt (t/m^3)	Density γsat (t/m^3)	Shearing Strength ϕ (deg.)	Shearing Strength C (t/m^2)	K (cm/sec)
I	Sand & Gravel	2.06	2.21	35°00'	0.0	1.9×10^{-4}
II	Filter (Intercepter)	2.16	2.35	35°00'	0.0	5.6×10^{-2}
III	Upper Layer of Foundation	1.63	2.06	32°00'	0.0	-
IV	Lower Layer of Foundation	1.98	2.26	35°00'	0.0	-

γt: wet density, γsat: saturated density, ϕ: angle of internal friction,
C: cohesion, K: coefficient of permeability

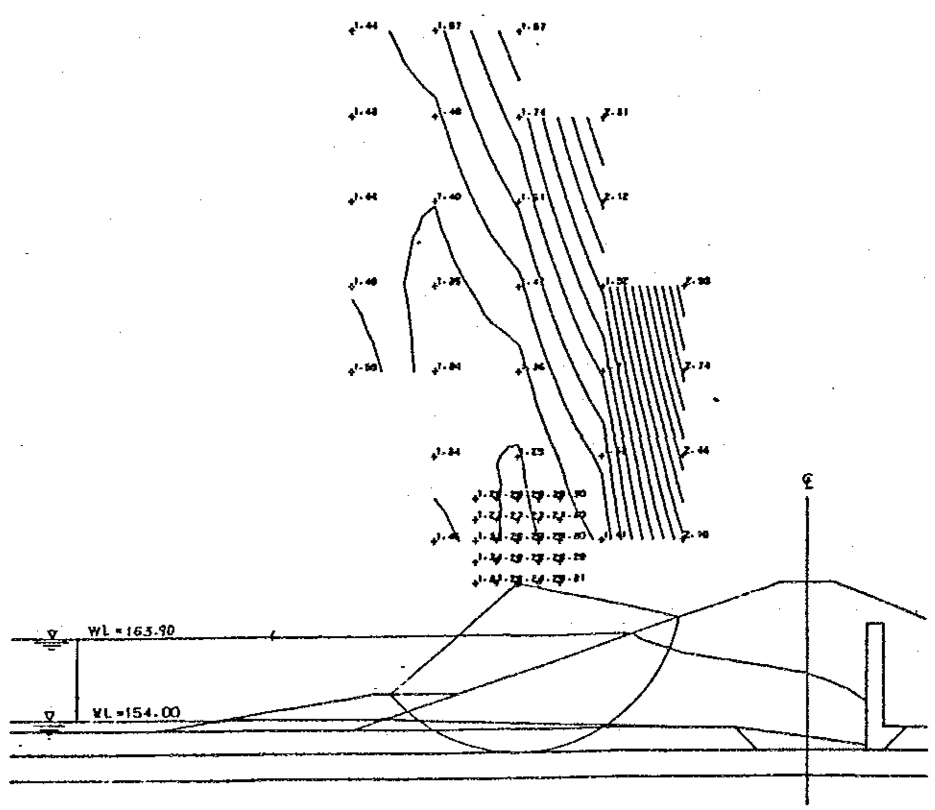
Results of Stability Analysis:

Reservoir Conditions	K	Factor of Safety Upstream Slope	Factor of Safety Downstream Slope
After Completion with F.W.S.	0.10	1.265	1.239
After Completion with M.W.S.	0.10	1.327	-
After Completion with L.W.S.	0.10	1.551	-
After Completion with H.W.S.	Nil	2.326	1.820
Immediately after Completion, Empty	0.05	2.090	1.569
Rapid Drawdown, F.W.S. to L.W.S.	0.05	1.241	-

Fig. D-10. RESULTS OF STABILITY ANALYSIS FOR DETENTION DAM (PART 2)

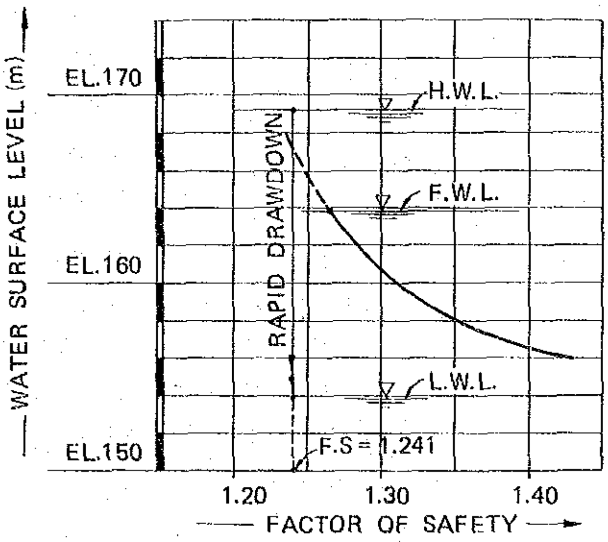


Case 7. Immediately After Completion
with Empty, Upstream Slope



Case 9. Rapid Drawdown F.W.S. to
L.W.S. Upstream Slope

And also, the factor of safety under various water surface elevations in the reservoir is shown in the right figure.



Against the sliding failure, the smallest value of safety factor is 1.241 in upstream with slope of 1 vertical to 3.00 horizontal under the rapid drawdown condition of the reservoir and 1.239 in downstream with slope of 1 vertical to 2.50 horizontal at the full water surface level of EL.163.90 m. The above factor of safety is considered reasonable judging from the proposed criteria, quality and quantity of soil tests and construction conditions. Besides, the above factor of safety will be increased taking into account that the spoil bank fills which are executed at the upstream and downstream sides of the dam body is effective.

4) Surface slope stability

It may sometimes happen that critical slip circle approaches to the surface of dam body in case that the dam body is constructed with cohesionless materials. In this case, the safety factor can be obtained by the following formula.

For upstream slope: $$F.S. = \frac{(1 - K \frac{\gamma sat}{\gamma sub} \cdot \tan\alpha)}{K \cdot \frac{\gamma sat}{\gamma sub} + \tan\alpha} \times \tan\phi$$

For downstream slope: $$F.S. = \frac{1 - K \cdot \tan\alpha}{K + \tan\alpha} \times \tan\phi$$

Where, S.F.: factor of safety

K: seismic coefficient, adopted by 0.10

γsat: saturated density of riprap material, adopted by 2.27 t/m^3 (refer to Table D-2)

γsub: submerged density of riprap material (γsub = γsat - 1), adopted by 1.27 t/m^3

α: tangential value of slope, adopted by 18°26' for upstream slope and 21°48' for downstream slope, respectively

ϕ: internal friction angle of riprap materials, adopted by 38°00' (refer to Table D-2)

Safety factor for the surface sliding is 1.435 in upstream with slope of 1 on 3.00 and 1.500 in downstream with slope of 1 on 2.50. From the above calculation results, the safety factor are more than 1.20 of the allowable one, there is no problem for the sliding failure of surface slopes at the detention dam body.

3.3. Seepage Analysis

1) General

According to the results of geological investigations at the detention dam site, the foundation of the dam has uneven and complicated seepage conditions varying in places and depths as mentioned below. From the above fact, the seepage analysis should be carried out against the piping failure and leakage discharge through the dam body and its foundation as a seepage continuum medium.

The seepage problem such as distributions of velocity, hydraulic gradient and seepage discharge in non-uniform and complicated foundation have been theoretically analyzed in applying the finite element method. The analysis is made on representative cross-section with saturated two-dimensional seepage flow by using electronic computer system.

2) Seepage condition

Geology of the foundation is composed of recent river deposits, terrace deposits and serpentine of the Cretaceous period in descending order. Permeability tests for the above formations were performed in the bore holes and test pits during the detailed design stage and the result is schematically shown in Fig. D-11. From the above figure, the permeability coefficient at the foundation is mainly classified into five groups as very low (less than 10 Lugeon), low (10 to 50 Lugeon), moderate (50 to 100 Lugeon) high (100 to 250 Lugeon) and very high (more than 1000 Lugeon). Although the coefficients are variable according to the places and depths, it seems that they are considerably correlated to depth and the permeability decreases in accordance with increase of depth.

3) Concept of analysis

According to the Darcy's law, the Motion Equation is given by the following formula (1).

$$\left.\begin{aligned} qx &= - Kx \frac{\partial \phi}{\partial x} \\ qy &= - Ky \frac{\partial \phi}{\partial y} \end{aligned}\right] \quad \cdots\cdots\cdots\cdots \quad (1)$$

While, the Equation of Continuity is:

$$- S \frac{\partial \phi}{\partial t} = \frac{\partial qx}{\partial x} + \frac{\partial qy}{\partial y} - q \quad \cdots\cdots\cdots\cdots \quad (2)$$

Therefore, based on the formula (1) and equation (2), the following equation (3) is derived.

$$S \frac{\partial \phi}{\partial t} = Kx \frac{\partial^2 \phi}{\partial x^2} + Ky \frac{\partial^2 \phi}{\partial y^2} + q \quad \cdots\cdots\cdots\cdots \quad (3)$$

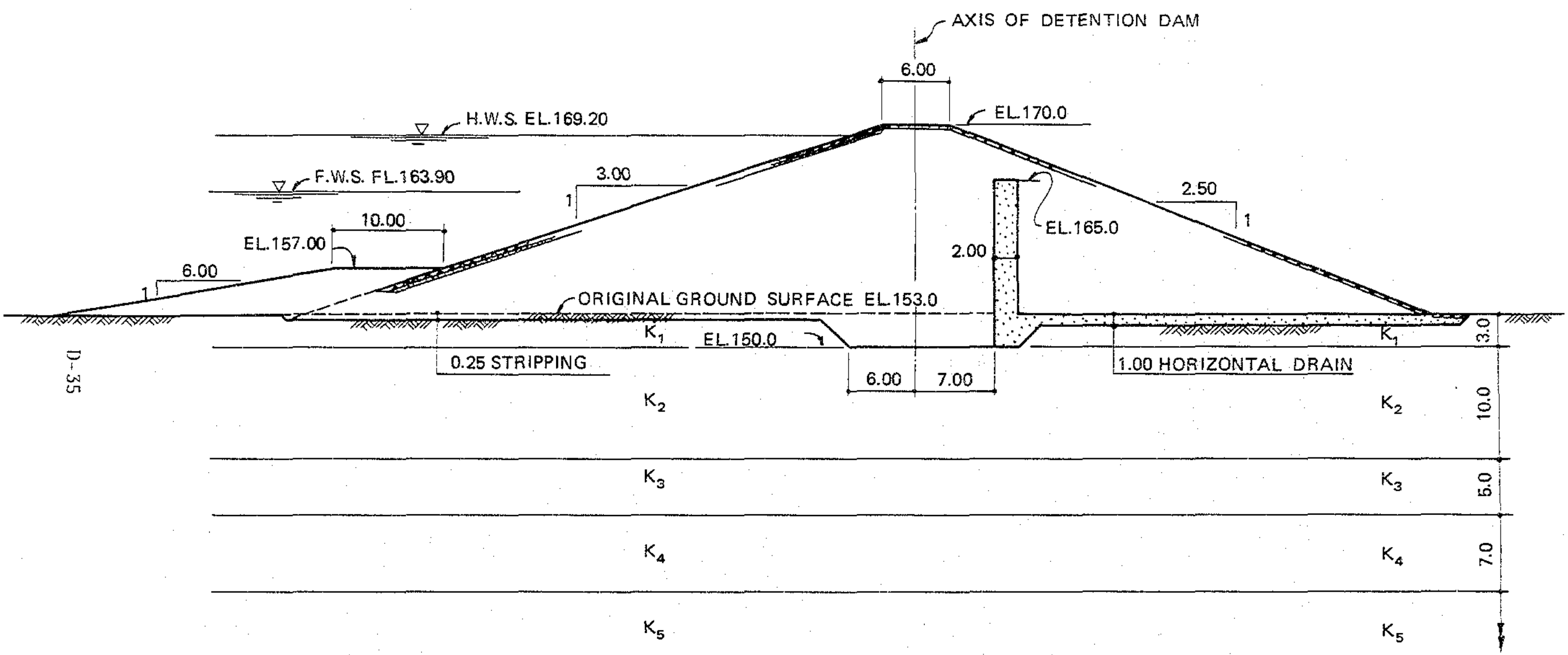
Where, S: a coefficient of storage

Kx, Ky: permeability along the axes

q: additional inflow from other domain

The finite element method is a direct analysis method to substitute the solution of calculus of variation which rules the equation (3) instead of direct solution of the equation (3).

Fig. D–11. EXEMPLARY MODEL OF PERMEABILITY FOR DETENTION DAM



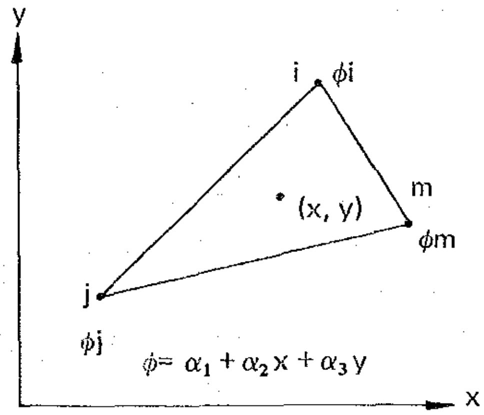
K_1 : Very high (more than 1000 Lugeon), recent river deposits.
K_2 : High (100 to 250 Lugeon), middle terrace deposits.
K_3 : Moderate (50 to 100 Lugeon), middle and upper terrace deposits.
K_4 : Low (10 to 50 Lugeon), upper terrace deposits.
K_5 : Very low (less than 10 Lugeon, upper terrace deposits and serpentine.

The variational function is given by the following equation.

$$E(\phi) = \frac{1}{2}\iint [Kx(\frac{\partial\phi}{\partial x})^2 + Ky(\frac{\partial\phi}{\partial y})^2 + q\phi]dxdy \quad \cdots\cdots\cdots\cdots \quad (4)$$

The whole objective domain is divided into pieces by triangles. The potential of each vertex of triangles is an unknown quantity.

Considering the triangle (given in the illustration) as one element which is picked out of the objective domain, the three unknown quantities at the vertex ϕ are ϕi, ϕj and ϕm, respectively.



Assuming that the potential changes linearly in the domain, ϕ is expressed by equation (5) and it continuously changes between each element.

$$\phi = \alpha_1 + \alpha_2 x + \alpha_3 y \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (5)$$

Therefore, ϕ at each vertex of a triangle is shown as follows:

$$\left.\begin{aligned} \phi i &= \alpha_1 + \alpha_2 xi + \alpha_3 yi \\ \phi j &= \alpha_1 + \alpha_2 xj + \alpha_3 yj \\ \phi m &= \alpha_1 + \alpha_2 xm + \alpha_3 ym \end{aligned}\right\} \cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (6)$$

In solving the above equation (6), the following assumptions are made.

$$\begin{aligned} \Delta &= \text{(twice of the area of triangle)} \\ &= xiyj - xjyi + xjym - xmyj + xmyi - xiym \quad \cdots\cdots \quad (7) \end{aligned}$$

$$\left.\begin{aligned} S_{11} &= (xjym - xmyi)/\Delta, \quad S_{21} = (yi - ym)/\Delta, \quad S_{31} = (xm - xj)/\Delta \\ S_{12} &= (xmyi - xiym)/\Delta, \quad S_{22} = (ym - yi)/\Delta, \quad S_{32} = (xi - xm)/\Delta \\ S_{13} &= (xiyj - xjyi)/\Delta, \quad S_{23} = (yi - yj)/\Delta, \quad S_{33} = (xj - xi)/\Delta \end{aligned}\right\} \cdots\cdots \quad (8)$$

Based on the equation (7) and (8), the equation (5) is given as follows:

$$\phi = (S_{11} + S_{21}x + S_{31}y)\phi i + (S_{12} + S_{22}x + S_{23}y)\phi j + (S_{13} + S_{23}x + S_{33}y)\phi m \quad \cdots\cdots (9)$$

According to the equation (5), the following equation (10) is available.

$$\left.\begin{aligned} \frac{\partial\phi}{\partial x} &= S_{21}\phi i + S_{22}\phi j + S_{23}\phi m \\ \frac{\partial\phi}{\partial y} &= S_{31}\phi i + S_{32}\phi j + S_{33}\phi m \end{aligned}\right\} \quad \cdots\cdots (10)$$

From the variation principle, the solution of equation (3) is equal to that of potential distribution when it makes the functional minimum in the equation (4). In order to estimate the condition in which $E(\phi)$ is minimum, the following equation (11) shall be solved with the condition in which $\frac{E(\phi)}{\partial(\phi)}$ is zero at each node (nodal point).

$$\frac{\partial E(\phi_1, \phi_2 \ldots \phi_n)}{\partial\phi i} = \iint\left[\frac{\partial}{\partial\phi i}\left(\frac{\partial\phi}{\partial x}\right)\left(Kx\frac{\partial\phi}{\partial x}\right) + \frac{\partial}{\partial\phi i}\left(\frac{\partial\phi}{\partial y}\right)\left(Ky\frac{\partial\phi}{\partial y}\right) + q\frac{\partial\phi}{\partial\phi i}\right]dxdy = 0 \quad \cdots\cdots (11)$$

In the equation (11), the quantities of $\frac{\partial}{\partial\phi i}\left(\frac{\partial\phi}{\partial x}\right)$, $\frac{\partial\phi}{\partial\phi i}\left(\frac{\partial\phi}{\partial y}\right)$ and $\frac{\partial\phi}{\partial\phi i}$ are S_{21}, S_{31} and $S_{11} + S_{21}x + S_{31}y$, respectively. And they have no any unknown quantity. And also, the quantities of $\frac{\partial\phi}{\partial x}$ and $\frac{\partial\phi}{\partial y}$ are expressed by the linear function of unknown quantities of ϕi, ϕj and ϕm in the equation (10). Therefore, the steady term in the equation (11) is expressed by the linear equation composed of those unknown quantities. The equation of $\phi\frac{\partial}{\partial\phi i}$ and $\Phi\frac{\partial\phi}{\partial\phi i}$ in the unsteady term are estimated by the following equations (12) and (13).

$$\int\phi\frac{\partial\phi}{\partial\phi i}dxdy = \int[\{(S_{11} + S_{21}x + S_{31}y)\phi i + (S_{21} + S_{22}x + S_{32}y)\phi j + (S_{31} + S_{32}x + S_{33}y)\phi m\} \times (S_{11} + S_{21}x + S_{31}y)]dxdy \quad ..(12)$$

$$\int\Phi\frac{\partial}{\partial\phi i}dxdy = \int[\{S_{11} + S_{21}x + S_{31}y)\Phi i + (S_{21} + S_{22}x + S_{32}y)\Phi j + (S_{31} + S_{32}x + S_{33}y)\Phi m\} \times (S_{11} + S_{21}x + S_{31}y)\}dxdy \quad ..(13)$$

Arranging the above equation (12) and (13), the following equations (14) and (15) are obtained.

$$\int \phi \frac{\partial \phi}{\partial \phi i} dxdy = \frac{\Delta}{2} \left(\frac{1}{6} \phi i + \frac{1}{12} \phi j + \frac{1}{12} \phi m \right) \quad \cdots\cdots\cdots\cdots \quad (14)$$

$$\int \Phi \frac{\partial \phi}{\partial \phi i} dxdy = \frac{\Delta}{2} \left(\frac{1}{6} \Phi i + \frac{1}{12} \Phi j + \frac{1}{12} \Phi m \right) \quad \cdots\cdots\cdots\cdots \quad (15)$$

The equation (11) is effected at the node (nodal point) where the quantity is unknown. And it is a linear equation with this unknown quantity.

The quantity is, therefore, computed by solving those simultaneous equations. The following matrix is available applying the equation (11) to the triangle elements.

$$\Delta \begin{bmatrix} KxS_{21}^2 + KyS_{31}^2 & KxS_{21}S_{22} + KyS_{31}S_{32} & KxS_{21}S_{23} + KyS_{31}S_{33} \\ & KxS_{22}^2 + KyS_{32}^2 & KxS_{22}S_{23} + KyS_{32}S_{33} \\ \text{Symmetrical} & & KxS_{23}^2 + KyS_{33}^2 \end{bmatrix}$$

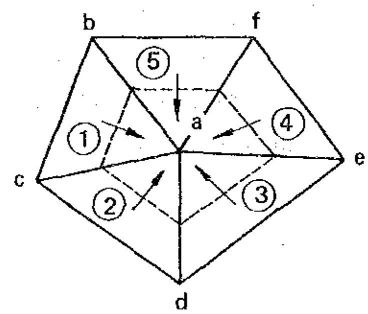
$$\times \begin{Bmatrix} \phi i \\ \phi j \\ \phi m \end{Bmatrix} = \begin{Bmatrix} qi \\ qj \\ qm \end{Bmatrix} \quad \cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots\cdots \quad (16)$$

The element equation (16) are made for all the elements, the permeability matrix is made for the whole analyzed domain applying the element equations.

In this manner, the simultaneous linear equations with the unknowns (ϕ) are solved and the distribution of potential (ϕ) is obtained.

It is clear through the equation (12) that the quantity of flow is already known at the node where the potential is to be obtained, the quantity of flow at the node where the potential is already given as the boundary condition can be obtained through the product of matrix. In short, it is a real meaning of the aforesaid analysis to estimate the distribution of water head at the node under given boundary condition and given distribution of permeability in order to establish the water balance at the environs of every node (viz, the Equation of Continuity is satisfied).

For instance in the right figure, the water head at nodes a, b ... and f are determined so that the following two factors can be equal in quality. Where, a factor is a half of the flow passing through the sides $\overline{bc}$, $\overline{cd}$, $\overline{de}$, $\overline{ef}$ and $\overline{fb}$ which encircle node a. Another factor is the amount of water which flows into or out of node a.



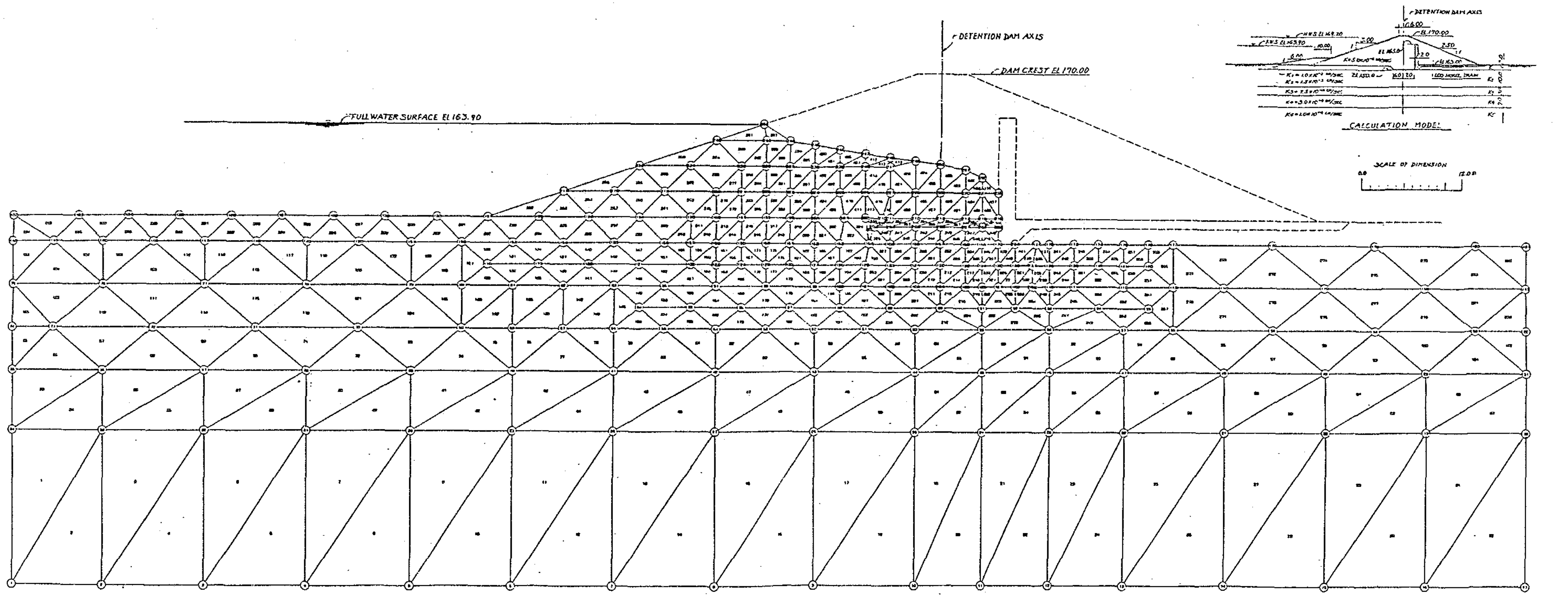
4) Condition of analysis

The seepage analysis for the detention dam is carried out on representative cross-section under the following conditions by using the finite element method.

- From the results of permeability tests, the calculation model of the detention dam to be applied for the seepage analysis is assumed as shown in Fig. D-11.

- Form of an element which possesses certain inherent advantage is a Cartesian coordinate element as shown in Fig. D-12.

Fig. D–12 DIVISION INTO ELEMENTS AND ARRANGEMENT OF NODES FOR DETENTION DAM



- Division into the elements and arrangement of the nodes is shown in Fig. D-12.

- Situation of the dam is that the embankment becomes stable some years after completion and the reservoir is at the full water level.

- Calculation for the hydraulic gradient, seepage velocity and leakage discharge is made for the following 3 cases.

  Case 1 ... Base of trench EL.152.0 m (in river deposit)
  Case 2 ... Base of trench EL.150.0 m (on terrace deposit)
  Case 3 ... Base of trench EL.147.5 m (in terrace deposit)

- Input constants of the permeability coefficient for the embankment and foundation materials at the detention dam are as follows:

Material			Permeability Coefficient	
Sand and gravel zone 1/			$K = 5x10^{-4}$ cm/sec,	50 Lugeon
Foundation 2/,	River Deposits,	Very High	$K_1 = 1x10-2$ "	1,000 "
"	Terrace Deposits,	High	$K_2 = 1.5x10^{-3}$ "	150 "
"	"	Moderate	$K_3 = 7.5x10^{-4}$ "	75 "
"	"	Low	$K_4 = 3.0x10^{-4}$ "	30 "
"	"	Very Low	$K_5 = 1x10^{-4}$ "	10 "

1/ This value is estimated based on the results of soil tests
2/ These values are estimated based on the data of geological investigations

5) Results of analysis

The calculation results for the hydraulic gradient, seepage velocity and leakage discharge on representative points under the above-mentioned conditions are shown in Table D-5.

Table D-5. RESULTS OF SEEPAGE ANALYSIS

Item		Case 1	Case 2	Case 3
Hydraulic Gradient				
Foundation	Upstream	0.245 (193)	0.270 (192)	0.460 (192)
	Trench Base	1.006 (349)	0.574 (336)	0.388 (201)
	Downstream	0.462 (439)	0.367 (439)	0.243 (226)
Dam Body		1.247 (359)	1.042 (349)	1.025 (349)
Seepage Velocity				
Foundation	Upstream	2.360 (323)	3.540 (326)	4.590 (323)
($\times 10^{-3}$ cm/sec)	Trench Base	10.100 (349)	0.862 (336)	0.582 (201)
	Downstream	0.693 (439)	0.564 (439)	0.364 (226)
Dam Body		4.160 (357)	6.830 (350)	4.460 (350)
Leakage Discharge	Dam Body	17.003	18.075	19.730
(m^3/day/m)	Foundation	6.969	5.306	4.024
	Total	23.972	23.381	23.754

Note: 1) The figures in a parenthesis express the number of elements.

2) Leakage discharges from the dam body include the leakage amount through the trench portion.

And also, the vectors of leakage discharge with equi-potential lines at the detention dam (Case 2) are shown in Fig. D-13.

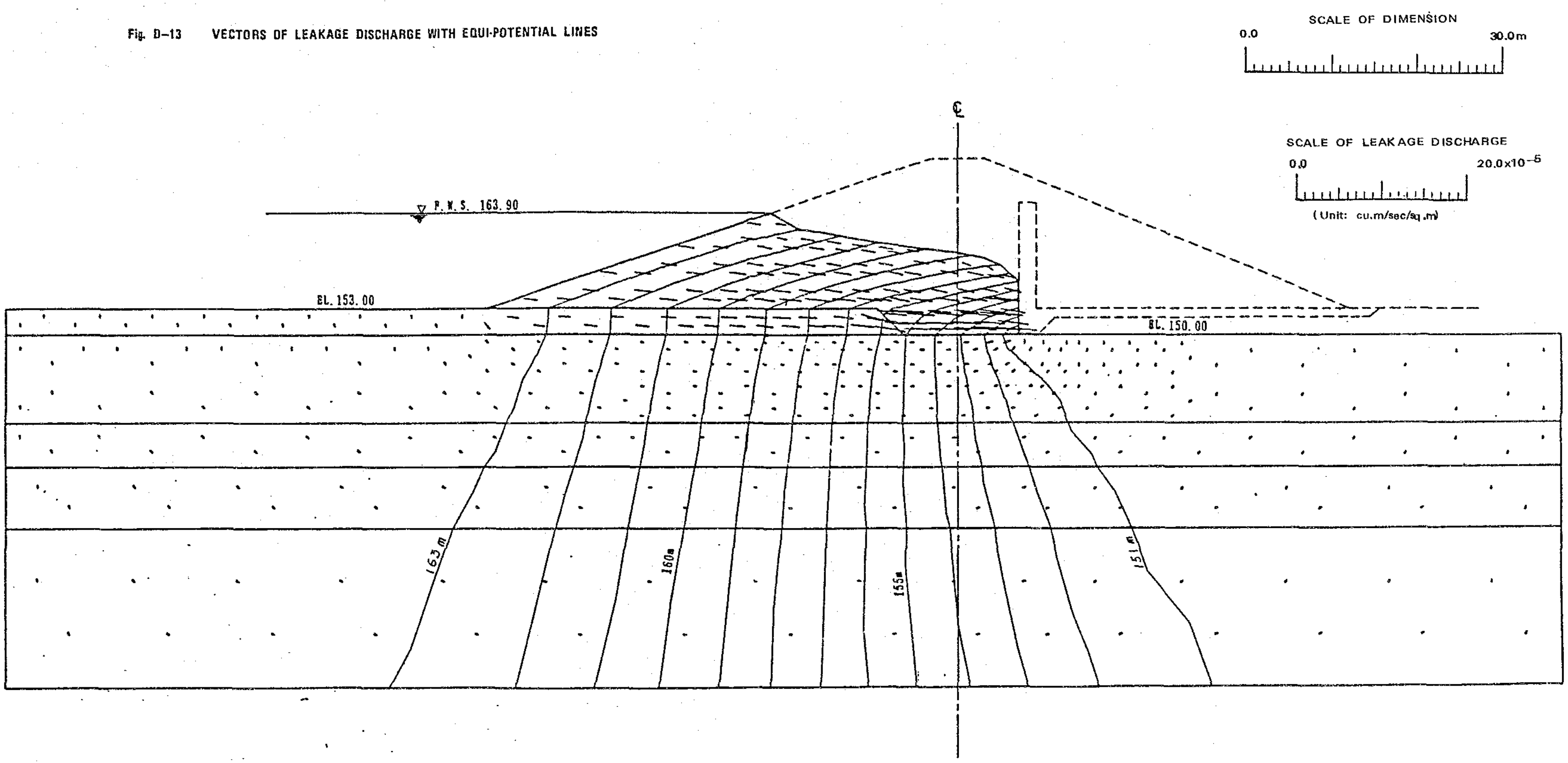
The effect of trench depth at the detention dam is shown in the following table.

Case	Trench Depth (m)	Max.Hydraulic Gradient	Max.Seepage Velocity (cm)	Leakage[1/] Discharge (m^3/day/m)
1	1.0	1.006	1.01×10^{-2}	23.972
2	3.0	0.574	3.54×10^{-3}	23.381
3	5.5	0.388	4.59×10^{-3}	23.754

In case 3, the leakage discharge through the dam body and foundation is slightly increased than that of Case 2 due to the anisotropic effect on the embankment materials. In the calculation of leakage discharge, the permeability coefficient of sand and gravel materials obtained by the soil tests should be modified by using the equivalent coefficient of $\sqrt{Kv \cdot Kh}$ taking into account the

1/ Total discharge leaks from the dam body and the foundation

Fig. D-13 VECTORS OF LEAKAGE DISCHARGE WITH EQUI-POTENTIAL LINES



permeability in anisotropic media and the result is 2.5×10^{-3} cm/sec under the condition of Kh = 25 Kv (refer to the following paragraph). On the other hand, the permeability coefficient of middle terrace deposits (K2 group) is represented by 1.5×10^{-3} cm/sec through the permeability tests in drilled holes.

In case that the middle terrace deposits is replaced by the sand and gravel fill, the leakage discharge will be increased gradually in accordance with increase of the depth replaced.

6) Evaluation of analysis

a) Safety against seepage failure

As a considerable amount of seepage water passes through the dam foundation, seepage water shall be controlled to prevent the piping failure in the foundation.

The evaluation method of piping failure may be classified into two groups, one is derived from the critical velocity based on the diameter of particle of the foundation material and the other derived from the critical hydraulic gradient.

Critical velocity

According to the Justin, the critical velocity for uniform particle material composed of the foundation is given by the following equation.

$$Vc = \sqrt{W g / A \cdot \gamma w}$$

Where, Vc: critical velocity

W: unit submerged weight of particle

g: acceleration of gravity (980 cm/sec^2)

A: sectional area of particle

γw: unit weight of water

In solving the above equation, a diameter of 0.074 mm which corresponds to the boundary value between silt and sand under the Unified Soil Classification System is assumed as an idealized particle diameter in the foundation. The critical velocity in the foundation is obtained as follows:

Sectional-area of particle: $A = (0.0074/2)^2 \cdot \pi = 4.30 \times 10^{-5} cm^2$

Unit submerged weight of particle: $W = 4/3 \cdot \pi\ (0.0074/2)^3\ \gamma sub$

($\gamma sub = Gs-1$, Gs: specific gravity, adopted by average value of 2.81)

$$Vc = \sqrt{3.84 \times 10^{-7} \times 980/4.30 \times 10^{-5} \times 1.0} = 8.75 \text{ cm/sec}$$

Assuming that the effective porosity in the foundation of the detention dam is 1.0 percent in safety side, the apparent critical velocity (Vca) is obtained as follows:

$$Vca = 8.75 \times 10^{-2} \text{ cm/sec}$$

From the results of seepage analysis, the maximum seepage velocity at the detention dam is 1.01×10^{-2} cm/sec in case 1 (3.54×10^{-3} cm/sec in case 2) and appears in the foundation around the trench base and this value is less than the critical velocity of 8.75×10^{-2} cm/sec. Consequently, the foundation has sufficient safety against the piping failure in view of the critical velocity. However, in this method, it is actually difficult to determine an idealized diameter of particle in the foundation due to the natural foundation consists of various diameter of the particles.

Critical hydraulic gradient

Usually, the evaluation of piping failure is made based on the critical hydraulic gradient which is derived from the potential gradient of flow-lines as shown in the following equation in order to eliminate the defect of critical velocity method.

$$\gamma w \frac{\partial \phi}{\partial z} = \gamma w \cdot ic \leqq W = \frac{Gs - 1}{1 + e} \cdot \gamma W$$

where, γw: unit weight of water

$\partial \phi / \partial z$: potential gradient of flow line

ic: critical hydraulic gradient

W: unit submerged weight of particle

Gs: specific gravity of particle, adopted by average value of 2.81 (results of soil tests)

e: void ratio of foundation material, adopted by average value of 0.580 (results of soil tests)

Since the density test of undisturbed samples for the foundation materials has not been executed, the unit submerged weight was estimated from the testing data of specific gravity and void ratio. Substituting these values into the above equation, the critical hydraulic gradient of the foundation materials at the detention dam can be obtained assuming the same safety factor of 1.2 as the stability analysis of the dam body as follows.

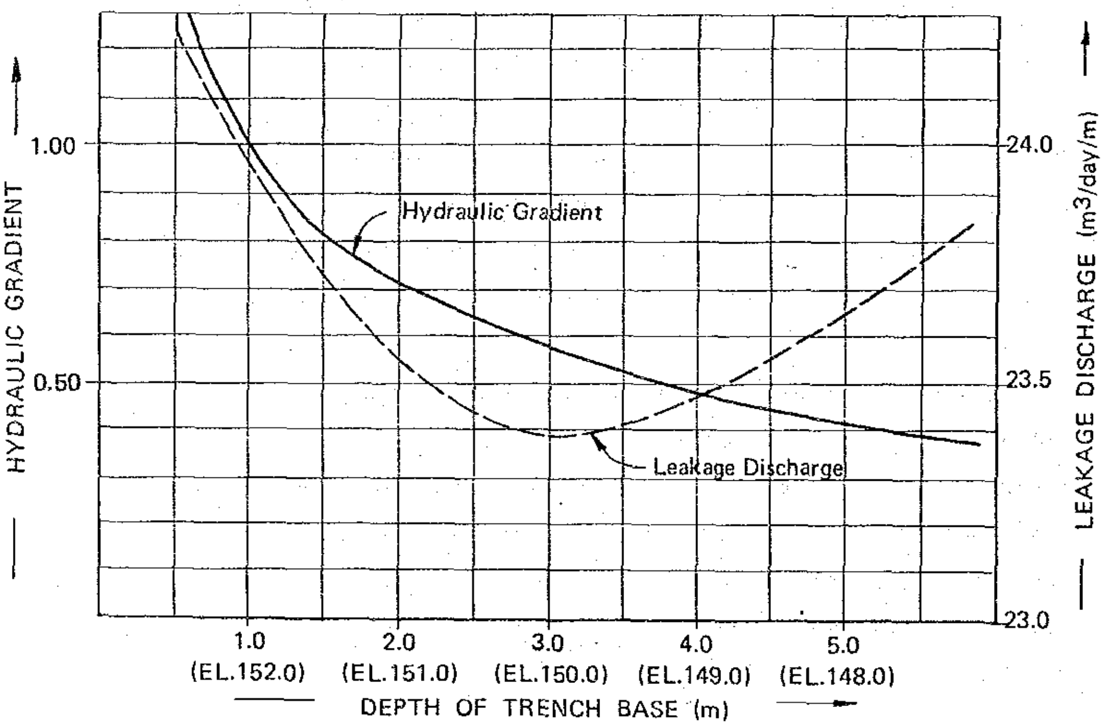
$$ic = \frac{1}{1.2} \cdot \frac{2.82 - 1.0}{1 + 0.58} = 0.955^{1/} \quad (GS = 2.81,\ e = 0.58)$$

The evaluation in terms of critical hydraulic gradient is made based on the assumed depth of trench base and the results is shown in Fig. D-14.

1/ In general, the critical hydraulic gradient of 0.5 to 0.8 is employed in case of fine cohesionless materials.

From the above figure, the trench base should be located at EL.150.0 m (Case 2, 3.0 m depth) taking into account the permeability conditions at the foundation, hydraulic gradient, critical velocity around the trench base and the possible piping phenomenon.

Fig. D–14. RELATION CURVE BETWEEN HYDRAULIC GRADIENT, DISCHARGE PER UNIT LENGTH AND DEPTH OF TRENCH BASE



Judging from the effect on the hydraulic gradient by depth of the trench base, it seems that there is no probability of the piping failure in the foundation because the hydraulic gradients appeared around the trench base are less than the critical one when the trench base is located at EL.150.0 m. Moreover, the element having the maximum value is solitarily distributed and the hydraulic gradients of the surrounding elements are remarkably less than that of the maximum.

b) Leakage discharge

According to the result of seepage analysis, the leakage discharge per unit length through dam body[1/] and foundation is estimated under the reservoir condition of full water level and the result is shown in Fig. D-14.

From the above figure, it can be understood that the trench base should be located at EL.150.0 m in order to minimize the leakage discharge at the detention dam. The total amount of leakage discharge from the dam body and foundation at the full water level is obtained by multiplying the leakage discharge per unit length by equivalent of dam length[2/] and the results are shown in the following table with the ratio to the total storage capacity of the detention dam.

Case	Trench Depth	Leakage Discharge Dam Body	Foundation	Total	Corresponding Ratio
	(m)	(m^3/day)	(m^3/day)	(m^3/day)	(%)
1	1.0	8,924.7	3,658.0	12,582.7	0.233
2	3.0	9,487.4	2,785.1	12,272.5	0.227
3	5.5	10,356.1	2,112.2	12,468.3	0.231

Judging from the above table, it seems that there is no leakage problem at the detention dam when the trench base is located at EL.150.0 m taking into account the purpose of detention dam, allowable leakage ratio of an ordinary storage reservoir which is generally 0.05 percent for the total storage capacity of the reservoir in Japan and the hydraulic gradient around the trench.

1/ The leakage discharge through the dam body is calculated under the condition of sand and gravel zone as effectively functioning semi-pervious zone.

2/ Equivalent of dam length means two-thirds crest length in axial direction of the dam.

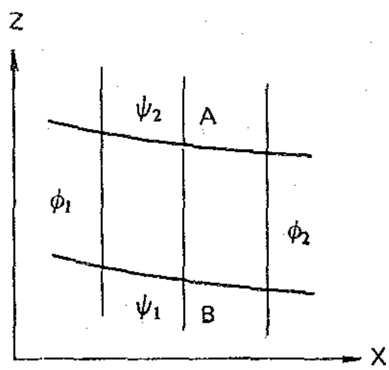
3.4. Leakage through Dam Body and Foundation

The quantity of leakage through the dam body and the foundation is analyzed as saturated two-dimensional seepage through continuum applying the finite element method at representative cross section of the detention dam. The details of the finite element method is described in the paragraph 3.3 Seepage Analysis.

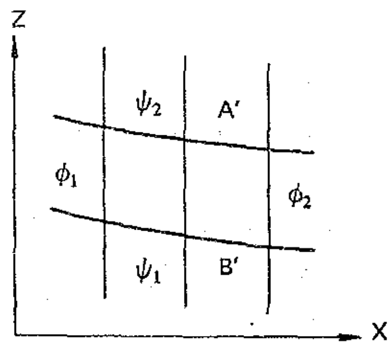
Since the compacted embankment is anisotropic, the seepage analysis should be made on the transformed system mentioned in the paragraph of phreatic line in this Chapter.

And also, the coefficient of permeability in anisotropic media is converted to the equivalent coefficient of permeability of $\sqrt{Kv \cdot Kh}$.

The above convert-ratio of $\sqrt{Kv \cdot Kh}$ is derived by the following procedure. At the prismatic element surrounded by equi-potential line of ϕ_1, ϕ_2 and flow line of ψ_1, ψ_2 on the transformed section as shown in the right figure, when taking Kx = K max. and Kz = k min, the sectional dimension of $\overline{AB}$ in parallel with the direction Z can be expressed as follow in the original section.



TRANSFORMED SECTION



ORIGINAL SECTION

$$\overline{A'B'} = \overline{AB} \sqrt{k\ min\ /\ k\ max}$$

The discharge flowing through $\overline{AB}$ should be equal to those through $\overline{A'B'}$ consequently, the discharge (q) flowing through the transformed section is shown in the following equation.

$$q = \overline{AB} \cdot k \text{ grad } h$$

In the original section, since the permeability coefficient in the direction x is k max, the discharge (q) can be shown as follows:

$$q = \overline{A'B'}\,k\,\max\ \mathrm{grad}\ h = k\,\max\sqrt{\frac{k\,\min}{k\,\max}}\ \overline{AB}\ \mathrm{grad}\ h$$

Therefore, the convert-ratio for equivalent coefficient of permeability is shown as $\sqrt{k\,\max \cdot k\,\min}$ (k max and k min is permeability coefficient of horizontal (Kh) and vertical (Kv) directions, respectively).

From the above analysis, the total amount of leakage through the dam body and the foundation is equal to 12,272.5 cu.m/day (dam body; 9,487.4 and foundation 2,785.1 cu.m/day) at the full water level in the reservoir and this quantity may be equivalent to approximate 0.227 percent of the total storage capacity of the detention dam, and this value is about 4.5 times of the allowable leakage amount at ordinary storage reservoir, which is generally 0.05 percent for the total storage capacity of the reservoir in Japan. There is no leakage problem at the detention dam (refer to 3.3 Seepage Analysis in this Report).

3.5. Extra Banking

The settlement of the fill dam is mainly caused by consolidation of the embankment materials. Since the detention dam is embanked by the sand and gravel materials, the consolidation of these materials will not take place after completion of the embankment. Therefore, very little settlement is expected in the dam body caused by the consolidation after completion of the embankment.

In such case, no theoretical analysis method can be applied for the settlement of the dam body. Accordingly, the following experimental formula is employed to presume the settlement of dam body. The formula is derived from the observed data of final settlement on the rock-fill dams. The final settlement referred to herein means the integrated settlement value until the annual settlement can reach below 0.02 percent of the dam height.

$$S = 0.001\ H^{3/2}\ (m)$$

Where, S: settlement of the dam body

H: height of the dam, adopted by 20 m

The settlement after completion of the embankment is computed to be about 0.1 m from the above formula, and another 0.2 m is added to this value considering the possible settlement in the foundation and spectacle of dam crest. The maximum extra banking is, therefore, fixed at 0.3 m corresponding to 1.5 percent of the dam height.

The height of extra banking is determined at zero meter on both abutment and 0.3 m on river-bed portion, and the profile of dam crest forms in trapezoidal shape.

3.6. Filter and Riprap

1) Filter

a) Filter function

In order to prevent the fine materials washing out from the sand and gravel zone, and to secure seepage water flowing-out from the zone, an intercepter (vertical filter zone) is planned in the downstream side of sand and gravel zone. The filter materials should be satisfied the relation of gradation between the intercepter and that of the sand and gravel zone as follows:

- $\dfrac{\text{15\% grain size of intercepter materials}}{\text{15\% grain size of sand and gravel materials } \underline{1/}} > 5$

- $\dfrac{\text{15\% grain size of intercepter materials}}{\text{85\% grain size of sand and gravel materials } \underline{1/}} < 5$

1/ 15% or 85% grain size means grain size of materials finer than respective percentage by weight of total volume of the materials.

- It is desirable that gradation curve of intercepter materials is approximately parallel to that of the sand and gravel one.

- If materials of sand and gravel zone contain coarse materials, above 2 equations shall be applied to the materials under 25 mm size.

- Intercepter materials shall not be cohesive and does not contain more than 5 percent of materials finer than 0.074 mm.

- The maximum grain size of intercepter materials is desirable to be 76 mm.

From the above definite relations, the intercepter zone (vertical filter zone) of the detention dam is recommendable to compose of a gradation within the limitation shown in Fig. D-15.

b) Thickness of filter

The thickness of filter zone can be made very thin through the calculation, however, the value of 2 m for the vertical filter and 1 m for the horizontal is employed actually in the existing dams.

In the right figure, the leakage discharge per unit length through the dambody and horizontal drain can roughly be obtained as follows:

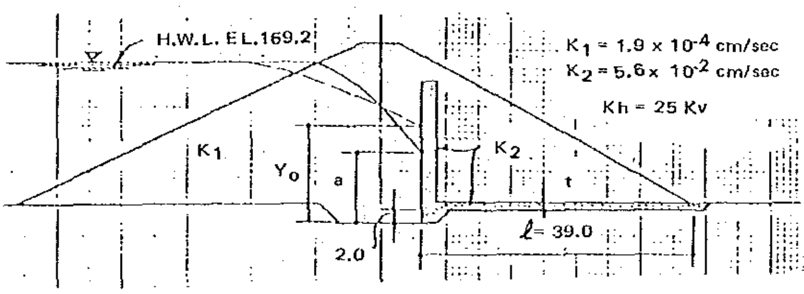
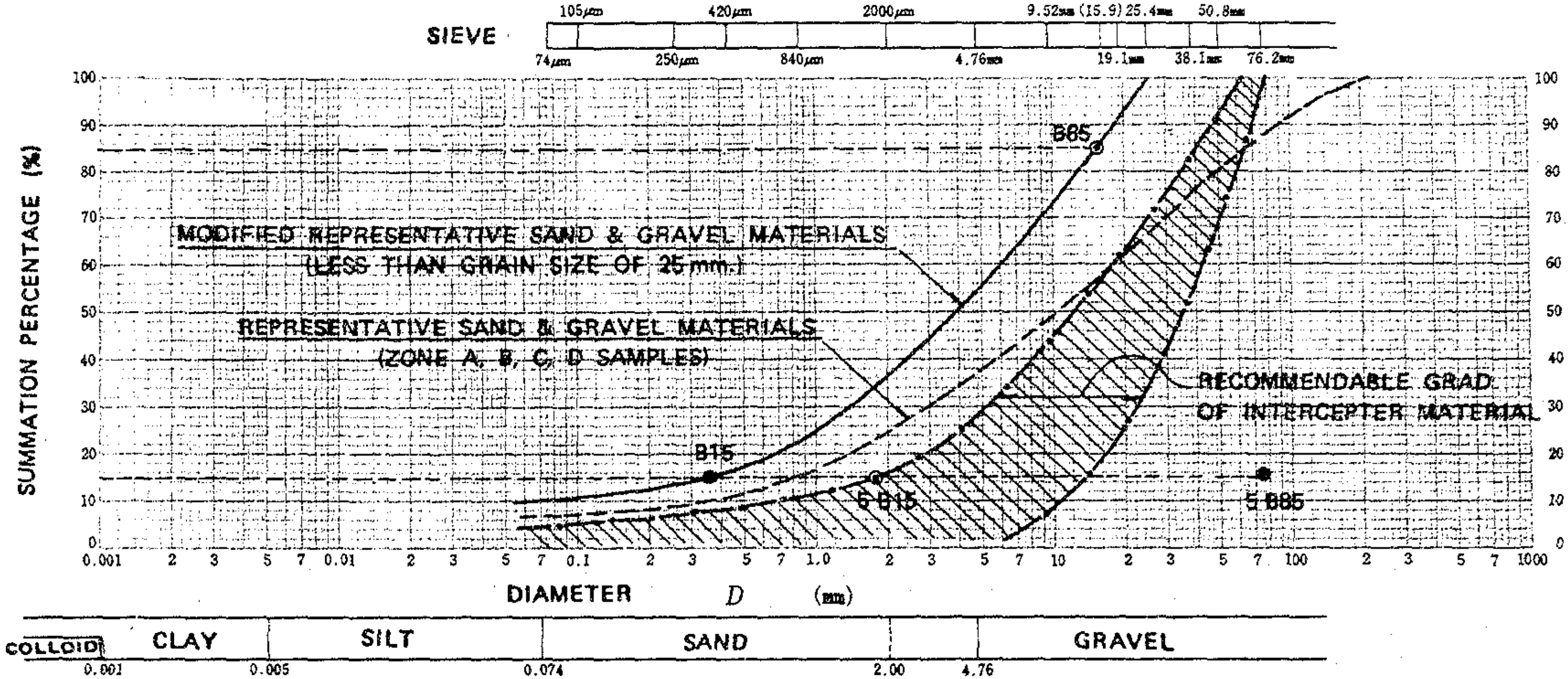


Fig. D–15 GRADING DISTRIBUTION OF INTERCEPTER MATERIALS



$$q = K_1' \cdot Y_o = K_2' \cdot \frac{(a - 2.0) - t}{\ell} \cdot t$$

Where, q: leakage discharge per unit length through dambody

K_1', K_2': permeability coefficient of dambody and horizontal drain in isotropic media and obtained by 9.5×10^{-4} cm/sec and 2.8×10^{-1} cm/sec, respectively

Y_o, a: height of seepage face and obtained by 14.16 m and 10.62 m respectively from Fig. D-6.

t: thickness of horizontal drain

ℓ: creep length and adopted by 39.0 m

From the above equation, the required thickness of horizontal drain is computed to be about 0.23 m, and another 0.77 m is added to this value considering the embankment conditions and available data on the existing dams.

Assuming that the hydraulic gradient in the intercepter is equal to 1.0, the required thickness of intercepter is expressed by the following equation.

$$t = \frac{K_1'}{K_2} \cdot Y_o$$

Where, t: required thickness of intercepter

K_1, Y_o: same in calculation as horizontal drain

K_2: permeability coefficient of intercepter in vertical direction and adopted by 5.6×10^{-2} cm/sec.

Substiluting these values into the above equation, the required thickness of the intercepter is obtained to be about 0.24 m, however, the value of 2 m is adopted taking into account the embankment method and available data on the existing dams.

2) Riprap

The embanked sand and gravel materials at the upstream slope should be protected by the hand-placed riprap in order to prevent the materials from moving and being washed away by wave action. The riprap materials must be hard and sound having enough durability against weathering.

The required weight of riprap in a piece can be obtained by the following equation.

$$W_T = \frac{K \cdot Wr \cdot f^3 \cdot Hw^3}{\left(\frac{Wr - W}{W}\right)^3 \cdot (f \cdot \cos\theta - \sin\theta)^3}$$

Where, W_T: weight of riprap in a piece (ton)

K: coefficient varies by slope and 0.036 is adopted

W_γ: unit weight of riprap rock and 2.70 t/m^3 is adopted by the results of rock tests

f: coefficient of friction in each riprap rock and 1.05 is adopted.

HW: height of wave, refer to Appendix I-1, Feasibility Report, and 0.80 m is adopted

W: unit weight of water

θ: grade of slope and 18°26' is adopted ($\cos\theta$ = 0.94868, $\sin\theta$ = 0.31623)

From the result of calculation, the weight of hand-placed riprap in a piece must not be less than 40 kg which is equivalent to the thickness about 0.45 m riprap layer with allowance of 1.5 assuming that the shape of riprap rock is globular form.

3.7. Foundation treatment

Since the detention dam is planned to recharge the groundwater through infiltration of the temporarily stored water by the dam into aquifers, there is no need to construct hydraulic barrier structures in its foundation from a view-point of the quantity of leakage discharge. But the foundation treatment is required to secure the sufficient bearing capacity and to ensure the stability against the piping failure and liquefaction.

1) Bearing capacity

The excavation of dam foundation consists of two parts, one is for the whole of dam body base and the other for the trench base along the dam axis. Judging from the deposit conditions at the dam foundation, the excavation depth for the dam body base is planned at 0.25 m on an average including the removal of objectionable materials such as top-soils, mud, organic materials, plants and roots.

The trench along the dam axis shall be excavated up to 3.0 m on an average where the N-value can be expected more than 100.

However, the excavation depth will vary from place to place according to the deposit condition.

2) Piping failure

The foundation of detention dam mainly consists of the recent river deposits with very loose sands and gravels distributed in the wadi channel and the terrace deposits with consolidated sands and gravels formed the both abutments and lain below the recent river deposits.

When the seepage water pressure exceeds a certain limit, collapse of the dam foundation may occur through the piping failure.

From the above facts, the analysis with saturated two-dimensional seepage flow by using the finite element method is carried out against the piping failure through the dam body and the foundation.

As a result of the analysis, the hydraulic gradients appeared in the dam foundation are less than the critical which derived from the specific gravity of particle and void-ratio for the foundation materials when the trench base is located at EL.150.0 m. Therefore, it seems that there is no possibility of the piping failure in the foundation (refer to 3.3. Seepage Analysis in this Report).

3) Liquefaction

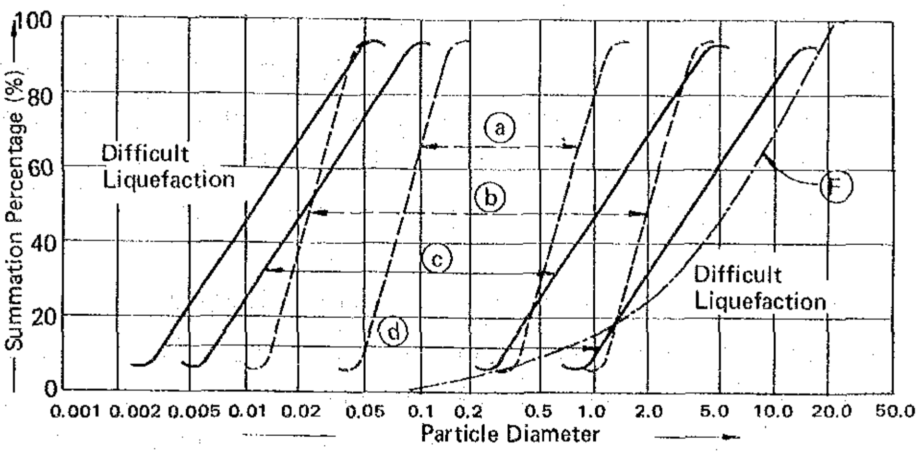
Since the recent river deposits at the dam foundation consists of very loose sand and gravel materials the study of liquefaction may be required so that whether or not it has sufficient resistability against earthquake vibrations.

It is well known that the liquefaction phenomenon take place occasionally in a certain place of the saturated sand foundation during the earthquake. There are several factors to be concerned with the liquefaction for the sandy materials, i.e., density, gradation distribution, strength characteristic, effective surcharge load, value and repeating number of shearing forces by the vibrations. Since the aforesaid factors, however, have relationship with each other, theoretical analysis of liquefaction is too intricate to be practised at presents, and the following method has been commonly used for judgement of liquefaction.

a) Gradation distribution

The liquefaction is closely related to the gradation distribution of the materials and it is likely to happen on small particles with low coefficient of uniformity (poor graded materials).

Judging from the following figure which makes from the data of earthquakes and dynamic model tests in Japan, it seems that liquefaction is unlikely to occur on the recent river deposits at the foundation of detention dam in view of the gradation distribution.



ⓐ Sand with low coefficient of uniformity; Very easy liquefaction
ⓑ — do — ; Possible liquefaction
ⓒ Sand with high coefficient of uniformity; Very easy liquefaction
ⓓ — do — ; Possible liquefaction
Ⓕ Gradation curve of recent river deposits at dam foundation

b) Relative density

The liquefaction can also be evaluated by the relative density of the materials as shown in the following table.

Seismic Acceleration	Very Easy Liquefaction	Liquefaction by Nature of Materials & Earthquake	Difficult Liquefaction
0.10 g	Dr < 33	33 < Dr < 54	Dr > 54
0.15 g	Dr < 48	48 < Dr < 73	Dr > 73
0.20 g	Dr < 60	60 < Dr < 85	Dr > 85
0.25 g	Dr < 70	70 < Dr < 92	Dr > 92

Where, Dr: relative density, $Dr = \frac{Dmax\ (D-Dmin)}{D\ (Dmax-Dmin)} \times 100$

Dmax: density in the most compact state

Dmin: density in the loosest compact state

D: density in-situ

In general, the relative density of 70 percent for the sand and gravel materials corresponds to about 93 percent of the maximum dry density in dry weight. From the results of soil test for the recent river deposit materials, the relative density for these materials can be presumed to be about 57 on an average.

Judging from the above fact, the recent river deposit materials at the dam foundation have a sufficient safety against the liquefaction in view of the relative density.

Moreover, the above safety against the liquefaction will be increased taking into account that the embankment of dam body is effective as the surcharge load.

As for the terrace deposits materials, there is no trouble against the liquefaction taking the gradation distribution (almost same the recent river deposits), consolidation degree, bulk density in-situ (presumed relative density is about 97 percent) and N-values (more than 100) into account.

4. Spillway

4.1. Type and Layout

An open-type spillway should be provided in principale to the fill type dam from the viewpoint of nonresistance against over topping caused by unexpected flood. And also, non-control type spillway, i.e. the overflow type spillway without gate, will be selected in order to eliminate possible dangerousness in gate operation if provided.

From the result of rough comparative study for the spillway layout, the construction costs of the dambody and spillway at the single and combined spillways are shown in the following table.

Item	Design Flood Discharge (cu.m/sec)				
	1,890	3,000	5,000	7,000	7,800
Single Spillway	2,720.6	2,856.1	4,785	7,635	8,975
Combined Spillway	-	-	3,425.3	5,097.6	5,881.1

(unit; x 10^3 O.R.)

The above table suggests that the combined spillway provided at the both banks is more favorable than that of single spillway at the right bank when the design flood discharge approaches to the value of around 5,000 cu.m/sec. And this value corresponds to the discharge of Standard Project Flood (SPF) as mentioned below.

As for the spillway located on the dambody, each of the dam height and overflow head is desirable to be restricted in a certain extent: for instance, 5.5 m and 0.7 m Southern dam, 6.25 m and 0.52 m in Northern dam respectively at the Wadi Al Khawad Aquifer Recharge Project in Oman, from the viewpoint of easy collapse of the dambody from erosion and scouring caused by running water with excessive energy through the spillway.

From the results of preliminary design for the spillway located on the dambody having the chute and stilling basin, the significant dimensions and construction costs are shown in the following table.

Item	Dambody-Type	Combined-Type	
Overflow Head (m)1/	4.17	Service	5.30
		Emergency	3.50
Dam Crest Elevation (m)	EL.168.90	EL.170.00	
Crest Width of Spillway (m)	550.0	Service	184.2
		Emergency	278.2
Excavation Volume of Spillway (m^3)	69,500	1,356,200	
Embank. Volume of Dambody (m^3)	471,700	670,600	
Construction cost ($x10^3$ R.O.)2/	5,530.0	5,246.0	
Cost-ratio	1.054	1.000	

Judging from the above table and hydraulic dimensions (14.18 cu.m/sec/m of discharge per unit length, 18.16 m/sec of velocity and 6.56 of Froude number at the end of chute) at the dambody type spillway, it seems that the combined type is reasonable for the spillway of Wadi Jizzi detention dam, because the spillway is located independently on the right side saddle where even if erosion and souring take place by the running water through the spillway, there will be no direct effect expected to the stability of the dambody.

The combined spillway consists of the service spillway located at the terrace plain on the right bank and the emergency spillway at the left bank where a small channel carved on the terrace plain that will be utilized as a tailrace of the spillway.

1/ Overflow head = Max. water surface elevation in reservoir - EL.163.90 (m)

2/ The direct construction costs of the spillway and dambody

As for the alignment of service spillway, a linear alignment is recommendable in order to eliminate the rising of water surface along the outer wall in the channel due to the cross-wave in the curved alignment.

4.2. Design Flood Discharges

According to the results of hydrological analysis in the Definitive Plan Report, 7,800 cu.m/sec was defined as the discharge of PMF with extreme safety taking into account the lack of hydrological data, dam type and importance of dam to the Project and also the discharge of SPF was determined at 60 percent of the PMF. The value of SPF is employed as the design flood discharge for the service spillway and the balance after decution of this value from the PMF is applied in the designing of the emergency spillway.

From the above, the design flood discharge for the service and emergency spillways are as follows.

° Design flood discharge for the service spillway:
  Qd = 7,800 cu.m/sec x 60 percent = 4,680 cu.m/sec
  ≒ 4,700 cu.m/sec

° Design flood discharge for the emergency spillway:
  Qd ≒ 7,800 cu.m/sec - 4,700 cu.m/sec = 3,100 cu.m/sec

4.3. Service Spillway

1) Hydraulic dimensions

a) Overflow head

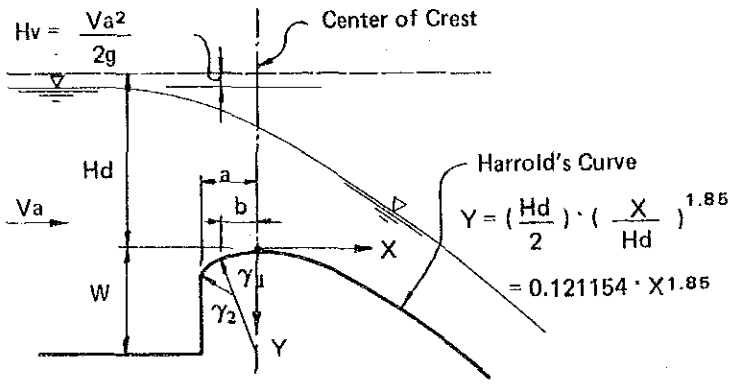
A rise of dam crest elevation and the construction cost of the spillway due to various overflow head are shown in the following table.

	Overflow Head of Spillway (m)			
	4.00	4.50	5.30	6.00
Crest Length of Spillway (m)	281.0	235.5	184.2	152.9
Dam Crest Elevation 1/ (m)	EL.168.7	EL.169.2	EL.170.0	EL.170.7
Construction Cost of Spillway ($x10^3$ R.O 2/)	3,590	2,860	2,010	1,520

As the overflow head becomes deeper, the construction costs of the spillway decreases and the crest elevation of dam will become higher contrarily. From the above table, it seems that the overflow head of 5.30 m is favorable in consideration of the restriction of topographical feature at the dam site.

b) Crest length

The dimension of a weir under complete overflow condition have a close relation with a shape of the weir. Provided that the Harrold's standard type shown in the following figure is employed for the overflow crest, the coefficient of discharge and the length of crest can be obtained from the following equations.



$r_1 = 0.5$ Hd

$r_2 = 0.2$ Hd

$a = 0.282$ Hd

$b = 0.175$ Hd

$L = QD/Cd \cdot Hd^{3/2}$

$Cd = 2.200 - 0.0416 \left(\frac{Hd}{W}\right)^{0.990}$

1/ Dam crest elevation = Overflow head + EL.163.90 + 0.80 (freeboard)

2/ The cost were estimated by rates of the Feasibility Study on January 1983.

$$C = 1.60 \cdot \frac{1 + 2\alpha(\frac{H}{Hd})}{1 + \alpha\ (\frac{H}{Hd})}$$

Where, L: length of crest

Qd: design flood discharge, adopted by 4,700 cu.m/sec

Cd: coefficient of discharge at design flood

Hd: overflow head at design flood, adopted by 5.30 m

W: depth of approach channel, adopted by 2.00 m

H: optional overflow head

α: constant which can be determined by the condition of H = Hd

Substituting those of already known values for the above equations, the coefficient of discharge (Cd, C) and the length of crest (L) can be obtained as follows:

$$Cd = 2.200 - 0.0416\ (\frac{5.30}{2.00})^{0.990} = 2.0908 \doteqdot 2.091$$

$$C = 1.60 \cdot \frac{1 + 0.886\ (\frac{H}{Hd})}{1 + 0.443\ (\frac{H}{Hd})}$$

In order to ensure the complete overflow condition at the weir, the top of downstream apron shall be located at the following elevation.

Top of downstream apron = EL.169.20 m - 1.7 Hd = EL.160.19 m $\doteqdot$ EL.160.20 m.

c) Flow condition

The water depth of chute and tailrace at the spillway is obtained by applying the following Bernoulli's theorem setting up the two control points at the center of crest (No.4 + 1.40) and the end of gabion protection (No.10 + 7.00).

$$D_1 \cos\theta + \frac{V_1{}^2}{2g} + \Delta x \cdot \tan\theta = D_2 \cos\theta + \frac{V_2{}^2}{2g} + \frac{n^2 \cdot Vm^2 \cdot \Delta \ell}{Rm^{4/3}}$$

Where, D_1, V_1: depth and velocity at the front section

D_2, V_2; depth and velocity at a section under consideration

θ: angle of bottom slope at the chute

g: acceleration of gravity, adopted by 9.8 m/sec^2

Δx: increment of distance

n: coefficient of roughness, adopted by 0.015, 0.035 and 0.030 for the concrete lining, gabion protection and no lining channels respectively.

Vm: mean velocity of flow, $Vm = 1/2\ (V_1 + V_2)$

$\Delta\ell$: increment of distance measured along the bottom of chute

Rm: mean hydraulic radius, $Rm = 1/2\ (R_1 + R_2)$

The results of calculation for the water depth, velocity and Froude number at the chute and tailrace are shown in the following table.

Station (m)	Bottom Elevation (m)	Bottom Width (m)	Water Depth (m)	Velocity (m/s)	Froude Number 1/	Remark	
No.4+1.40	EL.163.90	184.2	3.533	7.020	1.000	Crest	Supercritical flow
No.4+10.12	EL.160.2	184.2	2.175	11.526	2.497		
No.4+25.10	EL.160.2	184.2	2.206	11.362	2.444		
No.4+42.62	EL.160.2	184.2	2.281	10.980	2.322		
No.5+38.12	EL.159.448	158.018	3.091	9.347	1.698		
No.6+16.81	EL.158.974	141.509	3.760	8.494	1.399		
No.6+45.50	EL.158.50	125.00	4.682	7.604	1.123	End of Chute	
No.6+45.50	EL.158.50	125.00	6.779	5.130	0.629		Sub-critical flow
No.8+2.00	EL.158.50	125.00	6.564	5.310	0.662		
No.8+37.00	EL.158.50	125.00	6.326	5.525	0.702		
No.9+22.00	EL.158.50	125.00	6.000	5.845	0.762		
No.10+7.00	EL.158.50	125.00	5.127	6.909	1.000	End of gabion linin	

1/ Froude number, $Fr = V/\sqrt{gD}$

Judging from the Froude numbers in the above table, there is no difinite hydraulic jump occurring in the service spillway, except the undular jump accompanied by undulation or turbulance of the water surface at the end of chute (No.6 + 45.50 m). However, the running water through the spillway possess excessive energy to bring about erosion and scouring.

d) Erosion protection

Even though the service spillway is situated on the terrace deposits well consolidated, sufficient resistivity can not be expected against erosion and scouring caused by excessive velocity. Therefore, the protection works with gabion (partly concrete) should be executed at the chute and tailrace of the spillway.

Downstream of weir (No.4 + 10.12)

The required length of apron and riprap can be obtained by the following Bligh's formula.

Apron length: $L_1 = 0.6\ C\sqrt{De} = 13.849 \doteqdot 15.0$ m

Riprap length: $L_2 = 0.67\ C\sqrt{De \cdot q} - L_1 = 78.12 - 15.0$

$= 63.12 \doteqdot 63.0$ m

Where, C: Bligh's coefficient, adopted by 12 (coarse sand foundation)

De: height of weir, adopted by 3.7 m

q: discharge per unit length

End of chute (No.6 + 45.5)

Since the undular hydraulic jump will occur around the end of chute, the bottom of tailrace should be protected by the gabion. The conjugate depth and required length of gabion can be obtained by the following formula.

$$d_2 = d_1 \times 1/2\ (\sqrt{1 + 8F\gamma^2} - 1) = 5.455\text{m}$$

$$L = K \cdot d_2 = 18.547\text{m} \doteqdot 20.0\text{m}$$

Where, d_2: conjugate depth

d_1: water depth at the end of chute, adopted by 4.682 m

Fr: Froude number at the end of chute, adopted by 1.123

L: protection length of gabion

K: coefficient varied by Froude number, adopted by 3.4

e) Freeboard

Freeboards at the chute and tailrace with supercritical flow and subcritical flow conditions respectively are obtained by the following equations.

Chute - supercritical flow: $Fb = C\ V\ d^{1/2}$

Tailrace - subcritical flow: $Fb = 0.05d + Hv + (0.05 \sim 0.15)$

Where, Fb: freeboard

C: coefficient, adopted by 0.13 in trapezoidal shape

d: water depth at a section under consideration

hv: velocity head, $hv = V^2/2g$

The results of calculation for the freeboard and the top of protection at the chute and tailrace are shown in the following table.

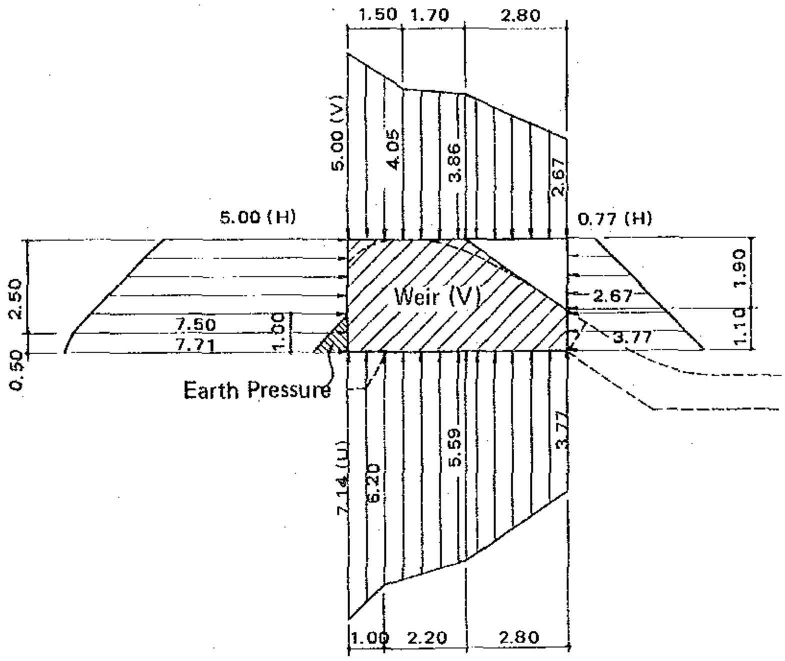
Station (m)	Velocity (m/sec)	Water Depth (m)	Freeboard (m)	Bottom Elevation (m)	Top Elevation (m)	Adopted Top Elevation (m)	
No.4+1.40	7.020	3.533	1.715	EL.163.9	EL.169.148	EL.170.0	Supercritical flow
No.4+10.12	11.526	2.175	2.210	EL.160.2	El.164.585	EL.167.9	
No.4+25.10	11.362	2.206	2.194	EL.160.2	EL.164.600	EL.167.0	
No.4+42.62	10.980	2.281	2.156	EL.160.2	EL.164.637	EL.167.0	
No.5+38.12	9.347	3.091	2.136	EL.159.448	EL.164.675	EL.167.0	
No.6+16.81	8.494	3.706	2.141	EL.158.974	EL.164.875	EL.167.0	
No.6+45.50	7.604	4.682	2.139	EL.158.50	EL.165.321	EL.167.0	
No.6+45.50	5.063	6.862	1.751	EL.158.50	EL.167.113	EL.167.0	Subcritical flow
No.8+2.00	5.310	6.564	1.867	EL.158.50	EL.166.931	EL.167.0	
No.8+37.00	5.525	6.326	1.973	EL.158.50	EL.166.799	EL.167.0	
No.9+22.00	5.845	6.000	2.143	EL.158.50	EL.166.643	EL.167.0	
No.10+7.00	6.909	5.127	2.791	EL.158.50	EL.166.418	EL.167.0	

2) Structural dimensions

a) Safety of weir-body

The loads[1/] to be considered for the stability analysis of the weir-body against reversing, sliding and bearing capacity are dead weight, hydrostatic pressure, earth pressure and uplift.

In order to simplify the stability calculation, the following trapezoidal section is employed for the weir-body.



From the external loads as shown in the above section and arm length in each load, the total of moment attributable to resultant force is obtained by the following equation.

1/ As for the earthquake force, there is no need to consider the simultaneous occurrence of design earthquake and Probable Maximum Flood (PMF), therefore, the earthquake force is neglected in this calculation.

$$\Sigma M = \Sigma(V \cdot X) + \Sigma(H \cdot Y) = 51.519 + 19.108 = 70.627 \text{ t·m}$$

Where, ΣM: total moment attributable to resultant force

$(V \cdot X)$: total moment attributable to vertical forces, obtained by 51.519 t·m through the calculation

$(H \cdot Y)$: total moment attributable to horizontal forces, obtained by 19.108 t·m through the calculation

Study on reversing

In order to stabilize the weir-body against the reversing, the eccentric distance must satisfy the following equation.

$$e = \left| \frac{\Sigma M}{\Sigma V} - \frac{\ell}{2} \right| \leqq \frac{\ell}{6}$$

Where, e: eccentric distance

ΣM: total moment attributable to resultant force, obtained by 70.627 t·m through the calculation

V: resultant of vertical forces, obtained by 21.785 ton through the calculation

ℓ: bottom length of weir, adopted by 6.00 m

Substituting those of already known values into the above equation, the eccentric distance is obtained by 0.242 m and this value is less than restrictive value of 1.0 m ($\frac{1}{6}\ell$), therefore, the weir-body has a sufficient safety against the reversing.

Study on sliding

Since the weir-body is situated on the sand and gravel deposits, the following equation should be satisfied to ensure the safety against sliding.

$$\frac{\Sigma H}{\Sigma V} < \frac{1}{1.2} \cdot f = 0.625$$

Where, ΣH: resultant of horizontal forces, adopted by 13.170 ton through the calculation

ΣV: resultant of vertical forces, adopted by 21.785 ton through the calculation

f: friction coefficient, adopted by 0.75

The calculation result ($\Sigma H/\Sigma V = 0.605$) shows that the weir-body has a sufficient resistivity against sliding.

Study on bearing capacity

The compression strength on the bottom surface of the weir-body is obtained by the following equation and this value should be less than the allowable bearing capacity of the foundation.

$$P = \frac{\Sigma V}{A} \cdot (1 \pm \frac{6e}{\ell}) = 4.509 \text{ t/m}^2 \text{ or } 2.752 \text{ t/m}^2$$

Where, P: compression strength at the weir base

ΣV: resultant of vertical forces, obtained by 21.785 ton through the calculation

A: bottom area of weir-body per unit length, $A = 6.0$ m^2

e: eccentric distance, $e = 0.242$ m obtained through calculation

ℓ: bottom length of weir, adopted by 6.00 m

Judging from the N-values of more than 100 at the foundation, there is no trouble for the bearing capacity and harmful settlement will not occur to the weir-body consequently.

b) Thickness of apron

The thickness of apron at the end of weir (No.4 + 10.12) is estimated by the following equation.

$$t = \alpha[0.1 + 0.1\ (q \cdot Fr)^{1/2}] = 0.898 \fallingdotseq 0.9\ m$$

Where, t: thickness of apron

α: coefficient by foundation materials, in sand and gravel foundation $\alpha = 1.0$

q: discharge per unit length, adopted by 25.516 $m^3/sec/m$

Fr: Froude number, adopted by 2.497

c) Thickness of gabion protection

Since there is no theoretical calculation method to determine the thickness of erosion protection in the canal against excessive velocity, the following empirical equations are employed to obtain a thickness of protective gabion.

$$F = K \cdot W \cdot \varepsilon \cdot A \frac{V^2}{2g}$$

$$R = f \cdot (Ws - W) \cdot U$$

Where, F: tractive force

K: coefficient by shape of protection, adopted by 1.0

W: unit weight of water

ε: blockout coefficient, in continuum protection $\varepsilon = 0.30 \sim 0.35$, adepted by 0.325

A: area of gabion against flow, adopted by 1.0 m^2 (A = 1.0 m x 0.5 m x 2) at double layers gabion

V: velocity

g: acceleration of gravity, adopted by 9.8 m/sec^2

R: resistant force against sliding

f: friction coefficient, adopted by 0.75

Ws: unit weight of gabion, Ws = (1 - e) Gs = 2.502 ton, e = 0.1 void ratio, Gs = 2.78 specific gravity of rock materials

U: volume of gabion, adopted by 2.0 m^3 (U = 2.0 m x 0.5 m x 1.0 m x 2) and 1.0 m^3 (U = 2.0 m x 0.5 m x 1.0 m x 1) for double layers gabion and single layer gabion, respectively.

The results of calculation for the tractive forces, resistant force and safety factors (R/F) are shown in the following table.

Station (m)	Velocity (m/s)	Tractive Force (ton)	Resistant Force (ton)	Safety Factor	Layer of Gabion
No.4+25.10	11.362	2.141	2.253	1.052	Double
No.4+42.62	10.986	1.999	2.253	1.121	"
No.5+38.12	9.347	1.449	2.253	1.555	"
No.6+16.81	8.494	1.196	2.253	1.884	"
No.6+45.50	7.604	0.959	2.253	2.349	"
No.8+2.00	5.310	0.468	1.127	2.408	Single
No.8+37.00	5.525	0.506	1.127	2.227	"
No.9+22.00	5.845	0.566	1.127	1.991	"
No.10+7.00	6.909	0.792	1.127	1.423	"

In case the placing of gabion, the long side of frame shall be laid in parallel with flow direction.

4.4. Emergency Spillway

1) Hydraulic dimensions

a) Overflow head

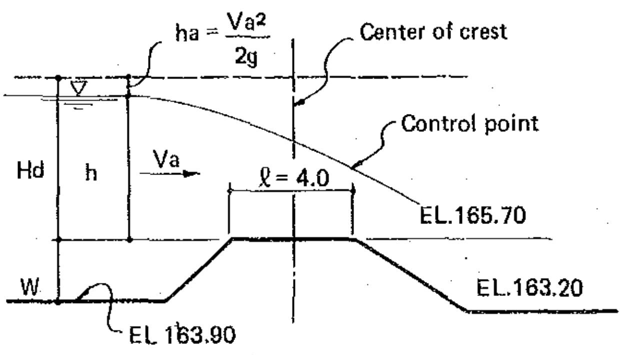
The following table shows the relationship between the crest length, construction cost and various overflow head of the spillway under the dam crest elevation is fixed at EL.170.0 m.

	Overflow Head of Spillway (m)		
	2.50	3.50	5.30
Crest Elevation of Spillway 1/ (m)	EL.166.70	EL.165.70	EL.163.90
Crest Length of Spillway (m)	461.0	278.2	149.4
Construction Cost of Spillway 2/ ($\times 10^3$ R.O)	1,776	971	760

From the above table suggests that the construction costs of spillway will become cheaper in accordance with the overflow head becomes deeper, however the figure of 3.5 m is reasonable as the suitable overflow head of the emergency spillway with the crest length of 278.2 m in consideration of the increasing ratio on the construction cost around the overflow head of 3.5 m, topographic restriction at the dam site and connection with existing small channel which will be utilized as a tailrace of the emergency spillway.

b) Crest length

A board crested weir type shown in the following figure is employed for the overflow crest of the emergency spillway, the coefficient of discharge and the length of crest can be obtained from the Beresinski's formula mentioned below.



$$L = Qd/cHd^{3/2}$$

$$0.6 < \ell/h < 2.5$$

$$C = 1.973 - 0.222\ (\ell/h)$$

$$2.5 < \ell/h < 10$$

$$C = 1.706 \cdot \frac{1 + 1.3\ (W/h)}{1 + 6.3\ (W/h)}$$

1/ Crest elevation of spillway = EL.170.0 - Overflow head - 0.80 (Freeboard)

2/ The costs were estimated by rate of the Definitive Plan Report on August 1985

Where, L: length of crest

C: coefficient of discharge

Hd: overflow head at design flood discharge, adopted by 3.50 m

ℓ: Width of crest, adopted by 4.0 m

h: overflow depth, h = Hd - ha, ha: approach velocity head

w: depth of approach channel, adopted by 1.80 m

In the design flood discharge Qd, assuming that the approach velocity head ha is equal to 0.23 m 1/: hence the coefficient of discharge C and length of crest L can be obtained as follows:

$$\ell/h = 1.223, \quad C = 1.973 - 0.222(\ell/h) = 1.7014$$
$$L = Qd/cHd^{3/2} = 278.262\text{m} \doteqdot 278.2\text{m}$$

In order to eliminate the submerged overflow condition at the weir which will reduce the coefficient of discharge, the surface of downstream apron should be located at the following elevation.

Surface of apron = EL.169.20 - 1.7 Hd = EL.163.25 m ≑ EL.163.20 m

c) Flow condition

The following Bernoulli's theorem is applied to estimate the water depth of chute and tailrace of the spillway setting up the two control points at the end of crest (No.'4 + 2.0) and the end of tailrace (No'6 + 38.0).

$$D_1\cos\theta + \frac{V_1{}^2}{2g} + \Delta x\cdot\tan\theta = D_2\cdot\cos\theta + \frac{V_2{}^2}{2g} + \frac{n^2 Vm^2\cdot\Delta\ell}{Rm^{4/3}}$$

The sings of the above theorem are same in the paragraph of service spillway.

1/ From the above crest length, the followings are estimated: the flow area A = 1,421.653 m^2 and approach velocity V = 2.181 m/sec. Then, the approach velocity head hv is obtained by 0.243 m and this value closely corresponds to the assumed value of ha.

The results of calculation for the water depth, velocity and Froude number at the chute and tailrace are shown in the following table.

Station (m)	Bottom Elevation (m)	Bottom Width (m)	Water Depth	Velocity (m/s)	Froude Number [1/]	Remark	
No'4+2.0	EL.165.70	278.20	2.333	4.717	1.000	End of Crest	Supercritical flow
No'4+9.0	EL.163.20	278.20	1.158	9.561	2.838		
No'4+21.0	EL.163.20	278.20	1.239	8.936	2.564		
No'4+31.0	EL.163.20	278.20	1.287	8.600	2.422		
No'5+1.0	EL.163.086	259.229	1.661	7.132	1.768		
No'5+16.0	EL.163.00	245.00	2.021	6.185	1.390	End of Chute	
No'5+16.0	EL.163.00	245.00	3.429	3.614	0.623		Subcritical flow
No'5+31.0	EL.163.00	245.00	3.340	3.712	0.649		
No'6+8.0	EL.163.00	245.00	3.144	3.948	0.711		
No'6+38.0	EL.163.00	245.00	2.511	4.963	1.000	End of Tailrace	

According to the Froude number in the above table, there is no definite hydraulic jump in the emergency spillway, except the undualr or weak jump accompanied by undulation, turblance and small rollers on the water surface at the end of chute (No'5 + 16.0). However, the running water through the spillway possess excessive energy to bring about the erosion and scouring at the side and bottom of the channel.

d) Erosion protection

Since the emergency spillway is constructed on the terrace deposits which has no sufficient resistivity against erosion and scouring, the protection works with gabion (partly wet masonry) should be executed at the chute and tailrace of the spillway.

Downstream of weir (No'4 + 9.0)

The following Bligh's formula is applied to obtain the required length of downstream apron and riprap.

1/ Froude number $Fr = V/\sqrt{gD}$

Apron length: $L_1 = 0.6\ C\sqrt{De} = 11.384\ m \doteqdot 12.0\ m$

Riprap length: $L_2 = 0.67\ C\sqrt{De \cdot q} - L_1 = 30.435m \doteqdot 30.0\ m$

Where, C: Bligh's coefficient, adopted by 12 (coarse sand foundation)

De: height of weir, adopted by 2.5 m

q: discharge per unit length

<u>End of Chute</u> (No'5 + 16.0)

Since the undular or weak hydraulic jump will occur around the end of chute, the bottom of tailrace should be protected by the gabion. The conjugate depth and required length of gabion can be obtained by the following formula.

$$d_2 = d_1 \times 1/2\ (\sqrt{1 + 8Fr^2} - 1) = 3.089m$$

$$L = K \cdot d_2 = 11.738 \doteqdot 12.0m$$

Where, d_2: conjugate depth

d_1: water depth at the end of chute, adopted by 2.021 m

Fr: Froude number at the end of chute, adopted by 1.390

L: protection length of gabion

K: coefficient varied by Froude number, adopted by 3.8

e) Freeboard

Freeboard at the chute and tailrace with supercritical flow and subcritical flow conditions are obtained by the following equations, respectively.

Chute-supercritical flow: $Fb = C\ V\ d^{1/2}$

Tailrace-subcritical flow: $Fb = 0.05\ d + hv + (0.05 \sim 0.15)$

The sings of the above equations are same in the paragraph of service spillway.

The results of calculation for the freeboard and top of protection at the chute and tailrace are shown in the following table.

Station (m)	Velocity (m/s)	Water Depth (m)	Freeboard (m)	Bottom Elevation (m)	Top Elevation (m)	Adopted Top Elevation	
No'4+2.0	4.717	2.333	0.937	EL.165.70	EL.168.97	EL.170.00	Supercritical
No'4+9.0	9.561	1.158	1.338	EL.163.20	EL.165.696	EL.167.50	
No'4+21.0	8.936	1.239	1.293	EL.163.20	EL.165.732	EL.167.50	
No'4+31.0	8.600	1.287	1.268	EL.163.20	EL.165.755	EL.167.50	
No'5+1.0	7.132	1.661	1.195	EL.163.086	EL.165.942	EL.167.50	
No'5+16.0	6.185	2.021	1.143	EL.163.00	EL.166.164	EL.167.50	
No'5+16.0	3.614	3.429	0.937	EL.163.00	EL.167.366	EL.167.50	Subcritical
No'5+31.0	3.712	3.340	0.970	EL.163.00	EL.167.310	EL.167.50	
No'6+8.0	3.948	3.144	1.052	EL.163.00	EL.167.196	EL. Existing	
No'6+38.0	4.963	2.511	1.483	EL.163.00	EL.166.994	EL. Ground Surface	

2) Structural dimensions

a) *Thickness of apron*

The thickness of apron at the end of weir (No'4 + 9.0) is estimated by the following equation.

$$t = \alpha\ [0.1 + 0.1\ (q.Fr)^{1/2}] = 0.662 \doteqdot 0.70\text{m}$$

Where, t: thickness of apron

α: coefficient by foundation materials, in sand and gravel foundation $\alpha = 1.0$

q; discharge per unit length, adopted by 11.143 m^3/sec/m

Fr: Froude number, adopted by 2.838

b) Thickness of gabion protection

The required thickness of gabion protection is made by using the following empirical equations.

$$F = K \cdot W \cdot \varepsilon \cdot A \frac{V^2}{2g}$$

$$R = f \cdot (Ws - W) \cdot U$$

The sings of the above equations are same in the paragraph of service spillway.

The calculation results for the tractive forces F, resistant forces R and safety factors (R/F) are shown in the following table.

Station (m)	Velocity (m/s)	Tractive Forces (ton)	Resistant Forces (ton)	Safety Factor	Layer of Gabion
No'4+21.0	8.936	1.324	2.253	1.702	Double
No'4+31.0	8.600	1.226	2.253	1.838	"
No'5+1.0	7.132	0.843	2.253	2.673	"
No'5+16.0	6.185	0.634	1.127	1.778	Single
No'5+31.0	3.712	0.228	1.127	4.943	"
No'6+8.0	3.948	0.258	1.127	4.368	"
No'6+38.0	4.963	0.408	1.127	2.762	"

In case the placing of gabion, the long side of flame shall be laid in parallel with the flow direction.

4.5. Flow-out capacity of Spillway

Influx of flood discharge into the reservoir will cause the spillage from the service spillway at frist, when the water surface exceeds the full water level of EL.163.90 m. And furthermore, when the rise of water level reaches EL.165.70 m, the spillage from the emergency spillway will take place. When the water level in the reservoir exceeds EL.165.70 m, the flood discharge will flow down through both the service and emergency spillways.

The flow-out capacity of the service and emergency spillways varies with the overflow head and coefficient of discharge, and also there is close relation between the overflow head and coefficient of discharge as follows.

° Service spillway $Q = C\ L\ H^{3/2}$

$$C = 1.60 \cdot \frac{1 + 0.886\ (\frac{H}{Hd})}{1 + 0.443\ (\frac{H}{Hd})}$$

Where, Q: flow-out capacity

C: coefficient of discharge

L: length of crest, adopted by 184.2 m

H: optional overflow head

Hd: overflow head at design flood discharge, adopted by 5.3 m

° Emergency spillway $Q = C\ L\ H^{3/2}$

$0.6 < \ell/h < 2.5$

$C = 1.973 - 0.222\ (\ell/h)$

$2.5 < \ell/h < 10$

$$C = 1.706 \cdot \frac{1 + 1.3\ (W/h)}{1 + 6.3\ (W/h)}$$

Where, Q: flow-out capacity

C: coefficient of discharge

L: length of crest, adopted by 278.2 m

H: optional overflow head

ℓ: width of crest, adopted by 4.0 m

h: overflow depth, h = H - ha, ha: approach velocity head

W: depth of approach channel, adopted by 1.80 m

From the above equations, the flow-out capacity of service and emergency spillways under various overflow heads are shown in the following tables and is drawn in Fig. D-16.

° Flow-out capacity of service spillway

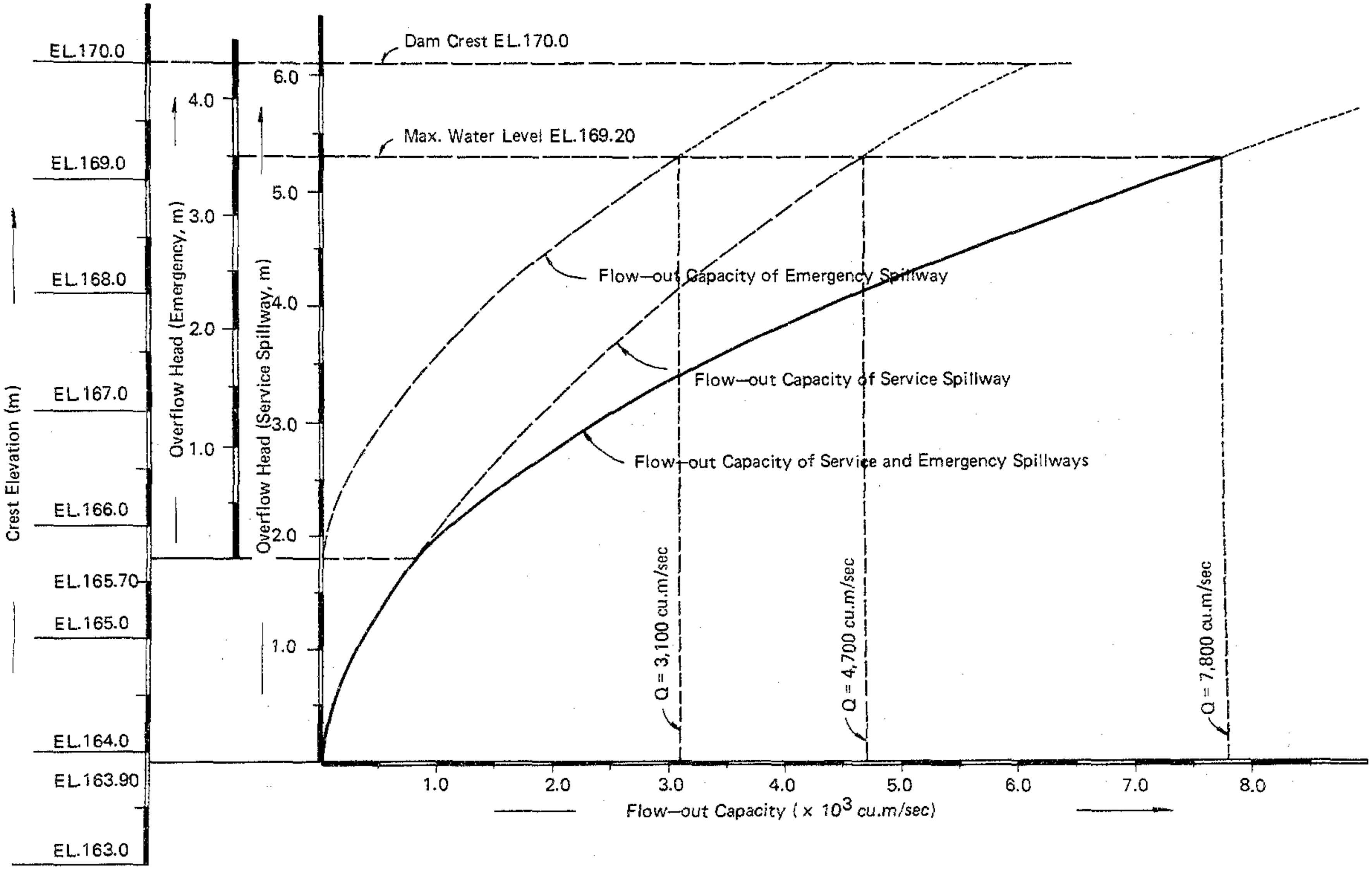
H	0.5 m	1.0 m	2.0 m	3.0 m	4.0 m	5.0 m	5.3 m
H/Hd	0.0943	0.1887	0.3774	0.5560	0.7547	0.9434	1.0000
C	1.664	1.723	1.829	1.921	2.001	2.072	2.091
Q (cu.m/sec)	108.37	317.38	952.90	1,838.65	2,948.67	4,267.12	4,700.0

° Flow-out capacity of emergency spillway

H	0.5 m	1.0 m	2.0 m	3.0 m	3.5 m
C	1.407	1.442	1.529	1.677	1.7014
Q (cu.m/sec)	138.39	401.16	1,203.12	2,424.22	3,100

If it is defined that the ultimate flow-out capacity of the spillway at the detention dam is equivalent to the discharge released through the spillway (service spillway and emergency spillway) when the reservoir water has arrived at the dam crest elevation of EL.170.0 m, the said capacity counts about 10,600 cu.m/sec from the Fig. D-16.

Fig. D-16. FLOW-OUT CAPACITY OF SPILLWAY (SERVICE & EMERGENCY SPILLWAYS)



5. Outlet Facilities

5.1. Function and Design Discharge

The outlet facilities of detention dam are composed of the service outlet and emergency outlet. The functions of those outlets aim to release the temporarily stored water in the reservoir by the sevice outlet in usual case and to release the stored water by emergency outlet in case of blockade at the service outlet caused by unexpected accident, respectively.

According to the result of hydrological analysis in the Definitive Plan Report (refer to A-4 in the Annex), the optimal release rate of the stored water in the detention dam was estimated at 13.0 cu.m/sec through the service outlet taking into account the recharge potential at the downstream wadi courses of the detention dam. Consequently, the release rate of 13.0 cu.m/sec is employed as the design discharge for the service outlet.

5.2. Service Outlet

The service outlet will be embedded beneath the dam body at the lowest portion of the river bed in order to extrude whole the water from the reservoir area. Otherwise, the standing water might breed mosquitoes which are media of Malarial desease.

1) Hydraulic design

A circular sectioned steel pipe is adopted as the inner shape of the service outlet taking into account the structural advantage and easy construction. The square shape with 2.4 m inner side-length of drop inlet equipped with trashrack is provided at the entrance of the conduit.

The outlet discharge of the service outlet as a pipeline is obtained for various inside diameter of the conduit under the full reservoir condition by the following equation.

$$Q = \frac{\sqrt{2g} \cdot A_1}{\sqrt{ft + fe + fr_1 + fb + fr_2 + Fv}} \cdot \sqrt{H}$$

Where, Q: outlet discharge of the service outlet as a pipeline flow

g: acceleration of gravity, adopted by 9.8 m/sec^2

A_1: flow area at the conduit, $A_1 = 0.7854\ D^2$

D: inside diameter of the conduit

ft: coefficient of trashrack loss varies by blockade rate of the trashrack, ft = 0.21 in case the blockade rate of 10%

fe: coefficient of entrance loss, adopted by 0.50

fr_1: coefficient of friction loss at the drop inlet $fr_1 = 19.6\ n^2/R^{4/3} \times L_1$,

n: coefficient of roughness, adopted by 0.015

R: hydraulic radius, $R = A_2/P = 0.60$

A_2: flow area at the drop inlet, adopted by 5.76 m^2

P: wetted perimeter at the drop inlet, adopted by 8.0 m

L_1: length of drop inlet, adopted by 2.94 m

fb: coefficient of bend loss, adopted by 0.99 (90° bend)

fr_2: coefficient of friction loss at the conduit $fr_2 = 124.5\ n^2/D^{4/3} \times L_2$

L_2: length of the conduit, adopted by 114.0 m

fv: coefficient of changing velocity loss, adopted by 1.00

H: total head, measured from top of crown at the end of conduit, adopted by 12.15 m

The calculation results are shown in the following table.

	Inside Diameter of Conduit (m)			
	1.40	1.45	1.50	1.55
Total Coefficient of Less (f)	4.572	4.482	4.394	4.317
$\sqrt{2g} \cdot A ; \sqrt{H}$	23.756	25.483	27.271	29.118
Outlet Discharge (cu.m/sec)	11.11	12.04	13.01	14.01

From the above table, the outlet discharge at the inside diameter of 1.50 m closely corresponds to the design discharge of the service outlet.

The outlet discharge for the service outlet with inside diameter of 1.5 m varies by the reservoir water levels and can be obtained by the following equation.

$$Q = \frac{\sqrt{2g} \cdot A}{\sqrt{\Sigma f}} \sqrt{H} = 3.7324\sqrt{H}$$

Where, g: acceleration of gravity, adopted by 9.8 m/sec^2

A: flow area of conduit, $A = 0.7854\ D^2 = 1.7672\ m^2$

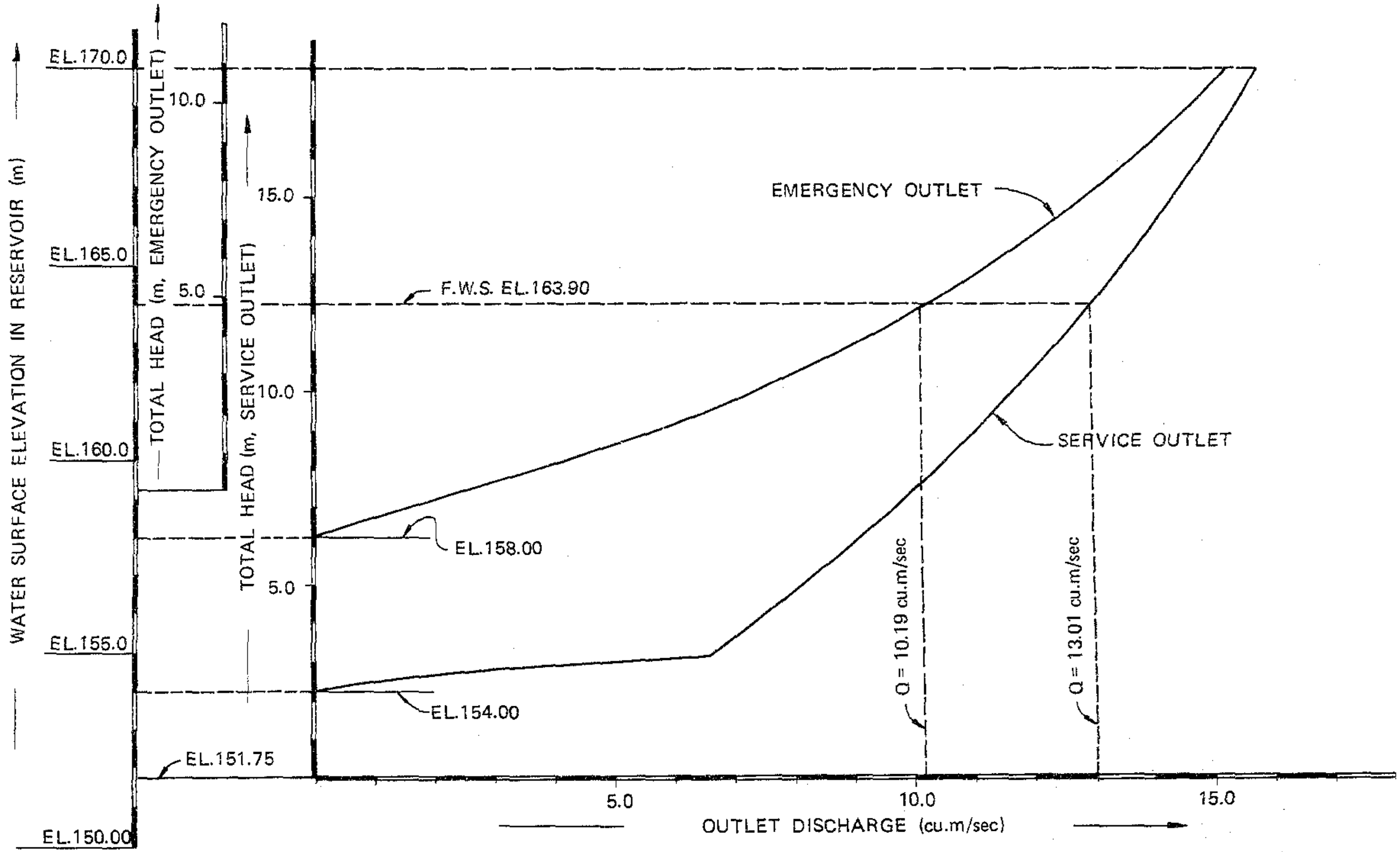
Σf: total coefficient of losses at the service outlet, $\Sigma f = 4.394$

H: total head, measured from top of crown at the end of conduit

The calculation results are shown in the following table and is drawn in Fig. D-17.

	Water Surface Elevation in Reservoir (m)					
	EL.155.00	EL.156.00	EL.158.00	EL.160.00	EL.162.00	EL.163.90
Total Head(m)	3.25	4.25	6.25	8.25	10.25	12.15
Outlet Discharge (cu.m/sec)	6.73	7.69	9.33	10.72	11.95	13.01
Velocity (m/sec)	3.81	4.35	5.28	6.07	6.75	7.36

Fig. D–17. FLOW–OUT CAPACITY OF OUTLET FACILITIES (SERVICE OUTLET & EMERGENCY OUTLET)



The running water through the service outlet has a relatively high velocity, therefore, the protection works with wet masonry and gabion should be planned in the adjacent downstream of the conduit and tailrace respectively in order to prevent the toe of dam body and the tailrace from erosion and scouring.

2) Structural design

The structural design for the service outlet is divided into two parts of the reinforced concrete structures, the one is for the conduit embedded beneath the dam body and the other for the drop inlet provided at the entrance of the conduit.

a) Conduit

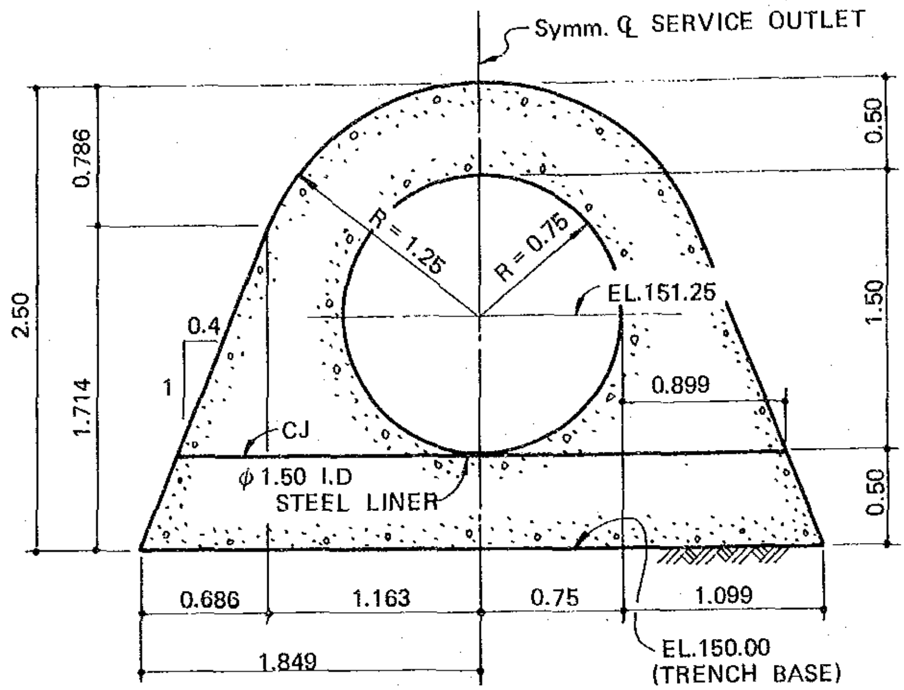
The outer section of the conduit has special shape as shown in Fig. D-18 in order to avoid cracks in the dam body caused by the differential settlement.

These cracks become leakage-passes of the stored water, induces piping phenomenon through the dam body, and finally may result in collapse of the dam. The analysis on the distribution of tensile stress in the reinforced concrete members has been carried out by using the finite element method.

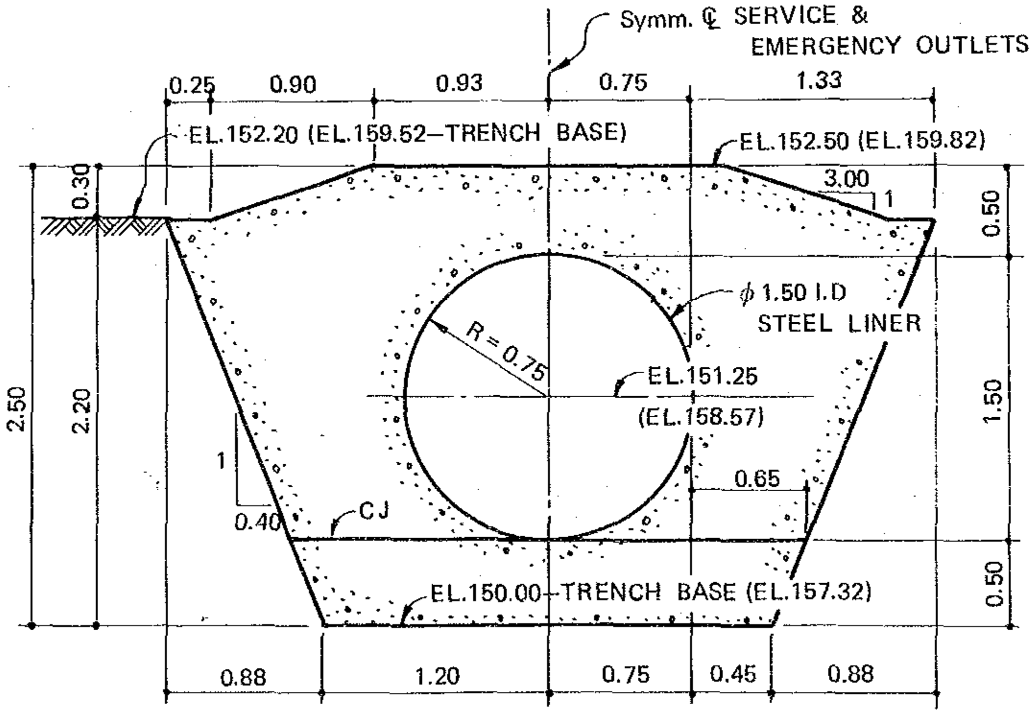
Methodology of analysis

The displacement method formulated with the radix of elasticity is introduced in this analysis to find the displacements and stresses for the conduit and its surroundings caused by the given embankment load.

Fig. D–18. TYPICAL SECTION OF OUTLET FACILITIES



(SERVICE OUTLET AT TRENCH PORTION)



(SERVICE & EMERGENCY OUTLETS)

The fig ıres in a parenthesis express the values of emergency outlet

Side-slope of 1 Vertical to 0.4 horizontal corresponds to the trimming criteria for the abutments slope of the fill type dam.

The basic idea that a continuous amount such as heat, pressure, displacement, etc. is approximated by the dispersion model which consists of continuous function group in the finited mumbers of divided sub-domain. The model of dispersion in the general field is composed as follows:

- ° to designate the limited numbers of nodal points in the domain

- ° to adopt the values of continuous amount at respective nodal points as the variables

- ° to divide the whole domain into the limited numbers of sub-domains called as elements. Those elements are united each other at the concurrents (nodes) and collectively approximate the shape (configuration) of the domain

- ° To approximate the continuous amount at the respective elements in adopting the polynomial expression defined by the nodal values (value of coordinates)

Nowadays, the finite element method is widely applied because of a variety of its original merits as shown below;

- ° It is not necessary that the specification of the adjoining elements must be the same. It means to make a possibility of the application of this method to the object which consists of the various substances.

- ° The irregular boundary is approximately expressed by the elements which have the boundaries. Therefore, the application of this method is not limited only to the neat configuration which has a simply boundary.

- The size of element can be arbitrarily changed. This character enables to make the grid size large or small as occasion demands.

- It easily expresses the boundary conditions such as discontinuous surface load, and the complex boundary conditions can be dealt with.

It is indispensable to provide an electric computer and its program for general use of the actual analysis.

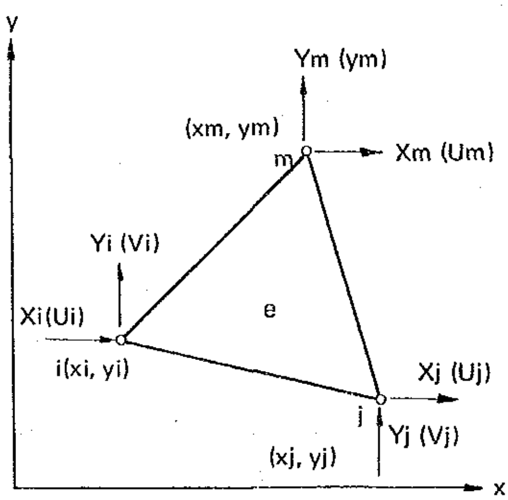
<u>Concept of analysis</u>

By using of the variation principle on the displacement function in an element, it is possible to obtain the structural property which have relations between the nodal forces and the nodal displacements induced there from.

A continuum is divided into limited numbers of triangular elements by a virtual boundary lines in two dimensional domain as shown in the right figure.

In considering a typical element (e) in the element group, the nodes are named i, j and m.

A function (displacement function) which defines the displacement in an arbitrary point in the element is shown in the following formula.



Stress or strain of triangular element or nodes displacement on two dimensional field.

$$\{f(x,y)\} = \begin{Bmatrix} U(x,y) \\ V(x,y) \end{Bmatrix} = [N]\{\delta\}^e = [Ni\ Nj\ Nm] \begin{Bmatrix} \delta i \\ \delta j \\ \delta m \end{Bmatrix} \cdots\cdots\cdots (1)$$

In this formula (1), a constituent of [N] is a function of ordinary coordinates (x, y), $\{\delta\}^e$ express a nodal displacement of an element, δi is a displacement of node (i) which has two constituents (Ui) and (Vi) in x and y directions, respectively. With summing up all the nodal displacements of all the elements in a whole continuum, the condition of compatibility of displacement is naturally satisfied.

$$\{\delta\} = [\ \delta_1\ \ \delta_2\ \ \cdots\cdots\cdots\ \ \delta_n\ ]^T \qquad \cdots\cdots\cdots (2)$$

On the other hand, a constituent of strain in the element is expressed by the following formula.

$$\{\varepsilon\} = \begin{Bmatrix} \varepsilon x \\ \varepsilon y \\ \gamma xy \end{Bmatrix} = \begin{Bmatrix} \partial u/\partial x \\ \partial v/\partial y \\ \partial u/\partial y + \partial u/\partial x \end{Bmatrix} = [B]\ \{\delta\}^e \qquad \cdots\cdots\cdots (3)$$

The above-mentioned matrix [B] forms a relation between nodal displacement and field of strain, and it is obtained as the function of Ni, Nj and Nm. If it is assumed that the material makes a linear-elastic movement in case that a material of the element receives the initial strain$\{\varepsilon_0\}$ produced by a temperature and a contraction, the relation between stress and strain is expressed as below:

$$\{\delta\} = \{\delta x\ \delta y\ \tau xy\}^T = [D]\ (\{\varepsilon\} - \{\varepsilon o\}) \qquad \cdots\cdots\cdots (4)$$

In this formula, the matrix [D] is an elasticity matrix which contains a material coefficient. The nodal force is expressed as follows which is acting on the nodes i, j and m of element e and is statically equivalent to a load on boundary and a distributed load in the element.

$$\{F\}^e = [Fi,\ Fj,\ Fm]^T,\ \{Fi\} = [Ui,\ Vi]^T \qquad \cdots\cdots\cdots (5)$$

The constituents U and V correspond to the direction U and V respectively.

In case that two constituents per unit volume are $\overline{X}$ and $\overline{Y}$, the force of object is expressed as below:

$$\{P\} = [\overline{X}, \overline{Y}]^T \qquad \cdots\cdots\cdot (6)$$

Then, in order to obtain the relationship between nodal force $\{F\}^e$ and nodal displacement $\{\delta\}^e$, the variation principle is adopted or the principle of virtual work is applied taking the virtual displacement into consideration.

Only the result is shown here omitting the process of the calculation.

$$\{F\}^e = [K]^e\{\delta\}^e + \{F\}_p^e + \{F\}_{\varepsilon o}^e \qquad \cdots\cdots\cdot (7)$$

Where, $[K]^e$: stiffness matrix of the element

$[F]_p^e$: nodal force being equivalent to the force of object

$[F]_{\varepsilon o}^e$: nodal force induced by the initial strain

These are obtained as below.

$$[K]^e = \int v[B]^T[D][B]d\ (VOL) \qquad \cdots\cdots\cdot\cdot (8)$$

$$[F]_p^e = -\int v[N]^T\{P\}d\ (VOL) \qquad \cdots\cdots\cdot\cdot (9)$$

$$[F]_{\varepsilon o}^e = -\int v[B]^T[D]\{\varepsilon o\}d\ (VOL) \qquad \cdots\cdots\cdot (10)$$

Moreover, in case that a boundary is affected by a distributed load of {g} per unit area, the load {g} shall be added to the node of the element which has a plane on the boundary. A nodal external force being equivalent to a distributed external force is obtained as below by the above-mentioned same process.

$$\{F\}^e g = -\int [N]^T \{g\} ds \qquad \text{..................} \quad (11)$$

From consideration of the relation (as called stiffness equation) between the nodal forces and their displacement in the whole structural system in which all the elements are united each other, the external force acting on each node is expressed by the formula (12).

$$\{R\} = [\ R_1\ R_2\ \text{.........}\ R_n\ ]^T \qquad \text{..................} \quad (12)$$

According to the balance of nodal force one each node, the following stiffness equation (13) is obtained.

$$\{R\} = [K]\{\delta\} + \{F\}_p + \{F\}_{\varepsilon o} \qquad \text{..................} \quad (13)$$

In this equation, $[K]$ is the stiffness matrix of a whole structural system, and $[K]$ is composed of the stiffness matrices $[K]^e$ of the respective elements. $\{F\}p$ and $\{F\}_{\varepsilon o}$ are obtained as the summations of the forces of surrounding elements in each node. From the equation (13), the stiffness equation regarding the elastic structure receiving the static loads given below:

$$[K]\{\delta\} = \{R\} - \{F\}_R - \{F\}_{\varepsilon o} = \{P\} \qquad \text{..............} \quad (14)$$

Therefore, the problem is solved by obtaining the solution of the following formula (15).

$$\{\delta\} = [K]^{-1}\{P\} \qquad \text{.................} \quad (15)$$

In the non-linear elasticity analysis, the modulus of elasticity or the Poisson's ratio corresponding to each stress level is re-valued in each calculation stage, and the new radix $[K]$ is reformed accordingly.

Condition of analysis

The special analysis has been carried out on the typical section of the conduit at the trench portion under the following conditions by using the finite element method. As for the condition of analysis at the conduit except the trench portion refer to the paragraph of the emergency outlet.

- The elastic modulus and Poisson's ratio for the embankment and foundation materials shall be handled as the linear materials.

- Stresses in term of strain are given by Hook's law in linear.

- From the results of geological investigations, the calculation model of the conduit at the trench portion to be applied for the special analysis is assumed as shown in Fig. D-19.

- Form of an element which possesses certain inherent advantage is a constant strain triangular element as shown in Fig. D-19.

- Division into the elements and arrangement of the nodes at the trench portion of the conduit are shown in Fig. D-19 and D-20 respectively.

- Case of analysis is set under the reservoir condition both full and empty.

- Input constants[1/] of the embankment and foundation materials and concrete members are assumed as shown in the following table.

1/ Since the effective in-situ tests of the input constants except density and shearing strength of the embankment materials have not been executed, those values were assumed based on the past data obtained in the similar materials in Japan.

Fig D-19. DIVISION INTO FINITE ELEMENT FOR CONDUIT AT SERVICE OUTLET (TRENCH PORTION)

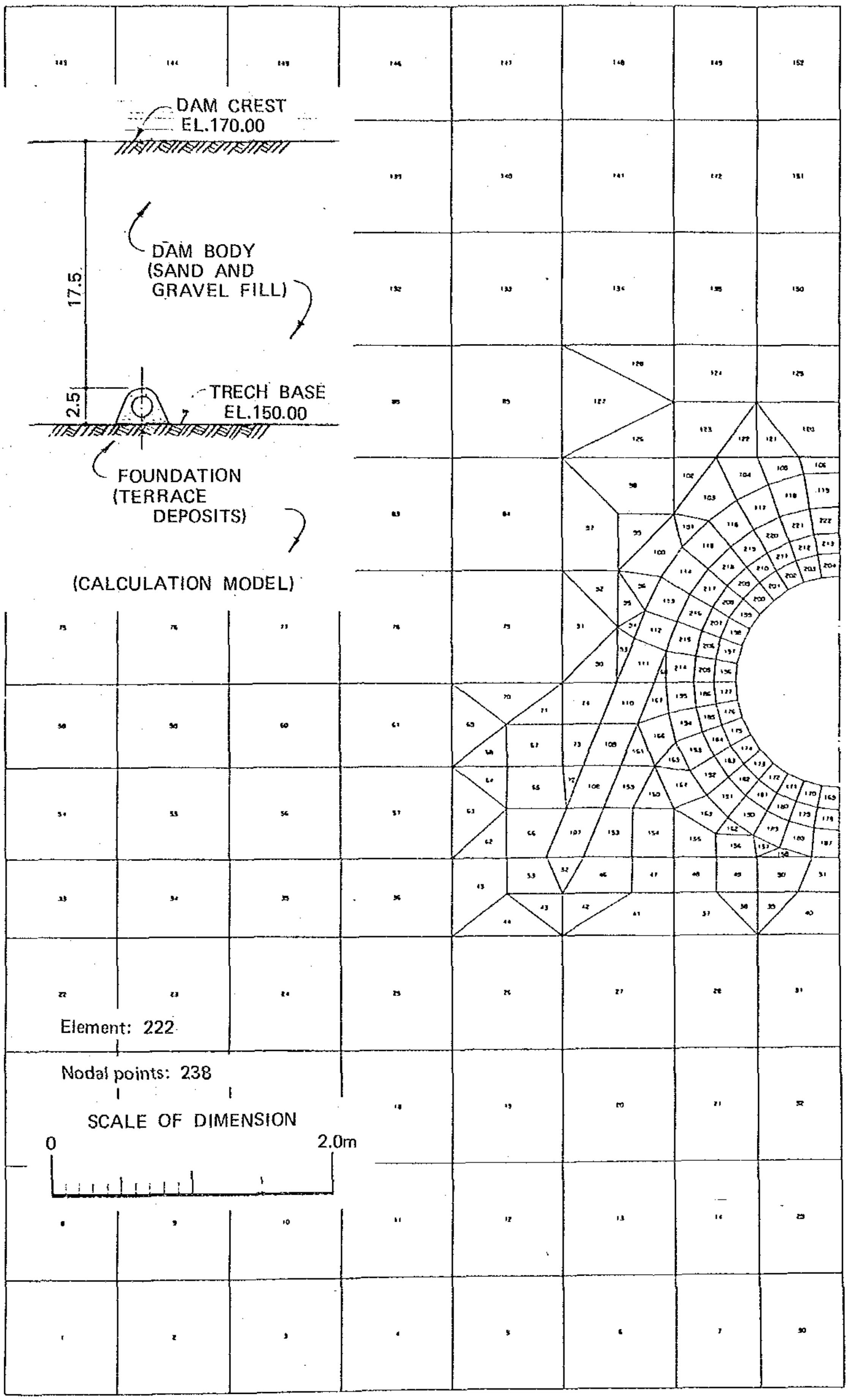
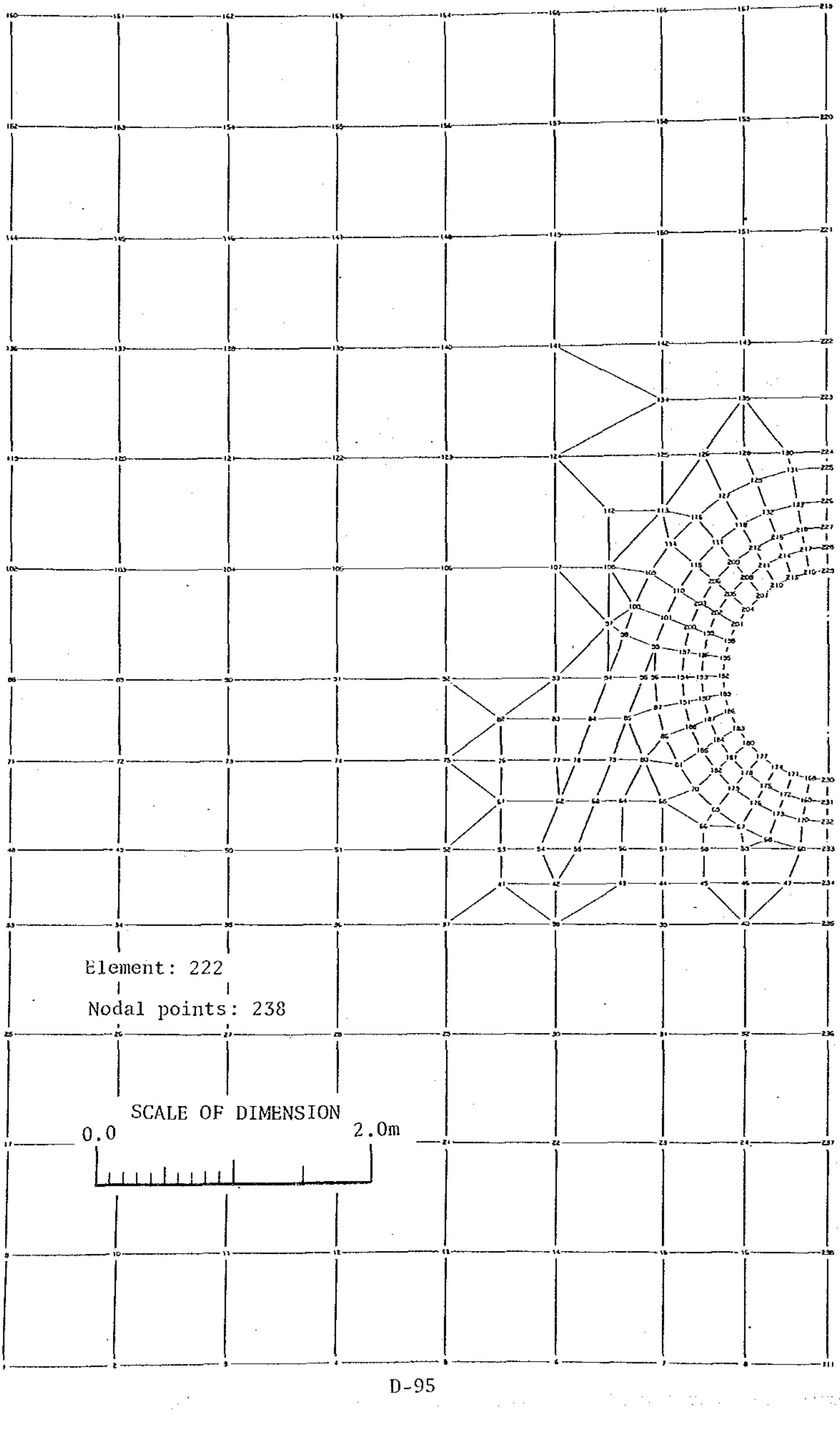


Fig. D-20. ARRANGEMENT OF NODAL POINTS FOR CONDUIT AT SERVICE OUTLET (TRENCH PORTION)



Section	Materials	Wet Density	Poisson's Ratio	Elastic Modulus	Shearing Strength ϕ	C
		(t/m^3)		(kg/cm^2)	(deg.)	(t/m^2)
Dam Body	Sand & Gravel Fill	2.06	0.38	800	35°00'	0.0
Foundation	Terrace Deposits	2.07	0.48	1,500	35°00'	0.0
Conduit	Concrete Member	2.40	0.17	210,000	33°00'	210.0

Result of analysis

The calculation results for the principal stress, safety factor of the element and displacement on representative points of the conduit section at the trench portion under the above-mentioned conditions are shown in Table D-6. The distributions of principal stress, tensile stress and safety factor of element for the conduit section at the trench portion are shown in Fig. D-21 and D-22, respectively.

Evaluation of analysis result

° Definition of terms

In this analysis, "the compressive stress" and "the tensile stress" means principal stress (δ_1, δ_3), and "the shear stress" means $1/2\ (\delta_1 - \delta_3)$ as shown in the right figure.

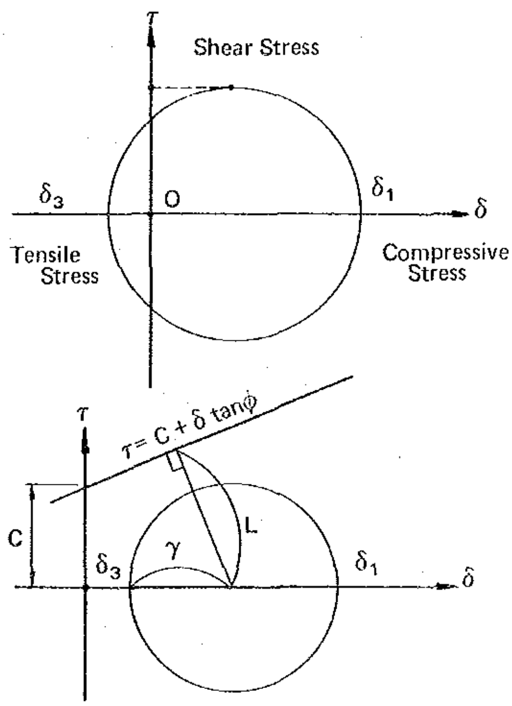
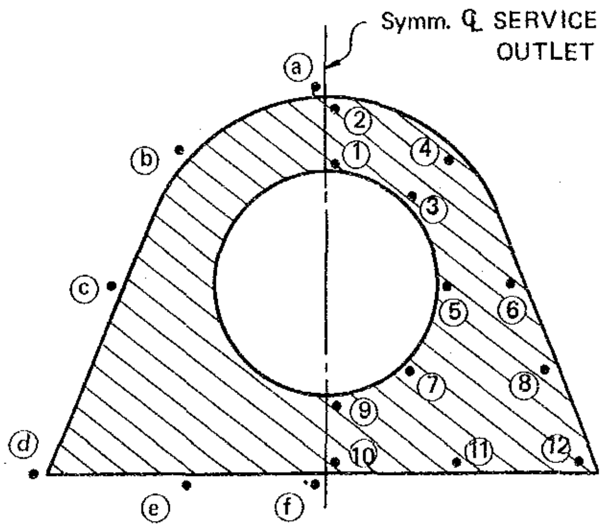


Table D-6. CALCULATION RESULTS OF CONDUIT AT SERVICE OUTLET (TRENCH PORTION)

No.	Element	Empty in Reservoir P-S 1/	Empty in Reservoir F.S. 2/	Empty in Reservoir Displacement 3/	Full Water in Reservoir P-S 1/	Full Water in Reservoir F.S. 2/	Full Water in Reservoir Displacement 3/
a	119	2.86	2.042	0.82	2.86	2.034	0.82
b	115	1.82	1.324	0.17	1.82	1.323	0.17
c	111	1.34	1.356	0.00	1.34	1.361	0.00
d	107	1.38	5.341	0.46	1.38	5.459	0.46
e	48	4.19	9.900	0.09	4.20	9.900	0.09
f	51	4.16	9.900	0.09	4.17	9.900	0.09
1	204	0.38	8.102	0.17	1.09	9.900	0.17
2	222	3.90	7.284	0.56	4.00	9.900	0.56
3	200	1.31	3.801	0.17	2.31	5.693	0.17
4	218	3.00	5.928	0.17	2.71	7.477	0.17
5	196	1.75	2.868	0.17	2.95	3.873	0.17
6	214	2.73	5.419	0.17	2.83	7.193	0.17
7	174	1.01	4.230	0.16	2.33	7.538	0.16
8	162	-0.49 (171)	5.629	1.46	-0.87 (169)	5.850	1.46
9	169or171	-0.29	6.246	0.16	-1.16	9.900	0.16
10	187	3.90	9.900	0.16	2.98	9.900	0.17
11	155	-2.04	5.560	1.24	-2.23	5.434	1.24
12	153	0.38	4.235	1.32	0.36	4.193	1.32



1/ Principal stress, symbols of + and - shown in compressive stress and tensile stress, respectively.

2/ Factor of safety for the elements

3/ Vertical displacement to the down direction

Unit of principal stress and displacement shows in kg/cm^2 and cm, respectively.

Fig. D–21. DISTRIBUTION OF PRINCIPAL STRESS AND TENSILE STRESS FOR CONDUIT AT SERVICE OUTLET (TRENCH PORTION)

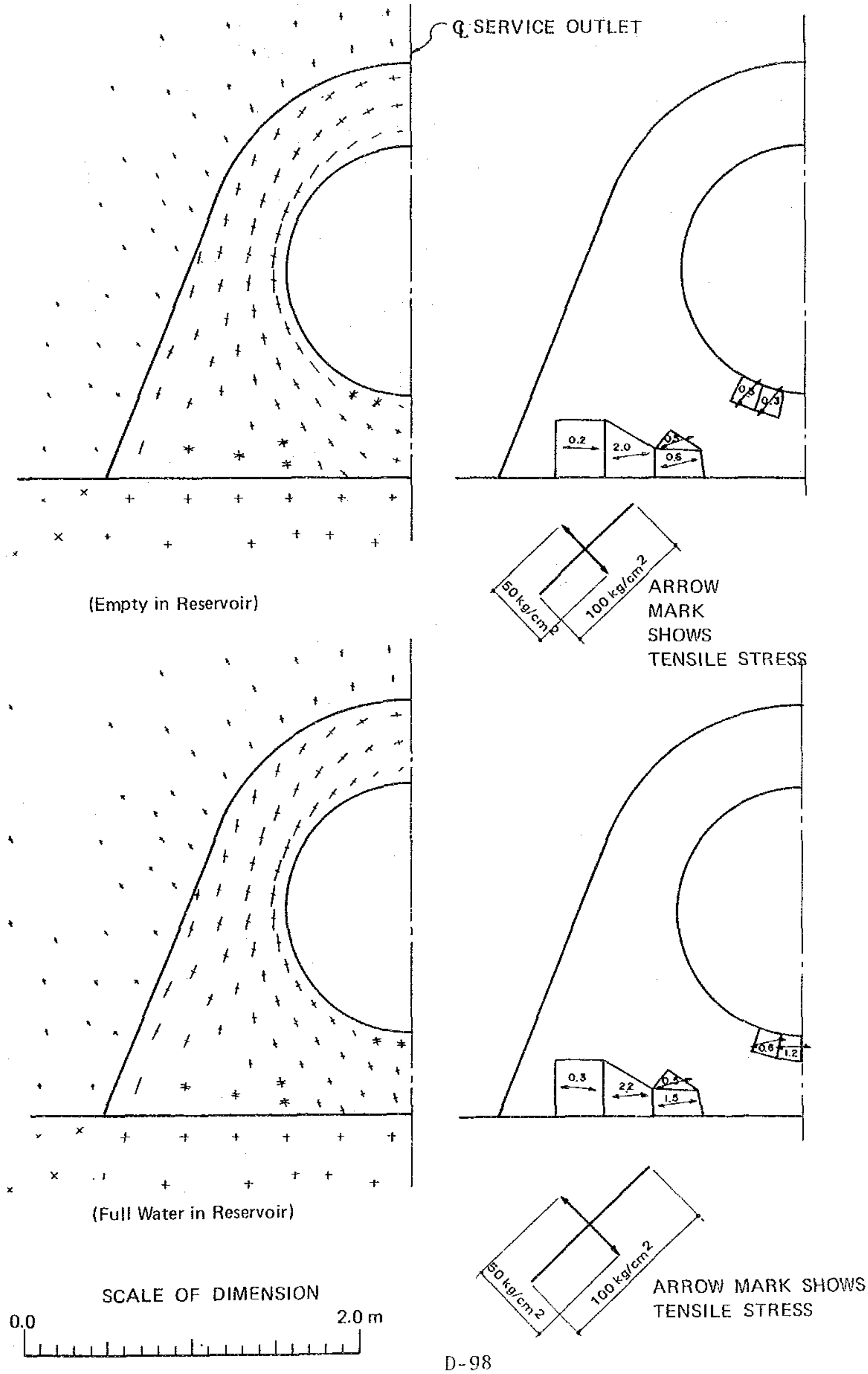
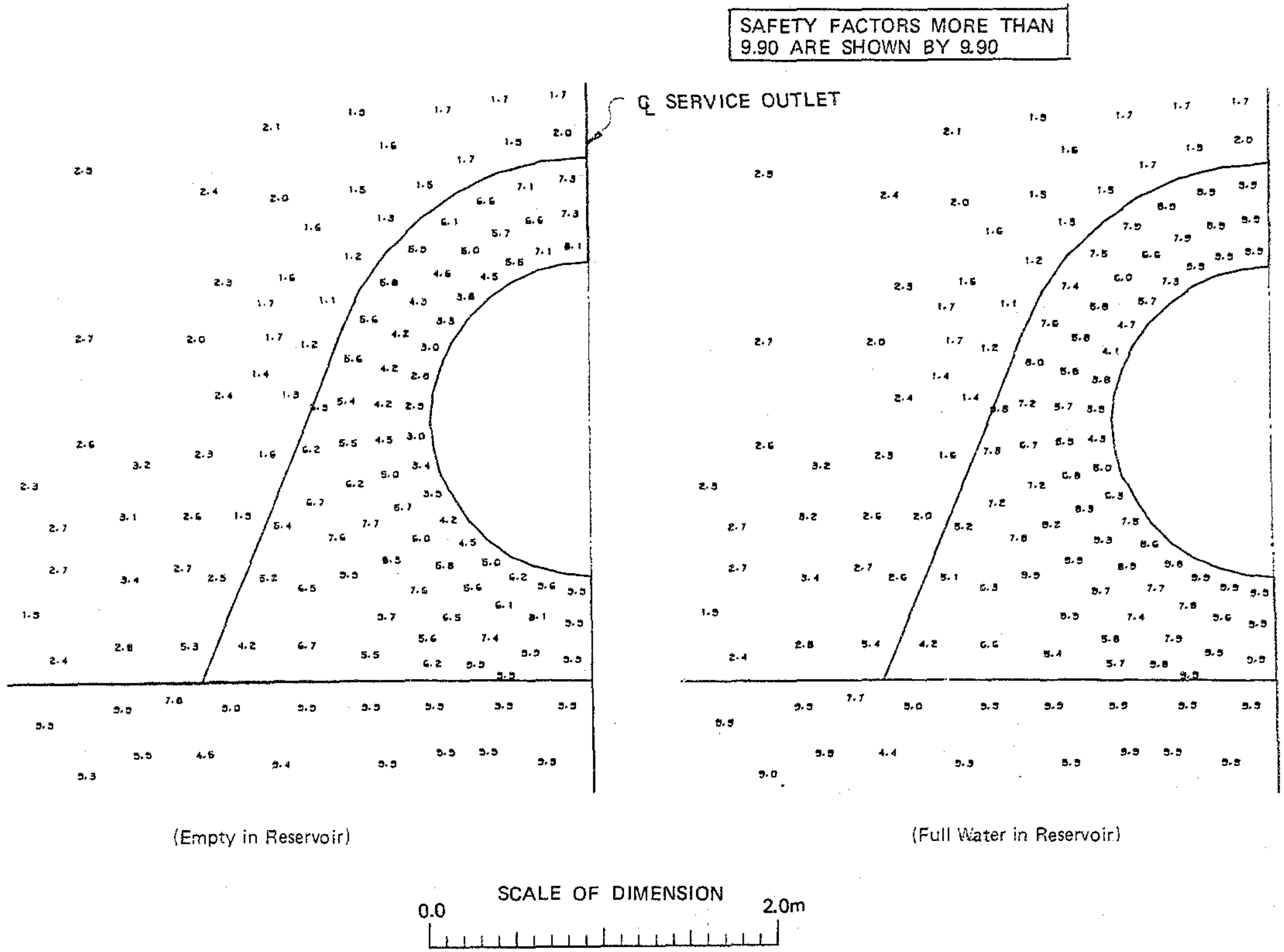


Fig. D–22. DISTRIBUTION OF SAFETY FACTOR FOR CONDUIT AT SERVICE OUTLET (TRENCH PORTION)



And also, the safety factor against shearing failure in a certain stress conditions is defined as the safety factor of element (S.F.) and given as below (refer to the above figure).

$$S.F. = \frac{L}{\gamma} = \frac{L}{1/2\ (\delta_1 - \delta_3)}$$

This formula can be transformed into the below.

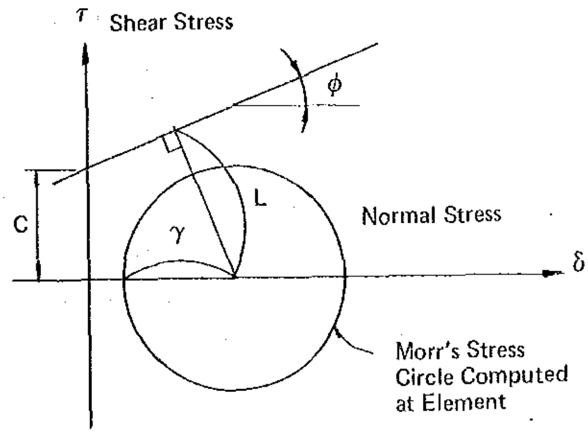
$$S.F. = \frac{2 \cdot C \cdot \mathrm{Cos}\,\phi + \mathrm{Sin}\phi\ (\delta_1 + \delta_3)}{(\delta_1 - \delta_3)}$$

° Evaluation method

Evaluation of the analysis results computed by the finite element method with two dimensional analysis can be made into two terms of shearing failure and tensile failure as mentioned below for the respective divided element.

Shearing failure

Shearing failure can be defined by the safety factor of element (S.F.) as follows by using the shearing strength: angle of internal friction (ϕ) and cohesion (c) in the right figure.

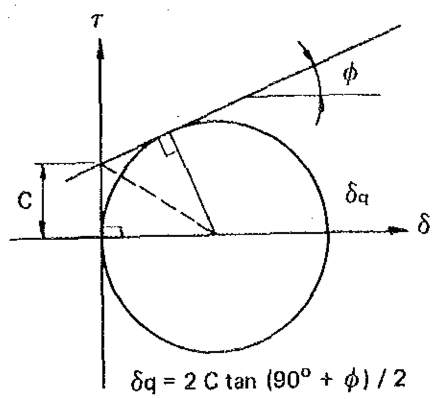


$$S.F. = \frac{L}{\gamma}$$

The safety factor of element defined by the above equation means that the shearing failure takes place when S.F. $\leq$ 1,0 is established.

Tensile failure

Basically, it is desirable that the tensile stress would not take place in the dam body and foundation. However, the tensile stress are occasionally observed in a certain place of the foundation adjoining to the conduit.



Judgement of result

As discussed above, the judgement can be made on the failure of the respective elements, whereas the stability of the whole dam body and foundation can not be defined. The reason that the finite element method analysis, which is made for the elasticity computation, will inevitably compute the tensile stress and/or a considerable shearing stress that are expected to happen in the place adjoining to a sharply varying slope and those with different physical properties.

Therefore, the stability for the dam body and foundation should be made taking into account the distribution of respective failure elements and those of concentration degree to be evaluated on an effect to the whole finite elements. In other words, the evaluation of analysis results will have to compare the data obtained past similar analysis since there is no definite standard.

° Conduit section at trench portion

From the distribution of tensile stress in the concrete members of the conduit at the trench portion (refer to Fig. D-21), the tensile stress develops partially at the invert members and the maximum value appears at outside face of the bottom portion. The required reinforcing bars at the trench portion of conduit can be estimated by using modified allowable stress of the bars which decreases from original value of 1,600 kg/cm^2 to 1,174 kg/cm^2 taking into account the acting inner head of 12.65 m and a circular shape of the conduit, and the results are shown in Table D-7.

Table D-7. ARRANGEMENT OF REINFORCING BARS FOR CONDUIT AT SERVICE OUTLET (TRENCE PORTION)

No.		No. of Element	Tensile Stress (kg/cm^2)	Tensile Force 1/ (kg)	Rein.Bar Requirement 2/ (cm^2)	Rein.Bar Arrangement 2/ (cm^2)
9	Inside	169	1.164	1,862.4	1.59	D13@300(2.38)
		170	0.621	993.6	0.84	"
		171	0.293	468.8	0.40	"
		154	0.277	969.5	0.83	"
10	Outside	155	2.228	5,792.8	4.93	D16@300(6.63)
		156	1.460)	3,521.2	3.00	"
		162	0.866			

From the above table, there is no special arrangement of the reinforcing bars in the most part of concrete members except the invert portion. However in the whole concrete members of the outlet conduit at the trench portion, the minimum arrangement of D16 @300mm as shown in Fig. D-23 is recommendable taking into account the values of input constants, calculation method and importance of outlet conduit in safety of the dam body.

1/ Tensile force obtains from multiply tensile stress by thickness of element.

2/ Reinforcing bar requirement and arrangement in the above table shows those of a length of 100 cm.

As for the arrangement of reinforcing bars at the conduit except the trench portion refer to the paragraph of the emergency conduit.

Since there is no failure element in any portion of the conduit section at the trench portion of the service outlet, there is no possibility to develop the shearing failure in the concrete members of the conduit.

° Surrounding of conduit

Since the tensile stress will not take place in the surrounding of the conduit and the foundation, there is no possibility to develop the tensile cracks around the conduit at the trench portion of the service outlet.

The result of shearing analysis revealed that there would not exist any elements with safety factor less than 1.0, the surrounding of conduit and the foundation at the trench portion of the service outlet has a sufficient shearing resistibility and there is no possibility of the shearing failure.

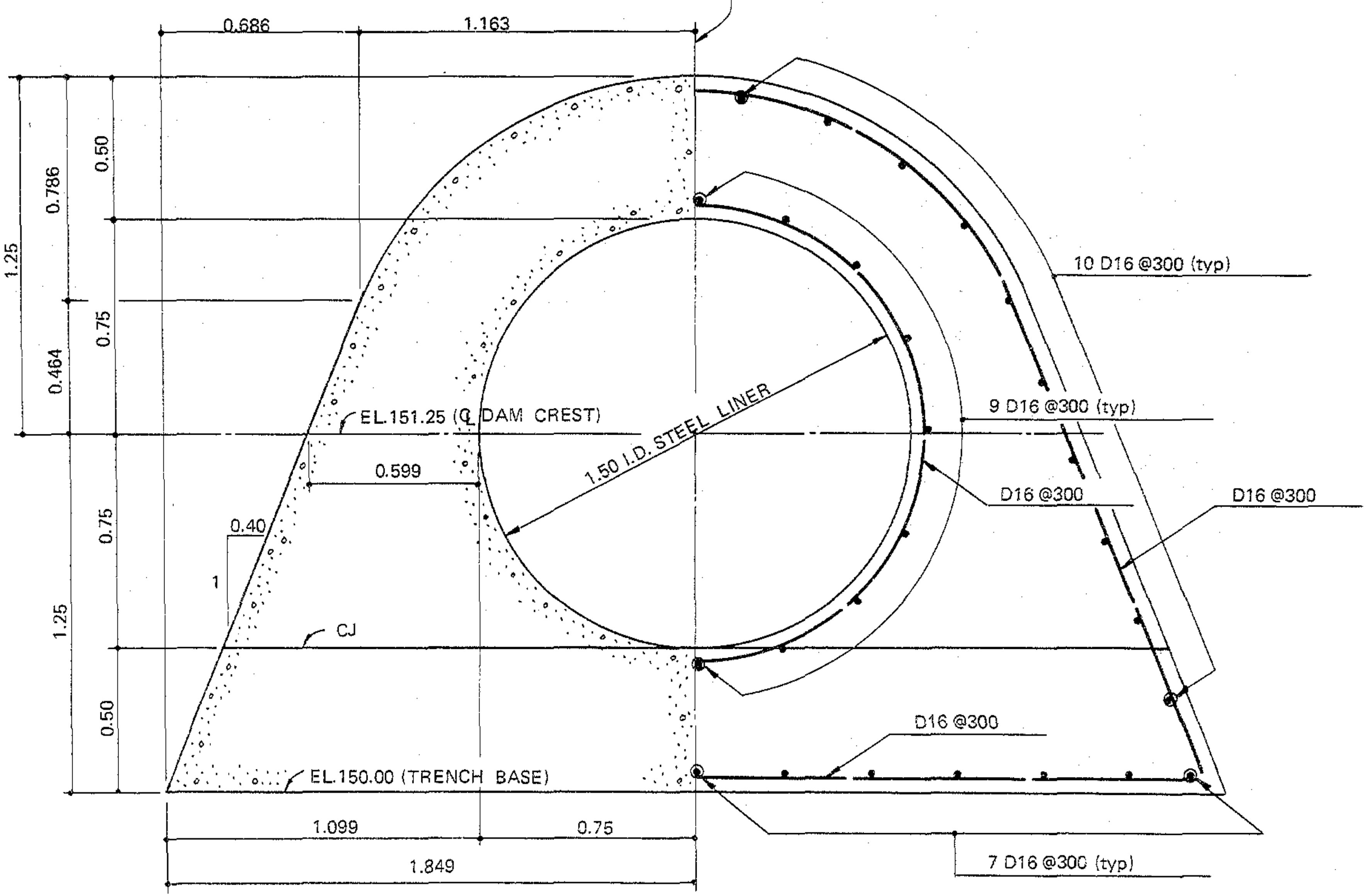
In order to cutoff the seepage water through the contact surface of dambody and conduit, the concrete cutoff wall should be placed at the trench base. The required length of cutoff wall can be obtained by the following formula.

$$C \cdot H < \frac{1}{f} \cdot (\ell + 2t), \quad t > \frac{1}{2} (\ell - f \cdot C \cdot H)$$

Where,
- t: length of cutoff wall
- f: safety factor and adopted by 1.2
- c: creep ratio and adopted by 4.0
- H: total head and adopted by 19.2 m (EL.169.20 - EL.150.0)
- ℓ: horizontal creep length and adopted by 94.0 m

Substituting these values into the above formula, the required length of cutoff wall is calculated to be 0.92 m, and another 0.08 m is added to this value, the length of cutoff wall is, therefore, fixed at 1.0 m.

Fig. D–23. ARRANGEMENT OF REINFORCING BARS FOR CONDUIT AT SERVICE OUTLET (TRENCH PORTION)



b) Drop inlet

The structural calculation for the drop inlet having 2.4 m square shape is made on the horizontal section at the bottom elevation of EL.150.81 m as a rigid-frame structure of the box culvert.

Load and bending moment

The loads to be taken into consideration in the structural calculation are hydrostatic pressure and earth pressure as follows:

Hydrostatic pressure $\quad Pw = w\ h = 3.69\ t/m^2$

Earth pressure $\quad Pe = we\ h\ \tan^2 (45° - \frac{\phi}{2}) = 2.06\ t/m^2$

Where, w: unit weight of water

h: water depth or back-fill depth, adopted by 3.69m

we: unit weight of back-fill material, adopted by $2.06\ t/m^3$

ϕ: internal friction angle of back-fill material, adopted by 35°

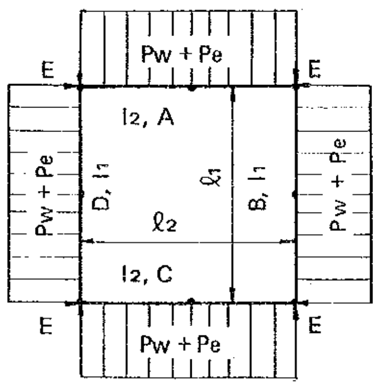
The bending moments at the points of A and E in the right figure are obtained by the following equations.

$$K = \frac{I_2}{I_1} \cdot \frac{\ell_2}{\ell_1} ,$$

$$M_E = - \frac{(Pw + Pe)}{12} \cdot \frac{\ell_1{}^2 + \ell_2{}^2 K}{1 + K}$$

$$M_A = Mc = \frac{(Pw + Pe)}{8} \ell_2{}^2 + M_E$$

$$M_B = M_D = \frac{(Pw + Pe)}{8} \ell_1{}^2 + M_E$$



Where, I_1, I_2: geometrical moment of inertia at members of B, D and A, C respectively.

ℓ_1 ℓ_2: span at members of B, D and A, C respectively

M_A, M_B, M_C, M_D, M_E: bending moments at points of A, B, C, D and E respectively.

In case of $I_1 = I_2$ and $\ell_1 = \ell_2$, the above equations can be transformed as follows:

$$M_E = -\frac{(Pw + Pe)}{12}\ell^2 = -4.030 \text{ t·m}$$

$$M_A = M_B = M_C = M_D = \frac{(Pw + Pe)}{8}\ell^2 + M_E = 2.015 \text{ t·m}$$

Where, M_E: bending moment at the end of span

Pw: hydrostatic pressure, adopted by 3.69 t/m^2

Pe: earth pressure, adopted by 2.06 t/m^2

ℓ: span, adopted by 2.90 m

M_A, M_B, M_C, M_d: bending moment at center of span

Arrangement of reinforcing bars

The required reinforcing bars can be estimated by the following equations on the condition that the allowable stresses for the concrete member and reinforcing bar are 70 kg/cm^2 (δca) and 1,800 kg/cm^2(δsa), respectively.

$$de = C_1 \sqrt{M/b} < d - d'$$

$$As = M / \delta sa \cdot j \cdot d$$

Where; de: effective thickness of member at section under consideration

C_1: coefficient relative to allowable stress of concrete member and reinforcing bar, adopted by 0.297

M: bending moment at section under consideration

b: unit width of concrete member, adopted by 100 cm

d: design thickness of member at section under consideration

d': covering thickness of reinforcing bar, adopted by 10 cm

As: area of tensile reinforcing bar

δsa: allowable tensile stress in reinforcing bar, adopted by 1,800 kg/cm^2

j; distance from computed center of compression to centroid of tensile reinforcing bar relative to effective thickness de, adopted by 7/8

The required reinforcing bars and effective thickness of the members at the points of center and end of the span are shown in the following table.

Calculation Point	Effective Thickness	d - d'	Bending Moment	Rein.Bars Requirement
	(cm)	(cm)	(t m)	(cm^2)
End of Span	18.85	40	4.030	6.40
Center of Span	13.33	40	2.015	3.20

From the above table, the arrangement of reinforcing bars at the drop inlet are planned to place D16 deformed bars with 30 cm intervals (As = 6.63 cm^2) at the both sides of the concrete members.

c) Steel liner

In order to prevent the corrosion and abrasion on the inside surface of the conduit and to ensure the watertight function, a steel liner with inside diameter of 1.50 m is installed at the whole length of the conduit. And also, this steel liner has a role as the inner form for placing concrete.

The minimum thickness of steel liner shell can be obtained by the following equation with assuming that the allowable tensile stress of the rolled steel is 1,300 kg/cm^2.

$$t = \frac{P \cdot D}{2\ \delta sa} + \varepsilon > 6\text{mm} \quad \text{(Minimum thickness)}$$

Where, t: thickness of steel liner shell (cm)

P: acting inner pressure, adopted by 1.29 kg/cm^2

D: inside diameter, adopted by 150 cm

δsa: allowable tensile stress of rolled steel, adopted by 1,300 kg/cm^2

ε: allowance of corrosion and abrasion, adopted by 0.15 cm

Regardless of the above calculation result (t = 2.24 mm), the thickness of steel liner shell is employed by 6 mm under no stiffeners condition.

5.3. Emergency Outlet

The emergency outlet will be embedded beneath the dam body at the left abutment in order to release the stored water through this outlet when the service outlet is blockaded by unexpected accidents. A circular section provided by steel pipe with 1.50 m inside diameter is planned as the conveyance conduit equipped with trashrack and slide gate at the inlet and outlet portals of the conduit, respectively. The slide gate is usually closed, however in case of blockade at the service outlet, this slide gate will be operated by manpower.

The inlet sill is located at elevation of 158.0 m to keep the facilities free from the sediment trouble.

1) Hydraulic design

The outlet discharge for the emergency outlet with inside diameter of 1.50 m varies by the reservoir water levels and can be obtained by the following equation.

$$Q = \frac{\sqrt{2g} \cdot A}{\sqrt{ft + fe + fr + fv}} \sqrt{H} = 4.6864 \sqrt{H}$$

Where, Q: outlet discharge of the emergency outlet as a pipeline flow

g: acceleration of gravity, adopted by 9.8 m/sec^2

A: flow area at the conduit, $A = 0.7854\ D^2 = 1.7672\ m^2$

D: inside diameter of conduit, adopted by 1.50 m

ft: coefficient of trashrack loss varies by blockade rate of the trashrack, ft = 0.21 in case the blockade rate of 10%

fe: coefficient of entrance loss, adopted by 0.50

fr: coefficient of friction loss at the conduit

$$fr = 124.5\ n^2/D^{4/3} \times L$$

n: coefficient of roughness, adopted by 0.015

L: length of the conduit, adopted by 66.0 m

fv: coefficient of changing velocity loss, adopted by 1.00

H: total head, measured from top of crown at the end of conduit

The calculation results under various water levels at the reservoir are shown in the following table and is drawn in Fig. D-17.

	Water Surface Elevation in Reservoir (m)				
	EL.160.17	EL.161.17	EL.162.17	EL.163.17	EL.163.90
Total Head (m)	1.00	2.00	3.00	4.00	4.73
Outlet Discharge (cu.m/sec)	4.69	6.63	8.12	9.37	10.19
Velocity (m/sec)	2.65	3.75	4.59	5.30	5.77

Judging from the velocity in the above table, the running water through the emergency outlet possesses some energy to bring about erosion and scouring, therefore, the protection works with wet masonry and gabion should be executed at the tailrace.

2) Structural analysis

a) Conduit

The outer section of the conduit has a special shape as shown in Fig. D-18 in order to eliminate the occurrence of the tensile cracks around the conduit, therefore, it may be necessary to execute a special analysis for the distribution of tensile and shearing stresses in or around the reinforced concrete members of the conduit by using the finite element method.

The methodology and concept of analyses are described in the previous paragraph of the service outlet.

Condition of analysis

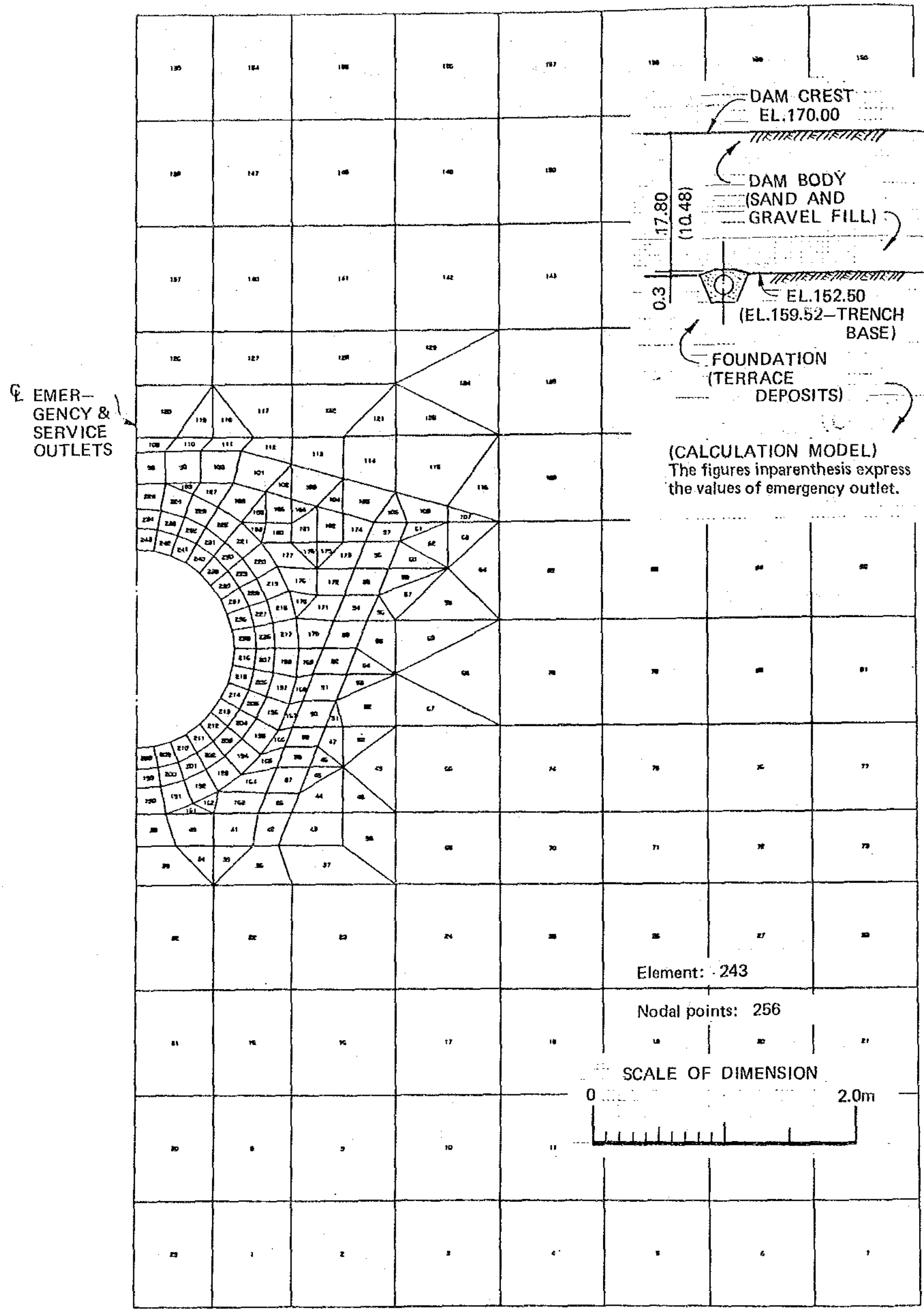
The special analysis has been carried out on the typical section of the conduit under the following conditions.

- ° The elastic modulus and Poisson's ratio for the embankment and foundation materials shall be handled as the linear materials.

- ° Stresses in terms of strain are given by Hook's law in linear.

- ° From the results of geological investigations, the calculation model of the conduit to applied for the special analysis is assumed as shown in Fig. D-24.

Fig. D-24 DIVISION INTO FINITE ELEMENT FOR CONDUIT AT EMERGENCY OUTLET



- Form of an element which possesses certain inherent advantage is a constant strain triangular element as shown in Fig. D-24.

- Division into the elements and arrangement of the nodes are shown in Fig. D-24 and D-25, respectively.

- Situation of the dam is considered as after completion of the embankment with empty and full water level at the reservoir.

- Input constants 1/ of embankment and foundation materials and concrete members are assumed as shown in the following table.

Section	Materials	Wet Density	Poissons's Ratio	Elastic Modulus	Shearing Strength ϕ	Shearing Strength C
		(t/m^3)		(kg/cm^2)	(deg.)	(t/m^2)
Dam Body	Sand & Gravel Fill	2.06	0.38	800	35°00'	0.0
Foundation	Terrace Deposits	2.07	0.48	1,500	35°00'	0.0
Conduit	Concrete Member	2.40	0.17	210,000	33°00'	210.0

Result of analysis

The calculations are made on the typical section of the service outlet and the results for the principal stress, safety factor of element and displacement on representative points of the conduit section under the above-mentioned conditions are shown in Table D-8. The distributions of principal stress and safety factor of element for the conduit section are shown in Fig. D-26 and D-27, respectively.

1/ Since the effective in-situ tests of the input constants except density and shearing strength of the embankment materials have not been executed, those values were assumed based on the past data obtained in the similar materials in Japan.

Fig. D-25 ARRANGEMENT OF NODAL POINTS FOR CONDUIT AT EMERGENCY OUTLET

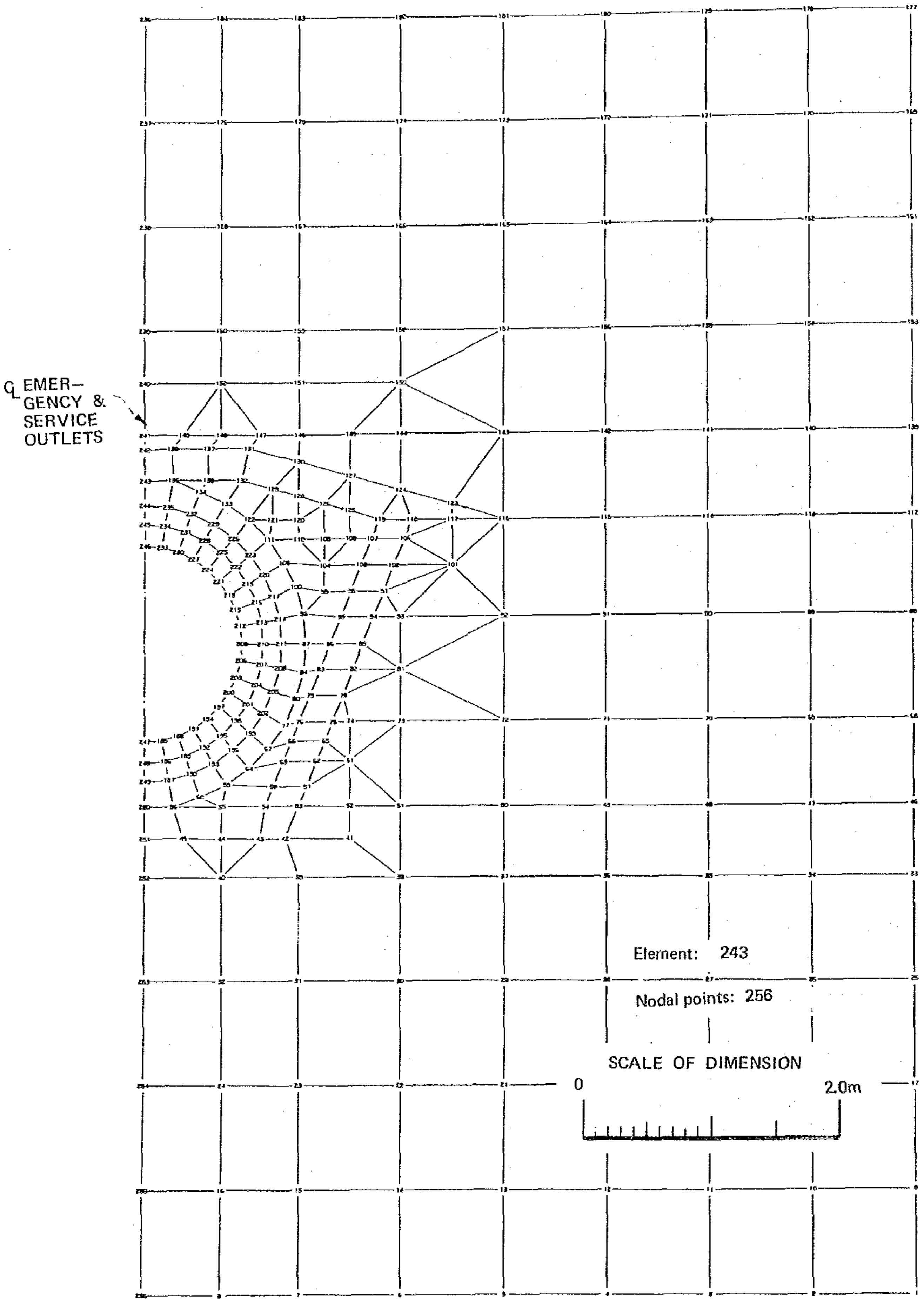
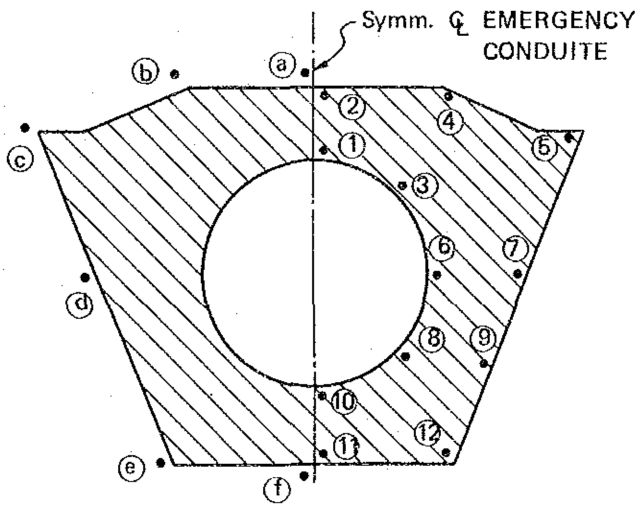


Table D-8. CALCULATION RESULTS OF CONDUIT AT EMERGENCY AND SERVICE OUTLETS

		Empty in Reservoir			Full Water in Reservoir		
No.	No. of Element	P-S 1/	F.S. 2/	Dis- 3/ placement	P-S 1/	F.S. 2/	Dis- 3/ placement
a	98	1.53	2.450	0.10	1.53	2.425	0.10
b	101	1.51	2.228	0.10	1.50	2.227	0.10
c	108	1.55	2.685	0.10	1.54	2.694	0.10
d	55	2.10	9.900	0.10	2.10	9.900	0.10
e	43	2.60	8.544	0.09	2.61	8.384	0.10
f	39	2.66	9.900	0.08	2.68	9.900	0.08
1	243	0.63	9.900	0.11	1.53	9.900	0.11
2	225	2.29	9.900	0.10	2.37	9.900	0.11
3	240	0.47	6.894	0.23	1.35	9.900	0.23
4	188	2.22	9.900	0.10	2.12	9.900	0.10
5	97	1.88	9.900	0.10	1.98	9.900	0.10
6	235	0.96	5.760	0.11	1.82	9.864	0.11
7	93	2.31	9.900	0.10	2.31	9.900	0.10
8	213	0.84	6.529	0.10	1.76	9.900	0.10
9	89	2.21	9.900	0.09	2.21	9.900	0.09
10	208	0.55	9.900	0.10	1.50	9.900	0.10
11	190	2.57	9.900	0.10	2.65	9.900	0.10
12	86	1.74	9.900	0.10	1.64	9.900	0.10



1/ Principal stress, symbols of + and - shown in compressive stress and tensile stress, respectively.

2/ Factor of safety for the elements

3/ Vertical displacement to the down direction

Unit of principal stress and displacement shows in kg/cm^2 and cm, respectively.

Fig. D-26 DISTRIBUTION OF PRINCIPAL STRESS FOR CONDUIT AT EMERGENCY OUTLET

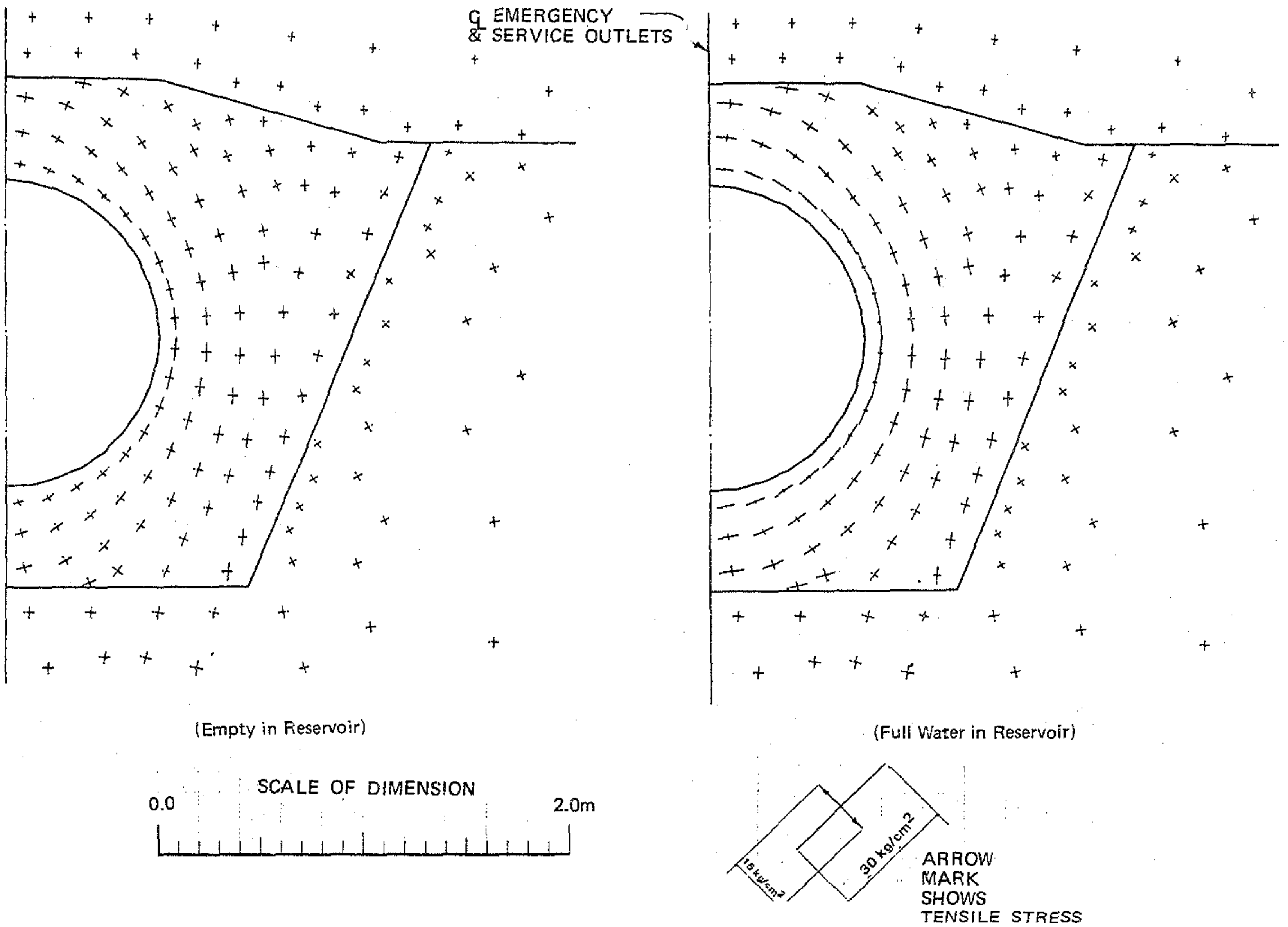
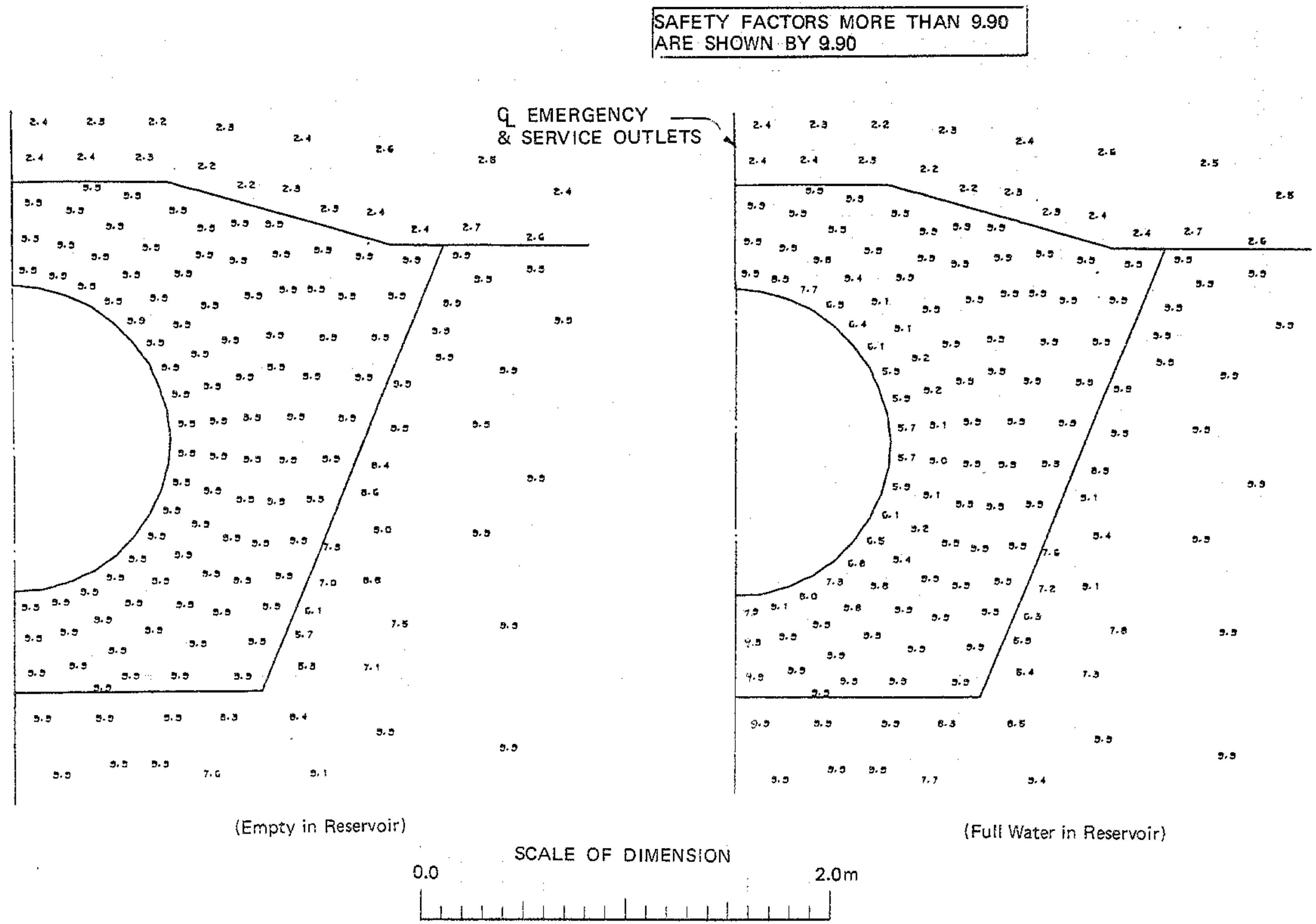


Fig. D–27 DISTRIBUTION OF SAFETY FACTOR FOR CONDUIT AT EMERGENCY OUTLET



Evaluation of results

The evaluation method of analysis results in terms of shearing failure and tensile failure is same as the service outlet described in the previous paragraph.

° Conduit section

From the distribution of principal stress in the concrete members of conduit (refer to Fig. D-26), the tensile stress would not develop any elements and there is no special arrangement of the reinforcing bars. However in the concrete members of the emergency outlet, the minimum arrangement of D13 @300 mm as shown in Fig. D-28 is desirable taking into account the values of input constants and calculation method. As for the arrangement of reinforcing bar at the service outlet except the trench portion, it is recommendable to place D16 deformed bars with 30 cm intervals at both sides of the conduit section as shown in Fig. D-28.

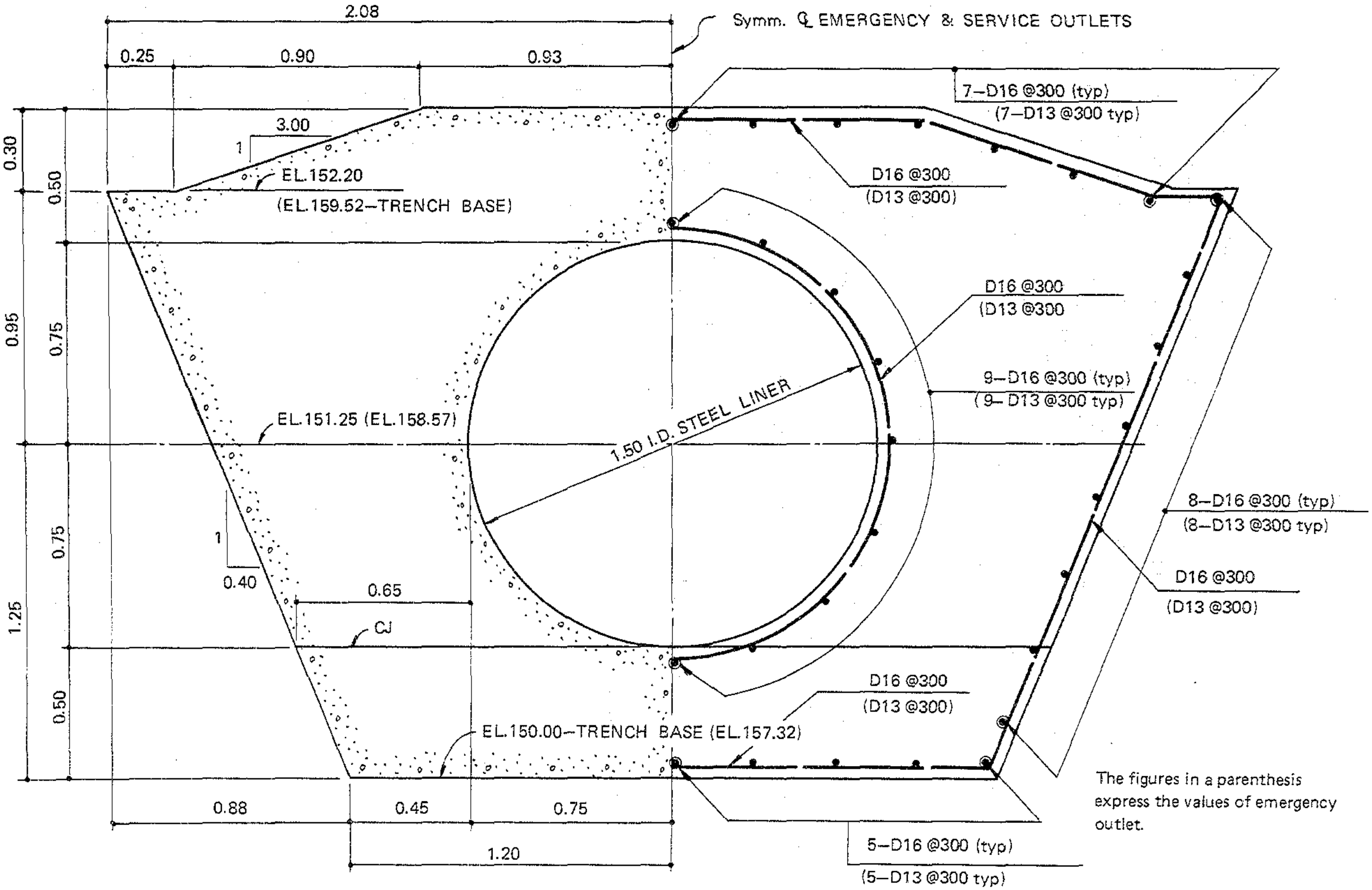
Since there is no failure element in any portion of the conduit section, it may be impossible to develop the shearing failure in the concrete members of the conduit.

° Surrounding of conduit

The tensile stress will not take place in the surrounding of the conduit, therefore, it seems that there is no development of the tensile cracks around the conduit.

The results of stress analysis revealed that there is no elements with safety factor less than 1.0, the surrounding of conduit and the foundation has a sufficient shearing resistibility and there is no possiblity of shearing failure.

Fig. D–28. ARRANGEMENT OF REINFORCING BARS AT EMERGENCY AND SERVICE OUTLETS



The figures in a parenthesis express the values of emergency outlet.

In order to cutoff the seepage water through the contact surface of dambody and emergency conduit, the concrete cutoff wall with 1.0 m length is placed at the trench base taking into account the calculation result of service outlet, acting head and creep length of the emergency outlet.

b) Steel liner

In order to prevent the inner surface of the conduit from corrosion and abrasion and to ensure the watertight function, a steel liner with inside diameter of 1.50 m is installed at the whole length of the conduit. And also, this steel liner has a role as the inner form for placing concrete.

The minimum thickness of steel liner shell is employed by 6 mm under no stiffeners condition taking the calculation result of service outlet and acting inner head of the emergency outlet into considerations.