4.1.2 Geotechnical Investigation

(1) Introduction

The following items were investigated in the period from 10th August to 19th September 1991:

1) 2-mechanical borings (ϕ100-75mm) on banks of the Canal; BH-1 boring of 50.5m was drilled at west side bank (15m south side of the Segment NO.464) and BH-2 boring of 65m was drilled at east side (15m north side of the Segment NO.908). Their locations are shown in Fig. 4.1.2.

2) 12-core drillings (ϕ100-65mm) from inside of the Tunnel: 3-cores of 2 to 3m depths (one vertical drill of in vert and two horizontal drills at walls) were respectively drilled at 4 sections (Segment Nos. 449,759,908 and 1232).

3) 3-non-core drillings (ϕ 22mm) at crown of the Tunnel: the back side of existing Segments, at Nos. 335, 632 and 915 were observed by a fiber scope instrument.

4) Other field tests: such as standard penetration test(SPT), PS wave logging and permeability test were performed.

5) Laboratory tests: unconfined compression test, etc. were carried out in Cairo.

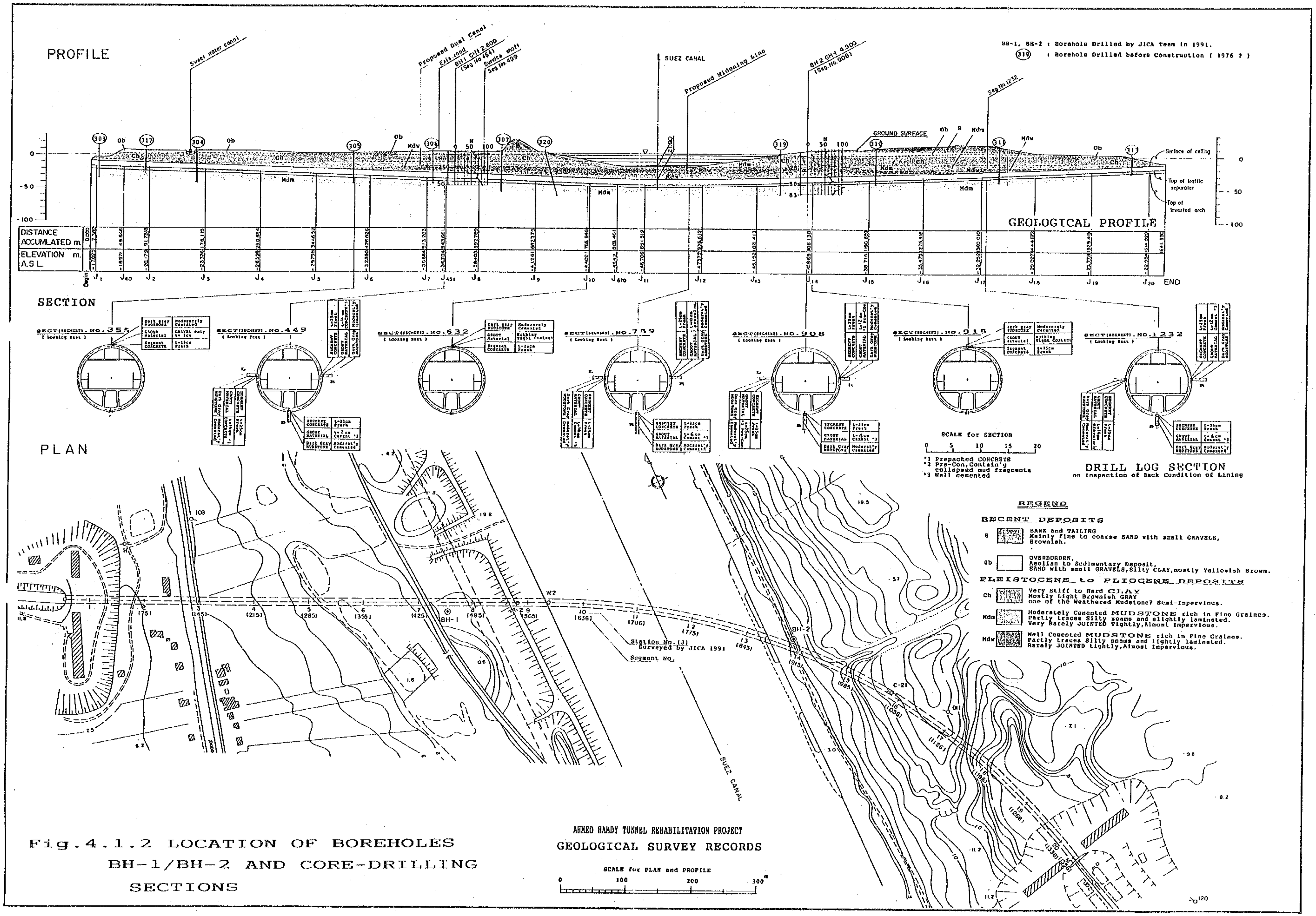


Fig.4.1.2 LOCATION OF BOREHOLES BH-1/BH-2 AND CORE-DRILLING SECTIONS

(2) Geology around the Tunnel

The fundamental stratification of the site is summarized as follows:

1) The thin surface layer of less than 10m consists of
   - alluvial desert aeolian sandy soil and
   - lacustrine silty soil
2) The underlying layers of more than 100m are considered to be mainly formed of mudstone which had been deposited in the periods of Pleistocene or Pliocene. The Tunnel is mainly situated in one of these mudstone layers.

Technical considerations are given to each layer by referring to the boring records as shown in Fig.4.1.3 and Fig.4.1.4.

Bank(B): There is a windbreak levee of approximaely 17m in hight along the west side of the Canal. Soil of the bank mainly consists of fine or medium sand brownish in colour and includes some rounded selicious pebbles and gravels. It is supposed that the levee was embanked with tailings of the Canal dredging. During the banking, the fine clay and silt which may have been included in the original have been blown away resulting into the existing sand with pebble and gravel layers.

Surface soil(Ob): The over burden mainly consists of desert aeolian sandy soils and lacustrine silty soils. On west side of the Canal, the soil is mainly laid by silt layers inter-bedded with thin sand bands. The sand bands are only of slight cementation even though the (S.P.T) N values is measured as high as around 25.

For both west side and east side of the Canal, the surface soil mainly consists of high permeable aeolian sand.

The ground water widely distributed in this overburden overlays on the impermeable mudstone. It is supposed that the groundwater is recharged from the Canal due to saline condition.

Hard clay(Ch): This layer is supposed to be a weathered zone of the underneath mudstone caused by the characteristic of high plasticity.

This layer shows semi-impervious condition even in the partly intercalated silty sand bands.

The thickness of this layer is thought to be 12 to 15m, which lies -12 to -15m in elevation height.

Moderately cemented mudstone(Mdm): This layer sandwiches a well cemented mudstone (Mdw) from upper and lower sides. The upper layer is dark gray with a thickness of 7 - 9m having an intercalated siltrich thin layer (several,mm) and has lateral laminations. The lower layer is supposed to be at the depth of - 30m to - 100m or more.

Siltrich thin layers are seen at the upper portion which includes shell fossil fragments and discolourationed thin layers are also intercalated having very high (SPT) N values of 60 to 70. The tunnel is mainly situated in this lower layer.

Well cemented mudstone (Mdw): This layer is nearly impervious with a thickness of 5m and (SPT) N value of 100 to 150.

The present canal base is situated in this layer. The following Figures 4.1.3. and 4.1.4. Drill Log of BH-1 & BH-2 provide a description of each of these layers.

DRILL LOG

HOLE NO. BH-1 SHEET NO. 1 OF 1

PROJECT	AHMED HAMDY TUNNEL REHABILITATION PROJECT	DEPTH	50.5m	ELEVATION	+2.8m
SITE	West(15m South of CL on Seg.No464)	INCLINATION	Vertical	DRILL RIG	Rotary
AVERAGE CORE RECOVERY	DATE 15/8 '91~26/8 '91	DRILLED	ARDAMAN-ASE	LOGGED	M.CHIDA

DATE	DEPTH	ELEVATION	ROCK TYPE OR FORMATION	DESCRIPTION
15/8	0.45	2.35	TAILING	Desert Sand or Tailing
	2.80	0.00	Aeolian~Alluvial SAND	Dense Desert SAND interlayers Stiff silty CLAY BROWNISH GRAY
	3.60	-0.80		
	15.00	-12.20	Hard Silty CLAY	Lightly cemented Silty CLAY, Very stiff. More than containing of 70 % of fine graines, Quite sticky. Slightly laminated and interlayers of silty bands YELLOWISH BROWN. Expecting Weathered zone of MUDSTONE. Rarely shown Banded cristal of Gypsums
	23.00	-20.20	Mode-rately Cemented MUDSTONE	Solid Clayey soft MUDSTONE DARK GRAY. In graine distribution, mainly consists of CLAY. Core recovered 60% by single wet drilling. Lateral laminations are often seen.
	29.00	-26.20	Well Cemented MUDSTONE	Well Cemented DARK GRAY MUDSTONE. Slight lamination often seen. High dipped tight faced joints rarely distributed. Fairly massive
26/8	50.50	-47.70	Mode-rately Cemented MUDSTONE	Moderately Cemented MUDSTONE DARK GRAY. Lateral laminations remarkablly seen, and silty band discoloured bands are often seen. Mainly consists of fine graines. High dipped and tight faced Joints rarely observed. Crystal Gypsum lenses banded in 43m in depth. High solidity and high Core recovery is shown. Discoloured seams often observed along the laminations with stripe tones.

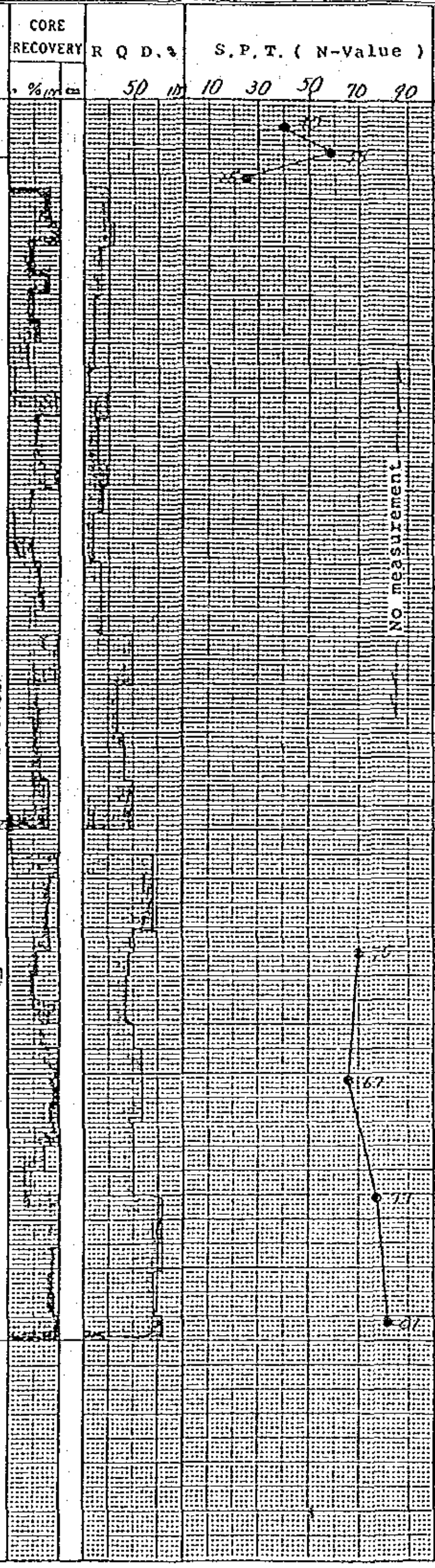
32.4 — Tunnel Formation — 44.0

Diameter drilled; 75mm to bottom
100mm casing to 15m.

ボーリングφ75ミリ

SAMPLE TESTING: 91:38.5; Tr.ax.1 31; Q4.53; Tr.ax2 37.0; Q1; Q3 41.8

GROUNDWATER LEVEL: GL-2.40; Stable during drilling; Equilibrium table; Packer; 27; 36.5; Test Dewatering



LOG FORM-B

HOLE NO. BH-1

NIPPON KOEI CO., LTD.
CONSULTING ENGINEERS, TOKYO.

*R.Q.D is Rock Quality Designation, R.Q.D = (Total length of cylindric cores longer than 10 cm)/(Total core length) x 100%
*LUCEON VALUE is l/min/m under injection water pressure of 10kg/cm²
*DEPTH and ELEVATION are in meter
*DIAMETER is in millimeter

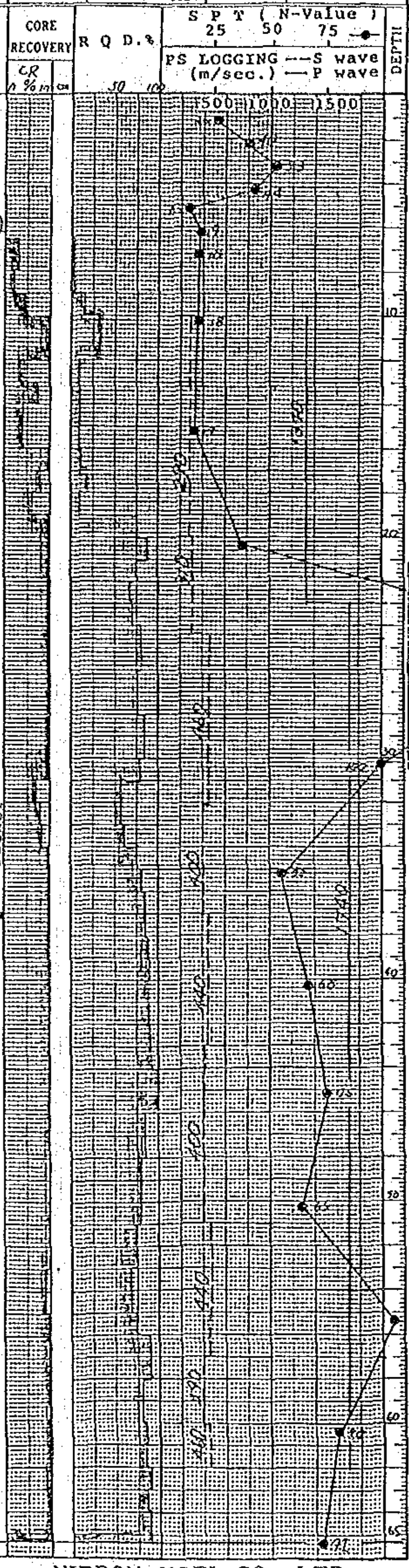
Fig.4.1.3 DRILL LOG OF BH-1

DRILL LOG

HOLE NO. BH-2 SHEET NO. 1 OF 1

PROJECT	AHMED HAMDY TUNNEL REHABILITATION PROJECT	DEPTH	65.5m	ELEVATION	+4.9m
SITE	East (15m North of CL on Seg. No.908)	INCLINATION	Vertical	DRILL RIG	Rotary
AVERAGE CORE RECOVERY	DATE 1/9'91~15/9.'91	DRILLED	ARDAMAN-ASE	LOGGED	M.CHIDA

DATE	DEPTH	ELEVATION	ROCK TYPE OR FORMATION	DESCRIPTION	SAMPLE TESTING	GROUNDWATER LEVEL
1/9	4.50	+0.40	TAILING or BANK SAND	Desert or Tailing SAND Dense, partly cemented. Yellowish BROWN		
	4.90	0.00				
	6.50	-1.60	Aeolian~Alluvial SAND	Desert SAND Inter-layers Stiff SILT Yellowish BROWN		GL -6
	20.00	-15.10	Hard Silty CLAY	Lightly cemented Silty CLAY, Very stiff and high plastisity. More than containing of 70 % of fine graines, Quite sticky. Slightly laminated and interlayers of silty bands. YELLOWISH BROWN. Expecting Weathered zone of MUDSTONE. Interlayered of 2cm Cristal Gypsum in 15m depth.		Stable during drilling
	24.00	-19.10	Moderately Cemented MUDSTONE	Solid Clayey soft MUDSTONE. DARK GRAY. Lateral laminations are often seen.		
	33.00	-31.10	DARK GRAY Well Cemented MUDSTONE	Well Cemented DARK GRAY MUDSTONE. Slight lamination often seen. High dipped tight faced joints rarely distributed. Fairly massive. In graine distribution, mainly consists of CLAY		Equilibrium table
12/9	65.00 (39.1 – 50.7 Tunnel Formation)	-60.60	Moderately Cemented MUDSTONE	From 33m little decreasing of solidity is seen. Moderately Cemented MUDSTONE DARK GRAY. Lateral laminations remarkablly seen, and silty band discoloured bands are often seen. Mainly consists of fine graines. High dipped and tight faced Joints rarely observed. Crystal Gypsum lenses banded in 43m in depth. High solidity and high Core recovery is shown. Discoloured seams often observed along the laminations with stripe. In 53m a little of leakages are seen while drilling even the table has kept approx.6m deep with stable, that will reason leak to the tunnel. Diameter drilled; 75mm to bottom 100mm casing to 12m.	36.7–38.0; 40.6–41.0; 44.3–45; 48.0–48.9; 53–53.2	Packer; Test, Dewatering



HOLE NO. BH-2

*R.Q.D is Rock Quality Designation. R.Q.D = (Total length of cylindric cores longer than 10 cm)/(Total core length) × 100%

LOG FORM-B

NIPPON KOEI CO., LTD.

Fig. 4.1.4 DRILL LOG OF BH-2

(3) Back-side of Segments

Loosening of the bedrock (Mdm) was observed in the depth of 0.5m to 1.0m for some core drillings to the side and to the bottom but not all. At the back-side of tunnel crown, the actual segment perimeter void was not observed but a trace of movement of mudstone to close the perimeter-void was found, therefore, it can be supposed that the geomechanicl loosening (relaxization) might have occurred all over the Tunnel.

(4) Results of Other Field Tests

1) Standard penetration tests: The results are shown in Fig.4.1.3 and Fig.4.1.4.

2) PS wave logging test: This test was carried out at BH-2 boring hole, and the travel velocities of seismic wave were 1.74km/sec, and 0.45km/sec. for Primary (Pv) and Secondary (Sv).

3) Water pressure tests: Test was made at the two boring holes to survey the permeability and pore pressure (Head Defferential) at the level corresponding to tunnel crown.
The results are as shown below:

	Pore pressure (H.D)	Permeability
BH-1	0.41 Kgf/cm^2 (4.1m)	3.38×10^{-7} cm/sec
BH-2	0.21 Kgf/cm^2 (2.1m)	3.86×10^{-7} cm/sec

During the test of BH-2, rapid increase of water level was observed due to water fracturing by outside water head which shows that the critical water pressure of this Mdm might be about 3 Kgf/cm^2.

(5) Main Laboratory Test

Unconfined compression tests show the following results:

(qu)	Unconfined compression strength	: 11.5 kgf/cm^2
(E)	Elastic modulus	: 1.138 Kgf/cm^2
(ED)	Deformation modulus	: 763 Kgf/cm^2

(6) Geomechanical Factors

The geomechanical factors were estimated by referring to the above field and laboratory tests. The previous Geotechnical Report Vol.3 (OSMAC) in 1977 shows the following:

Factor	Recommendable Value	
1) Bulk Density (Yt)	1.9-2.0	kgf/cm^3
2) Shearing strength (C)	2-4	kgf/cm^2
3) Internal Friction Angle (ϕ)	1.5-20	deg.
4) Elastic Modulus (E)	1,000-2,000	kgf/cm^2
5) Deformation Modulus (ED)	700-1,500	kgf/cm^2
6) Volume Compressibility (Mv)	$5\text{-}9 \times 10^{-4}$	cm^2/kgf
(mv')	$1\text{-}2 \times 10^{-3}$	cm^2/kgf
7) Coefficient of subgrade Reaction (K)	2-4	kgf/cm^3
8) Lateral Pressure Ratio	0.7-0.8	--
9) Poisson's Ratio	0.4-0.46	--

(7) Considerations

The following considerations are given to the geotechnical investigations:

1) The ground of the Tunnel was found to be composed of a kind of soft base rock which has not stopped its slow plastic deformation.

2) The existence of Mdw layer works effectively to increase stability and an impervious layer around the Tunnel. However, in future, this Mdw layer must be excavated for deepening of the Canal. Therefore, it is afraid that such unfavorable and complex phenomena as seepage and/or loosening of the ground of the Tunnel would occur.

3) The following methods are proposed based on geotechnical considerations:
   a. Additional back fill grouting to behind the segments
   b. Placing of a flexible layer to absorb the deformation of existing segments
   c. Consolidation of grouting to stabilize the back ground of the Tunnel.

4.1.3. Tunnel Section Survey

(1) Objectives

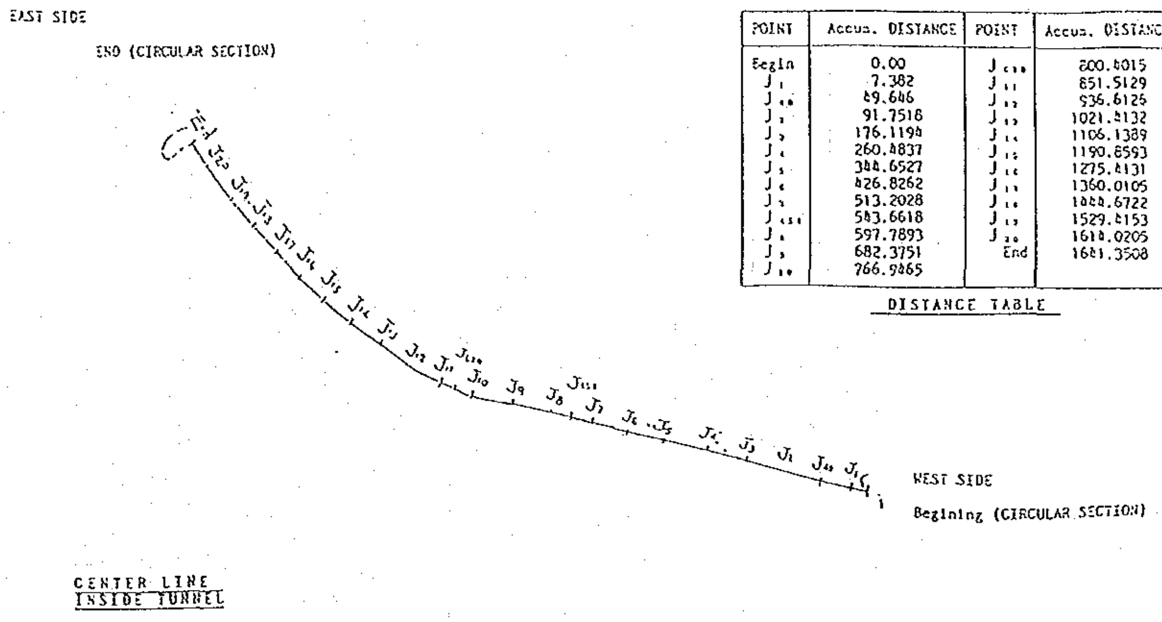
The main objectives of the tunnel section survey are to examine the tendency of segment ring deformation and to secure the horizontal/vertical clearance for the vehicles.

(2) Methodology

A total of 24 points were marked by nails on the medium-strip including three points (J40, J451, J610) which were surveyed by the Task Team in 1989. The location of each point is shown by accumulated distance from the entrance of the Tunnel in Fig. 4.1.5. The distance of 14 points from A to N were measured by tape. The positions of these points are illustrated in Fig. 4.1.6.

(3) Results of survey

Survey results of each section are shown in Table 4.1.5. The difference compared with the segment ring radius of 5.200m was considered to be the deflection of the Tunnel, and the survey results of B,D,F and H were converted to the lengths from the ring center which is shown in the Table 4.1.6.



POINT	Accum. DISTANCE	POINT	Accum. DISTANCE
Begin	0.00	J_{10a}	800.4015
J_1	7.382	J_{11}	851.5129
J_{1a}	49.646	J_{12}	936.6125
J_2	91.7518	J_{13}	1021.4132
J_3	176.1194	J_{14}	1106.1389
J_4	260.4837	J_{15}	1190.8593
J_5	344.6527	J_{16}	1275.4131
J_6	426.8262	J_{17}	1360.0105
J_7	513.2028	J_{18}	1444.6722
J_{7a}	543.6618	J_{19}	1529.4153
J_8	597.7893	J_{20}	1614.0205
J_9	682.3751	End	1641.3508
J_{10}	766.9465		

DISTANCE TABLE

. CROSS SECTION WAS MEASURED AT EACH POINT(J)

POINT(J) IS MARKED BY EXPANSIVE NAIL & PAINT ON THE CENTER LINE MEDIAN

AHMED HAMDI TUNNEL
SUEZ CITY

Fig.4.1.5 TUNNEL SECTION SURVEY POINTS

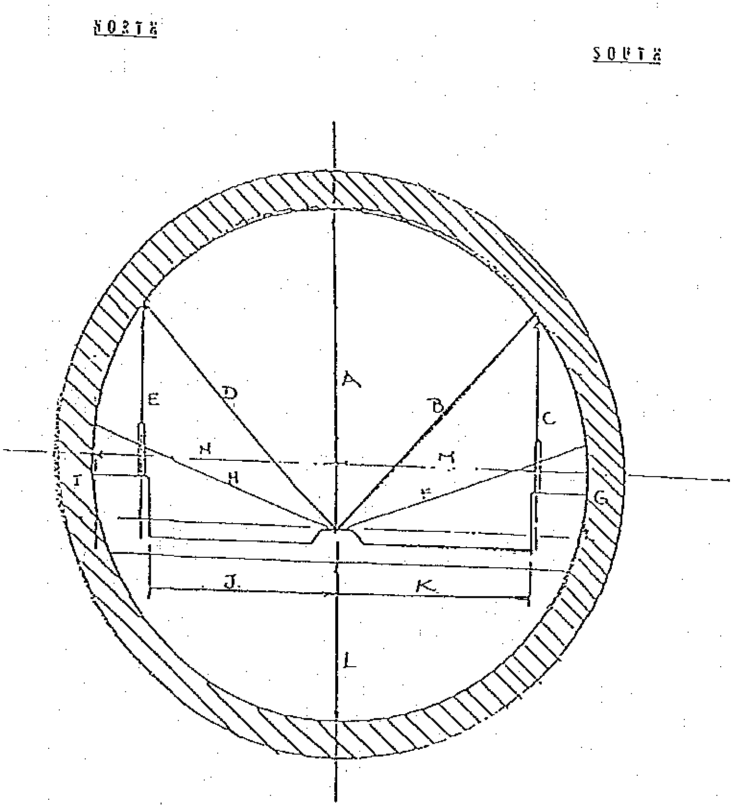


Fig.4.1.6 DISTANCE MEASURE LINES A-N

Table 4.1.5 SURVEY RESULTS OF A-N

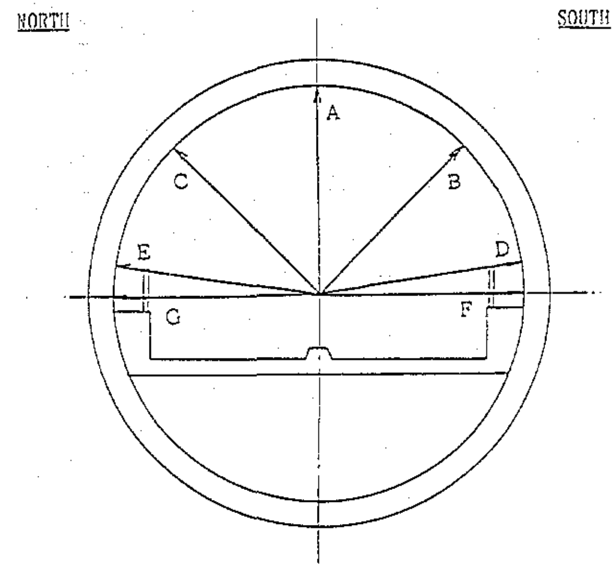
Location	A	B	C	D	E	F	G	H	I	J	K	L	M	N
J_1	—	6.472	4.962	6.505	5.100	5.61	2.169	5.58	2.150	3.753	3.754	2.870	5.168	5.185
J_{40}	6.890	6.452	4.961	6.490	4.940	5.575	2.09	5.605	2.010	3.746	3.741	2.930	5.15	5.233
J_2	6.930	6.496	4.985	6.500	4.990	5.655	2.187	5.632	2.129	3.745	3.744	2.840	5.20	5.202
J_3	6.980	6.554	5.053	6.544	5.149	5.653	2.142	5.597	2.170	3.750	3.738	2.810	5.205	5.153
J_4	6.970	6.504	5.000	6.566	5.019	5.612	2.194	5.615	2.028	3.755	3.738	2.825	5.18	5.19
J_5	6.990	6.436	4.904	6.514	5.000	5.574	2.188	5.626	2.134	3.748	3.741	2.880	5.135	5.225
J_6	7.065	6.553	4.980	6.517	5.084	5.550	2.197	5.594	2.140	3.745	3.750	2.820	5.2	5.15
J_7	7.065	6.478	5.040	6.563	4.885	5.557	2.153	5.670	2.128	3.757	3.732	2.760	5.13	5.244
J_{451}	7.089	6.433	4.900	6.553	4.965	5.555	2.113	5.682	2.216	3.755	3.746	2.793	5.149	5.225
J_8	7.025	6.482	5.055	6.503	4.890	5.573	2.118	5.855	2.836	3.753	3.743	2.815	5.125	5.235
J_9	6.990	6.443	4.9197	6.523	4.990	5.614	2.130	5.625	2.080	3.748	3.740	2.810	5.206	5.211
J_{10}	7.095	6.468	4.859	6.522	5.022	5.607	2.068	5.604	2.192	3.744	3.742	2.770	5.176	5.196
J_{670}	6.980	6.505	4.917	6.545	5.090	5.651	2.075	5.606	2.229	3.750	3.748	2.809	5.235	5.162
J_{11}	7.000	6.424	4.829	6.502	5.0248	5.603	2.0656	5.640	2.227	3.727	3.750	2.840	5.19	5.203
J_{12}	7.035	6.490	4.876	6.514	5.120	5.617	2.000	5.610	2.266	3.756	3.734	2.850	5.241	5.15
J_{13}	6.985	6.563	5.070	6.548	5.137	5.622	2.075	5.637	2.260	3.734	3.760	2.790	5.205	5.197
J_{14}	7.045	6.498	4.959	6.525	5.020	5.620	2.176	5.654	2.259	3.755	3.738	2.805	5.167	5.205
J_{15}	7.000	6.470	4.903	—	—	5.567	1.946	—	—	3.743	3.766	2.786	5.184	5.19
J_{16}	7.015	6.502	5.000	6.525	5.177	5.597	2.042	5.648	2.222	3.746	3.760	2.815	5.202	5.209
J_{17}	7.015	6.575	5.040	6.506	5.152	5.701	2.185	5.605	2.308	3.750	3.750	2.810	5.28	5.145
J_{18}	6.960	6.505	5.010	6.602	5.180	5.600	2.105	5.686	2.292	3.755	3.750	2.85	5.185	5.204
J_{19}	6.960	6.495	4.963	6.533	5.127	5.643	2.142	5.653	2.256	3.745	3.747	2.82	5.22	5.204
J_{20}	—	6.526	5.072	—	—	5.676	2.335	—	—	3.729	3.746	2.785	5.202	5.192

Table 4.1.6 CALCULATED DEFORMATIONS

DEFLECTION TABLE

MEASUREMENT IN M.M

Point	A	B	C	D	E	F	G
J_1	–	–	–	–	–	−32	−15
J_{40}	−20	−21	+27	−25	+35	−50	+33
J_2	−45	+40	+43	+29	+24	0	+2
J_3	−33	+81	+36	+42	−29	+5	−47
J_4	−33	+38	+108	−21	+40	−20	−10
J_5	+5	−36	+27	−63	+13	−65	+25
J_6	+13	+78	−4	−92	−24	0	−50
J_7	−17	−19	+140	−77	+64	−70	+44
J_{451}	+11	−42	+84	−56	+44	−51	+25
J_8	−10	−23	+61	−37	+18	−75	+35
J_9	−30	−11	+63	+3	+33	+6	+11
J_{10}	+3	+21	+31	+16	−31	−24	−4
J_{670}	−36	+69	+59	+63	−39	+35	−38
J_{11}	−10	−15	+12	+13	−4	−10	+3
J_{12}	+17	+34	−23	+51	−53	+41	−50
J_{13}	−43	+93	+51	+32	−26	+5	−3
J_{14}	−5	+28	+40	−8	0	−33	+65
J_{15}	−37	+32	–	+17	–	−16	−10
J_{16}	−15	+24	−9	+15	+7	+2	+9
J_{17}	−17	+102	−23	+77	−70	+80	−55
J_{18}	−25	+31	+92	−3	+25	−50	+4
J_{19}	−40	+44	+34	+31	+2	+20	+4
J_{20}	–	–	–	–	–	+2	−8



– MARK MEANS NO MEASUREMENT DUE TO OBSTACLE

4.1.4 Study on the Present Condition of Tunnel Deterioration

(1) Tunnel Structure

The safety of the tunnel structure is still gradually going down by the following reasons. So, it shall be decided for the all related parties to start the Rehabilitation Works as soon as possible, by the sense of "earlier is better".

1) The deterioration of the concrete segment such as carbonation of concrete, and cracking and chipping of concrete, and cracking and chipping of concrete slice due to corrosion and expansion of the steel bar of the element are still going on in the same way as it was observed/recognized in February/March 1990 (ref. in Final Report of May 1990).

a. According to the previous 4 years records by S.C.A., cracking and chipping of concrete slice were found for approximately 94 percent of the concrete segments of the tunnel crown, which is much higher than that of the deck (in total 14927) as shown in Fig. 4.1.7.
So, it is no longer practical to repair the Tunnel partially.

Furthermore, further deterioration of the concrete segments located approximately 200m from both end of the tunnel entrances were observed in this field survey.
For instance, in Fig. 4.1.8, the surface concrete of the segment No.1232(4S) at the Tunnel's south side 200m from the east entrance, has been spalled with concrete with a maximum of 5cm in thickness at the rib and disappeared the reinforcing steel bar already at the same place.

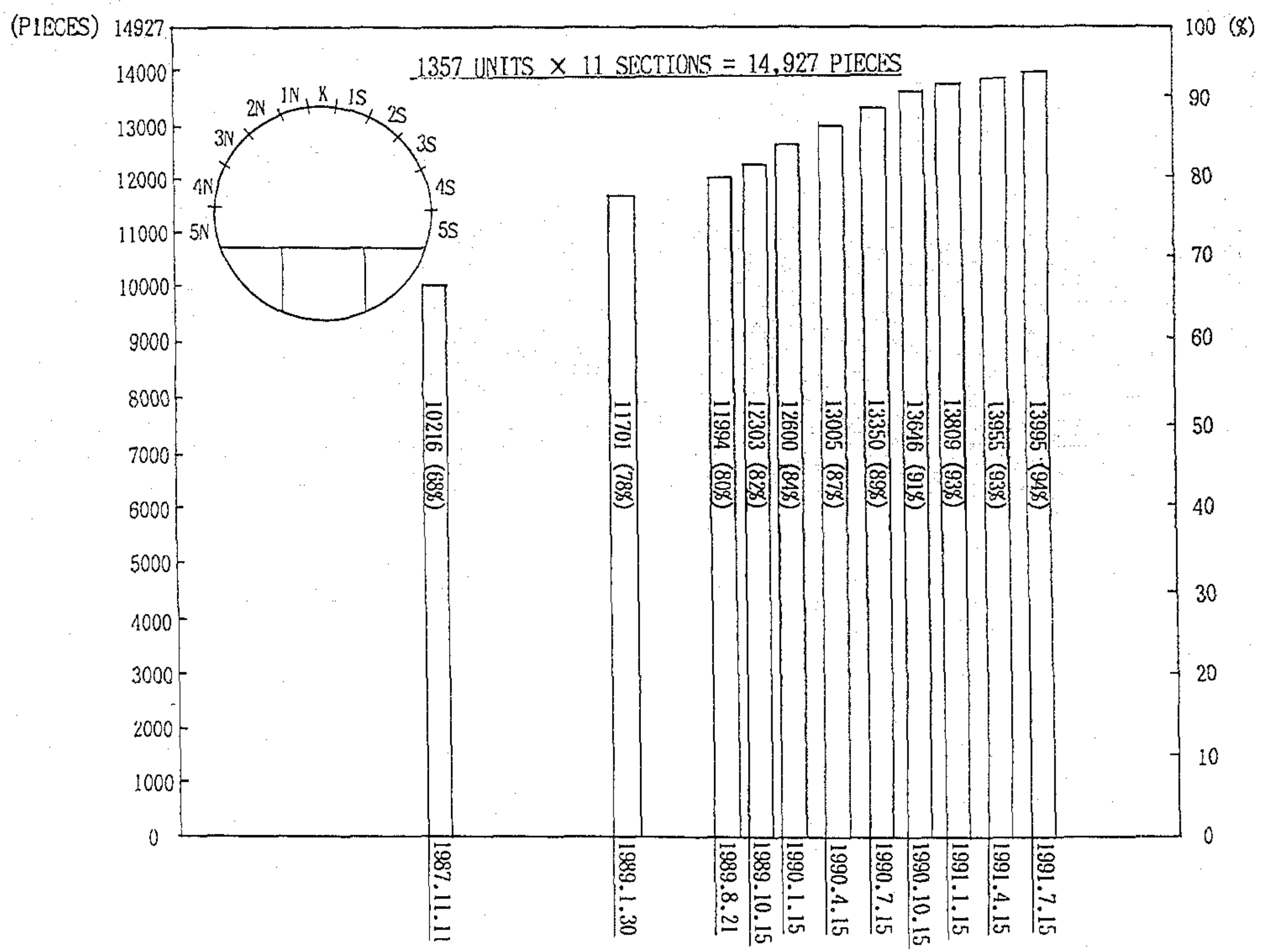


Fig.4.1.7 Number of Segment with Cracks in the Tunnel Crown Observed by S.C.A.

Key

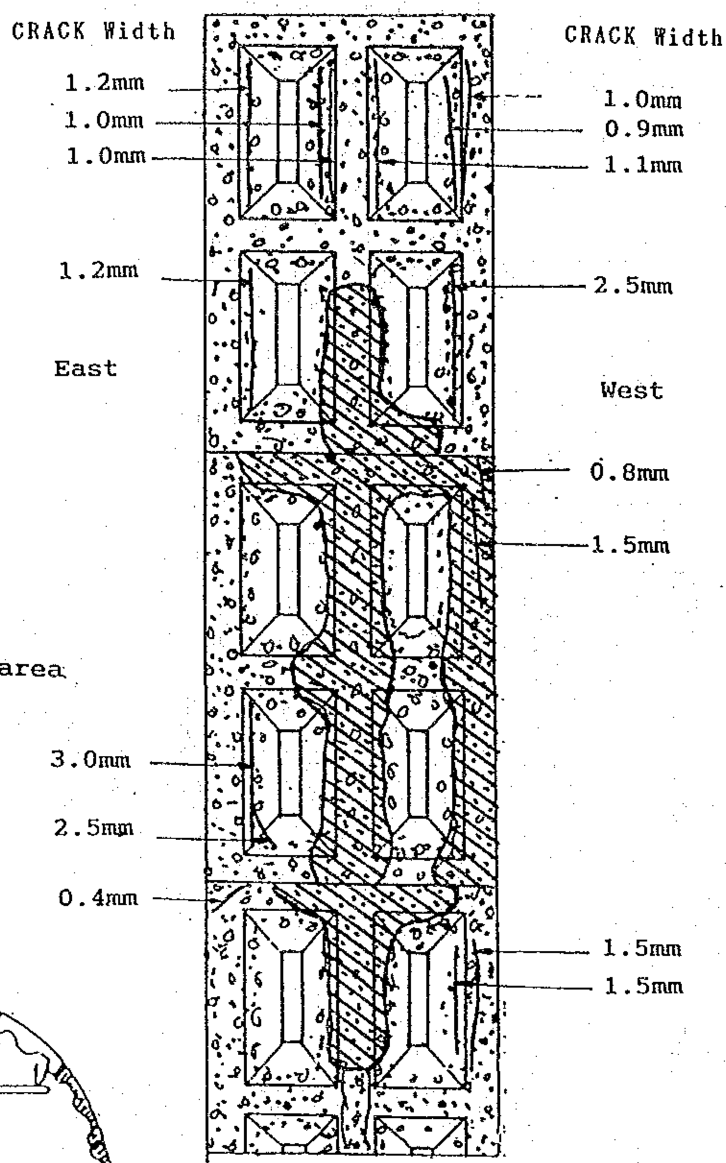
Cracks

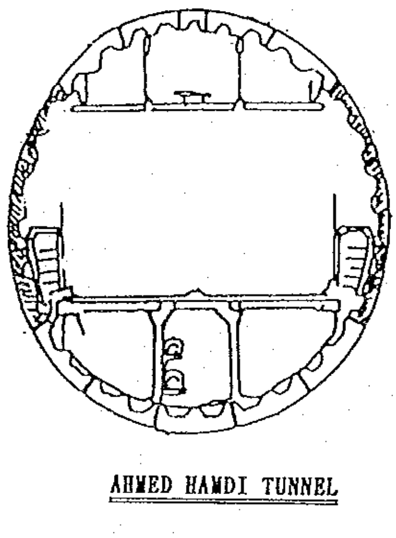
Broken area

Spoien area

Salted area

Steel rusted area





AHMED HAMDI TUNNEL

Date 7-7-91

Time 11am

By Sayed Faheem

Ring No. 135

Segment No. 3S, 4S, 5S

Fig. 4.1.8 Deteriorated Typical Sample on the Segment

b. Concerning the concrete segment under the fresh air supply ducts and through service duct, deterioration is not serious.

Because of the continuous wet condition of the concrete surface, the carbonation of concrete and the corrosion of steel in the concrete are slowly going on.

Based on the above situation, "The earlier, the better" should be a policy adopted by all parties related to the Rehabilitaion Works.

2) It shall be noted that the tunnel structure has smaller Ring Strength than usual for the following reasons:
   - the segment connection system is in cross checked pattern instead of staggered pattern.
   - Usually, with a diameter of 10-11 meters, the number of the segments of a tunnel structure should be 10-11. Best in this case, there are more segments by some reasons.

3) It may be assumed that the stress concentration and the deformation might be bigger than usual because of lack of roundness on the tunnel structure due to the wide gap between segment joints.

4) It may be assumed that the following weak points might be existing now by the soil moving into the Tunnel together with the leaking water.
   - the tunnel bedrock support condition around the segment ring becoming worse
   - the one side earth pressure acting to the segment ring

5) It should be pointed out that the K-ring at the segment top has the tendency to drop off from the position to the inside of the Tunnel by the following reasons:
   - the section of K-ring joints being not in wedge shape/(even the K-ring to be inserted into the axis direction)
   - no segment joint bolts for the connection between each ring

6) The stress of the reinforcing steel bar has been measured by the strain gage as shown in Table 4.1.7. According to this survey measurement, the stress levels of the reinforcing steel bar are as follows.
   - In the healthy reinforcing steel bar without any corrosion: 700-1,000 kgf/cm^2
   - In the reinforcing steel bar with approx. 20 percent corrosion: 1,500 kgf/cm^2

   It shall be noted that the section area of some reinforcing steel bars is only approx. 1/2-1/3 of the original, and at these portions the stress level be assumed to become 1,400-2,100 kgf/cm^2 .

7) It may be assumed that the soil around the Tunnel may become weak*1, and the earth pressure may be changed by the organizing of water path along side of the Tunnel outside surface.

Many horizontal cracks with 0.5-2.0 mm in width on the surface of the concrete segment have been found. It could be assumed that:

- The cracks have been produced at the time of the tunnel erection by the bolt clamping and the jack driving force (gap) by shearing stress from view point of cracks' visual inspection*2.
- So, the cracks have not been produced by the bending moment caused by the tunnel outside load.

  However, it shall be essential to keep the visual inspection on the cracks and its recording. Because even these cracks may become the reason for loosing of the Tunnel's safety.

*1 Deck gray clay has the tendency to reduce the strength when saturated in water.

*2 Cracks are passing through in the segment thickness direction.

Table 4.1.7 Measurement of Reinforcing Bar's Stress

Point No.	Ring No.	Segment No.[*1]	Stress(kg/cm^2)	Expectable ratio(%)
1	134	N1	- 704	100
2	266	N1	- 357	100
3	379	N1	- 281	100
4	402	N1	- 279	100
5	531	N1	- 331	100
6	531	S1	- 122	100
7	670	N1	- 295	100
8	762	N1	- 376	100
9	762	S1	- 622	100
10	800	N1	- 999	90
11	931	N1	- 395	100
12	1070	N1	-1155	100
13	1070	S1	- 265	100
14	1200	N1	- 425	100
15	1337	N1	- 443	100
16	1037	S1	- 628	70
17	1002	S1	-1187	100
18	618	N4	165	70
19	134	N7	- 454	100
20	266	N7	- 111	100
21	379	N7	-1484	80
22	402	N7	- 508	100
23	531	N7	20	100
24	670	N7	-1045	80
25	762	N7	- 672	100
26	800	N7	-1385	70
27	931	N7	- 992	70
28	1070	N7	- 743	100
29	1200	N7	- 919	100
30	1337	N7	-1108	100
31	260	S7	- 63	100
32	379	S7	- 333	100
33	449	S7	- 396	100
34	549	S7	- 595	80
35	636	S7	325	45
36	782	S7	- 425	70
37	799	S7	- 184	100
38	893	S7	- 473	80
39	993	S7	851 *	100
40	1193	S7	- 749	100
41	1086	N1	- 454	90
42	986	N1	-2099	60
43	802	N1	- 243	100
44	759	S1	-1276	100
45	744	S1	- 335	100

* : Average of 8 points

(2) Safety for Road Deck and Support Wall

The Safety for the Road Deck and the Support Wall is going down also for the following reasons. Therefore, it is recommended to all the related parties that the Rehabilitation Works should start as soon as possible.

1) The road deck and the support wall which are all precasted concrete have been damaged by the carbonation and deterioration, and this damage is still going on daily. According to the previous 4 years records by S.C.A., the deterioration is in the form of cracking and chipping of concrete slice with approximately 71 percents of the total 7,458 units (=678 units x 11 sections) as shown in Fig 4.1.9.

2) Reduction of section area of some the reinforcing steel bar at the road deck by the deterioration/rusting has been found.

   Some partial chipping off of the covering concrete may also occur in future (ref. Fig 4.1.11(b)).

3) The base concrete at the foot of the road support walls has deteriorated severely due to the carbonation and salty water penetration. In this situation, it can be assumed that the base concrete strength has been weakened because the base is becoming porous place by place. (ref. Fig .4.1.10)

   To avoid any accident such as the road deck falling down in the near future, it is essential to take the following actions.

- Partial Repairing according to "6.1.3(6) Procedure on Repairing Method on the Base Concrete Portions at the Foot of Road Support Walls".
- Execution of Traffic Speed Limit with 20km/h (the exact control).

4) The support walls have been damaged approximately 70 percents the carbonation and salty water penetration especially in the Fresh Air Duct areas resulting in cracking, chipping off/spalling off and removing of covering concrete. Even though many partial repairs have been made for the damages, the phenomenon is still going on and it can be possible that the Road Deck supporting condition may be further deteriorated (ref. Fig.4. 1.11(a)).

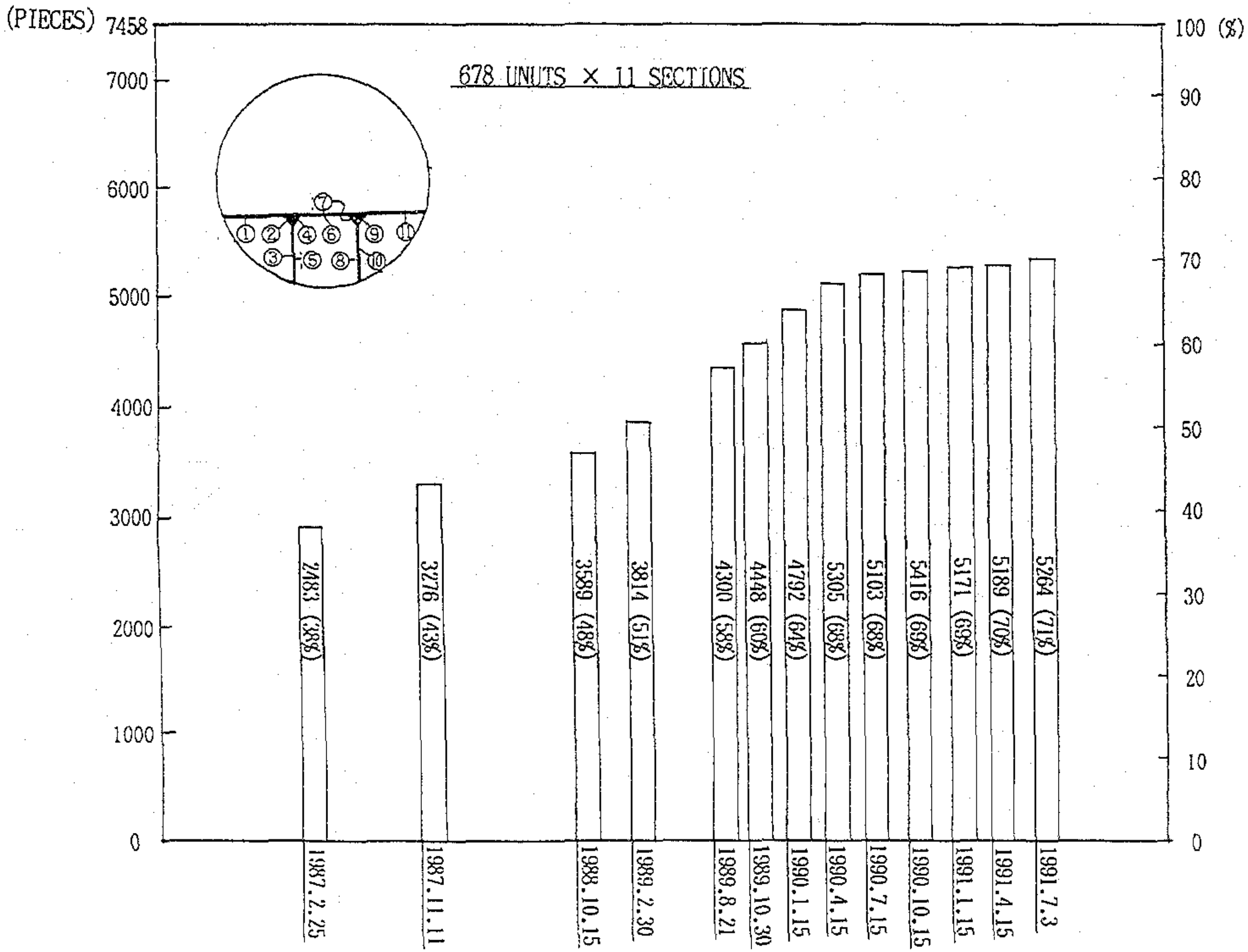


Fig.4.1.9 Number of Places with Observed Cracks at Road Decks and Support Walls (Below Road Deck Units) by S.C.A.

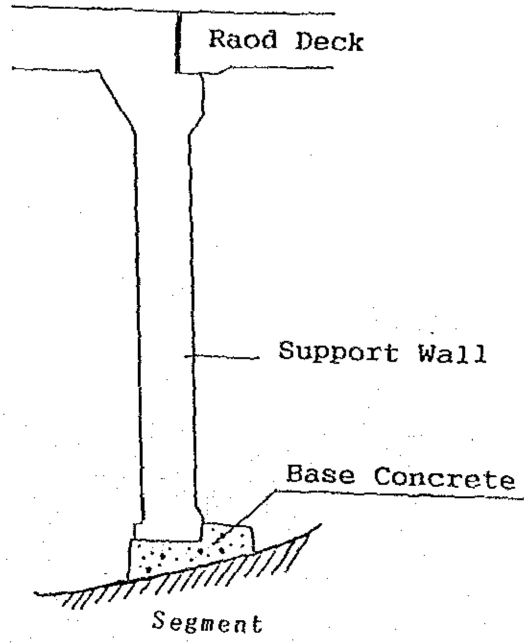


Fig 4.1.10 Base Concrete at The Foot of Support Wall

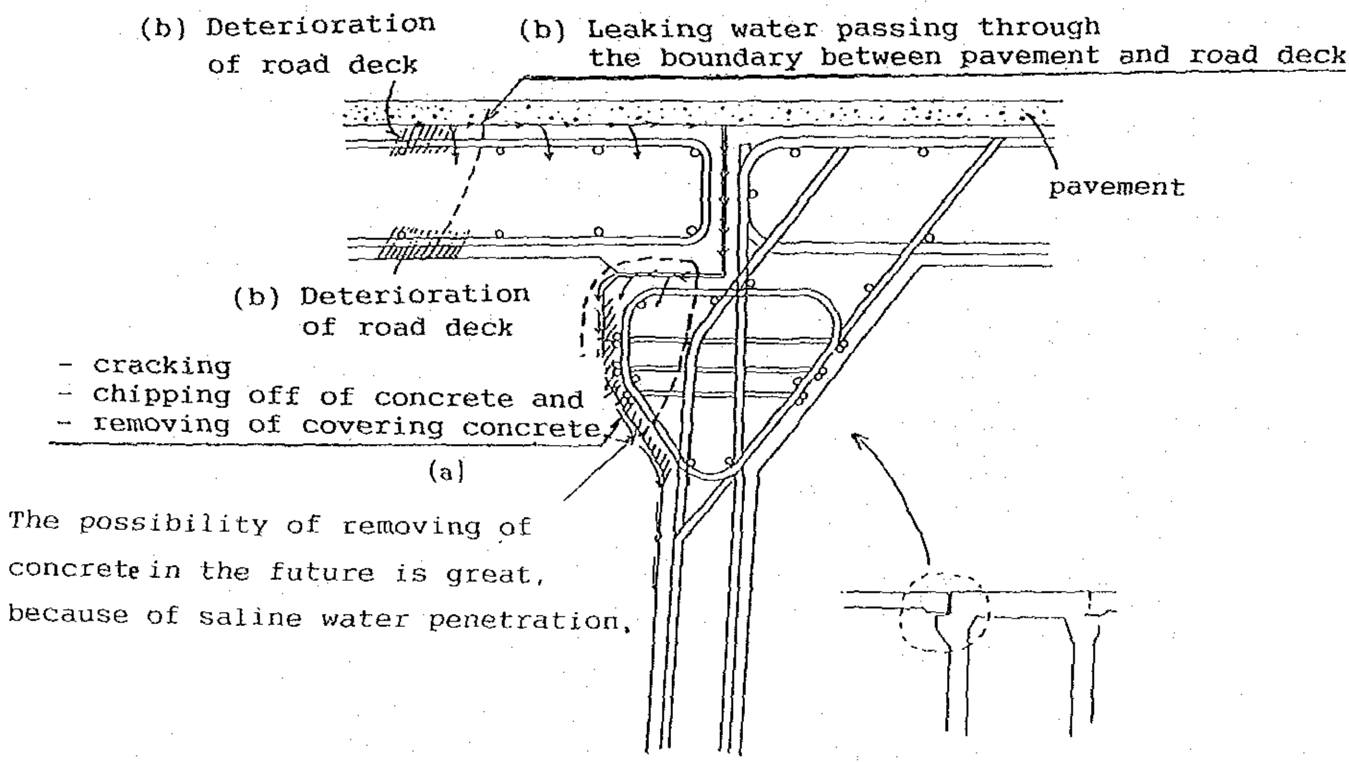


Fig.4.1.11 Covering Concrete Damage at Road Deck and Support Wall

4.1.5 Study on the Related Facilities

(1) Method of Survey

1) Sequence of survey activity

The survey was performed in accordance with the following steps in order to observe the present condition precisely:

STEP 1. To study the completed drawings and documentations on the tunnel facilities and equipment to catch their form, arrangement and specification and so on

STEP 2. To visually inspect the present conditions of the tunnel facilities and equipment guided by counterparts and to fill in inspection sheets prepared to asses the degree of their damages

STEP 3. To recheck the inspection sheets filled in comparing with the damage records in the past

2) Inspection form

The inspection form was prepared by the JICA Study Team prior to performing the assessment survey in order to distinguish between the reuse and replacement in the Rehabilitation Works of each tunnel ancillary. See Table 4.1.8.

3) Result of field survey

The survey was performed in the order stated above and the results of observation were written in the sheets provided on every component of structure or equipment. The objective of this survey is firstly to grasp the possibility of reuse so that only summary/overall evaluation of the sheet is arranged in Table 4.1.9 and 4.1.10.

Table 4.1.8 Inspection Sheet

First Category ; Name of structure or equipment
Second Category ; Name of component or part

		Result of survey	Evaluation
Specification	Scale	Principal scale and specification	
	Quantity	Estimated quantity	
From a damage and deterioration view point	Present condition	The degree of soundness or deficiency	Mark in accordance with legend
	During demolition and rebuilding	Presumption on the degree of damage caused by demolition and rebuild	〃
	During storage	Presumption on the dederionation degree during storage	〃
Possibility for reuse	Work efficiency	Consideration on the possibility for reuse from work efficiency	〃
	Economy	Consideration on the possibility for reuse from the economic view point	〃
From the design view point	Relation to lining	Consideration on the effect from rehabilitation lining	〃
	Basic design	Design condition for basic design affected by new lining	〃
	Detailed design	Design condition for detailed design affected by new lining	〃
Summary/Overall Evaluation		Observing the above mentioned considerations, the final evaluation of the reuse or replacement shall be made herein.	〃
Remarks (Legend)		◎ : Excellent. ○ : Good. } This might be considerably reused by way of repairing or overhauling damaged parts. △ : Possible. This might be partially reused by way of replacing defective parts with new parts. × ; Impossible. This must be totally replaced with new parts.	

Table 4.1.9 Summary/Overall Evaluation Related to Electrical Facilities

(1/2)

	Reuse or replace	Degree of reusability	Description of summary/overall evaluation given from the inspection sheet	Remarks
1. Lighting Equipment 1) Illuminator	Replace	×	• This equipment is close to the life end and will probably be seriously damaged during the construction period. The introduction of new-brand system will be desirable for the existing one.	
2) Transformer and Switch-Box	Remove	×	• This will be unadaptable for the new lighting system.	
3) Cable	Replace	×	• It might not be unavailable for reuse because of deterioration and shortage of cable length.	
2. Communications Equipment 1) Telecommunication Equipment • Emergency Telephone	Reuse	○	• It might be available for reuse after certifying the performance by test and adjustment.	
• Cable	Replace	×	• Same reason as 1-3).	
2) Radiotelephone • Cable	Reuse	○	• It might be available for reuse.	
3) ITV Equipment •Camera	Reuse	○	• Same reason as 2-1).	
•Cable	Replace	×	• Same reason as 1-3).	

Table 4.1.9 Summary/Overall Evaluation Related to Electrical Facilities

(2/2)

	Reuse or replace	Degree of reusability	Description of summary/overall evaluation given from the inspection sheet	Remarks
3. Equipment for the prevention of fire disaster 1) CO Monitor • CO Monitor	Reuse	○	• Same reason as 2-1).	
• Cable	Replace	×	• Same reason as 1-3).	
2) VI Monitor • VI Monitor	Reuse	○	• Same reason as 2-1).	
• Cable	Replace	×	• Same reason as 1-3).	
3) Fire-Extingushing Equipment • Fire Hydrant	Reuse	○	• Same reason as 2-1).	
• Cable	Replace	×	• Same reason as 1-3).	
4. Control Cable (Wire) • Control Cable	Replace	×	• Same reason as 1-3).	
5. Electric Power • Cable	Reuse	○	• Same reason as 2-1).	2-11 kv Cables

Table 4.1.10 Summary/Overall Evaluation Related to Ancillary

(1/2)

	Reuse or replace	Degree of reusability	Description of summary/overall evaluation given from the inspection sheet	Remarks
1. Ceiling				
1) Ceiling Panel	Replace	×	• There is difference in panel size due to Rehabilitation lining.	
2) Duct Cowling	Reuse	◎	• This can be totally transferred onto the new ceiling. • It's essential to remove them carefully and keep them in the warehouse.	
3) Hanger	Reuse	△	• A turnbackle might be reused except one with the damaged screw. • The eyebar shall be replaced with new one because of length shortage.	
4) Supporting Frame	Replace	×	• The T-shaped steel of middle hanger shall be abolished because of adherence of sealing mortar on the surface of it. • The supporting structure has been changed to alternative so that the existing steel is made useless.	
5) Covering from wind leaking	Replace	×	• This material is useless since it is not fire-proof.	
6) Center Diaphragm	Replace	×	• Same reason as 1-5).	
2. Wall Panels	Replace	×	• This material shall be replaced with newly specified one since this does not have the necessary properties of incombustibity stipulated in Japan Building Structure Standard.	

Table 4.1.10 Summary/Overall Evaluation Related to Ancillary

(2/2)

	Reuse or replace	Degree of reusability	Description of summary/overall evaluation given from the inspection sheet	Remarks
3. Walkways				
1) Steel Frame	Replace	×	• This must be newly prefabricated except minor reusable parts because of difference in size of external shape due to the Rehabilitation Lining. • The same size member such as the hand rail and the longitudinal member might be used again in new walkway.	
2) Deck plate	Reuse	△	• This precast concrete panel might be available for the new walkway by cutting off one end to meet the new size.	Tis idea will be examined in detail in the coming further study.
3) Side Panel	Replace/Reuse	△	• This member might be reused by replacing the seriously damaged one.	• Upper plastic→galvanized steel
4. Life line				
1) Water Main(2-ϕ500)	Reuse	○	• This pipe might be available for reuse after giving some appropriate treatment such as taking off rust at joint part by burnishing and replacing the gasket with a new one. • It's require to make an elaborate examination of the joint and perform water pressure test prior to laying.	
2) Pump Main(1-ϕ200)	Replace	×	• It's presumed difficult to refit a joint making it water-tight because of the occurance of serious damage at the joint caused by running condensed salty water in it.	
3) Exhaust Pipe	Replace	×	• This pipe has serious damage caused by salty water running along concrete segment.	
4) Fine Hydrant Pipe	Reuse	○	• It's presumed that there will be minor damage at the joint by reason of fresh water in it. • This pipe will be useful with polishing joint area and replacing the gasket.	

4.1.6 Study on the Ventilation System

(1) Introduction

1) The existing ventilation system is a full transverse ventilation system with 8 fans having the maximum capacities of 616 m^3/sec. for air supply and 648 m^3/sec. for air exhaust.

2) The present operation level is at the level 3 of the prepared levels from 1 to 7. This level gives a supply volume of 118m^3/sec. against the present traffic volume of about 1,500 vehicles. per day with 36 percent of diesel car ratio.

3) Maintenance service workers are continuously cleaning the interior surface of the crown segments by removing the wind leakage stopper in the exhaust duct, therefore, the duct has not been completely functionable.

4) Although the existing monitoring equipment for CO gas and Visibility (VI) are in satisfactory functional condition with periodical calibration by S.C.A., the maintenance removal of the wall panels caused some of VI monitors' accuracy problem. Because a slight displacement of the monitoring equipment for maintenance caused shifting of the laser beam line between the monitoring equipment.

5) The layout of the present ventilation facilities is shown in the Fig. 4.1.12.

(2) Review of Required Ventilation Volume

The required air volume was reviewed by the latest PIARC standards and calculated to be at 410 m^3/sec., which is decreased to less than 70 percent of the existing maximum capacity volume of 616 m^3/sec. This decrease is due to the average gas emission being gradually decreased from the time of the tunnel design. Although the duct area would be reduced to about 70 percent of the existing area after the tunnel rehabilitation, the existing ventilation fans may be utilized because only its output needs to be adjusted.

Details of the calculation by the PIARC (1987) is attached in Appendix-5, Ventilations System Plan, the letter to S.C.A. dated 11th October 1991.

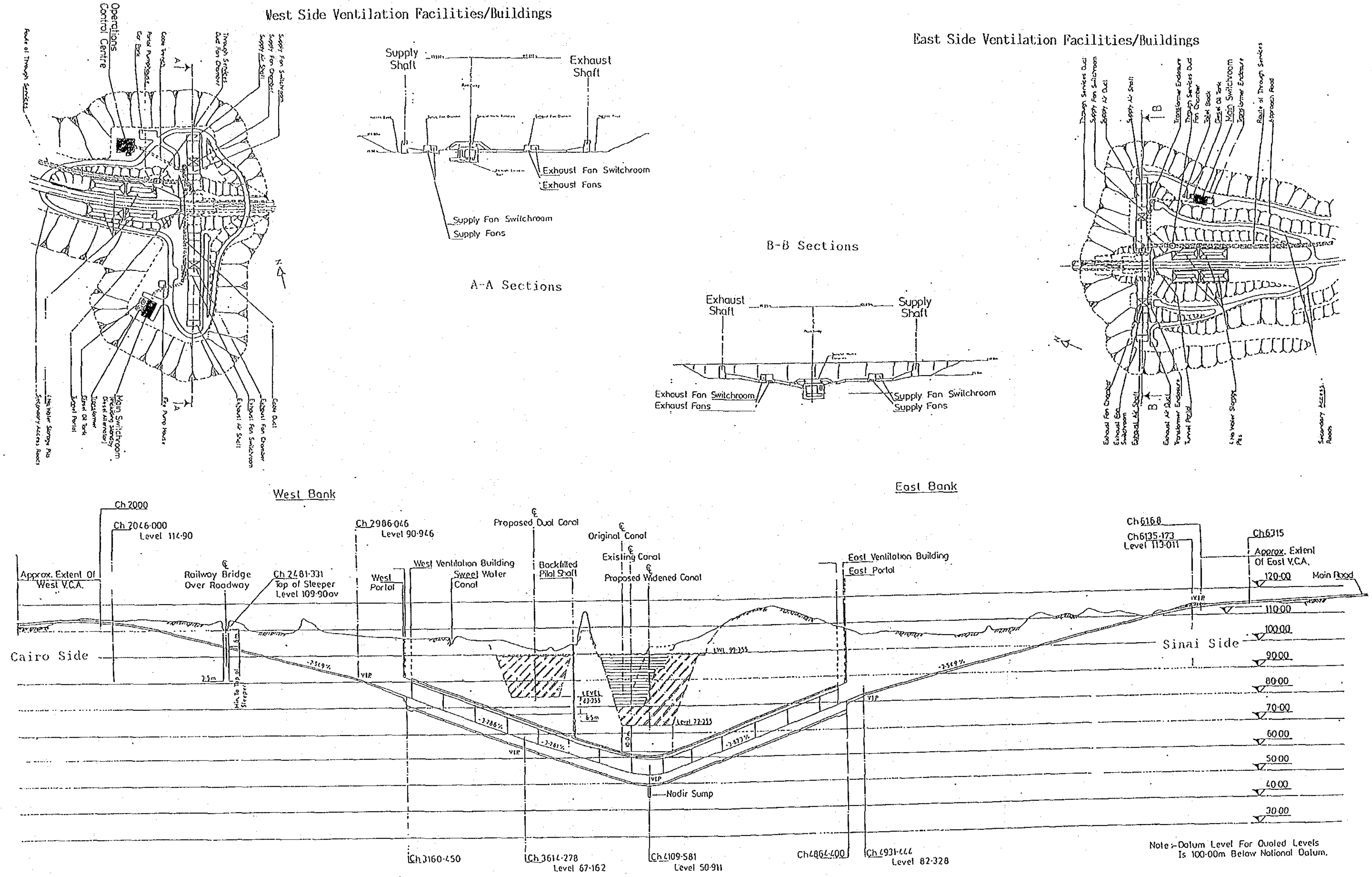


Fig.4.1.12 LAYOUT OF THE EXISTING VENTILATION FACILITIES

4.1.7 Survey/Study on Construction Materials

(1) Introduction

Site investigations were performed in 1988*(1) and 1990 *(2) to determine the cause of salty water leakage in the Ahmed Hamdi Tunnel in the Arab Republic of Egypt, which was completed in 1983. However, the properties of deteriorated concrete were not fully analyzed during those investigations because the survey was limited to visual inspection of the existing tunnel condition.

The site investigation and laboratory analyses contained in this report were conducted recently, prior to the start of the Rehabilitation Works, with the objective of finding out the causes of deterioration to prevent the repetition of such deterioration. The investigation included analysis of the properties of concrete specimens collected from various Tunnel components and determination of the quality of aggregate, cement, and mixing water that will be used for the Rehabilitation of the Tunnel.

The scope of the site investigation and laboratory analyses were as follows:

1) Site investigation

Site investigation was performed by visual inspection of deteriorated concrete components and rust formation of metallic parts, including bolts and nuts.

2) Sampling

Specimens were collected from the concrete components of Tunnel and local construction materials, including aggregate, cement and mixing water.

3) Laboratory Analyses of Concrete Specimens

a. Depth of carbonation measurement
b. Observation by polarizing microscope
c. Salt content analysis
d. Element distribution analysis
e. Powder X-ray diffraction analysis of hardened concrete
f. Corrosion rate analysis of reinforcing steel

g. Unconfined compressive strength test
h. Dynamic and static moduli of elasticity measurement
i. Mix proportion analysis (including alkali content analysis)
j. Tensile strength and Young's modulus of reinforcing steel

4) Laboratory Analyses on Construction Materials

a. Aggregate (coarse and fine aggregates)

a-1. Sieve analysis (JIS A1102)
a-2. Specific gravity and absorption rate (JIS A1109 - A1110)
a-3. Decantation test (JIS A 1103)
a-4. Unit volumetric weight (JIS A1104)
a-5. Clay lump test (JIS A 1137)
a-6. Soft particle proportion test (JIS A 1126)
a-7. Salinity analysis (JASS ST 202)
a-8. Weight of particle floating at specific gravity 1.95 (JCEA Method)
a-9. Alkaline-silica reaction test (JISA 5308 Apdx.7 chemical methods)
a-10. Powder X-ray diffraction analysis (JCI-DD3,-DD4)
a-11. Polarizing microscope observation (JCI-DD1,-DD4)

b. Cement

b-1. Specific gravity (JIS R 5201)

b-2. Fineness test (JIS R 5201)

b-3. Setting test (JIS R 5201)

b-4. Strength test (bending and compression)

b-5. Heat of hydration

b-6. False setting

b-7. Chemical analysis for: ignition loss, insoluble residue, SiO_2, Al_2O_3, Fe_2O_3, CaO, MgO, SO_3, K_2O, TiO_2, P_2O_5, MnO, S, free lime, Cl, and Na_2O

c. Mixing Water

c-1. Physiochemical tests (Article 4 of the Waterworks Law)

c-2. Water quality test (JASS 5 T-301)

c-3. Chemical analysis for: Na_2O, K_2O, CaO, MgO, SO_2, S, and Cl

(2) Summary of Investigation and Analyses

1) Analyses of Quality of Concrete in Tunnel Components

a. Depth of Carbonation

Purpose: This analysis is to measure the depth of carbonation on concrete surface.

Results: Depth of carbonation was measured by spraying a sawcut surface of a concrete sample with a red phenophthalein indicator dye and measuring the depth of concrete that was not colored. Readings (depths of carbonation) were 1 to 3 mm for wet surfaces and 9 to 10 mm for dry surfaces.

b. Observation by Polarizing Microscope

Purpose: Analysis is made with a polarizing microscope to observe the tesseral and mineral systems of aggregates, and hardness of the cement paste (including carbonation portion).

Results: As a result of observations by polarizing microscopes, 30% by weight was found to be coarse aggregate composed of calcareous dolomite, and fine aggregate was sand. It was observed that calcium hydroxide was prominently formed in the hardened cement paste, showing characteristics of hydrated cement that had been alkalized by salt. Air content was only 1%.

c. Salinity Analysis

Purpose: This is to analyze the salinity concrete. Excessive salinity can greatly degrade the quality of concrete.

Results: The concrete collected from dry areas contained less salt than that from wet areas. Salt was mostly concentrated to a depth of 2 or 3 cm. Chlorine ion concentration measured at the core of samples was 1 to 2 kg/m^3. The salinity concentration in the concrete from wet locations was higher on the surface but lower in the core. The concentration measured at a depth of 5 cm ranged from 6 to 8 kg/m^3. It is estimated that salt reached a very deep level in concrete. Since the maximum chlorine ion concentration allowed in reinforced concrete in Japan is only 0.60 kg/m^3 (also in England) (3), these values are very significant.

d. Element Distribution Analysis (Ref.EPMA)

Purpose: Cementitious Na, K, Cl, and S can be thickened or diffused if concrete is deteriorated. This method analyzed the distribution of elements over the cut surface of concrete.

Results: The distribution of elements was checked by analyzing the secondary X-rays peculiar to elements with an electron probe micro-analyzer (EPMA). Results show K was distributed in deep areas (5 to 10 cm), and Cl, Na, and S were mostly distributed to depths of 7 to 8 cm for concrete from dry areas. For the concrete from wet areas, Cl and Na were distributed to depths from of 7 to

8 cm. It was estimated that Cl distributed to a depth of 10 cm could cause rust on reinforcing steel.

e. Powder X-ray Diffraction Analysis of Hardened Concrete

Purpose: Concrete deteriorates when hardened cement paste decomposes. The condition of hardened cement paste can be evaluated by checking the minerals in the concrete.

Results: As a result of analyzing the concrete samples to see minerals produced by the decomposition of cement paste by using the powder X-ray diffraction method, portlandite ($Ca(OH)_2$) and calcium carbonate ($CaCO_3$) caused by carbonation were detected, but cementitious minerals that can degrade the concrete were not noted.

f. Corrosion Rate Analysis of the Reinforcing Steel

Purpose: This is to check the rust and corrosion formed on reinforcing bars in concrete.

Results: Reinforcing bars in concrete cores collected from the Tunnel by drilling were visually inspected to check corrosion. It was noted that rust had formed on the surface of most reinforcing bars to a concrete cover depth of 7 or 8 cm, notwithstanding the sampling locations, dry or wet. More rust was formed on reinforcing bars under thinner cover.

g. Unconfined Compressive Strength Tests

Purpose: This is to check the strength of concrete.

Results: Concrete specimens showed an unconfined compressive strength between 415 and 650 kgf/cm^2, all being within the permissible value.

h. Dynamic and Static Moduli of Elasticity

Purpose: This is to check the quality of concrete.

Results: The dynamic moduli of elasticity ranged from 3.30 to 5.19 x 10^5 kgf/cm^2 and the static moduli of elasticity ranged from 2.63 to 3.75 x 10^5 kgf/cm^2. These values are smaller than that of normal concrete of this compressive strength. The modulus of elasticity reduces when the density of concrete lowers.

i. Mix Proportion Analysis

Purpose: This is to determine the mix proportions (especially the unit cement ratio) of concrete at the time of placement.

Results: The unit cement ratio ranged from 388 to 447 kg/m^3, and it is assumed that the alkali content was between 0.40 and 0.54%, expressed as Na_2O. Both the unit cement ratio and the amount of alkali are within permissible range.

J. Reinforcing Steel

Purpose: This is to check the quality of reinforcing steel bar.

Table 4.1.11 Results of Physical Test on Aggregate

Methods of Test		Coarse Aggregate: Gravel Gineifa Quarry	Coarse Aggregate: Crushed Stone Suez Canal Authority	Coarse Aggregate: Crushed Stone Arab Contractor Authority	Coarse Aggregate: Crushed Stone Company	Fine Aggregate: Sand Gineifa Quarry	Fine Aggregate: Crushed Sand Suez Canal Authority	Fine Aggregate: Crushed Sand Arab Contractor Company	Standard Value: Coarse Aggregate	Standard Value: Fine Aggregate
JIS A 1102 Method of Test for Sieve Analysis of Aggregate — Nominal Sieve Size (mm)	+ 60	-	-	-	-	-	-	-	See Separate Table	
	60 – 40	-	-	-	-	-	-	-		
	40 – 25	-	30	-	-	-	-	-		
	25 – 20	18	61	-	23	-	-	-		
	20 – 15	17	8	62	73	-	-	-		
	15 – 10	18	-	32	4	-	7	27		
	10 – 5	46	-	6	-	1	86	65		100
	5 – 2.5	1	-	-	-	3	7	8		100 – 90
	2.5 – 1.2	-	-	-	-	8	-	-		100 – 80
	1.2 – 0.6	-	-	-	-	27	-	-		90 - 50
	0.6 – 0.3	-	-	-	-	41	-	-		65 – 25
	0.3 – 0.15	-	-	-	-	16	-	-		35 – 10
	- 0.15	-	-	-	-	4	-	-		15 – 2
	Total	100	100	100	100	100	100	100		
JIS A 1109 [4]	Specific gravity in saturated surface-dry condition	-	-	-	-	2.50	2.70	2.71	-	-
	Absolute dry weight	-	-	-	-	2.58	2.66	2.68	-	2.5 min
	Absorption rate (%)	-	-	-	-	0.53	1.42	1.04	-	3.5 max
JIS A 1110 [5]	Specific gravity in saturated surface-dry condition	2.57	2.73	2.68	2.70	-	-	-	-	-
	Absolute dry weight	2.55	2.70	2.64	2.67	-	-	-	2.5 min	-
	Absorption rate (%)	0.54	0.78	1.50	1.01	-	-	-	3.5 max	-
JIS A 1108 [3]	Percent by weight of particles passing 74 mm sieve	0.42	0.28	0.41	0.28	1.85	0.31	0.45	1.0 max	7.0 max
JIS A 1104 [2]	Unit Volumetric Weight	1.66	1.57	1.57	1.56	1.68	1.54	1.49	-	-
	Percentage of absolute volume (%)	64.9	58.0	59.5	58.4	65.0	57.8	55.6	55 min	43 min
JIS A 1137 [7]	Clay lumps (%)	0.9	0.2	0.3	0.2	1.3	0.3	0.4	0.25 max	1.0 max
JIS A 1126 [6]	Soft stone percent by weight (%)	0.0	0.0	1.6	1.1	-	-	-	-	-
JASS 5T-202 [10]	Salinity (NaCl) percent by weight (%)	-	-	-	-	0.04	0.03	0.06	-	-
JIS A 5308 [8]	Percent by weight of particles floating on liquid at specific gravity 1.95	0.2	0.0	0.0	0.0	0.7	0.0	0.0	> 0.5 – 1.0	> 0.5 – 1.0
JIS A 5308 Appendix 7 [9]		Detrimental	Harmless	Harmless	Harmless	Detrimental	Harmless	Harmless	-	-

Note
1) JIS A 1103 Method of TEST FOR Amount of Material Passing Standard Sieve 74 mm in Aggregates
2) JIS A 1104 Method of Test for Unit Weight of Aggregate and Solid Content in Aggregate
3) JIS A 1108 Method of Test for Compressive Strength of Concrete
4) JIS A 1109 Method of Test for Specific Gravity and Absorption of Coarse Aggregate
5) JIS A 1110 Method of Test for Specific Gravity and Absorption of Coarse Aggregate
6) JIS A 1126 Method of Test for Soft Particles in Coarse Aggregate by Use of Scratch Tester
7) JIS A 1137 Method of Test for Clay Contained in Aggregate
8) JIS A 5308 Appendix 2 "Method of Test for Particles Floating on Liquid at Specific Gravity 1.96"
9) JIS A 5308 Appendix 7 "Alkaline-Silica Reaction Test for Aggregate (Chemical Method)"
10) JASS 5T-202 Analysis of Salinity in Normal Fine Aggregate

Results: The tensile strength of reinforcing steel bar in the cpncrete is 51.1 kg/mm^2, and Young's modute is 2.34 x 10^6 kgf/cm^2. The measured readings are all within permissible values.

2) Laboratory Analyses on Construction Materials

a. Aggregate

Table 4.1.11 shows the results of laboratory analyses made on four samples of coarse aggregate and three samples of fine aggregate collected on the site. The gravel and sand contained a large amount of clay lumps. The material has been judged as detrimental from the results of alkaline-silica reaction tests (by the chemical method). However, a large value for clay lumps was partially caused by the large number of sand lumps, which was broken up in water. There will be no problem if it is understood in advance that the sand lumps become loose and the gradation may change. It was understood from the results of alkaline-silica reaction tests that the gravel and sand were composed of chert containing a small amount of quartz, siliceous schist, or agate, and that these materials are detrimental. However, these materials will not have a great influence on the quality of concrete because exceptionally high alkali is not contained in cement made in Egypt.

b. Cement

The results of chemical analyses and the specific gravity, fineness, setting, heat of hydration, and false setting for three types of portland cement are shown in Table 4.1.12. Test results were compared with Japanese cement for the alkali equivalent value Na_2O, with the following results: Helwan cement, 0.79%; Tora cement, 0.77%; Japanese cement, 0.65 to 0.7%.

Table 4.1.12 Results of Analysis of Normal Portland Cement Made in Egypt

Test Item \ Specimen			Helwan	El Suez	Tora
Physical Test	Specific Gravity		3.14	3.17	3.13
	Fineness	Specific Surface (Brane Law)	3320 cm^2/g	3540 cm^2/g	3290 cm^2/g
	Setting	Water Quantity	22.5%	25.0%	22.5%
		Initial Setting	1-05	1-35	1-10
		Final Setting	2-10	2-40	2-25
	Stability	Boiling Method	Good	Good	Good
	Strength / Bending Strength	3 days	23 kgf/cm^2	30 kgf/cm^2	23 kgf/cm^2
		7 days	32 "	41 "	31 "
		28 days	46 "	66 "	46 "
	Strength / Compressive Strength	3 days	82 kgf/cm^2	112 kgf/cm^2	81 kgf/cm^2
		7 days	118 "	166 "	126 "
		28 days	180 "	251 "	184 "
	Heat of Hydration	3 days	- cal/g	- cal/g	- cal/g
		7 days	61.5 "	62.2 "	61.0 "
		28 days	72.5 "	74.9 "	71.3 "
	Fault	After 5 min	Reference Bar 35: 1	Reference Bar 34: 3	Reference Bar 35: 1
		After 10 min	1	1	1
Chemical Analysis	Ingredient	Ig-loss	2.88%	1.91%	2.51%
		Insoluble Remnant	1.02%	0.47%	0.93%
		SiO_2	20.41%	21.56%	21.01%
		Al_2O_3	6.62%	6.08%	6.29%
		Fe_2O_3	1.61%	1.73%	1.64%
		CaO	60.62%	61.72%	62.00%
		MgO	2.84%	1.57%	2.68%
		TiO_2	0.62%	0.44%	0.61%
		MnO	0.05%	0.02%	0.05%
		SO_3	0.65%	0.79%	0.59%
		S	0.00%	0.00%	0.00%
		P_2O_3	0.03%	0.04%	0.03%
		Extricate CaO	0.97%	0.28%	1.33%
		Na_2O	0.59%	0.47%	0.57%
		K_2O	0.31%	0.31%	0.31%
		Na_2Oeq	0.79%	0.67%	0.77%

c. Mixing Water

The analytical results of water are shown in Table 4.1.13. All the samples were drinking water that can be used for mixing concrete.

Table 4.1.13 Results of Analysis of Mixing Water

Sample		Suspended solid (gram/ltr)	Dissolved solid (gram/ltr)	Chloride ion (ppm)	Na (mg/l)	K (mg/l)	Ca (mg/l)	Mg (mg/l)	SO_2 (mg/l)	SO_4 (mg/l)
Suez Sweet Canal	0.5 m	0.06	0.42	153	109	7.91	4.21	0.50	0.00	0.08
	1.0 m	0.07	0.38	153	108	7.88	4.13	0.50	0.00	0.08
	2.0 m	0.08	0.37	153	107	7.66	4.02	0.48	0.00	0.08
Tunnel Site		0.06	0.40	146	104	7.64	1.04	0.51	0.00	0.08
Suez Water Plant		0.01	0.42	170	125	8.12	4.59	0.03	0.00	0.10
Standard Value		2 max	1 max	200 max	-	-	-	-	-	-

3) Discussion

The following facts have been learned from the site survey of the existing tunnel's concrete components and analyses of materials being used for the Rehabilitation of the Tunnel.

a. Deterioration of Concrete Components

a-1. At Locations of Leakage

Segments:

- Segment ribs cracked, spalled, became loose, or were separated by the expansion of rusted steel. Concrete cover with an average thickness of 5 cm was observed completely removed at numerous locations.

- It was noted that calcareous dolomite had started to leach on severely deteriorated concrete surfaces.

- Bolts and nuts were severely corroded.

Culvert Support Wall

- Wall surfaces had started to spall at numerous locations.

Culvert Support Base

- The base was extremely deteriorated and formed into broken pieces.

a-2. At Locations Not Leaking

Segment(s)

- Cracks along the lines of reinforcing bars were noted.

- Most bolts and nuts became loose.

- Rust was formed on steel with a 5 cm concrete cover. This phennomena was caused by on Cl removing and concentration in concrete by carbonation of cement.

b. Results of Analyses of the Concrete

b-1. The proportion of the coarse aggregate composed of calcareous dolomite was 30% or more. Fine aggregate consisted of sand. Cement was a sulfate resistant type. 1 to 2 kg/mp^3 of chlorine ion, was detected even deep in the concrete. Since high salinity in concrete not only causes reinforcing steel rust, but also Nacl tends to interfere the dehydration of cement, it is required to control the quality of mixing water at the time of construction.

b-2. The unit cement ratio in concrete ranged from 388 to 447 kg/m^3; alkali content in cement was 0.40 to 0.54%, expressed as Na_2O; the strength was 415 to 650 kgf/cm^2, the dynamic elastic modulus was 3.30 to 5.19 x 10^5 kgf/cm^2, and the static elastic modulus was 2.63 to 3.19 x 10^5 kgf/cm^2. The elasticity was relatively low for the strength. The results indicated that the density of the concrete was lower than that of the common concrete of today. Water and can easily penetrate into concrete with a hardened cement past of low density.

b-3. The depth of carbonation was measured with a solution of phenolphthalein which indicated carbonation depths of 1 to 3 mm for the concrete from wet areas, and 9 to 10 mm for that from dry areas. However, Na reached the depths of 7 to 8 cm, and CL reached the depth of 10 cm for concrete from wet areas. Salinity analysis showed Cl between 6 and 8 kg/mp^3 at the depth of 5 cm. It was observed that K was concentrated at a depth of 5 to 10 cm, and Cl, Na, and S were concentrated at a depth of from 2 to 8 cm.

b-4. Rust was formed on the surface of reinforcing bars in concrete to a depth of 7 or 8 cm, notwithstanding the source of sampling. The tensile strength of reinforcing bars 20mm in diameter was 51.1 kg/cm^2, and the measured Young's modulas was 2.34 kfg/cm2. Both are within permiable values.

c. Discussion

High salinity in concrete not only causes reinforcing steel to rust, But NaCl tends to interfere the dehydration of cement. NaCl increases the alkali content in cement paste and tends to produce hardened cement paste of low density with the $Ca(OH)_2$ formed by Ca separated from cement *(4). On the other hand, it is said that sulfate resisting portland cement used contained little calcium alminate to protect the concrete from expansion by calcium alminate reaction occurring in cement, caused by penetrated sulfate and that hardened cement paste was low in density. Water and air can easily penetrate into concrete with a hardened cement paste of low density. It is presumed that the Tunnel has concrete with these characteristics.

Leaked water containing salt infiltrates into the concrete and the water tends to evaporate in the Tunnel. When the salinity in the infiltrated water exceeds the solubility, salt is crystallized and the concrete starts to separate.
Salty water tends to dissolve calcareous dolomite in the aggregates or to cause rust to form on steel deep in the doncrete. Corrosion of steel can also be caused by the mass transfer of chlorine in dry areas.

Therefore, major causes for the deterioration of the concrete, except for that caused by construction deficiencies, can be concluded as mentioned below, as a result of the analyses of the construction materials.

- Mixing water or aggregate containing salt was used in concrete.

- Sulfate resisting portland cement was used.

- Calcareous dolomite aggregates were used.

- Based on the results of analysis described above, it is recommended that the following precautions be observed in the selection of construction materirals.

- Salt is not contained in concrete mixing water.

- Either normal portland cement or portland blast furnace cement is selected.

- Aggregate other than limestone or calcareous dolomite is selected.

d. Analyses of Materials for the Rehabilitation Works

d-1. Aggregate

Since the existing concrete was made with calcareous dolomite aggregates and the concrete performed poorly, it is desirable that aggregates other than calcareous dolomite or lime-stone be used in the Rehabilitation Works. Gravel and sand in the samples were relatively good in quality, but the alkaline-silica reaction test (by chemical method) and the test for clay lumps indicated high values. These test results indicated characteristics that were judged as "aggregate considered dileterious".

However, large sanf lumps, were contained in the gravel and sand that indicated the high values, but clay minerals detrimental to concrete was not found. Therefore, it should be understood in advance that the gradation of sand can vary when sand lumps become broken up in water.

It is specified in JIS that any material judged as "Aggregate considered dileterious" detrimental when tested by the chemical method of alkaline-silica reaction test must be retested by the mortar bar method. Local gravel and sand are mainly composed of chert containing a small amount of quartz, siliceous schist, or agate, and it can be assumed from the mineralogical point of view that they will be deemed as harmless if the materials are subject to mortar bar test.

Alkalized aggregate may be improved if the gross weight of alkali is controlled. The alkali content in concrete, equivalent to Na_2O, if reduced to 3 kg/m^3 or less, is acceptable. Normal portland cement in Egypt contains alkali between 0.67 and 0.79%. These rates are within the design unit cement ratio of 300 kg/m^3 and considered fully acceptable.

d-2. Cement

Chemical and physical analyses were made on three types of portland cement made in Egypt. The results of analyses did not indicate significant problems. Alkali content equivalent to Na_2O ranged from 0.67% to 0.79%, slightly higher than that of Japanese portland cement (Na_2O = 0.67 to 0.7%).

d-3. Mixing Water

The samples collected from three locations are all fresh water that can be used for mixing concrete.

<u>Appendix and Technical Data</u>

*(1) Sir William Halcrow & Partners Ltd. in conjunction with Tarmac Overseas Ltd. and the Arab Contractors (Osman Ahmed Osman & Co.): Ahmed Hamdi Tunnel CONDITION SURVEY FACTUAL REPORT, Vol. 1 Text, July 1988

*(2) JICA: Report of Analyses on Concrete Samples of Ahmed Hamdi Tunnel, Arab Republic of Egypt,S 1990

*(3) Ministry of Construction, Technical Bulletin No. 285, Standard of Gross Weight Control for Chloride Content in Concrete (Civil Structures),S 1986

*(4) Mori, H, Minegi, K, Ohta, I, and Akiba, T: Alkaline Effect on Microstructure of C3S Hardened Concrete,S Cement Technical Annual, Vol. 25, pp. 40 to 47, 1971

*(5) Kobayashi.K, Shiraki.R, Uno.Y, Kawai.K,: "Carbonation and Concentration of Chrorine in Concrete Containing Chloride(II)", Report of the Institute of Industrial science, Vol.41,No4,pp234-236,1989

4.2 Design Criteria and Policy

"Analysis in Japan" is the second phase coming after the Field Survey in the Basic Design study. This includes:

- Evaluation of Project
- Basic Design Study
- Construction Planning

as shown in Fig. 4.1.1.

Design Criteria and Policy based on the field survey are important key functions for the execution of the Basic Design Study.

And, the design policy for the Rehabilitation Works has been confirmed with S.C.A., we should specify the following points.

4.2.1. Tunnel Structure

(1) The tunnel structure shall go through no further deterioration and similar Rehabilitation Works shall not be required again. For this, the followings are to be considered.

1) The Rehabilitation Works for the whole length (1650m) of the Tunnel shall be implemented by the method of reinforced concrete lining after applying waterproofing sheet including the road deck within the existing concrete segments.
Water proofing sheets and fleece installed between the segment of the actual lining and the reinforced concrete lining should be adequately selected for the material, the thickness, and the flexibility of the installation.

2) The lining design shall be strong enough to stand soil and water pressure which is always present.
3) The material selection such as cement for lining and road deck concreting shall be strictly safe against salt deterioration.
4) The waterproofing sheet shall be installed before lining to prevent salty water leakage penetration which is the cause of the deterioration.
5) Enough thickness of concrete cover shall be maintained to guard against burst of reinforcing steel bar.
6) After the Rehabilitation Works, the Tunnel shall be maintained through regular inspection and slight partial repair works only.

(2) The above mentioned rehabilitation design for the Tunnel shall allow the implementation of future widening, deepening and doubling projects of the Suez Canal.

1) In the process future widening, deepening and doubling of Suez Canal, soil sodium deformation can be anticipated.
2) The soil medium deformation will be transmitted to the new lining through the concrete segment ring, fleece and waterproofing sheet. By analysis of this transmission, the design shall be safe against the deformation.

(3) The existing road capacity and limit shall be secured.
- The road limit shall be equal to the existing one, which is 5.0m in height and 7.5m in width.

(4) Concerning the drainage system, salty ground water shall be directed to the drain at the invert and collected into Nadir sump tank.

4.2.2. Related Facilities

(1) Basis of Ventilation System Plan

1) The existing transverse system consists of a 4 fan chamber with shaft, 16 axial reversible fans, air supply and exhaust ducts and a central control system. Present value of the system could be estimated at L.E.15.0 Million. These valuable existing facilities for the ventilation system must be utilized as much as possible without being damaged.

2) The present traffic volume is about 1,500 vehicles per day. To forecast a correct future traffic volumn is quite difficult due to the unstable development in Sinai area. Therefore, the capacity of ventilation facilities shall be based on the maximum traffic volume which was calculated at 1,500 vehicle per hour with diesel car ratio of 36 percent.

3) The required ventilation volume shall be reviewed based on the maximum emission of gas given in the Category of Standard D of PIARC (1987) which includes the countries having no legislative restriction on the emission of gas for each vehicle.

(2) Reuse of (Utilization of) Traffic Safety Facilities

1) As a principle, all equipment in the Tunnel, such as fire hydrant, telephone, TV camera, CO/VI monitor shall be reused after the rehabilitation. However, the illumination system shall be changed from the existing fluorescence type to sodium type.

2) The existing electrical wires on the racks will not be reused due to deterioration in quality during removal. However, 2-11 KV power cables from the west to the east will be reused.

(3) Ancillary Works in the Tunnel

1) New ceiling panel will be installed to protect the exhaust duct area as much as possible. The existing corner-cover to stop wind leakage will be replaced by a new corner support block.

2) The existing secondery lining plate will be replaced by an in-combustible material for safety of human beings.

3) Walkway width will be narrowed about 300mm. Therefore, the number of supporting angle frames for the racks will be increased to install the same number of wires under the walkways.

4) Pipes for drainage and Nadir sump will be replaced due to deterioration and change in lengths. However, pipes for fire hydrant will be reused.

4.2.3. Fresh Water Supply Pipeline

The Permanent Fresh Water Supply Pipelines (two lines) shall be installed outside the Tunnel crossing the bottom of the Canal in order to secure water supply to Sinai Peninsula and to facilitate the Rehabilitation Works.

The present situation of the pipelines are as follows:

(1) Pipes in Tunnel : 2 x 500mm NID

- Ductile Iron Pipe Spigot and Socket spun with neoprene rubber gasket, Class K9 (25 bar PN), BS4772
- Installed in the Through Service Duct in vertical parallel as in the DWG No.AC8501

(2) Scope of Work for S.C.A

The scope of work of Water Supply Pipelines by S.C.A. is shown in Fig.4.2.1.

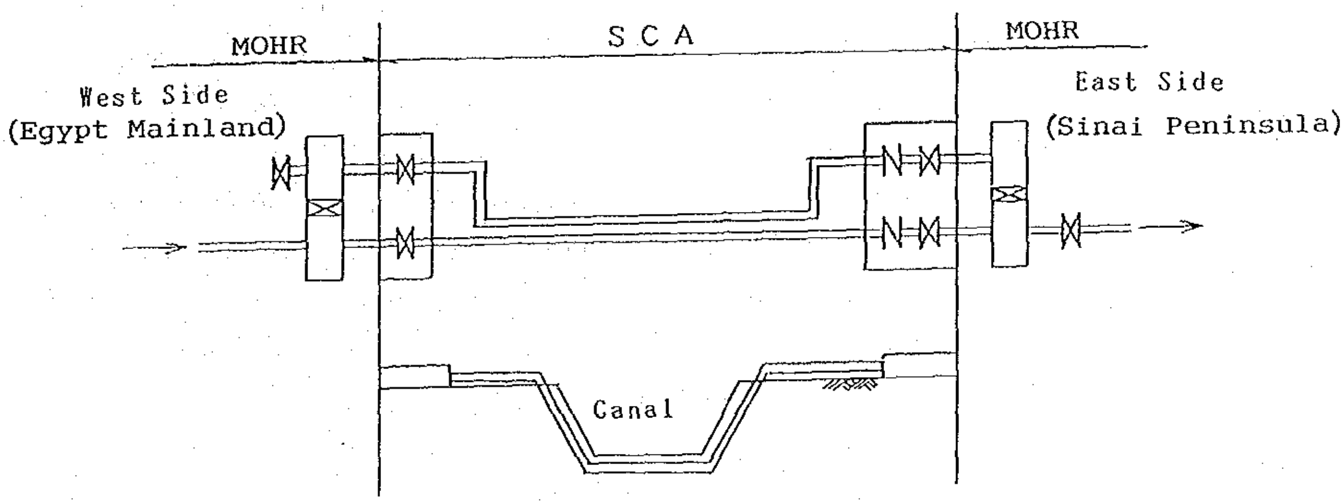


Fig.4.2.1 S.C.A. Scope of Work for Water Pipelines

The water treatment and the pipelines except for S.C.A. scope are under the control of MOHR (Minister of Housing and Reconstruction).

(3) The water treatment capacity :

at present : 150 l/sec = 540 m^3/hour

in future : 400 l/sec = 1440 m^3/hour

(4) The water supply capacity and actual operation :

- capacity 8 bar PN - 848m^3/hour
- actual 8 bar PN - 324M^3/hour

Operation* 17hours : 6:00 - 23:00 Summer (5-9)

10hours : 6:00 - 16:00 Winter (10-4)

* depends on the demand/reservoir in Sinai
* one pipe to be under operation out of two pipes

(5) Switch from one pipe to another is available without suspension (by valve handling) when necessary.(ref. DWG No.AC2118 and AC2121)

(6) Future increase in water demand in the coming 5 years will be expected to be double. The water supply capacity of 500mm NID pipe is 848 m^3/hour and it is approximately 2.5 times of the present actual supply of 324 m^3/hour. This should be enough for the increase in future demand. However, according to Dr. A.A.Gowely, Local Governor of Ismailia, there is a big project in Sinai Peninsula of increasing the population through huge agricultural expansion plan. So, the water supply to Sinai Peninsula is one of the most important subjects under concern.

4.3 Basic Plan

4.3.1 Tunnel Structure

4.3.1.1. Tunnel Lining

The following Loads which should be taken into consideration in the design of lining for this Tunnel are decided:

a. Earth Pressure
b. Water Pressure
c. Dead Load
d. Buoyancy
e. Soil Medium Deformation due to the Canal Expansion Work
f. Seismic Load
g. Other Load(s)

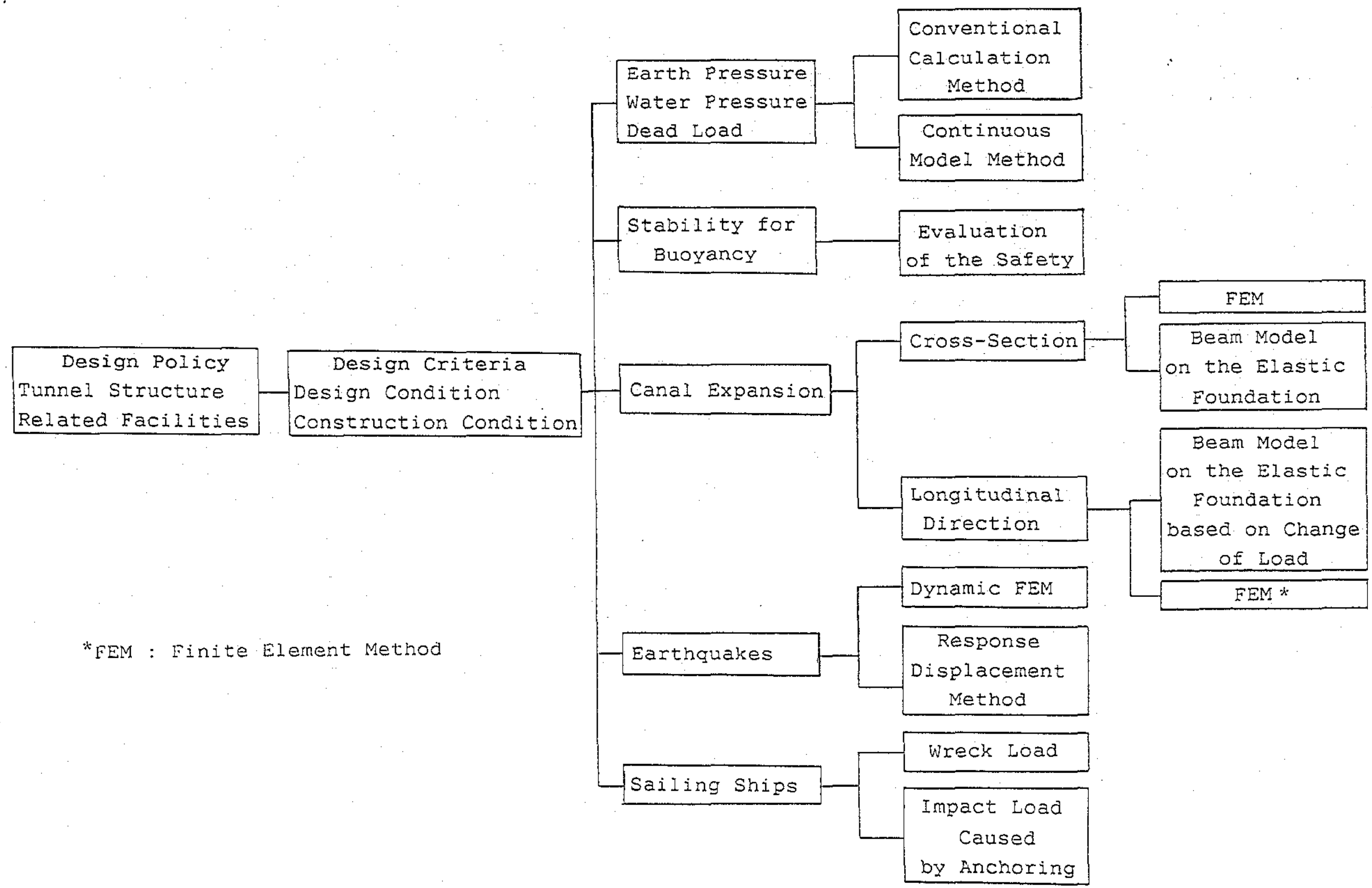
Among them, "a, b and c" are listed as the conventional design conditions. "d and e" are the special design conditions of this project as for the evaluation/analysis on the reduction of the overburden and the soil medium displacement due to the Canal expansion work. "f" is derived from the historical seismic data in areas adjacent to the project site. "g" shall be added for the analysis on the influence by cruising ships.

To correctly evaluate items "a, e, f and g" among these Loads, it is necessary to take into consideration interaction between lining and ground in addition to the soil conditions and construction conditions.

Accordingly, it has beed decided to confirm the safety of lining by analizing using all methods currently available in this basic design.

The flow for the design of the tunnel lining is shown in Fig.4.3.1.

The following is a main summary of the design of the tunnel lining including the deign methods, design conditions and design results. The details are shown in Appendix-6.



*FEM : Finite Element Method

Fig.4.3.1 **FLOW FOR DESIGN OF TUNNEL LINING**

1) Design Method

For the design method of tunnel lining, the following two methods shall be applied:

-Allowable stress design method (Service Ability Condition)	:To be applied for the conventional design conditions*1)
-Limit state design method (Ultra Limit State Condition)	:To be applied under the extremely special situation*2) or the most severe design condition

By the above two methods, it has been so designed that the tunnel shall not collapse even under the worst condition.

Notes for *1) and *2) marks;

*1): Since the Tunnel has been planned as a drainage type tunnel, it can be basically considered that no water pressure is applied to the lining.
In other words, under the normal condition, only earth pressure and dead load are to work/apply on the tunnel lining.

*2): In case of the tunnel drainage system being broken, which shall be a very rare/special case,
it has been decided that water pressure in addition to earth pressure and dead load are to apply on the tunnel lining.

2) Design Condition of the Tunnel Lining Section

Desgin lining thickness of the Tunnel is to be 400mm and the lining thickness as constructed is to be 450mm in consideration of the tunnel's deformation as shown in Fig.4.3.2.

The Basic reinforcement steel bar is as follows:

-Sectionnal direction----- D22ctc200 (See Fig.4.3.3)
(Deformed Bar, Diameter:22mm, Interval:200mm)

-Longitudinal direction--- D19ctc200
(Deformed Bar, Diameter:19mm, Interval:200mm)

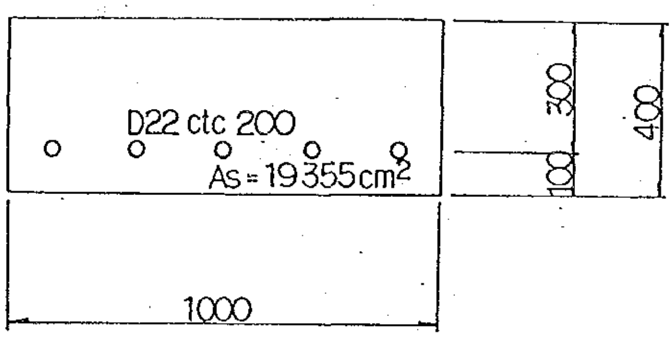
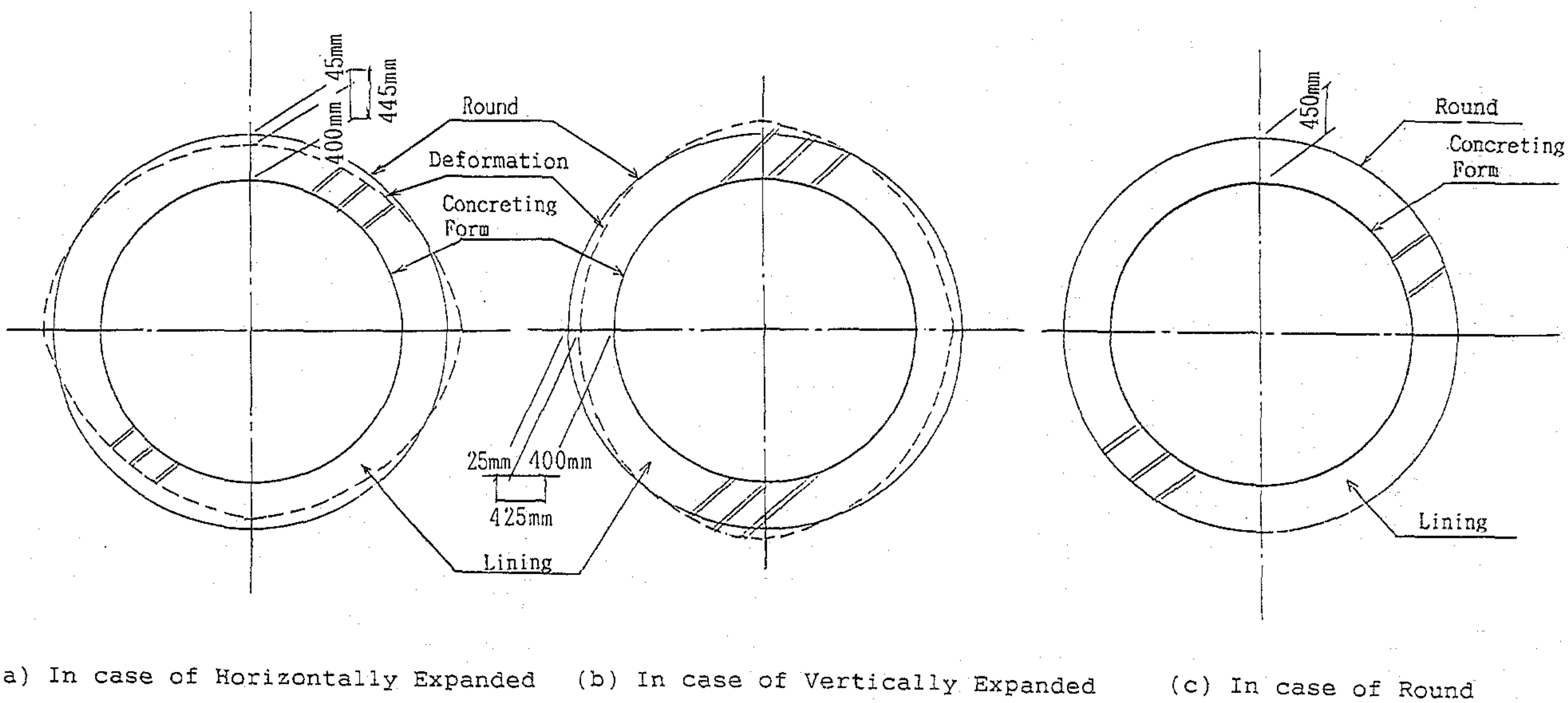


Fig.4.3.3 <u>Design Condition of the Tunnel Lining Section (Thickness and Reinforcement Steel Bar)</u>



※ Note: Deformation mode of segment lining derived by Instrumental Measurement.

Fig.4.3.2 Required Thickness of Lining for Construction

3) Design Conditions and Results

a. Design Relative to Earth and Water Pressure, and Dead Load

a-1. Allowable stress design method

This method has been applied for the conventional design conditions. The sectional force was calculated by applying a trapezoid distributed load used in the conventional calculation method for a structural model in which lining was evaluated by ring structure with uniform rigidity and ground reaction by springs around the ring structure as shown in Fig.4.3.4 "Analytical Model of Lining for the Earth Pressure and Water Pressure (Conventional Method)".

The design result can be summarized in Fig.4.3.5 "Allowable Strength, Ultimate Strength and Sectional Force of Lining" and it has been confirmed that the tunnel lining shall be safe.

Since the road deck and lining are incorporated into one at this time, the road deck was taken into consideration as a structural member for calculation.

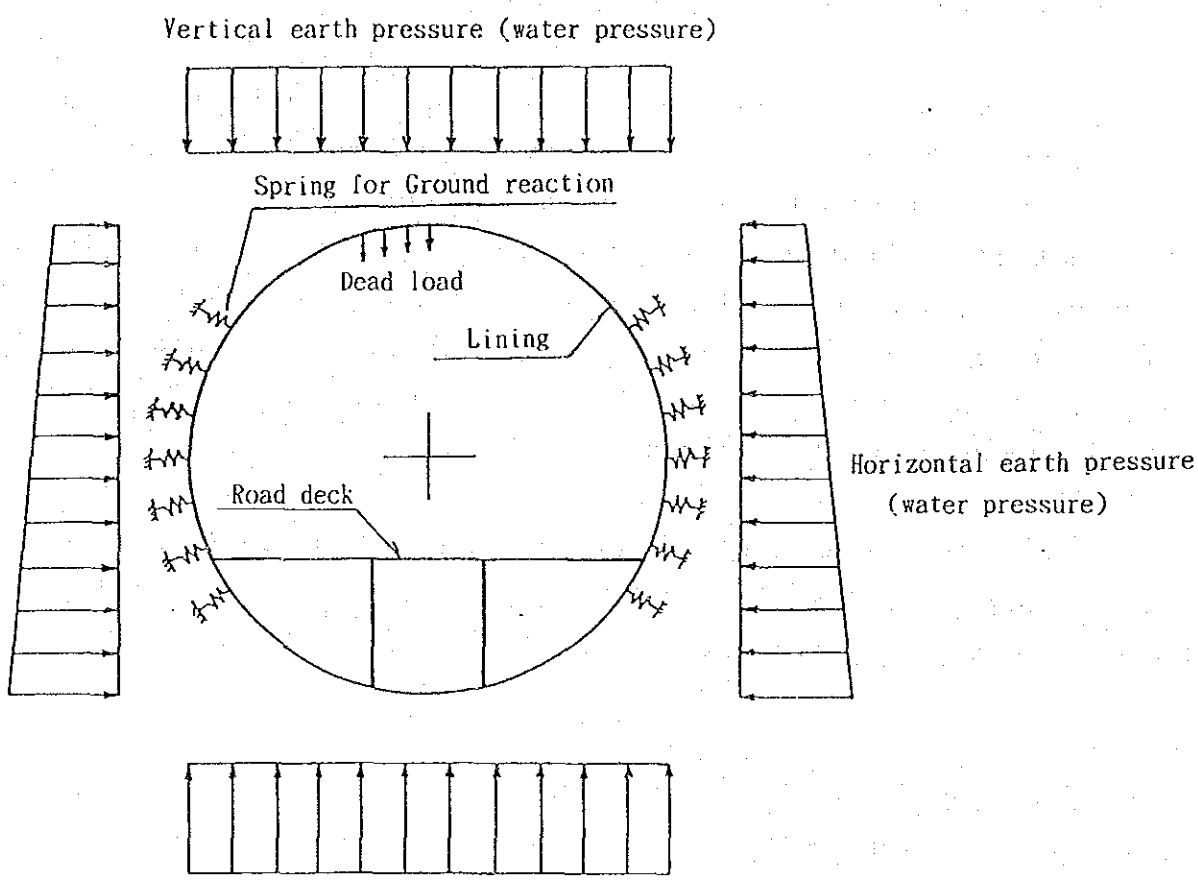


Fig.4.3.4 Analytical Model of Lining for the Earth Pressure and Water Pressure (Conventional Method)

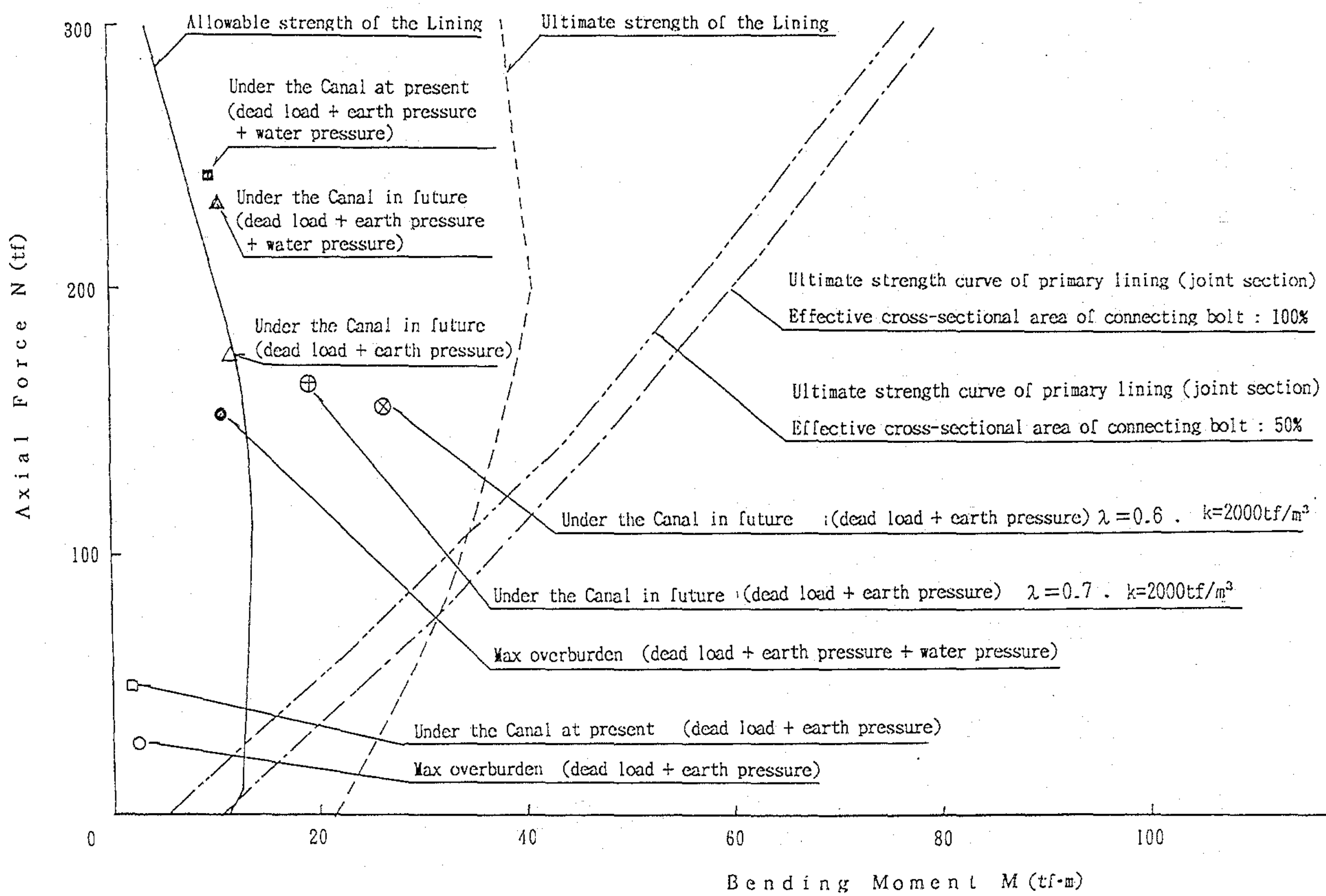


Fig.4.3.5 **Allowable Strength, Ultimate Strength and Sectional Force of Lining**

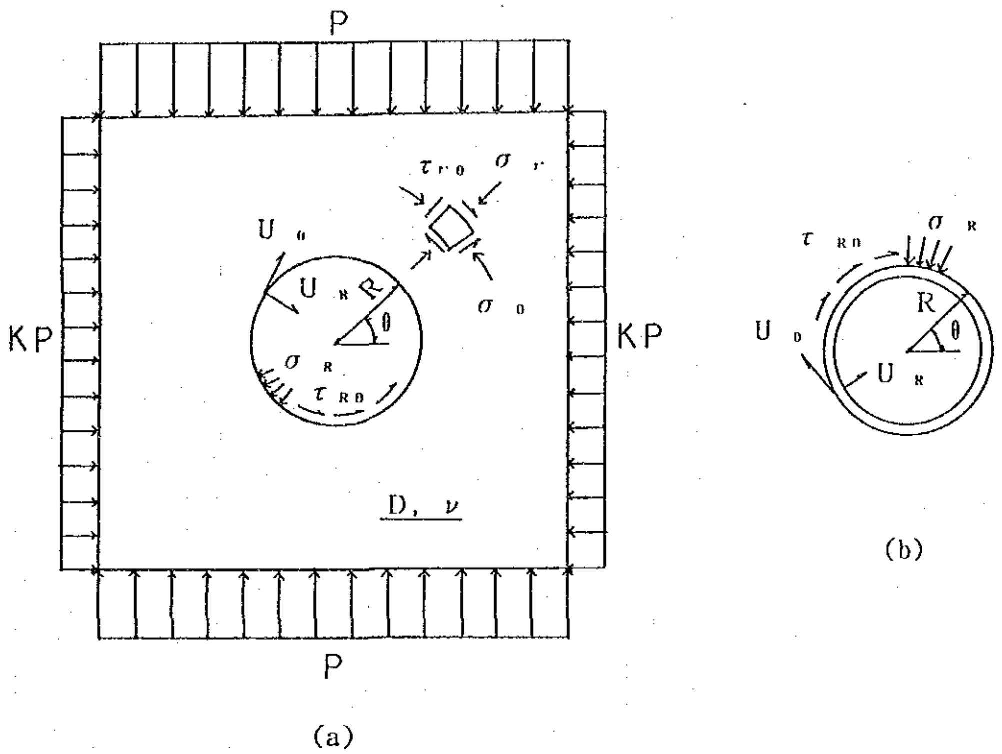
a-2. Limit State design method (Continuum Model Method)

This method has been applied under extremely special situation or the most severe design condition.

An analytical method using a continuum model can be used when the earth presure and water pressure are applied to the lining and soil medium at the same time.

In this model, it is assumed that P(= vertical load) and Kp(= horizontal load) are working to soil medium after the excavation and tunnel lining as shown in Fig.4.3.6.

The evaluation of safety was confirmed by the limit state design method as shown in Fig.4.3.7.



Where σ_R, σ_θ : Ground Stress of Lining Surface

$\tau_{R\theta}$: Ground Shearing Force of Lining Surface

U_R, U_θ : Ground Displacement of Lining Surface

Herbert H.Einstein, M.ASCE and Charles W.Schwartz, A.M.ASCE [Simplified Analysis for Tunnel Supports] Journal of the Geotechnical Engineeering Division, April 1979,pp.499-518.

Fig.4.3.6 Analytical Model Based on the Theory of Elasticity
(Continuum Model Method)

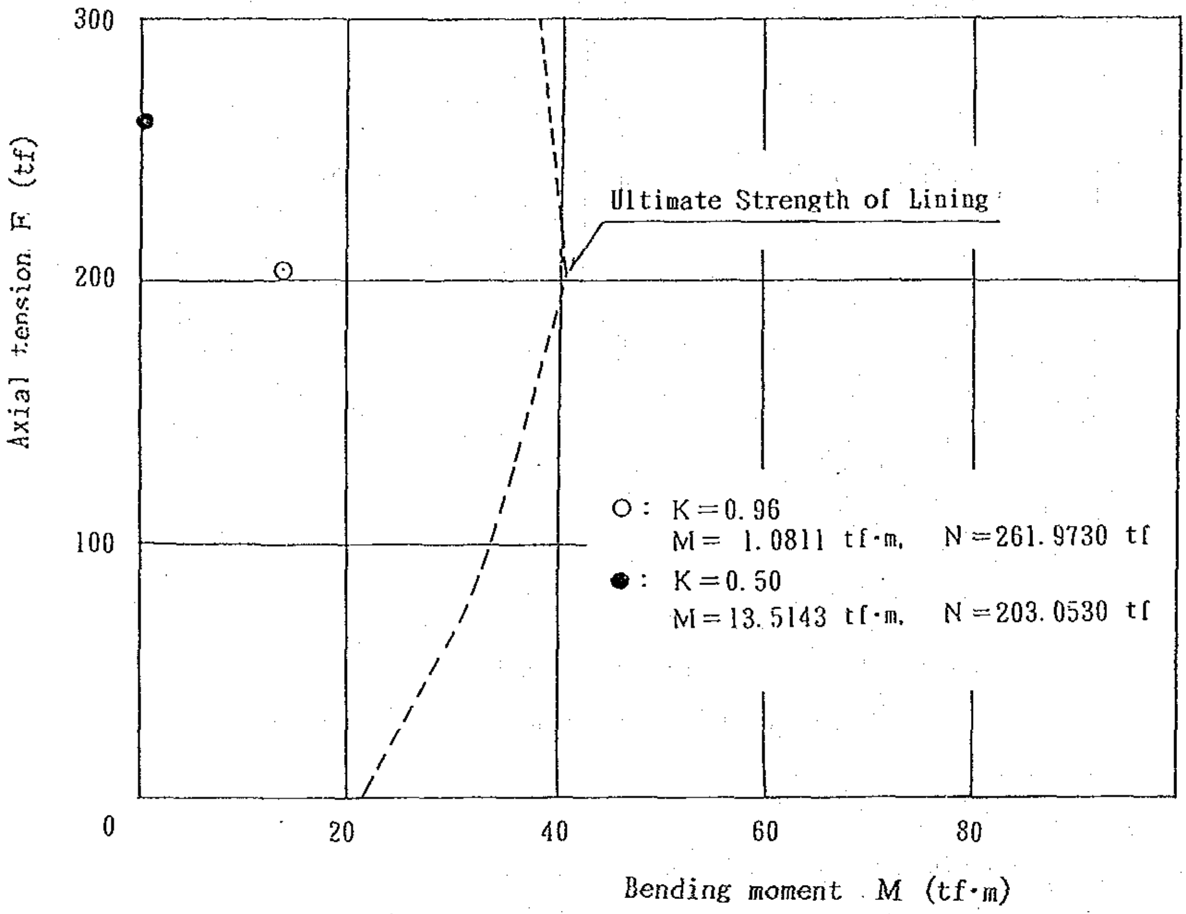


Fig.4.3.7 Strength of Lining and Sectional Force by Limit State Design Method

b. Stability Relative to Buoyance

Since this Tunnel is a drainage type tunnel, it is not necessary to consider that 100% of the theoretical buoyancy is applied to linnig. To secure safety, however, the design was kmade on the assumption that 100% of the theoretical buoyance operates.

As regards to stability of the Tunnel, the shearing resistance force (S) of the overburden portion will also be taken into consideration as shown in Fig.4.3.8 and if the relationship exists, among buoyancy (F) applied to the Tunnel, weight (P) of the overburden and the tunnel dead load (w_1:Segment ring, W_2:Lining), stability can be secured.

$$\alpha=(W_1+W_2+2S+P)/F$$

Where

α : Safety Factor

W_1: Weight of Segment Ring

W_2: Weight of Lining

P : Weight of the Overburden

S : Shearing Resistance of Ground

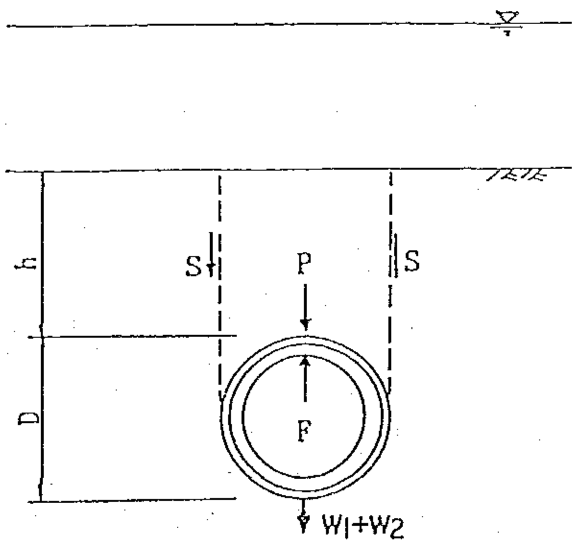


Fig.4.3.8 Relationship among Bouyancy and Other Loads that Acts upon the Tunnel and Resisting Forces

$\alpha=(41.71+2\times133.48+73.08)/110.97=3.72$

The stability relative to buoyance has been with the safety factor of 3.72 through the above analysis.

c. Design Relative to Influence of the Canal Expansion Work

It is assumed that the most severe condition analysis is required when the Canal expansion work reaches 27.0m in depth. The over burden remains only 7.0m at the top of the Tunnel under this condition.

c-1. Design Relative to Influence of the Canal Expansion Work (Cross-section)

For the design relative to influence of the Canal expansion work in cross-section, the Finite Element Method (FEM) has been used. The analytical model FEM is shown in Fig.4.3.9.

The rebound of the ground affected by the Canal expansion work was calculated by applying it to a double-deck ring model, in which the segment ring and linng were evaluated by girder, as shown in Fig.4.3.10.

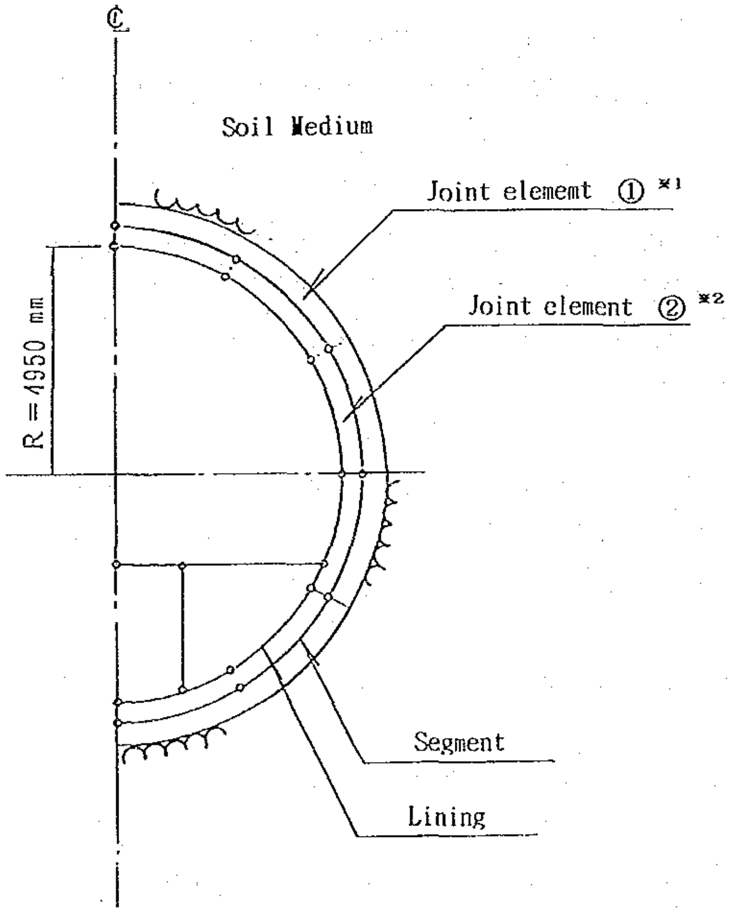
The amount of rebound produced by the Canal excavation at the depth of 27.0m and the lining section force is approximately 200mm as shown in Fig.4.3.11.

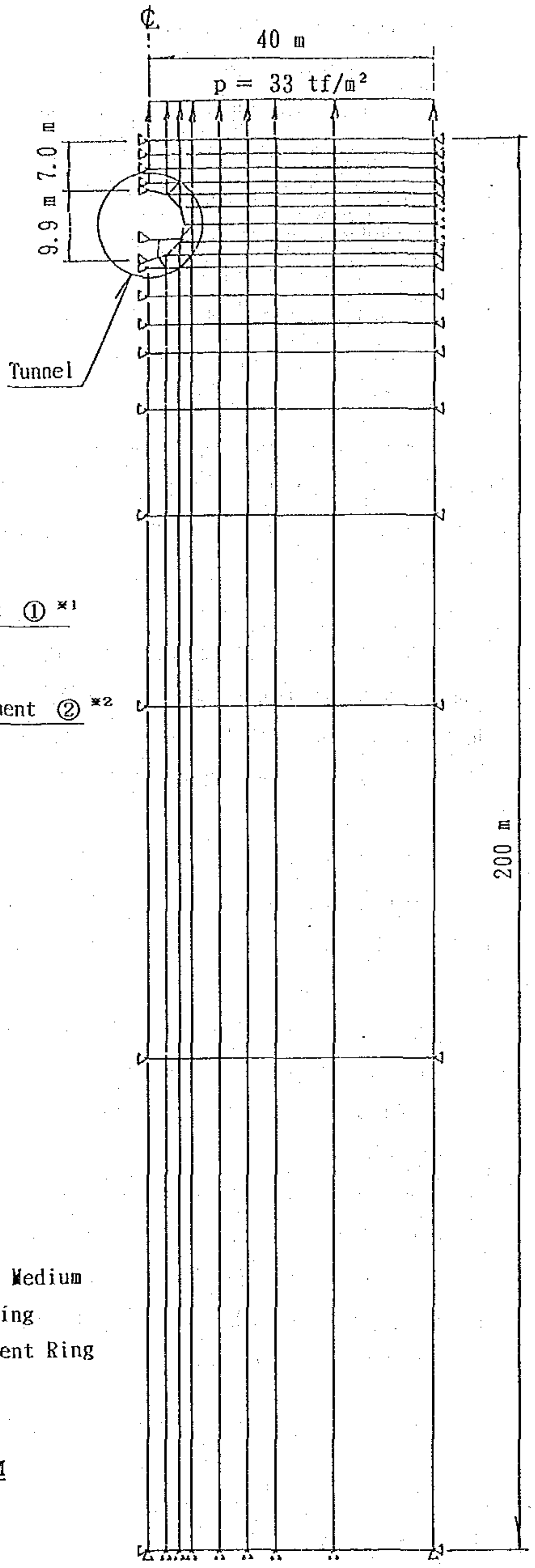
The safety was evaluated by the ultimate strength of the limit state design method.

Verification of the results shows that the ultimate strength exceeds that of the single reinforcement with fleece 7mm in thickness and is within the ultimate strength with fleece 14mm in thickness as shown in Fig.4.3.12.

From this fact, it can be seen that fleece material of 14mm (=7mm x 2) is effective against the influence of the Canal expantion work.

Detail of tunnel element

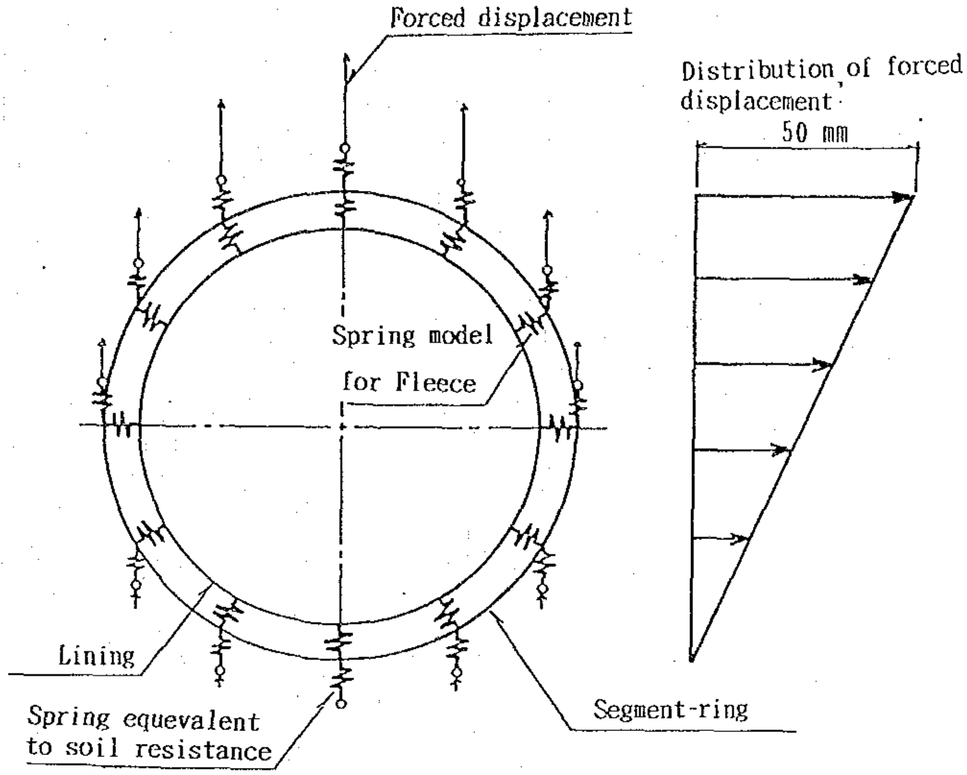




※ 1 Interaction between Soil Medium and Segment Ring and Lining

※ 2 Interaction between Segment Ring and Lining

Fig.4.3.9 Analytical Model of FEM



Segment-ring,

$E = 3500000\ tf/m^2$

$I = 0.01131 * \eta = 0.005655\ m^4 \quad (\eta = 0.5)$

$A = 0.3859\ m^2$

note for μ: effective flexural rigity of segment ring with reduced stiffness joint.

Fig.4.3.10 Analytical Model with a Double-Deck Ring

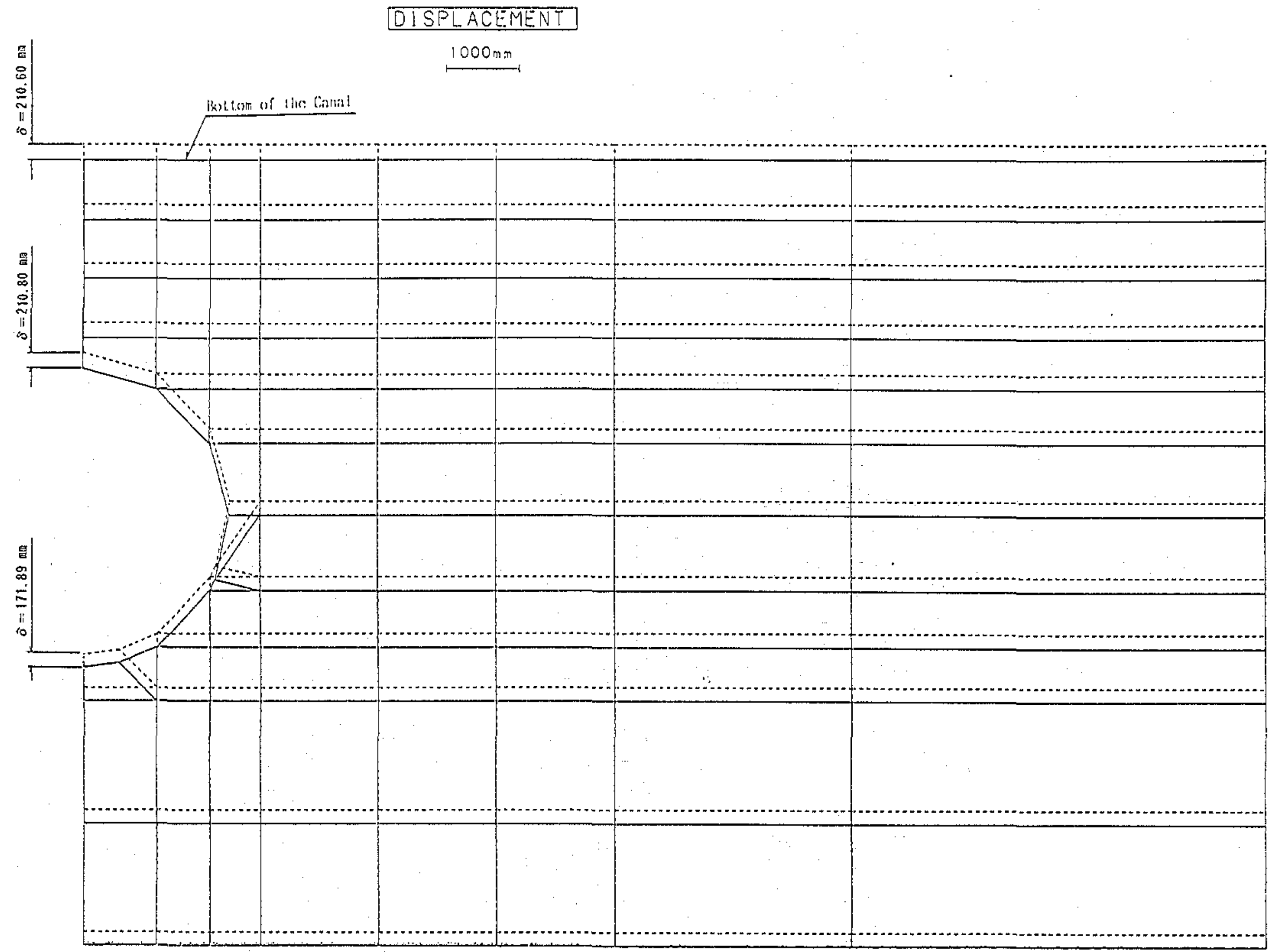


Fig.4.3.11 **Diagram of Ground Displacement**

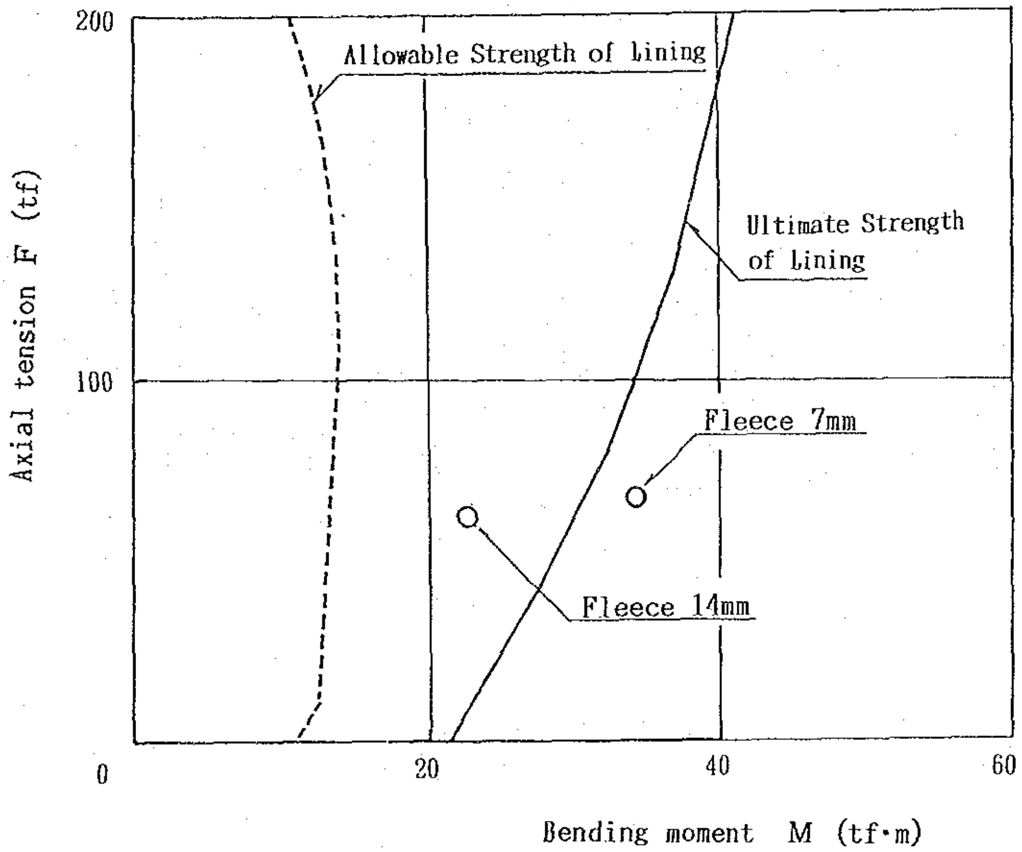


Fig.4.3.12 Strength and Sectional Force of Lining

c-2. Design Relative to Influence of the Canal Expansion Work (Longitudinal direction)

The vertical load, which applies to the Tunnel longitudinal direction due to the Canal expansion work, produces found out difference between the current overburden and the overburden after the Canal excavation as shown in Fig.4.3.13.

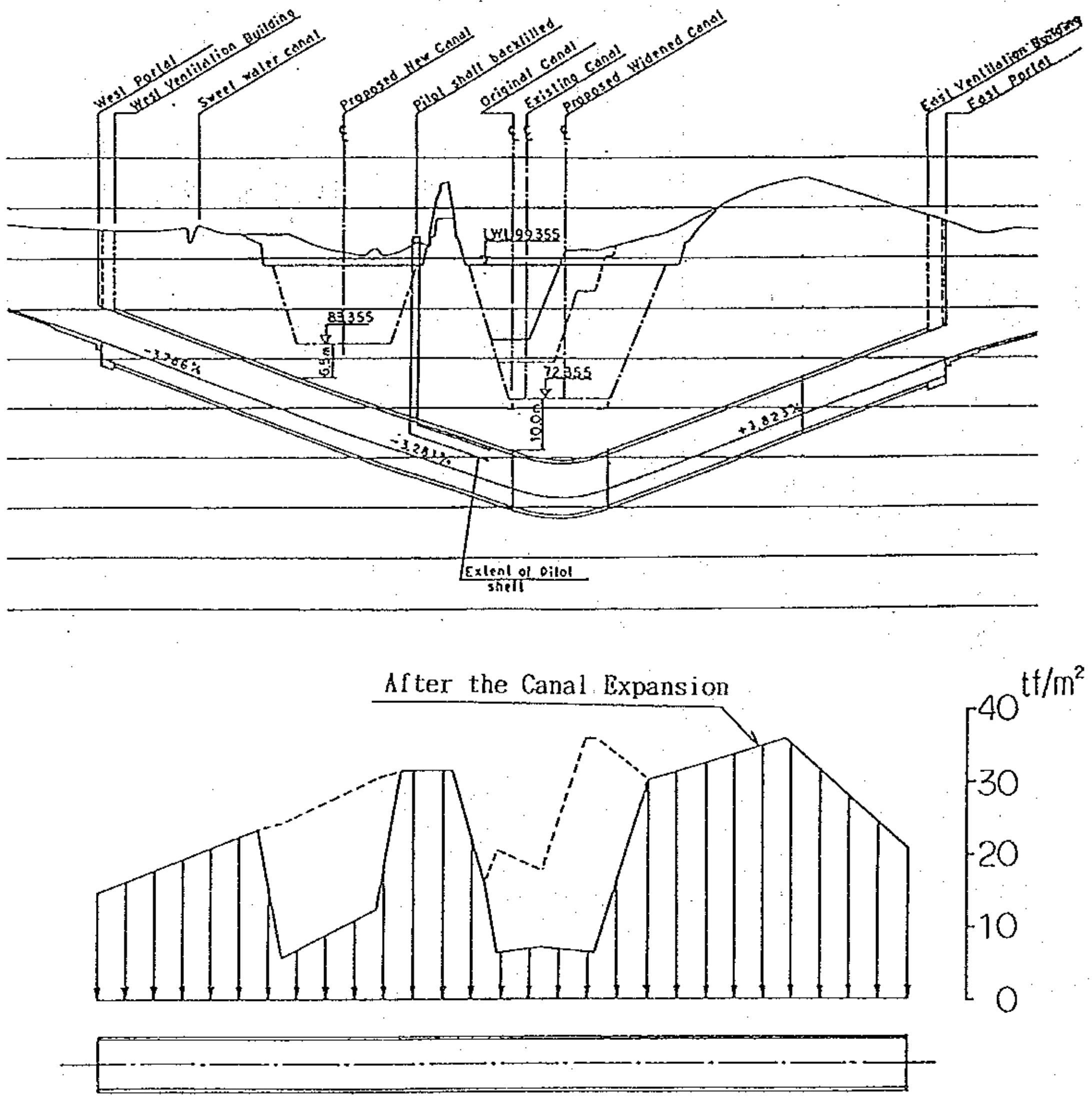


Fig.4.3.13 Distribution of Vertical Load in the Longitudinal Direction of Tunnel

The sectional force due to the Canal expansion work was calculated by Analytical Beam Model having uniform rigidity in the longitudinal direction and the ground reaction as shown in Fig.4.3.14.

Design is carried out by using FEM taking into consideration the influence of the Canal expansion in the Tunnel's longitudinal direction. A FEM analytical model is shown in Fig.4.3.15.

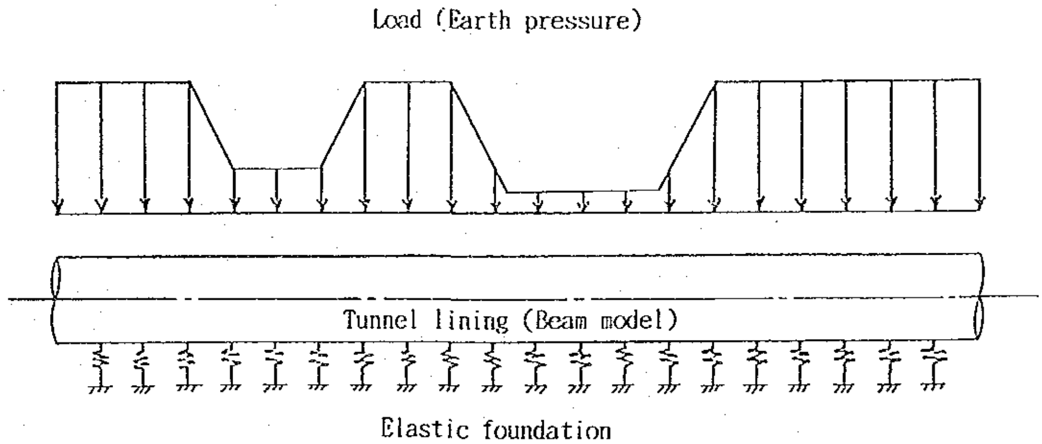


Fig.4.3.14 Analytical Beam Model for the Canal Expansion

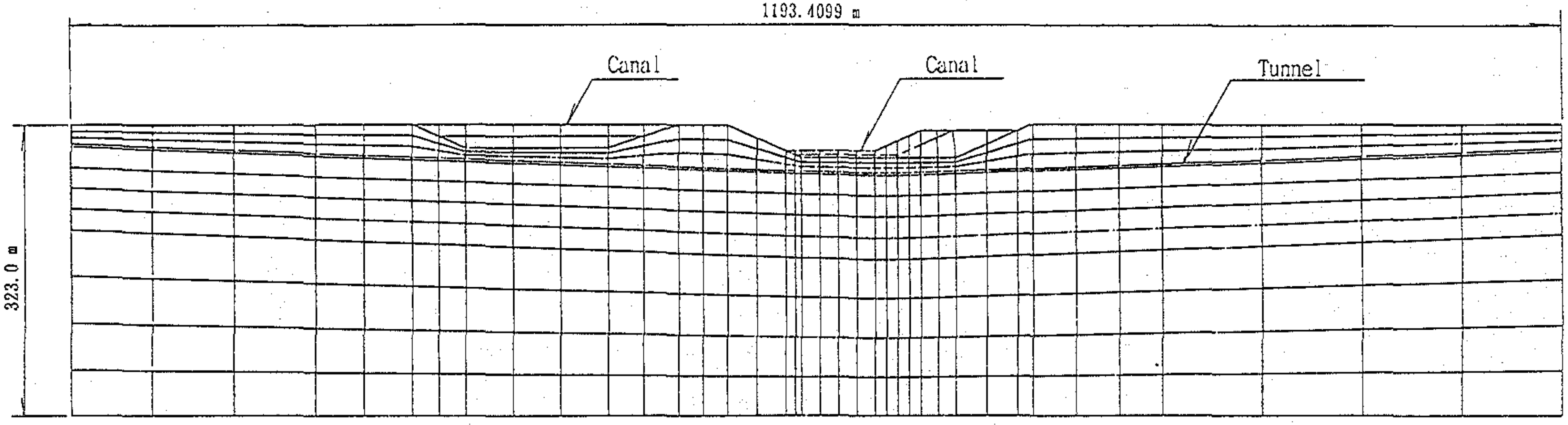


Fig.4.3.15 Analytical Model by FEM

By the applicatioin of FEM analytical model, it has been confirmed that the results of calculation are within the allowable stress intensity as in Table 4.3.1.

Table.4.3.1 Result of Calculation on the Stress Intntensity

	σ c	σca	σ s	σsa
K=3000 tf/m2	14.8	90.0	1310.7	2100.0
K=2000 *) tf/m2	21.2	90.0	1863.8	2100.0

σ_c : Calculated Stress of the Concrete

σ_{ca} : Allowable Stress of the Concrete

σ_s : Calculated Stress of the Reinforcement Steel Bar

σ_{sa} : Allowable Stress of the Reinfrocement Steel Bar

Note for *): As regards the coefficient of ground reaction among these, to confirm that even if against a coefficient of ground reaction of 3,000 t/m^3 to be estimated from the geological survey result, and its value reduces to about 2/3, the ground will not be broken, comparative design is performed using k=2,000 t/m^3.

Safety against the ultimate strength is confirmed by using the limit state design method. The ultimate strength is calculated in two cases of single reinforcement and double reinforcement, the results of which are as follows:

In case of single reinforcement:	Mud= 7615.9	tf·m
In case fo double reinforcement:	Mud=14908.9	tf·m
The moment estimated to occur :	M= 7910.0	tf·m

(Confirmation result)

In case of Single reinforcement

M=7910.0 tf·m > Mud=7615.9 tf·m NO

In case of double reinforcement

M=7910.0 tf·m < Mud=14908.9 tf·m OK

Where M: Bending Moment

Mud: Ultimate Strength

In case of single reinforcement, ultimate strength is beyond the safety margin sepcified, and in case of double reinforcement, ultimate strength is below the safety margin specified.

Judging from the above calculation, double reinforcement is necessary for reinforced tunnel lining directly under the Canal.

d. Seismic Design of Tunnel Sturcture

The input earthquake ground motion for seismic design analyses is generally determined based on the magnitude of earthquakes, epicentral distance, and dynamic characteristics of ground, which are assumed that the past records of strong earthquakes occurred in areas adjacent to the structures of the design analyses.

The historical data of strong earthquakes in areas adjacent to the Tunnel have been obtained from the Focus Catalogue of the Meteorological Agency of Japan, as shown in Fig.4.3.16. The data was extracted from earthquakes with a magnitude (M) of 5 or stronger occurred in the selected areas for 91 years from 1900 to 1991.

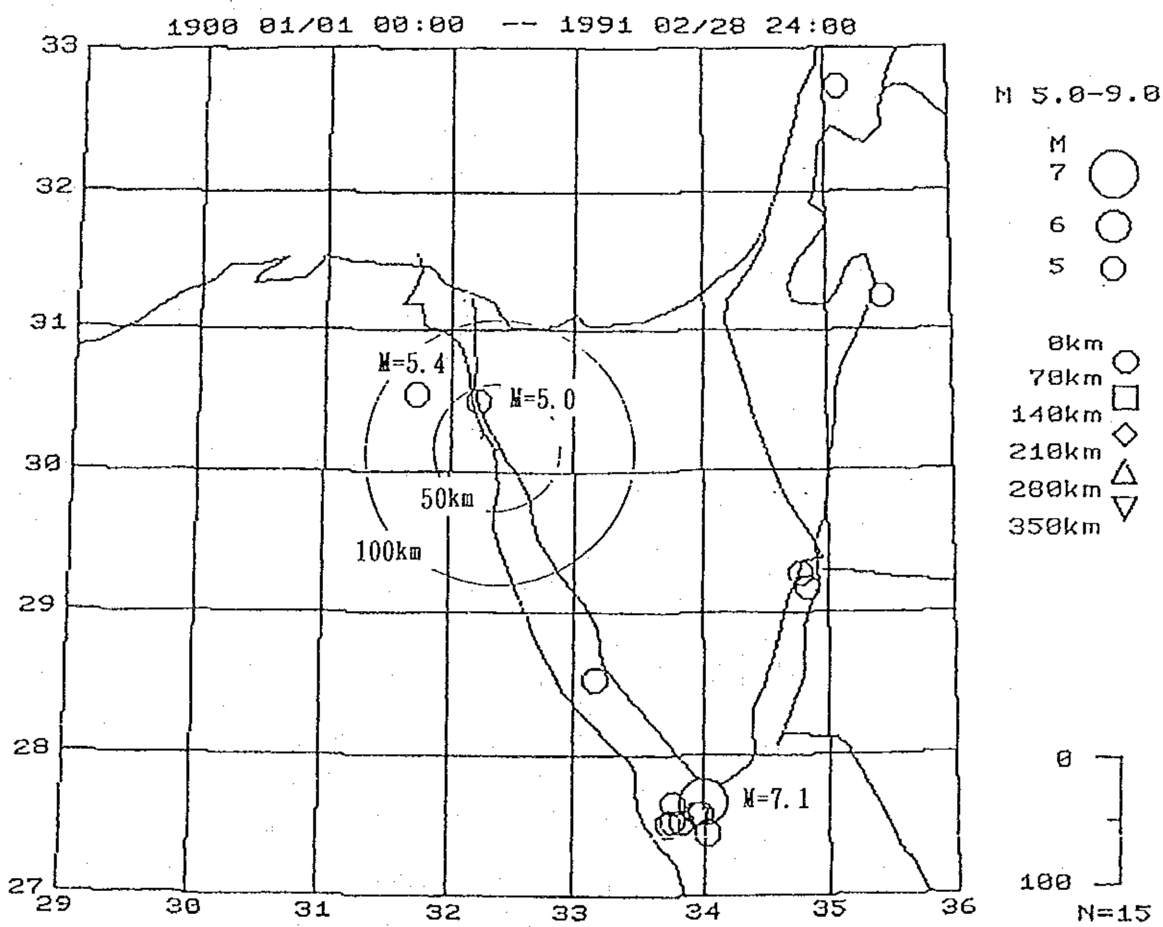


Fig.4.3.16 Past Strong Earthquakes in Areas Adjacent to the Project Site

In this analysis, the following condidtions are decided.

Epicenter : immediately below the structure
Focus depth : 30 km (Effective distance: 30 km)
Magnitude : M = 7.0

The type of earthquake is assumed to be vertical shocks produced from a source directly under the structure.

The peak ground acceleration for the design against earthquake ground motion is normally determined based on the assumed relationship between the magnitude and the epicentral distance. The methods of estimating the peak acceleration on bed rock, are based on the data from strong earthquakes recorded at ports and harbors in Japan, which are shown in Fig.4.3.17.

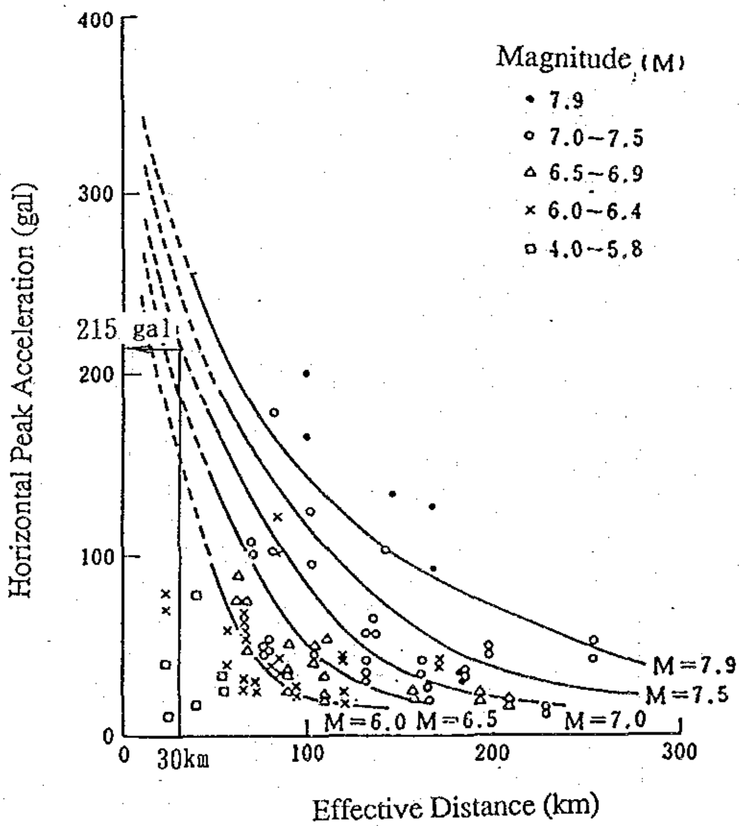


Fig.4.3.17 Distance Damping Curves at Peak Acceleration of Bed Rock

Based on the curves shown in Fig.4.3.17, the peak acceleration (αmax) of foundations during earthquakes of the magnitude assumed above will be:

$\alpha max = 215$

for which M = 7 and effective distance = 30 km

Seismic analyses are to be made by both the dynamic FEM (the Finite Element Method) and the Response Displacement Method as stated below.

d-1. Seismic Analysis by Dynamic FEM

The dynamic FEM program is named "FLUSH" on a three-demensional pseudo model, and the concept of this model is shown in Fig.4.3.18.

According to the results of analyses by FEM, the largest strain value (max) of soil medium surrounding the Tunnel is,

max: 4,400 - 4,500μ

This value is exceeding the yield strain for reinforcing steel bars (ϵsy = 2000μ), and will cause the residual strain. However, this (max) is smaller than the restoring limit strain (ϵsh = 13,000μ), and after the conpletion of earthquake action, the deformed Tunnel will be restored to its original structural situation. So, the safety will be verified.

Furthermore, it can be determined that the Tunnel will be safe by incorporating the following conditions.

- Leakage can be prevented by waterproofing sheets, even if cracks occur on concrete surfaces.

- Since corrosion resistant and waterproofing sheets concrete are to be inserted between the segment rings and lining, isolation effects can be expected. It can be assumed that the relative strains occuring in this analysis will not be conveyed to the lining concrete.

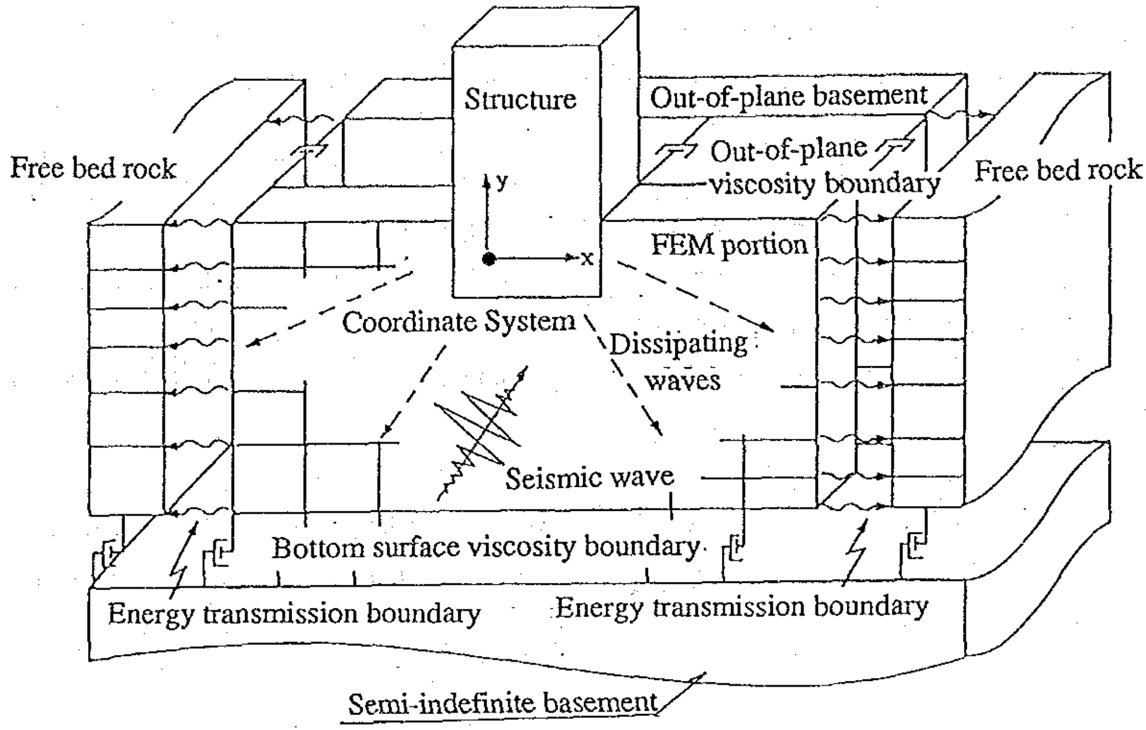


Fig.4.3.18 Distance Damping Curves at Peak Acceleration of Bed rock

d-2. Seismic Analysis by Response Displacement Method

The response displacement method based on the "Design Guidelines for Underground Multi-Purpose Duct" of the Japan Road Association has been applied for this analysis.

The response displacement method is a method of design based on the concept that underground structures, like shielding tunnels, have characteristics displacement determined by the relative displacement or strains of the bed rock in adjacent areas, not by the inertia force of earthquake ground motion.

According to the results of analysis by the response disiplacement method, the safety at maximum ultimate durability will be verified by the critical state design method.

The moment estimated to occur (M) is,

2106.0 tf·m

The maximum ultimate durability (Mud) is,

7615.9 tf·m

Therefore confirmatioin result will be;

$$M = 2106.0 \text{ tf·m} < Mud = 7615.9 \text{ tf·m}$$

e. Design for Effects of Cruising Ships

In addition to the above items a. through d., the effects of cruising ships against the loads both by sinking ship (wreck load) and dropping anchor have been evaluated in the design. Of course, it is definitely prohibited to stop cruising ships and drop anchors in the Canal. It has been concluded that the Tunnel would safely withstand tankers with a dead weight of up to 250 thousand tons.

However, there are still some possibilities which may cause accidents to occur which may destroy the tunnel lining. Therefore, adequate Canal operation and maintenance shall be necessarily carried out.

4) Conclusion on the Tunnel Lining Design

The conclusions on the Tunnel Structural designe are as follows:

PM5

a. The design lining thickness at the section of the lining is to be 400 mm (lining thickness as constructed: 450mm), consisting of D 22 ctc 200 single reinforcement steel bar. (See Fig.4.3.19)
This section's thickness is the maximum lining thickness within the clearance limit.

b. The reinforcement steel bar used along the vertical section of the Tunnel is D 19 ctc 200 single reinforcement. (See Fig.4.3.19)

c. The safety of the lining against soil/water pressure has been confirmed based on the allowable unit stress level.

d. The stability of the Tunnel has been confirmed against the buoyancy after the Canal expansion work.

e. The safety of the lining has been confirmed against the impact of the Canal expansion work.

f. It is confirmed that eqrthquake does not cause any breakage of tunnel lining.

g. Considering how the Tunnel would be affected by cruising ships when they sink or anchored, it has been concluded that the Tunnel would safely withstand tankers of up to 250 thousand tons.
It shouold be noted, however, that the conclustion is based on the hypotheses for designing, leaving the possibility that there are some possible phenomena that have not been evaluated in obtaining the conclusion.

Comprehensive examination of the above conclusions leads us to judge that basically single reinforcement attaining 400 mm of the lining thickness, D 22 ctc 200 for the main reinforcement, and D 19 ctc 200 for the reinforcement along the axis direction could secure the safety of the Tunnel.

However, various assumptions in sect.c. "Design Relative to Influence of the Canal Expansion Work" are involved in the design to cope with the effects of the Canal expansion work and cruising ships in evaluating loads and in analyzing methods.

Therefore, the following measures should be taken to sufficiently secure the safety of the Tunnel lining:

a. To reinforce the Tunnel by applying double reinforcement steel bar (See Fig.4.3.20 and Refer to Table 4.3.2), and

b. To review/confirm the safety of Tunnel by field measurement.

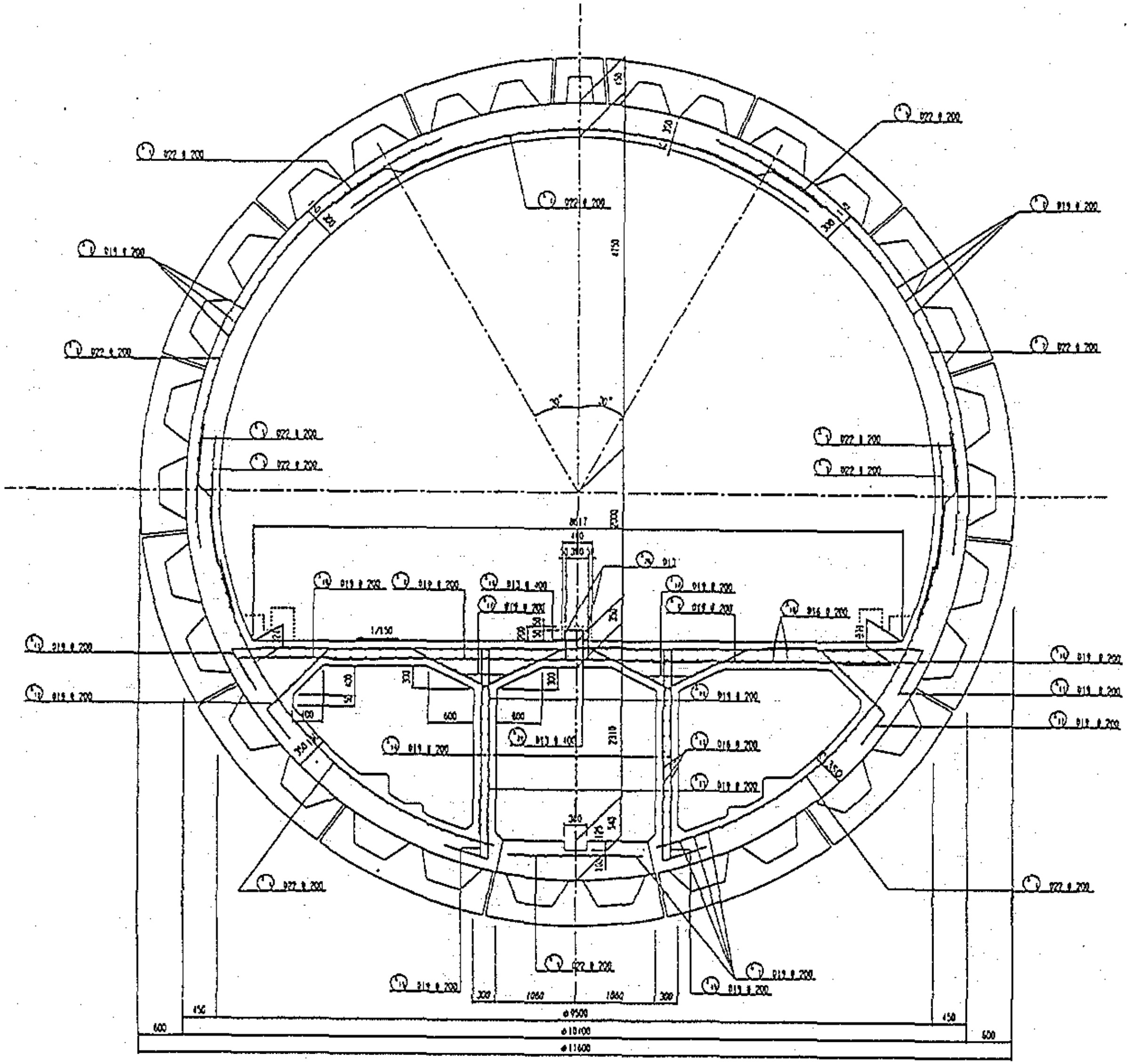
Additionally, the above mentioned safety measurement will provide significant information for examining the future Canal expansion work.

The field measurement of the Tunnel should apply the followings.

a. Displacement of the Tunnel.

b. Strain of the reinforcing steel bar of the segments.

Table 4.3.2 is a summary of our protection plan of the Tunnel's structure against future expansion of the Canal.



LIST OF REINFORCEMENT

MARK	DIA	LENGTH	EACH	[illegible]	[illegible]	WEIGHT	REMARKS
PER ONE SEGMENT (L = 12,500 m)							
A 1	φ22	6 620	52	3.04	20.1	1 045	
2	"	6 820	104	"	20.8	2 163	"
3	"	2 460	104	"	7.48	778	"
4	"	5 580	104	"	17.0	1 768	"
5	"	1 820	208	"	5.53	1 150	
6	"	1 790	52	"	5.44	283	
7	φ19	10 200	180	2.25	23.0	4 140	"
8	"	4 910	104	"	11.0	1 144	"
9	"	3 180	104	"	7.16	745	"
10	"	2 500	104	"	5.63	586	
11	"	1 500	104	"	3.38	352	"
12	"	1 670	104	"	3.76	391	
13	"	2 310	104	"	[illegible]	548	
14	"	2 470	104	"	5.56	578	
15	"	1 460	104	"	3.29	342	
16	"	1 470	104	"	3.31	344	
17	"	1 170	104	"	2.63	274	
18	φ16	10 200	142	1.56	15.9	2 258	
19	φ13	400	54	0.995	0.398	21	
20	"	500	27	"	0.498	13	
					φ22	7 187 kg	
					φ19	9 484 "	
					φ16	2 258 "	
					φ13	34 "	
					TOTAL	18 963 kg	

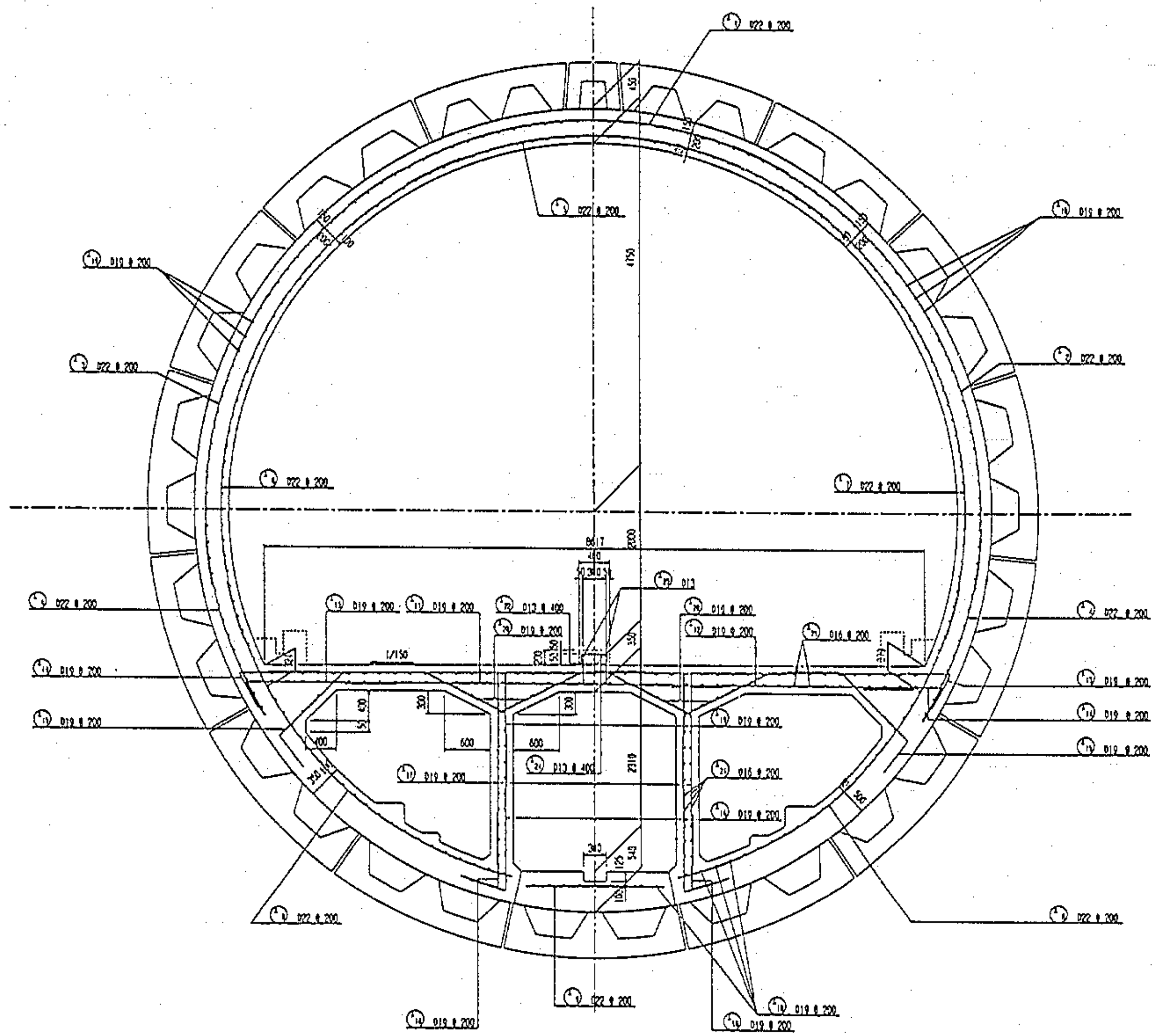
AHAMED HAMDI TUNNEL

GENERAL ARRANGEMENT OF REINFORCING STEEL BAR

Scale		Date		Review	

JICA

Fig.4.3.19 General Arrangement of Reinforcing Steel Bar



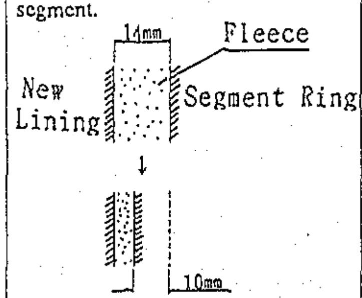
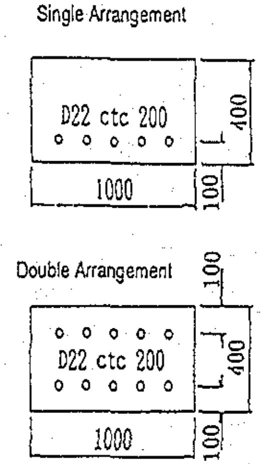
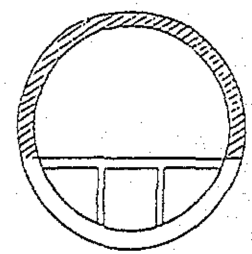
LIST OF REINFORCEMENT

MARK	DIA	LENGTH	EACH	[illegible]	[illegible]	WEIGHT	REMARKS
PER ONE SENTERING (L = 10,500 m)							
A 1	D22	6 060	52	3.04	18.4	957	
2	〃	7 080	52	〃	21.5	1 118	〃
3	〃	6 310	52	〃	19.2	998	〃
4	〃	2 650	104	〃	8.06	838	〃
5	〃	5 850	52	〃	17.8	926	〃
6	〃	7 560	52	〃	23.0	1 196	〃
7	〃	6 780	52	〃	20.6	1 071	〃
8	〃	5 590	104	〃	17.0	1 768	〃
9	〃	1 790	52	〃	5.44	283	
10	D19	10 200	241	2.25	23.0	5 543	〃
11	〃	4 910	104	〃	11.0	1 144	〃
12	〃	3 180	104	〃	7.16	745	〃
13	〃	2 500	104	〃	5.63	586	
14	〃	1 500	104	〃	3.38	352	〃
15	〃	1 670	104	〃	3.76	391	
16	〃	2 510	104	〃	5.65	588	
17	〃	2 470	104	〃	5.56	578	
18	〃	1 460	104	〃	3.29	342	
19	〃	1 470	104	〃	3.31	344	
20	〃	1 170	104	〃	2.63	274	
21	D16	10 200	142	1.56	15.9	2 258	
22	D13	400	54	0.995	0.398	21	
23	〃	500	27	〃	0.498	13	
24	〃	10 200	2	〃	10.1	20	
					D22	9 155 kg	
					D19	10 887 〃	
					D16	2 258 〃	
					D13	54 〃	
					TOTAL	22 354 kg	

AHAMED HAMDI TUNNEL		
ARRANGEMENT OF REINFORCING STEEL BAR UNDER THE CANAL		
Scale	Date	Review
JICA.		

Fig.4.3.20 Arrangement of Reinforcing Steel Bar Under the Canal

Table 4.3.2 Protection Plan of Tunnel Structure from Canal Expansion Project

Alternative Plan	Backfilling	Reinforcing Bedrock by Injection Method	Thicken Non-woven cloth (Fleece)	Increase Reinforcing Steel (double-row arrangement)	Use SFRC	Use GFC
Estimated Achievements	The adhesion between tunnel liners and bedrock will increase and proper subgrade reaction will be secured by backfilling the rear side of existing segments.	Bedrock will be reinforced and can be protected from slips that may be caused by the canal expansion if cracks are all filled up.	The impact on secondary liners from the canal expansion project can be reduced with the displacement capability of non-woven cloth under the conditions that existing segments have bearing force.	Transverse and longitudinal bearing strength will increase with reinforcing bars arranged in double rows. Transverse (M/N = 0.2 m) Single: Mu = 35.44 tf·m Double: Mu = 39.23 tf·m Longitudinal direction: Single: Mu = 7615.88 tf·m Double: Mu = 14,908.90 tf·m	Bending and tensile strengths that are weak points of reinforcing steel can increase, and crack and impact resisting strength can also increase with steel fiber reinforced concrete (SFRC). This will be a very efficient measure for resisting the impacts of the canal expansion project.	The loading capacity of fiberglass reinforced concrete (GFC) is basically similar to that of SFRC. This has advantage in corrosion resistance because of the material used.
Record of Performance and Problem Areas	Backfilling must be performed prior to placing waterproofing sheets. Additional work of sealing and caulking will be required because of the condition of existing joints.	Prior to repair by this method backfilling must be provided. Extreme care is required during performance of work because segment rings can be deformed by excessive pressures that may act on the tunnel during the work.	While the loading capacity of existing tunnel is lowered, segments are required to have a proper bearing capacity. As a result of analyzing the bearing power of existing segments, the relative displacement of segment and secondary liner was 10 mm. Since fleece of 14 mm thickness normally has a transformation capacity of only 7 mm compression rate 50% and may not absorb the displacement of segment. 	There are a number records of performance, but the volume of reinforcement can be limited by the case of construction. (maximum volume will be: D 22 ctc 200). Therefore, cracks cannot be prevented with this method. 	This method has been used for lining ECL in Europe. It has been reported that efficient corrosion protection was provided by this method, but examples of research are very scarce. Liner surface of steel fiber must be coated to prevent corrosion.  Hatched portions where bending moment is larger will be reinforced by SFRC.	Where pumping is required for placing concrete, the delivery of concrete can often be difficult. Record of performance is very scarce.
Construction Schedule	The construction schedule will not be affected.	-	The construction schedule will not be affected.	The construction schedule will not be affected.	The construction schedule will not be affected.	The construction schedule will not be affected.
Estimated Cost (for construction area 650 m)	△	-	○	○	△	×
General Evaluation	△	-	○	○	○	△

4.3.1.2. Road Deck

Road deck will be designed according to the Design Code of NIHON DORO KODAN.

1) Design criteria

The design criteria are shown in Table 4.3.3

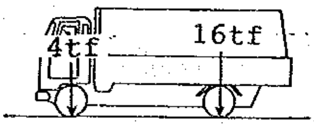
Table 4.3.3 Design Criteria of Road Deck

Items		Value	Remarks
Thickness : t(cm)		35	Since cross sectional gradient is 1.5%, thickness of the central part is 35cm, and that of the edge part is 29cm
Concrete cover for reinforcement : D(cm)		10	
Allow-able stress	Concrete Design strength $\sigma ck(kgf/cm^2)$	270	
	Compressive stress $\sigma ca(kgf/cm^2)$	90	
	Tensile stress $\tau ca(kgf/cm^2)$	9.5	
	Reinforcement Tensile stress $\sigma sa(kgf/cm^2)$	1,400	
Vehicle load *1)	One rare wheel load T-20	8	Total weight : 20tf
	TT-43(trailer)	6.5	Total weight : 43tf

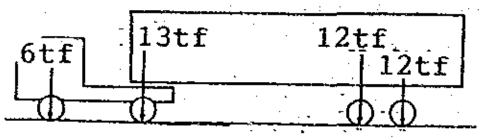
*1)From the load shown in Fig.4.3.21(1), one rear wheel maximum load is obtained as follows.

T-20: 16tf/2=8tf

TT-43: 13tf/2=6.5tf



T-20



TT-43(Trailer)

Fig.4.3.21(1) Wheel Load for road Deck Design

The Vehicle Load Condition in the Egyptian Road Specification in shown in Fig.4.3.21(2)

According to this specification, the one rane wheel maximum load is 5tf(=10tf/2) and this is smaller than the design condition shown in 3)Result and conclusion, the safty should be secured.

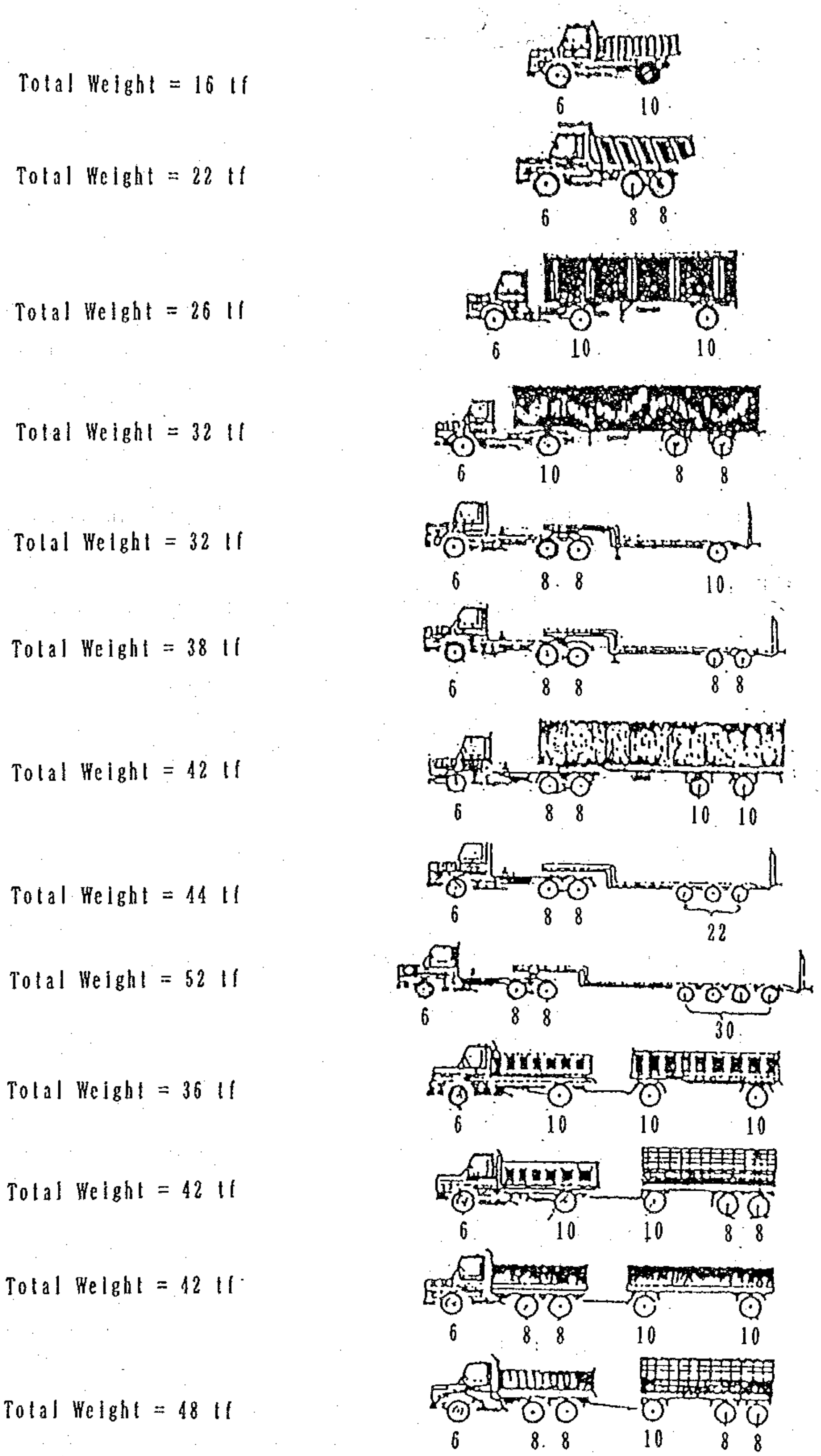


Fig.4.3.21(2) Wheel Load (Two Wheel) by Egyptian Road Specification

2) Calculation formula of sectional force

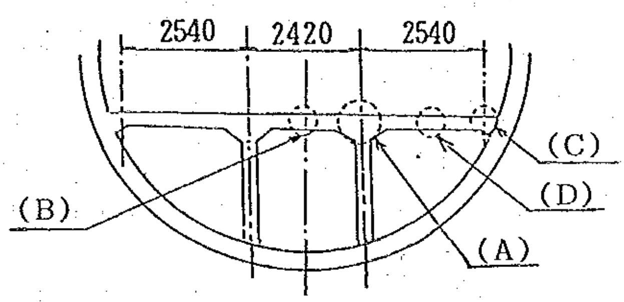
a. Bending moment

The calculation formulas of bending moment, according to the Design Code of NIHON DORO KODAN, are shown in Table 4.3.4.

Table 4.3.4 Calculation Formula of Bending Moment

	Supporting Point (A, C) *1)	Center of Span *2) (B, D) *1)	Remarks
Load of Vehicle	Mmax = $-(0.15\times L+0.125)\times P$	Mmax = $(0.12\times L+0.07)\times 0.8\times P$	L: Span of load deck p: One rear wheel load of vehicle
Dead Load (slab, pavement)	$M = -W\times L^2/10$	$M = -W\times L^2/14$	L: Span of load deck W: Weight of slab and pavement

*1) Each position is shown in the following figure.



*2) The additional load for trailer will be done according to Table 4.3.5.

Table 4.3.5 Coefficient of Additional Rate for Trailer Load

(Center of Span)

ℓ (m)	2.5	2.6	2.7	2.8	2.9	3.0	3.1	3.3	3.5	3.7	3.8	4.0	5.0	6.0
K	1.04	1.05	1.06	1.07	1.075	1.08	1.09	1.10	1.12	1.13	1.14	1.16	1.20	1.25

ℓ : Span of road deck (m)

b. Shearing force

The calculation formula of the punching shear unit stress of road deck against motor vehicle load is shown in the following formula (4.3.1).

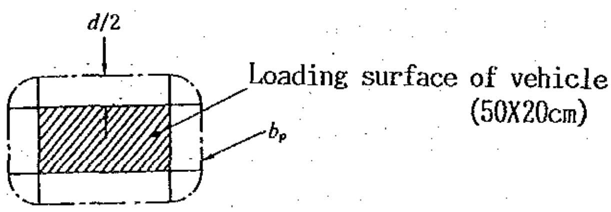
$$\tau_p = \frac{P}{b_p \cdot d} \quad \cdots\cdots \text{formula (4.3.1)}$$

where, τ_p : punching shear stress (kgf/cm^2)

P : load (kgf/cm^2)

b_p : Length of the periphery of the shape, projecting the distribution shape of section into the face that is at the distance of half of the effective height of a member, at an angle of 45 degree,

d : effective height of the section of a member



3) Results and conclusion

a. Bending moment

Checking of the unit stress by bending moment is shown in Table 4.3.6. As a result, sufficient safety is proved by the following values.

$$\sigma c = 24.7 \sim 51.2 \text{ kgf/cm}^2 < \sigma ca = 90 \text{ kgf/cm}^2$$
$$\sigma s = 832 \sim 1242 \text{ kgf/cm}^2 < \sigma ca = 1400 \text{ kgf/cm}^2$$

b. Shearing force

The punching shearing force caused by motor vehicle load is calculated by the formula (4.3.1). As a result, sufficient safety is proved in the following formulas.

$$\tau p = (8.0 \times 1.2 \times 1000) / (218.5 \times 25)$$
$$= 1.76 \text{kgf/cm}^2 < \tau pa = 9.5 \text{ kgf/cm}^2$$

However, the detailed structure of the contact points between road deck and pavement and the connecting method of reinforcement will be performed at the time of detail design stage.

Table 4.3.6 The Calculation Result of Road Deck Sectional Force

Items \ Position		Supporting Point		Center of Span	
		A	C	B	D
Bending Moment M(tf·m)		-5.50	-5.50	3.37	3.46
Width B(cm)		100	100	100	100
Height H(cm)		53	43	35	31
Concrete cover for reinforcement D(cm)		10	10	10	10
Amount of Reinforcement	As	D19ctc200	D19ctc200	D19ctc200	D19ctc200
	As'	D19ctc200	D19ctc200	D19ctc200	D19ctc200
Stress of Concrete $\sigma c(kgf/cm^2)$		24.7	37.9	36.9	51.2
Stress of Reinforcement $\sigma s(kgf/cm^2)$		832.2	1,076.8	1,037.8	1,242.1

4.3.1.3. Drainage system

The drainage of the Tunnel is classified into the following two types:

1) Drainage of inflow water between the segment and the lining

2) Drainage of surface water

The drainage system of the Tunnel is shown in Fig.4.3.22 (cross-sectional view) and Fig.4.3.23 (longitudinal section).

Water proofing sheets and fleece should be efficiently and safely implemented particularly while casting the lining that should not be exposed to breaking, fracture or affecting the aim of its implementation ; as to intercept leaking water containing high percentage of chloride and to prevent any deterioration of the tunnel lining.

1) Drainage system for inflow water

The inflow water between segments and linig is treated by the following system.

a. Water penetrating into the concrete segment is intercepted by waterproofing sheets. Then the water is gathered into the drainage gutter of invert through fleeces installed at the back of the lining.

b. The water is gathered into Nadir sump tank through the drain gutter, having a sufficient capacity for maintenance.

c. Then, the water is pumped out of the Tunnel.

2) Surface drainage system

Water on a road is treated by the following drainage system. (ref. Fig.4.3.22)

a. Water on the road is gathered into both ends of the surface edge due to crossfall.

b. The water flowing at the both sides of a surface edge is gathered to the deepest part of the Tunnel.

c. Then the water, through a drain gutter of invert part, is gathered into Nadir sump tank by drain pipes installed at the deepest part of the Tunnel.

d. Finally, the water is pumped out of the Tunnel.

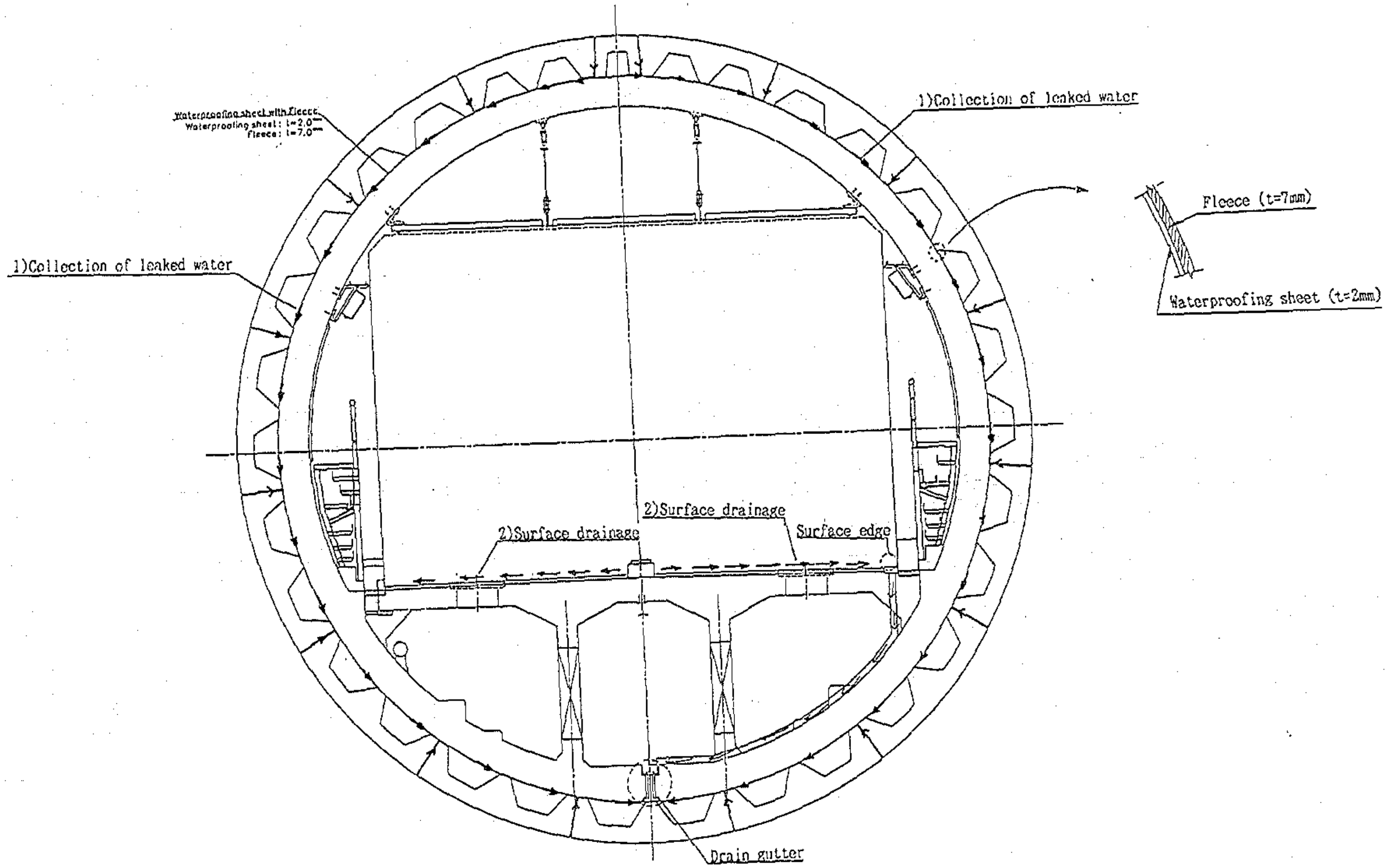


Fig.4.3.22 Cross-sectional of Drainage System

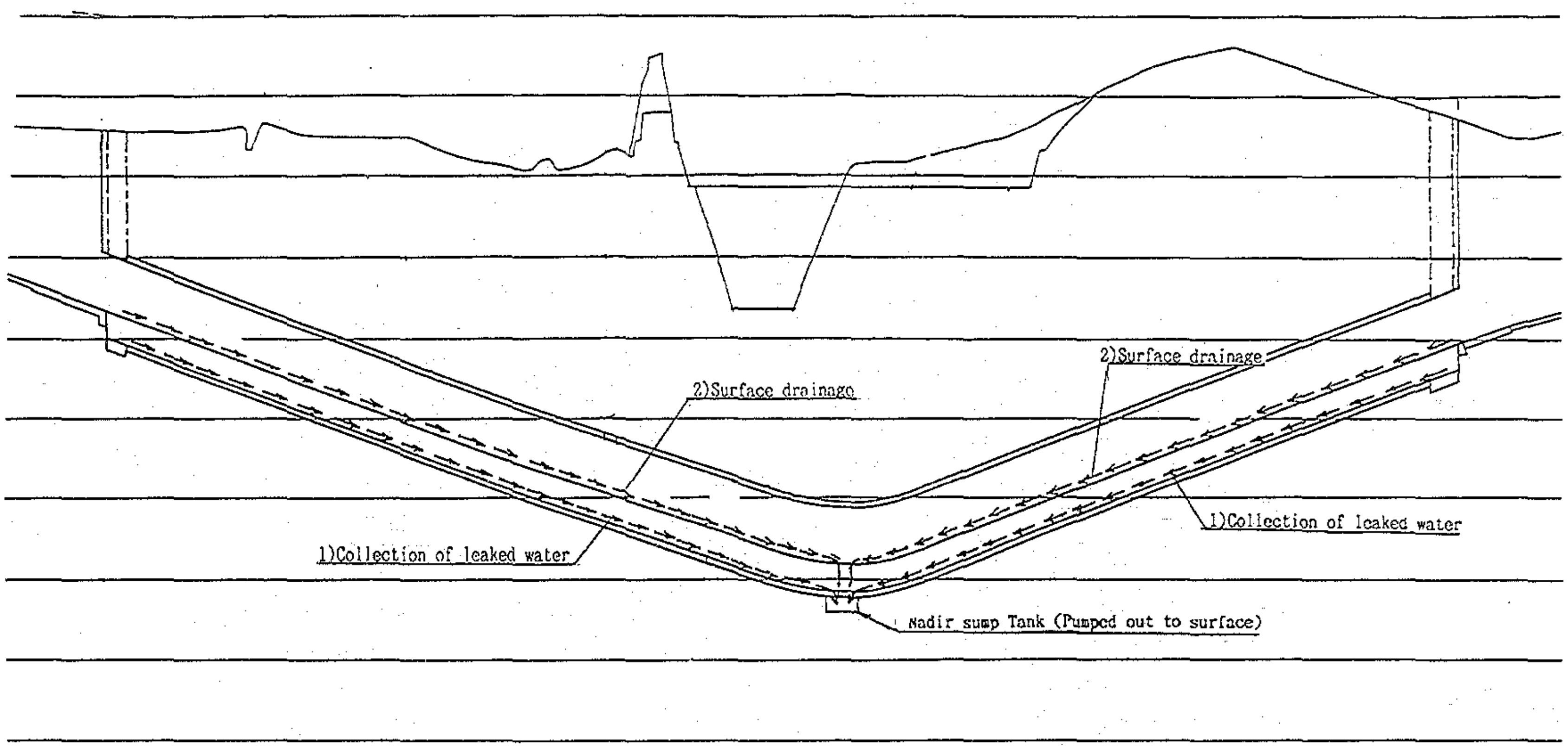


Fig.4.3.23 Longitudinal Section of Drainage System

a. Waterproofing sheet

a-1. Purpose

For the following purpose, waterproofing sheets are installed between the segments and the lining.

- In order to prevent the deterioration of the lining, waterproofing sheets are to intercept leaking water containing chloride.

- At the time of the widening of the Canal, isolating segments by waterproofing sheets will minimize the effect on the Lining.

a-2. Construction material

The construction materials for waterproofing sheets are of various kinds: plastic group, rubberized asphalt group, synthetic rubber group etc. For this Tunnel, ECB (Ethylene Co-polymer Bituminous) seemes to be suitable for the following reason:

- At the time of joining, it has a wide range of melt temperature.

- Single-layer processing is possible (an white thin sheet is coated on the surface of a black sheet). If it is damaged, the black sheet under the white one appears. That is to say, even a small damaged part can be detected by visual inspection.

a-3. Thickness

The thickness will be 2mm according to practices in European countries and Japan.

b. Fleece

b-1. Purpose

From the following purpose, fleece is installed at the back of waterproofing sheet.

- Fleece at the back of waterproofing sheet is to improve the permeability of water, and it leads penetrated water to lower drain gutter speedily.

- At the time of the widening of Suez Canal, it minimizes the effects on the Lining.

b-2. Construction material

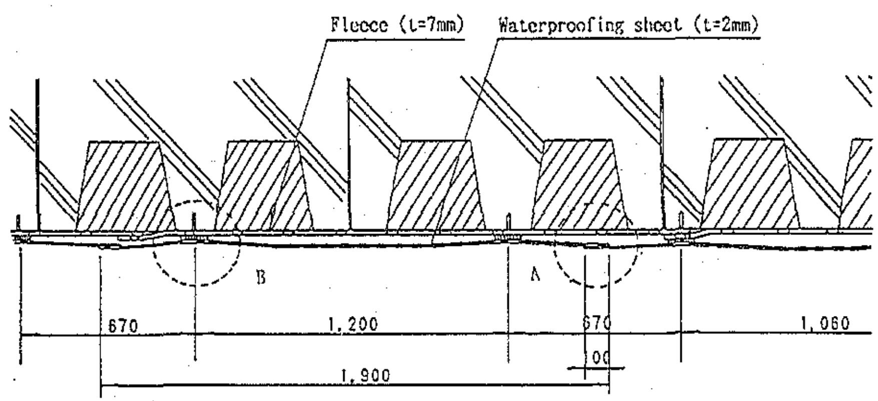
For construction materials, highly alkali-proof polypropylene is used.

b-3. Thickness

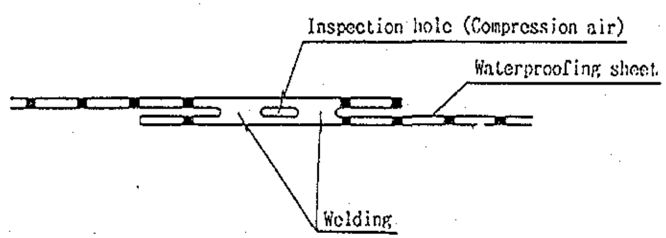
Since it is used also as a cushioning material at the time of the widening of the Canal, the thickness will be 7mm, which is a little thicker than that of general practice or than in a normal situation.

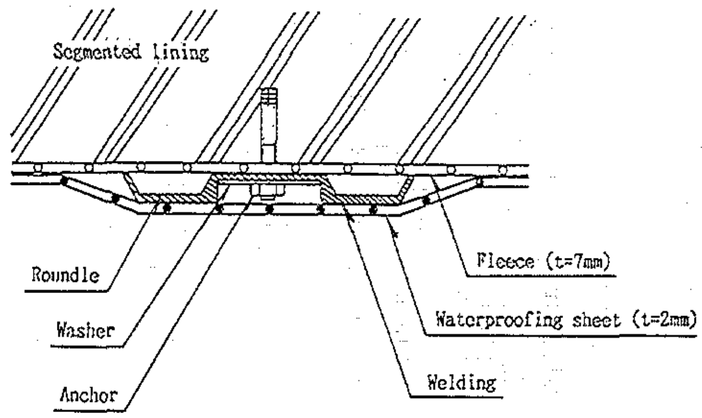
c. Installing method of sheets and fleeces, and connecting method for sheets

The installing method of sheets and fleeces, and connecting method for sheets are shown in Fig.4.3.24.



Detail (A)





Detail (B)

Fig.4.3.24 Application of the Waterproofing Membrance

d. Drain gutter

The detailed drawing of drain gutter is shown in Fig.4.3.25 and Fig.4.3.26.

The gathered water generally flows in the lower drainage gutter.

Both lower and upper drain gutter are connected by drainage pipes installed at the interval of 2m. The pipes are covered so that nothing will get into them.

The size of drain gutter was studied. The results are shown as follows.

Possible drainage capacity Q is shown by the formula (4.3.3) and the formula (4.3.4).

$$Q = A \cdot v \qquad \text{formula (4.3.3)}$$

$$v = 1/n \cdot R^{2/3} \cdot I^{1/3} \qquad \text{formula (4.3.4)}$$

where, Q: quantity of flow
A: sectional area
R: A/S
S: perimeter
n: coefficient of roughness
I: hydraulic gradient

$$v = 1/0.01 \cdot (0.036)^{2/3} \cdot (0.038)^{1/3}$$
$$= 0.14\text{m/sec}$$
$$Q = 0.25 \cdot 0.05 \cdot 3.67$$
$$= 0.0018\ \text{m}^3\text{/sec}$$

If the quantity of leakage of present Tunnel Q'is assumed to be from one side of the Tunnel, the quantity is obtained as follows.

$$Q' = 50\text{m}^3\text{/day/2}$$
$$= 1.04\text{m}^3\text{/hr}$$
$$= 0.0003\text{m}^3\text{/sec} << Q$$

As a result, the size of the drain gutter can well treat the present leaked water.

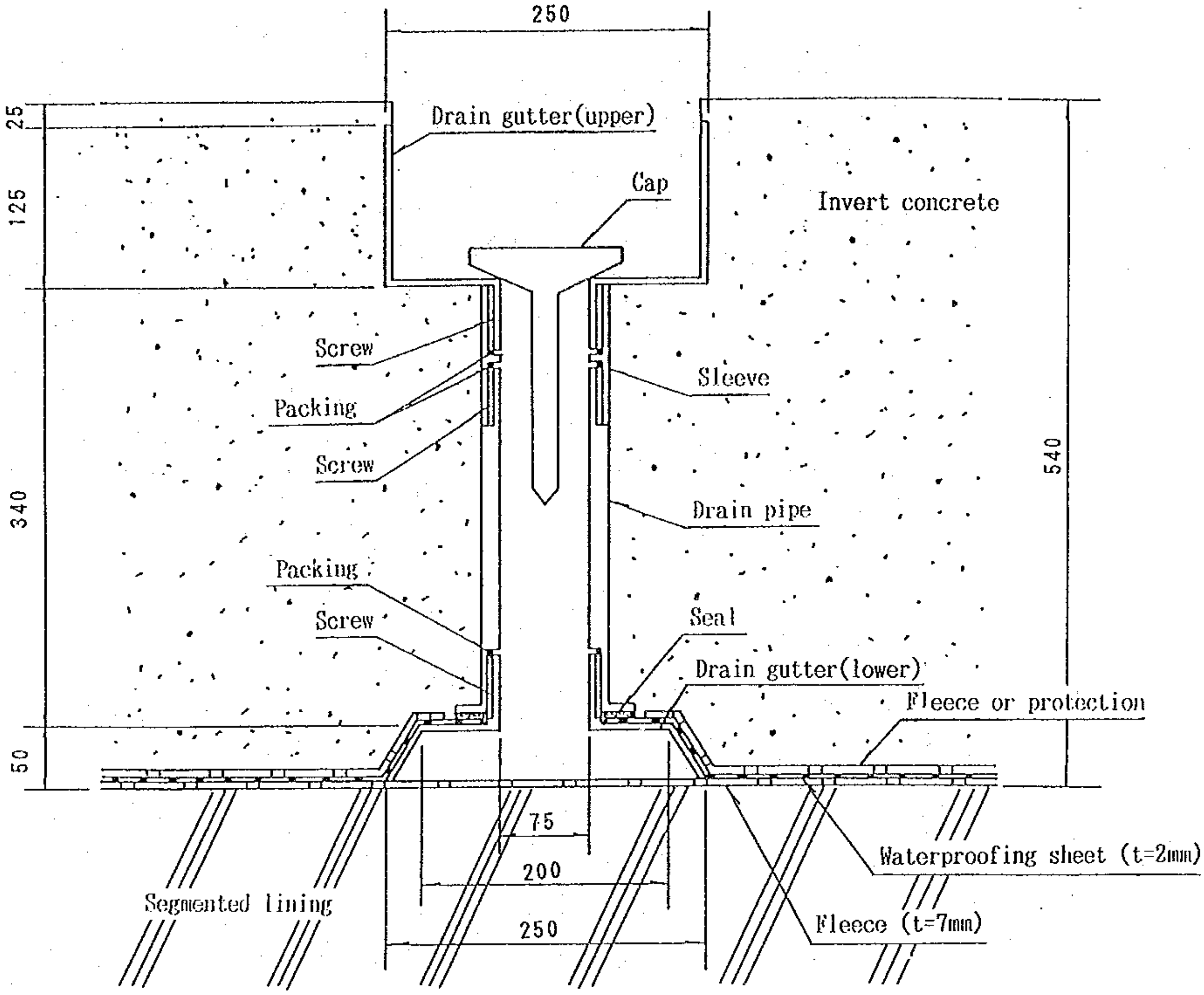


Fig.4.3.25 Detail Drawing of Drainage Gutter

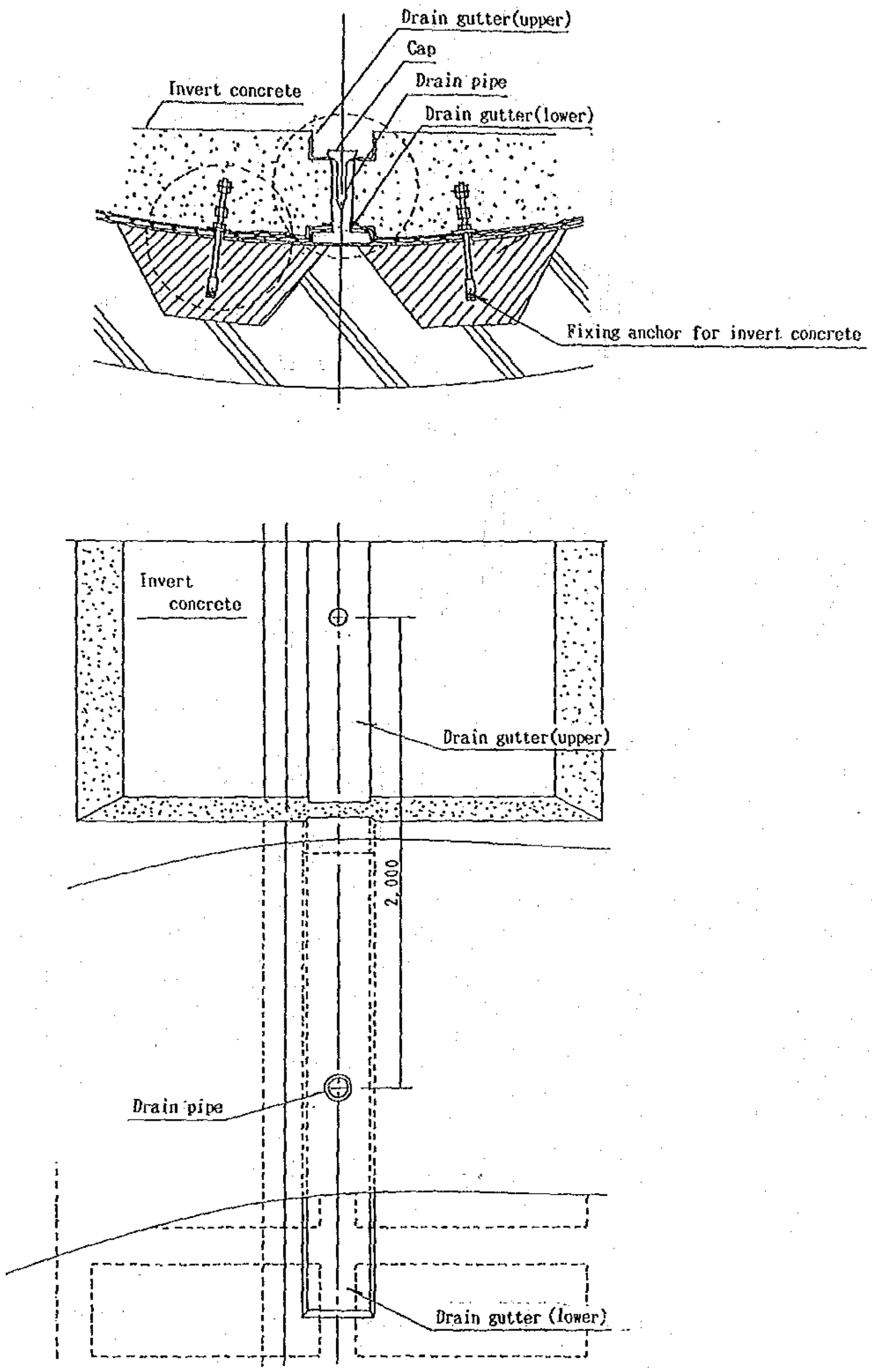


Fig.4.3.26 Structural Drawing of Drainage