Appendix-6

Basic Plan for Tunnel Lining

Appendix-6

Basic Plan for 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. Soil Medium deformation due to the Canal expansion work
e. Seismic Load
f. Influence Load

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

However, as we can see from the comparisons with the bridge field shown in Table 1, there are currently many unsolved problems in the Tunnel field, and the evaluation methods for them have not yet been established.

Accordingly, we have decided to confirm the safety of lining by analyzing using all methods currently available in this basic design.

Flow for Design of Tunnel Lining is shown in Fig.1.

Table 1 Comparison of calculation Models

Source : H.DUDDECK, "THE SAFETY PROBLEMS IN THE DESIGNING OF A TUNNEL STRUCTURE" MEET INT. TUNNELING ASSOC. 3rd. modified by our experience.

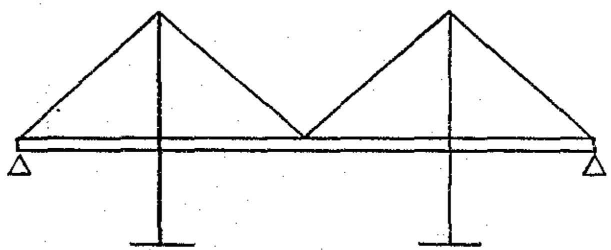
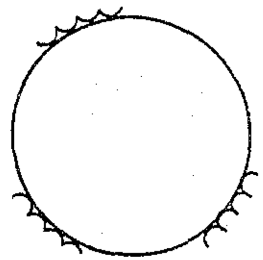
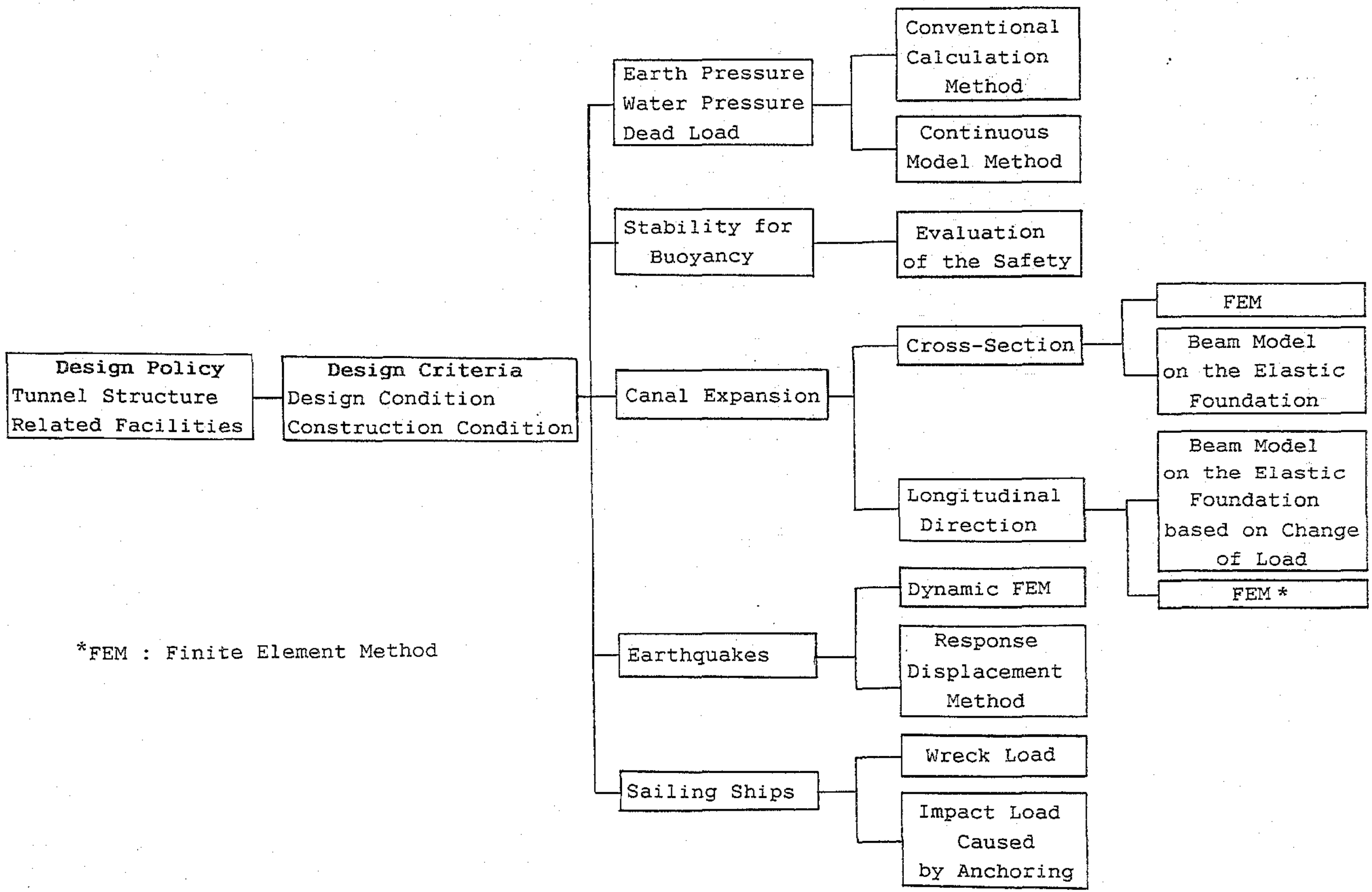
	BRIDGE	TUNNEL
		
Loads:	Specified in codes	Primary stress etc., Probing
Strength:	Given standards	Ground characteristics, Probing
Analysis:	Theory of elasticity or plasticity, fixed in codes	Open for the engineer
Limit states:	Defined	experience
Design criteria:	Prescribed : Mu, Bu etc.	experuence
Safety margin:	Prescribed factors	experience
Verification:	Test of specimens, Experience	In situ measurements (Not sufficient)

Fig. 1 FLOW FOR DESIGN OF TUNNEL LINING



*FEM : Finite Element Method

1. Design relative to earth and water pressures

The structural calculation for tunnel lining is classified into the following two methods concerning the evaluation method for load:

a. Conventional calculation method

Loads will be evaluated on loose earth pressure based on the ultimate equilibrium theory. The structural calculation is performed by applying the loose earth pressure to a dynamical model.

b. Continuous model method

This method calculates with the soil medium and tunnel lining as a compound system, that is, a sectional force occurring in the lining depends upon the soil medium rigidity and lining rigidity.

The method in item "a." is mostly used in Japan, and the method in item "b." is used in European countries. We will use both methods for the designing of lining.

1.1. Conventional calculation method

Lining for earth and water pressures was designed for three cases shown on Table 1.1.1 by varying the overburden and groundwater level and the Canal water level.

Table 1.1.1 Design Contents

CASE	Condition	Height of Overburden	Water Level
1	Max. Overburden	40 m	24 m
2	Under the Canal (at Present)	18 m	37 m
3	Under the Canal (in Future)	7 m	37 m

The objects of the three respective cases are those positions shown in Fig.1.1.1 below.

Further, the respective cases were designed both when the dead load and earth pressure were applied and when the dead load, earth and water pressures were applied.

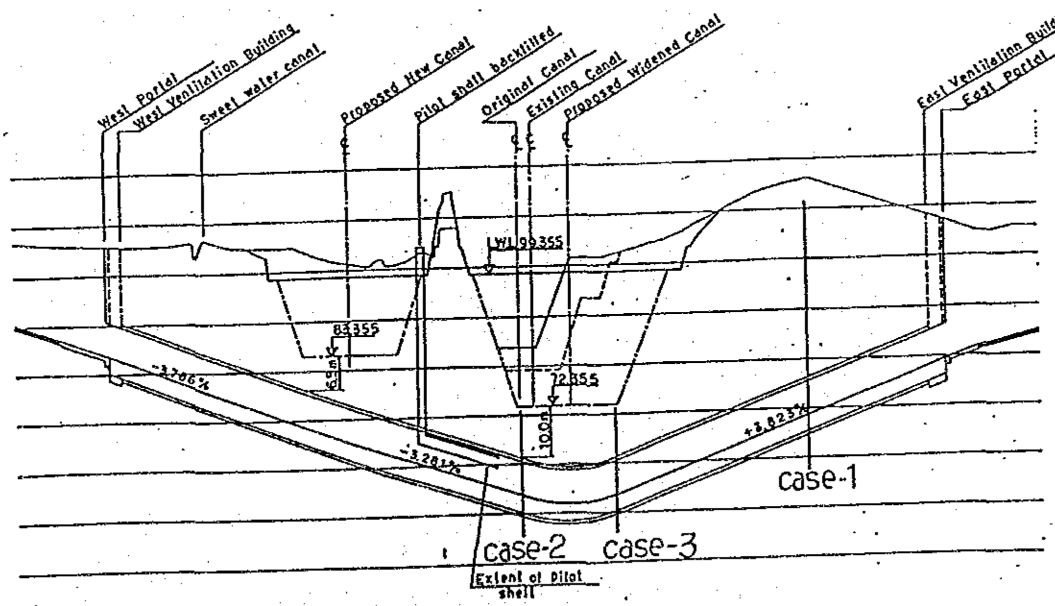


Fig.1.1.1 Longitudinal section of the Tunnel

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.

Concerning safety of the lining, therefore, we confirmed by using the **allowable stress design method** in which no water pressure is applied, and by using the **limit state design method** in which the lining would not be broken even if water pressure should be applied on it.

For the limit state design, the Standard Specification for Design and Construction of Concrete Structures, prepared by Japan Society of Civil Engineers is used.

The conditions in which study was carried out are shown in Tables 1.1.2 and 1.1.3.

The thickness of the lining has been decided as 400mm from viewpoint of structural Basic Design.

To keep this design thickness through the concreting works, the actual planned thickness shall be 450mm. The reason comes from the consideration of the difference between the inside concreting form and the actual roundness/straightness of the Tunnel.

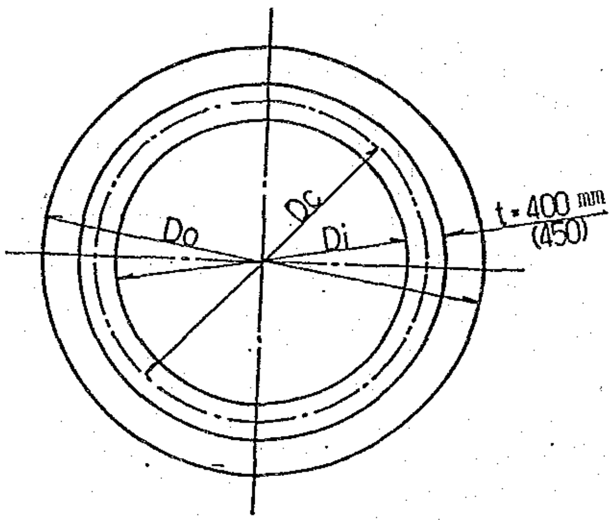


Fig.1.1.2 Cross Sectioin of the Tunnel

The inside radius of the Concreting Form is fixed through out the lining work. However, the actual measured tunnel size is different places by places and the maximum diameter deformation from the theoretical round is recorded as 45mm by the result of measurement as in item 4.1.3 "Tunnel Section Survey".

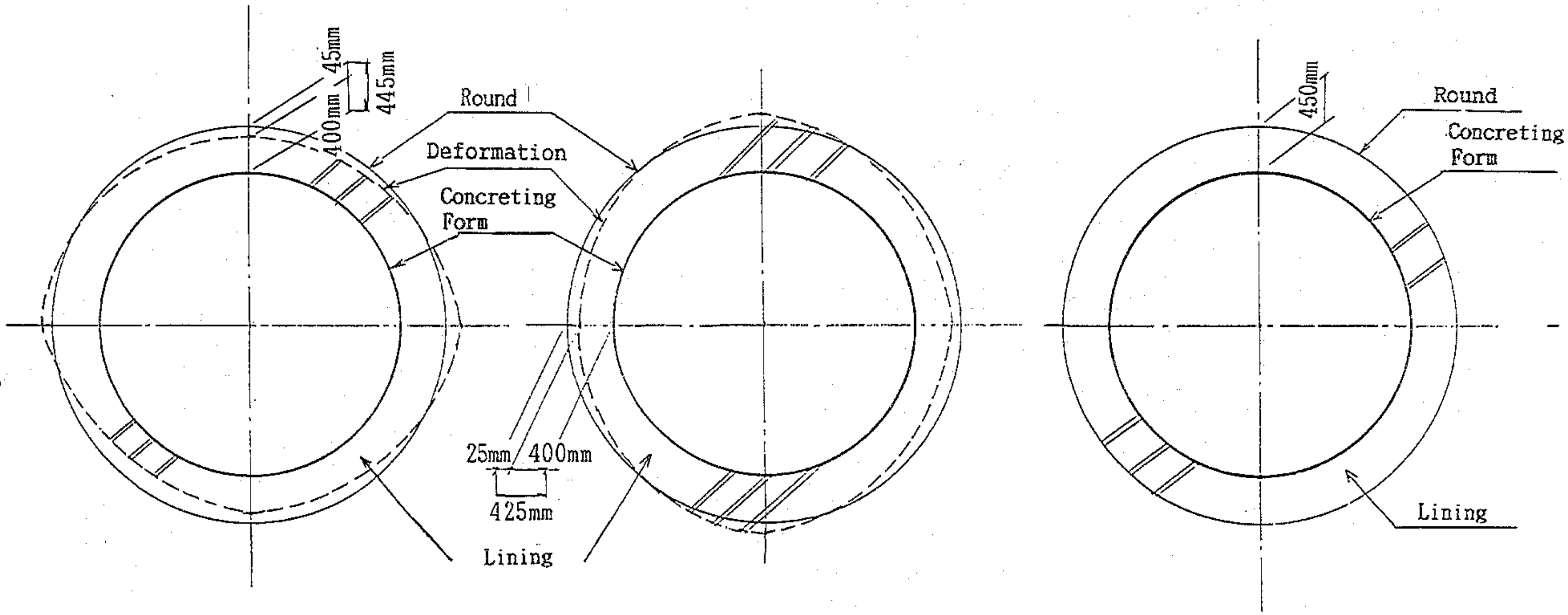
For reinforcement, this specification was adopted because reinforcement steel bar available in Arab Republic of Egypt is SD40 for D19 or more.

Table 1.1.2 Soil Conditions

Item	Condition
Density of Soil	1.95 tf/m^3
Density of Water	1.05 tf/m^3
Density of Soil in Water	0.90 tf/m^3
Coefficient of Lateral Pressure	0.8
Coefficient of Ground Reaction	3,000 tf/m^3
Internal Friction Angle of Soil	15.0 °
Soil Cohesion	19.0 tf/m^2

Table 1.1.3 Structural Condition of Tunnel Lining

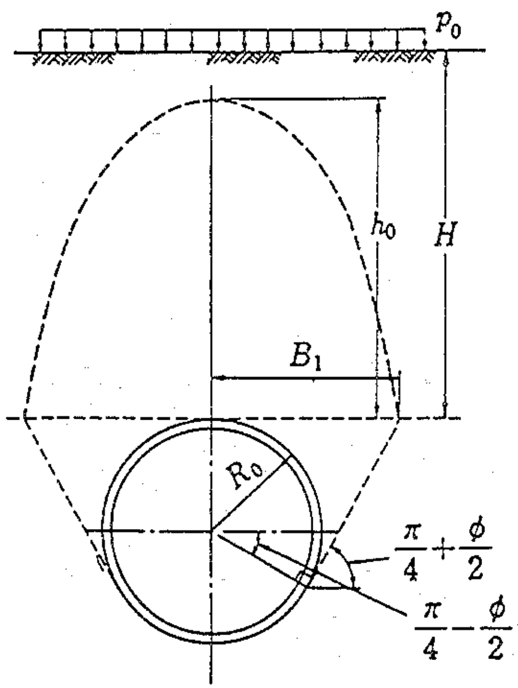
Item	Condition
Tunnel Outside Diameter(segment)	11.60 m
Tunnel Inside Diameter	9.50 m
Centroid Radius	4.95 m
Lining Thickness	0.40 m
Coefficient of Ground Reaction	2,650,000 tf/m^2
Cross Section	0.40 m^2/m
Moment of Inertia	0.005333 m^4/m
Design Strength of Concrete	270.0 kgf/cm^2
Allowable Compressive Strength of Concrete	90.0 kgf/cm^2
Tensile Yielding Strength of Reinforcing Bar	(SD40) 4,000.0 kgf/cm^2
Allowable Tensile Stress of Reinforcing Bar	2,100.0 kgf/cm^2



(a) In case of Horizontally Expanded (b) In case of Vertically Expanded (c) In case of Round

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

Fig.1.1.3 Required Thickness of Lining for construction:



$$h_0=\frac{B_1(1-C/B_1/\gamma)}{K_0\tan\phi}\cdot(1-e^{-K_0\tan\phi\cdot H/B_1})+\frac{p_0}{\gamma}\cdot e^{-K_0\tan\phi\cdot H/B_1}$$

$$B_1=R_0\cdot\cot\left(\frac{\pi/4+\phi/2}{2}\right)$$

h_0 : Loosening height of soil
K_0 : Ratio between horizontal and vertical earth pressure
ϕ : Internal friction angle of soil
p_0 : Surcharge Load
γ : Unit volume weight of soil
c : Cohesion of soil

However, When P_0/γ is small in comparison with H , following equation can be used :

$$h_0=\frac{B_1(1-C/B_1/\gamma)}{K_0\cdot\tan\phi}\cdot(1-e^{-K_0\tan\phi\cdot H_1/B_1})$$

where, H_1 : reduced overburden

$$\left(H+\frac{p_0}{\gamma}\right)\text{(m)}$$

Fig.1.1.4 The Loosening Height of Soil by Terzaghi

1.1.1. Calculation of the Load

Regarding the evaluation method for load, the load was calculated on the basis of the definition by Terzaghi's loosening height of soil as shown in Fig.1.1.4., which has been generally used in the conventional calculation method of lining.

While calculating the loosening height under the Canal, the canal water pressure was treated as a placed load.

The loosening height calculated for each case and the results for the vertical load and horizontal load calculated on the basis thereof are shown in Table 1.1.4.

While calculating the earth pressure in Case 3, it was assumed that the weight of water existing on the Canal bed was applied to the tunnel as a placed load in addition to the full earth covering thickness (7m) because the loosening height exceeded the full overburden height.

Table 1.1.4 Result of Calculated Load

		Case1	Case2	Case3
Overburden	(m)	40.0	18.0	7.0
Ground Water Level	(m)	24.0	37.0	37.0
Height of Loose Zone	(m)	4.5	9.4	7.0
Vertical Earth Pressure	(tf/m2)	4.05	8.46	37.80
Horizontal Earth Pressure	(tf/m2)			
at Tunnel Crown		3.24	6.77	30.24
at Tunnel Bottom		12.88	16.80	38.59
Water Pressure	(tf/m2)			
at Tunnel Crown		25.20	38.85	38.85
at Tunnel Bottom		37.38	51.03	51.03

1.1.2. Calculation of the Sectional Force

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.1.1.5. 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.

The ground reaction was ensured to be applied only to a portion of the lining which has been deformed outside (ground side) from the initial state. Assuming that no shearing force transmits because waterproofing sheets and fleece exist around the lining, the ground reaction was ensured to be applied only in the ring normal line direction.

Also, judging that the lining will receive sufficient reaction from the surrounding ground through the segment rings during construction, the ground reaction was also taken into consideration against the dead load of lining.

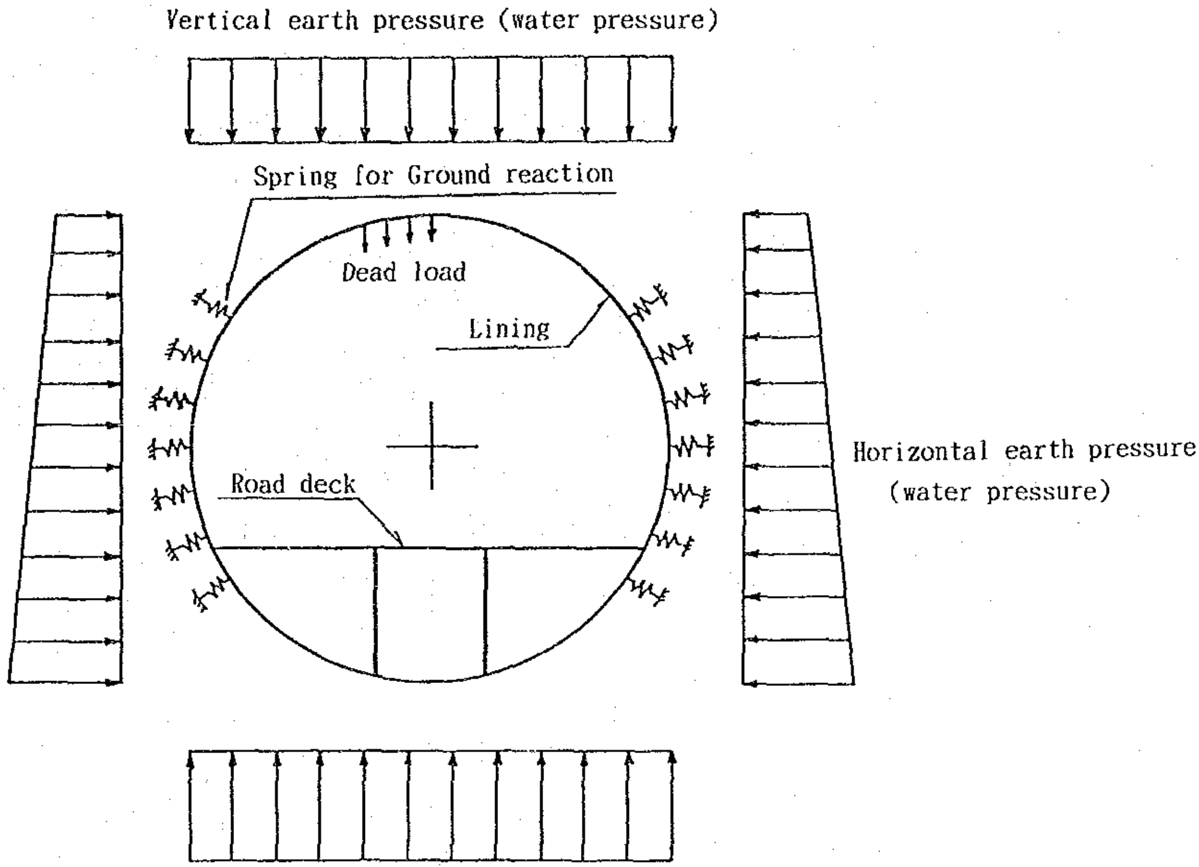


Fig.1.1.5 Analytical Model of Lining for the Earth Pressure and Water Pressure

The calculation result of the sectional force is shown in Table 1.1.5., in which the dead load of the new lining is taken into consideration.

Table 1.1.5 Calculation Result of Cross Sectional Force

Types of Load		Maximum Bending Moment tf·m		Axial Force tf	
		$k=3000tf/m^3$ = 0.8	$k=5000tf/m^3$ = 0.5 *1	$k=3000tf/m^3$ = 0.8	$k=5000tf/m^3$ = 0.5 *1
CASE-1	Earth Pressure	2.280	2.288	27.960	20.869
	Earth Pressure + Water Pressure	10.890	6.738	156.396	153.150
CASE-2	Earth Pressure	1.859	5.253	50.773	37.915
	Earth Pressure + Water Pressure	9.994	3.938	242.329	240.436
CASE-3	Earth Pressure	11.180	24.890	174.677	151.454
	Earth Pressure + Water Pressure	10.686	4.178	232.422	228.479

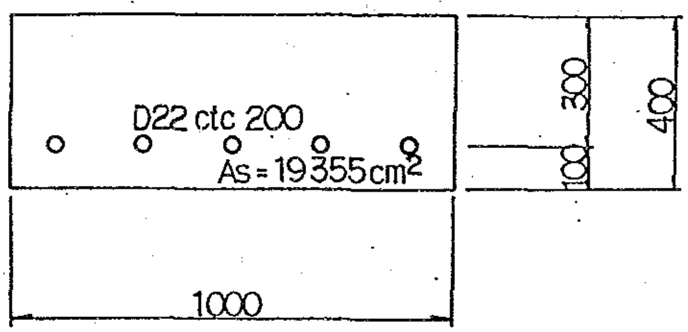
*1 According to the past record in Japan, K = 5,000 tf/m^3 and K = 0.5 are considered, and therefore these were fixed values.

1.1.3. Evaluation of Safety

Out of the calculation results for the sectional force, the stress intensity was verified by using the allowable stress design method both in a stationary state of loading, in which only the earth pressure is applied, and in a non-stationary* state of loading in which earth and water pressures are applied.

The stress intensity for a section shown in the following figure was calculated.

Note for * : in case of the Drain System being not effective



The calculation result for stress intensity is shown in Table 1.1.6.

Table 1.1.6 <u>The Result of Compressive and Tensile Stress for Lining</u>

		Earth Pressure		Earth Pressure + Water Pressure	
		σc	σs	σc	σs
CASE-1	k=3000tf/m3, =0.8	15.6	—	80.5	—
	k=5000tf/m3, =0.5	15.1	16.0	64.3	—
CASE-2	k=3000tf/m3, =0.8	20.0	—	99.4	—
	k=5000tf/m3, =0.5	34.8	146.7	76.6	—
CASE-3	k=3000tf/m3, =0.8	86.3	—	99.4	—
	k=5000tf/m3, =0.5	167.5	1129.1	74.4	—

Regarding the calculation results of the sectional force when earth and water pressures were applied and when k=5,000m^3 and tf/m^3=0.5 were used, it is a very unusual case that water pressure is applied to the Tunnel lining, and k=5,000tf/m^3 and K=0.5 are not basic values in this design. Therefore, safety was confirmed by the ultimate capacity of lining as shown in Fig.1.1.6.

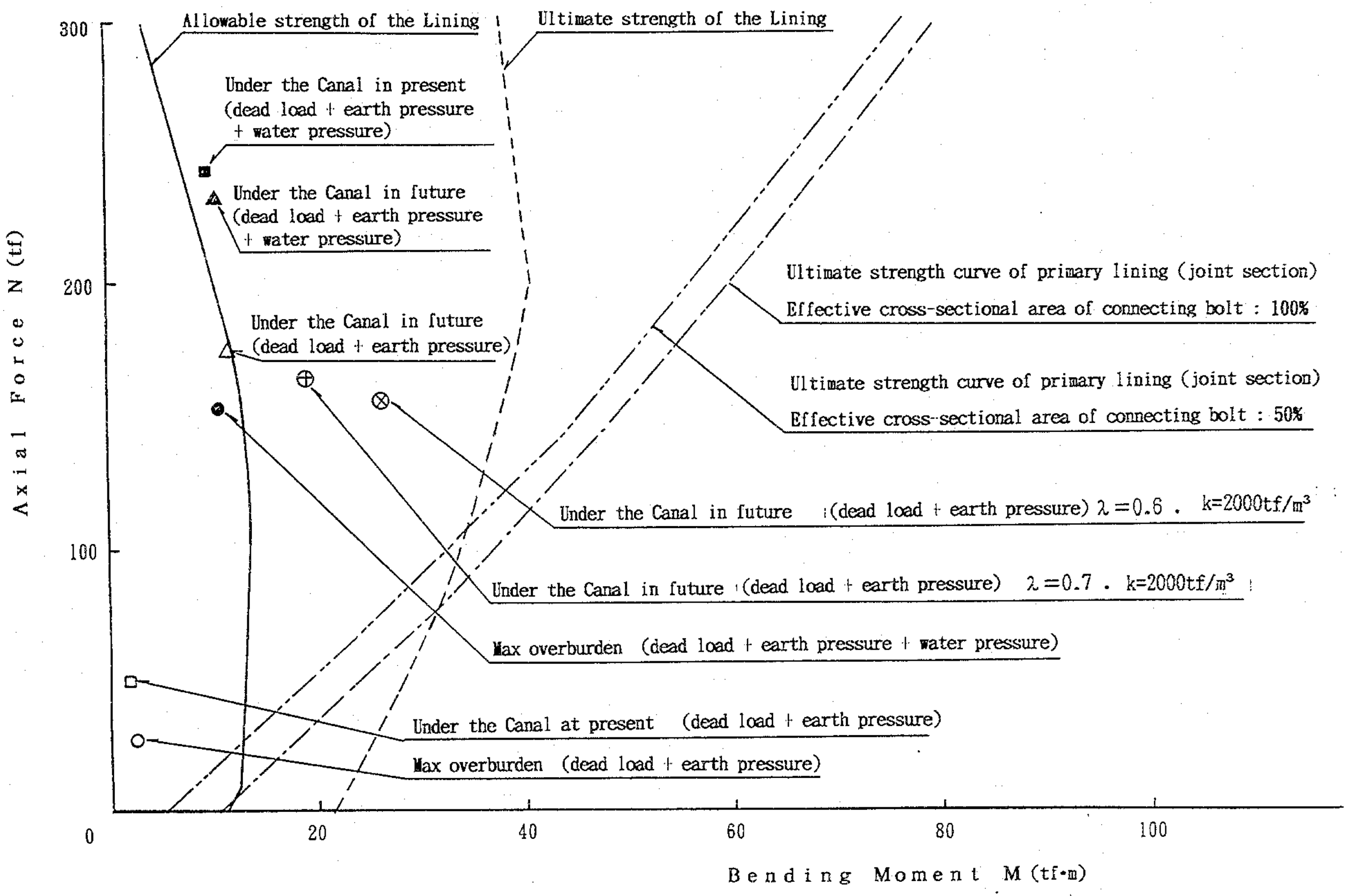


Fig 1.1.6 Allowable Strength, Ultimate Strength and Sectional Force of Lining

160

Table 1.1.7 Result of Stress Intensity Calculation (dead load+earth pressure)

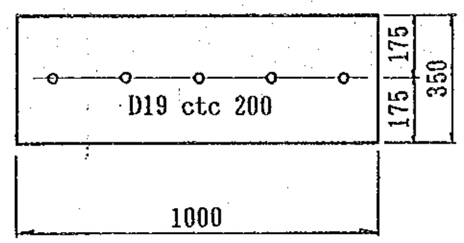
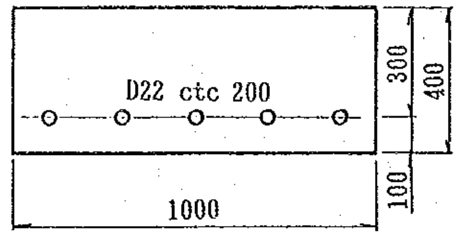
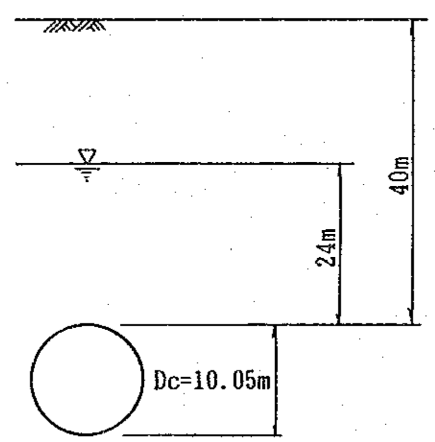
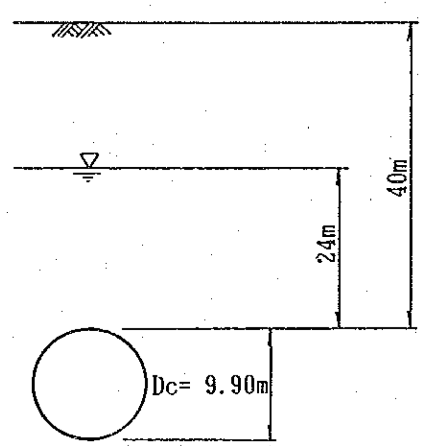
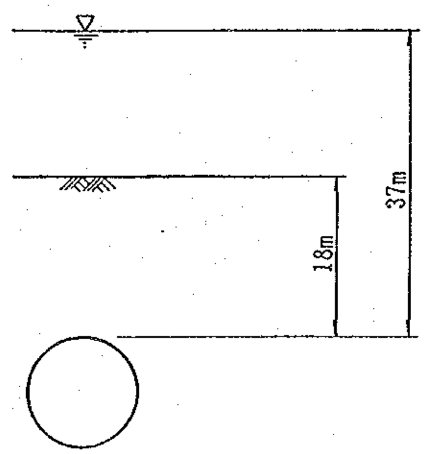
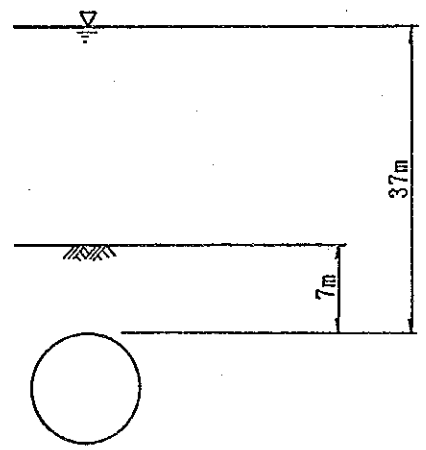
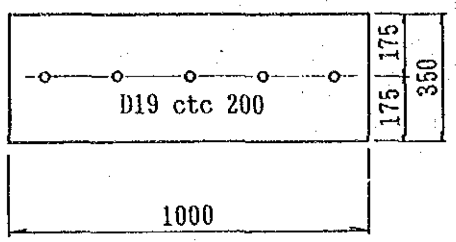
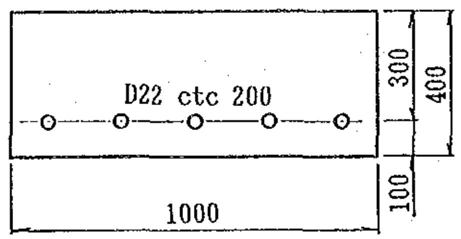
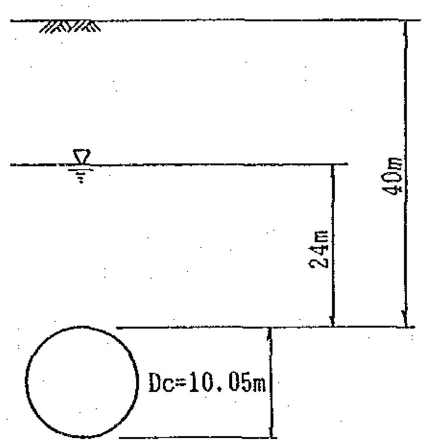
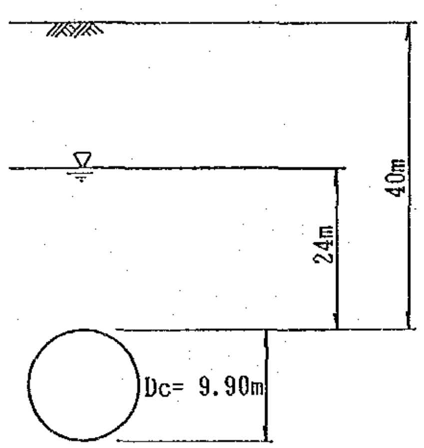
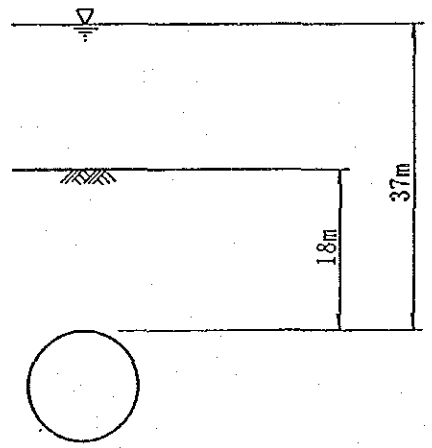
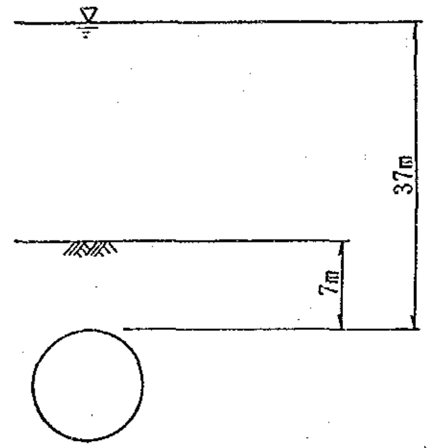
	Task Team Report May, 1990	Basic Design					
	Max overburden	Max overburden		Under the Canal at present		Under the Canal in future	
Cross-section of Lining							
Overburden and Ground-water level							
Cohesion c (tf/m^2)	c' = 2.74	cu= 19		cu= 19		cu= 19	
Internal friction angle ϕ (deg.)	ϕ' = 27	ϕu= 15		ϕu= 15		ϕu= 15	
Height of loosening ho (m)	15.2	4.5		9.4		7.0	
Coefficient of ground reaction K (tf/m^3)	2000	3000	5000	3000	5000	3000	5000
Coefficient of lateral earth pressure λ	0.7	0.8	0.5	0.8	0.5	0.8	0.5
Mmax (tf·m)	7.500	2.280	2.288	1.859	5.253	11.180	24.890
N (tf)	73.000	27.960	20.869	50.773	37.915	174.677	151.454
σc (kgf/cm^2)	56.4	15.6	15.1	20.0	34.8	86.3	167.5
σs (kgf/cm^2)	——	——	16.0	——	146.7	——	1129.1

Table 1.1.8 Result of Stress Intensity Calculation dead load+earth perssure+water pressure

	Task Team Report May, 1990	Basic Design					
	Max overburden	Max overburden		Under the Canal at present		Under the Canal in future	
Cross section of Lining							
Overburden and Ground water level							
Cohesion c (tf/m²)	c' = 2.74	cu= 19		cu= 19		cu= 19	
Internal friction angle ϕ (deg.)	ϕ' = 27	ϕu= 15		ϕu= 15		ϕu= 15	
Height of loosening ho (m)	15.2	4.5		9.4		7.0	
Coefficient of ground reaction K (tf/m³)	2000	3000	5000	3000	5000	3000	5000
Coefficient of lateral earth pressure λ	0.7	0.8	0.5	0.8	0.5	0.8	0.5
Mmax (tf·m)	3.700	10.890	6.738	9.994	3.938	10.686	4.178
N (tf)	200.000	156.396	153.150	242.329	240.436	232.422	228.479
σc (kgf/cm²)	72.0	80.5	64.3	99.4	76.6	99.4	74.4
σs (kgf/cm²)	―	―	―	―	―	―	―

1.2. Continuous Model Method

An analytical method using a continuous model was advocated by Schmid H. in 1926, and numerous proposals have been made so far. Since, however, there are not much differences among methods as long as a flexible lining is concerned, we have decided to use for our design the proposal by Einstein H, et al as shown below.

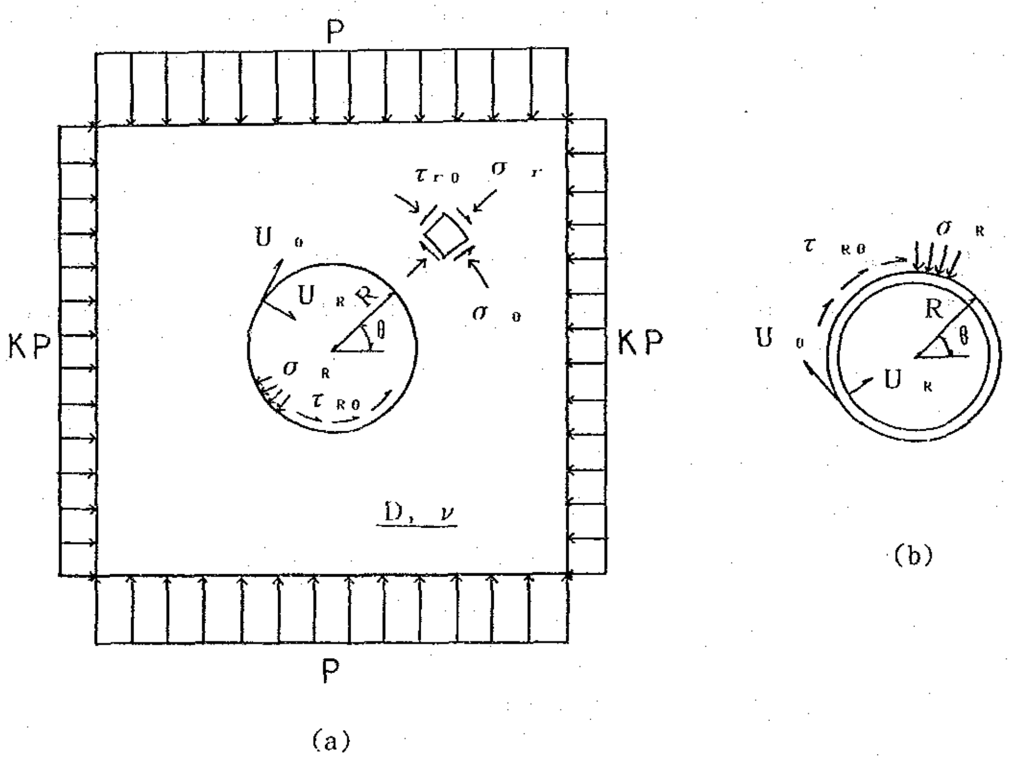
* Since the rigidity ratio of the ground to the lining is as great as 100 times, it can be regarded as a flexible lining.

$$\alpha = EgR^3/EcJ \div 100$$

Where, E : Elastic modulus of ground
Ec: Elastic modulus of lining
R : Radius of tunnel lining
J : Inertia of lining

This model is used to determine displacement in the Tunnel Lining surface and sectional force produced in the Lining.

In this model, it is assumed that P(= vertical load) and Kp (= horizontal load) are working to soil medium after excavation and lining of tunnel as shown in Fig.1.2.1.



Herbert H. Einstein, M. ASCE and Charles W. Schwartz, A. M. ASCE
「Simplified Anaysis for Tunnel Supports」
Journal of the Geotehnical Engineering Division, April 1979, pp. 499～518.

Fig.1.2.1 Analytical Model Based on the Theory of Elasticity

The calculation formulas are as follows :

The compressibility ratio, C^*, is defined as

$$C^* = \frac{E\ R^3\ (1-\nu_s^2)}{E_s A_s\ (1-\nu^2)}$$

The flexibility ratio, F^*, is similarly defined as

$$F^* = \frac{E\ R^3\ (1-\nu_s^2)}{E_s I_s\ (1-\nu^2)}$$

The lining displacements (in dimensionless form):

$$\frac{u_s E}{PR\ (1+\nu)} = \frac{1}{2}\ (1+K)\ a_0^* - (1-K)\ [\,(5-6\nu)\ a_2^* - (1-\nu)\,]\cos 2\theta$$

$$\frac{v_s E}{PR\ (1+\nu)} = \frac{1}{2}\ (1-K)\ [\,(5-6\nu)\ a_2^* - (1-\nu)\,]\cos 2\theta$$

The lining sectional forces (in dimensionless form):

$$\frac{N}{PR} = \frac{1}{2}\ (1+K)\ (1-a_0^*) + \frac{1}{2}\ (1-K)\ (1-2a_2^*)\cos 2\theta$$

$$\frac{M}{PR^2} = \frac{1}{2}\ (1-K)\ (1-2a_2^*)\cos 2\theta$$

in which

$$a_0^* = \frac{C^* F^*\ (1-\nu)}{C^* + F^* + C^* F^*\ (1-\nu)}$$

$$a_2^* = \frac{(F^*+6)\ (1-\nu)}{2F^*\ (1-\nu) + 6\ (5-6\nu)}$$

1.2.1. Calculation of the Load

For the design condition, the same condition as in Case 3 with the greatest sectional force was used in the conventional calculation method.

The load was calculated by using the following procedure.

Vertical load : P

$$P_e = 12.8\,\text{m} \times (1.95 - 1.05)\,\text{t/m}^3 = 11.52\,\text{t/m}^2$$

$$P_w = 42.8\,\text{m} \times 1.05\,\text{t/m}^3 = 44.94\,\text{t/m}^2$$

$$\therefore\; P = P_e + P_w = 11.52 + 44.94 = 56.46\,\text{t/m}^2$$

Lateral pressur: K

Here, calculation is done using apparent pressure cofficient K'

Coefficient of earth pressure λ : $\lambda = 0.8$

$$\therefore\; \lambda P_e = 0.8 \times 11.52\,\text{t/m}^2 = 9.216\,\text{t/m}^2$$

Coefficient of water pressure λ_w : $\lambda_w = 1.0$

$$\therefore\; \lambda_w P_w = 1.0 \times 44.94\,\text{t/m}^2 = 44.94\,\text{t/m}^2$$

$$KP = \lambda P_e + \lambda_w P_w = 9.216 + 44.94 = 54.156\,\text{t/m}^2$$

$$K' = KP / P = 54.156/56.46 = 0.96$$

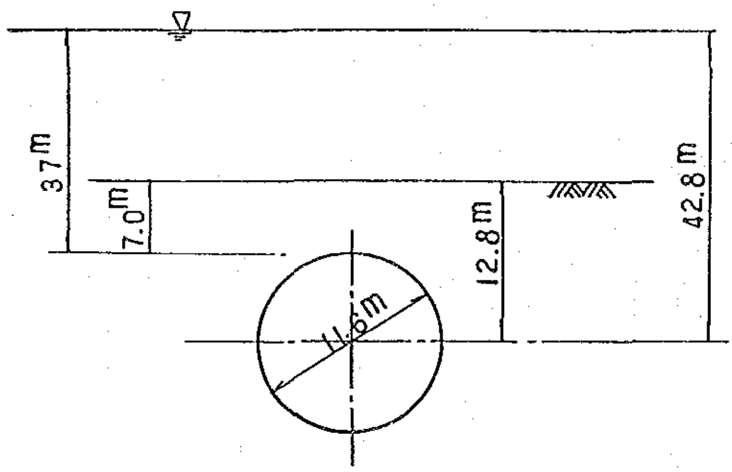


Fig.1.2.2 Tunnel Section for Calculation

1.2.2. Calculation of the Sectional Force

The input constant used to calculate the sectional force is shown in Table 1.2.1.

The calculation results for the sectional force is shown in Table 1.2.2.

Table 1.2.1 Conditions for Sectional Force Calculation

Item	Conditions
Elastic Coefficient of Ground	15,000.0 tf/m^3
Elastic Coefficient of Lining	3,500,000.0 tf/m^3
Poisson's Ratio of Ground	0.45
Poisson's Ratio of Lining	0.20
Radius of Lining	4.95 m
Cross Sectional Area of Lining	0.40 m^2
Moment of Inertia of Lining	0.00533 m^4
Coefficient of Lateral Pressure	0.96/0.50
Vertical Load	56.46 tf/m^2

Table 1.2.2 Calculation Results of Cross Sectional Force
(Continuous Model)

Angle	Bending Moment (tf·m)		Axial Force (tf)		Remarks
	K = 0.96	K = 0.50	K = 0.96	K = 0.50	
0	1.08	13.51	261.97	203.05	
10	1.02	12.70	261.96	202.89	
20	0.83	10.35	261.92	202.41	
30	0.54	6.76	261.86	201.69	
40	0.19	2.35	261.79	200.80	
50	-0.19	-2.35	261.72	199.85	
60	-0.54	-6.76	261.65	198.96	
70	-0.83	-10.35	261.59	198.23	
80	-1.02	-12.70	261.55	197.76	
90	-1.08	-13.51	261.54	197.59	

1.2.3. Evaluation of Safety

The evaluation of safety was confirmed by the limit state design method as shown in Fig.1.2.3. For the verified sectional force position, the angle in Table 1.2.2 was assumed to be 0 degree. As a result, it could be confirmed that it is sufficiently safe.

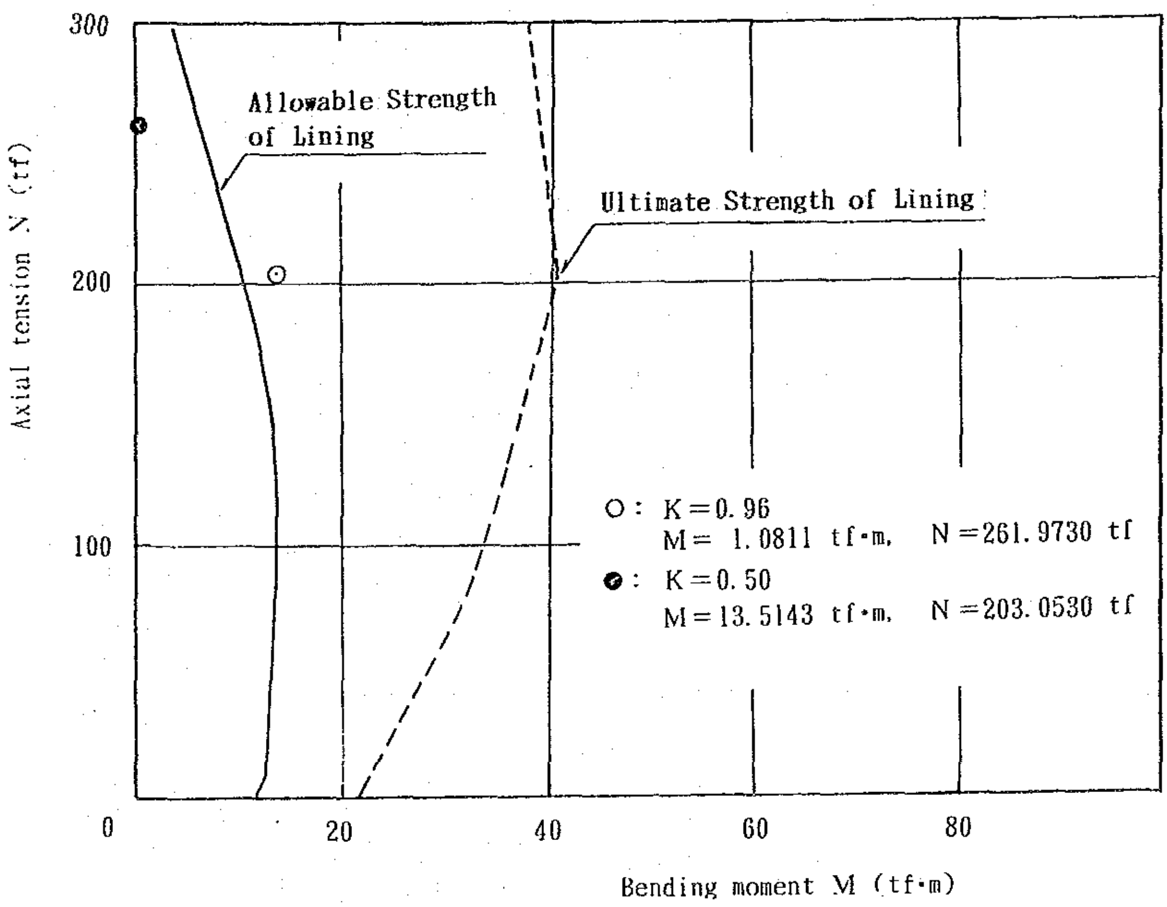


Fig.1.2.3 <u>Strength of Lining and Sectional Force</u>

2. Design 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 lining. To secure safety, however, the design was made 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.2.1.1, and if the relationship exists between 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.

$$W_1 + W_2 + 2S + P \geq \alpha \cdot F$$

(α : Safety Factor 1.25)

where,

Weight of Segment Ring : $W_1 = 41.71$ tf

Weight of Lining : $W_2 = \pi \cdot Dc \cdot t \cdot rc$

$= \pi \cdot 9.9 \cdot 0.4 \cdot 2.5$

$= 31.10$ tf

Weight of the overburden :

$P = Do \cdot h \cdot (r - rw)$

$= 11.6 \cdot 7 \cdot (1.95 - 1.05)$

$= 73.08$ tf

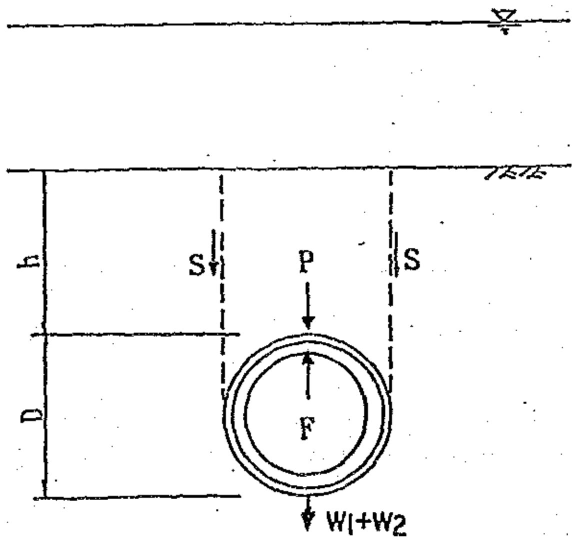


Fig.2.1.1. Relation ship between Buoyancy that acts upon the Tunnel and Resisting Forces

Shearing resistance force of ground :

$$S = h \cdot c' + k/2\ (\gamma - \gamma_w) \cdot h^2 \cdot \tan\phi$$
$$= 7 \times 19 + 0.589/2 \times (1.95-1.05) \times 7^2 \times \tan 15$$
$$= 133 + 3.48$$
$$= 133.48\text{tf}$$

Where, k : Coefficient of active earth pressure

$$k = \tan^2(45 - \phi/2)$$
$$= \tan^2(45 - 15/2)$$
$$= 0.589$$

Buoyancy : $F = \pi/4 \cdot D_0 \times \gamma_w$

$$= (\pi/4) \times 11.6^2 \times 1.05$$
$$= 110.97\text{tf}$$

$$W_1 + W_2 + 2S + P \geqq \alpha \cdot F$$

$$41.71 + 31.10 + 2 \times 133.48 + 73.08 \geqq 1.25 \times 110.97$$

$$412.85 \quad \geqq \quad 138.71 \qquad \text{OK.}$$

3. Design relative to influence of the Canal expansion work (Cross-section)

The ground at the Canal's bottom is predicted to rebound by the Canal expansion work. It is considered that the Tunnel, which crosses the Canal's bottom, will be affected by this rebound.

Accordingly the design will be made using FEM and a Beam on the elastic foundation model.

3.1. Design using FEM(Finite Element Method)

3.1.1. Design condition

The influence of the ground rebound due to the Canal expansion on the lining cross-section was studied using FEM.

The analytical model of FEM is as shown in Fig.3.1.1, and to view the influence of excavation, only an excavation load (P= 33 tf/m^2) will be applied.

The Tunnel's cross-section will be studied at the center of the Canal. The excavation load for the Canal is not infinite in the Tunnel's depth direction, but it is essentially necessary to make three-dimensional analysis. Although two-dimensional analysis will be made for simplification's sake this time, the ground layer thickness will be a problem in the calculation of displacement at this time. In the analytical model, the ground layer thickness was assumed to be 200 m to allow the amount of rebound to meet that of FEM in the longitudinal direction as to be mentioned later.

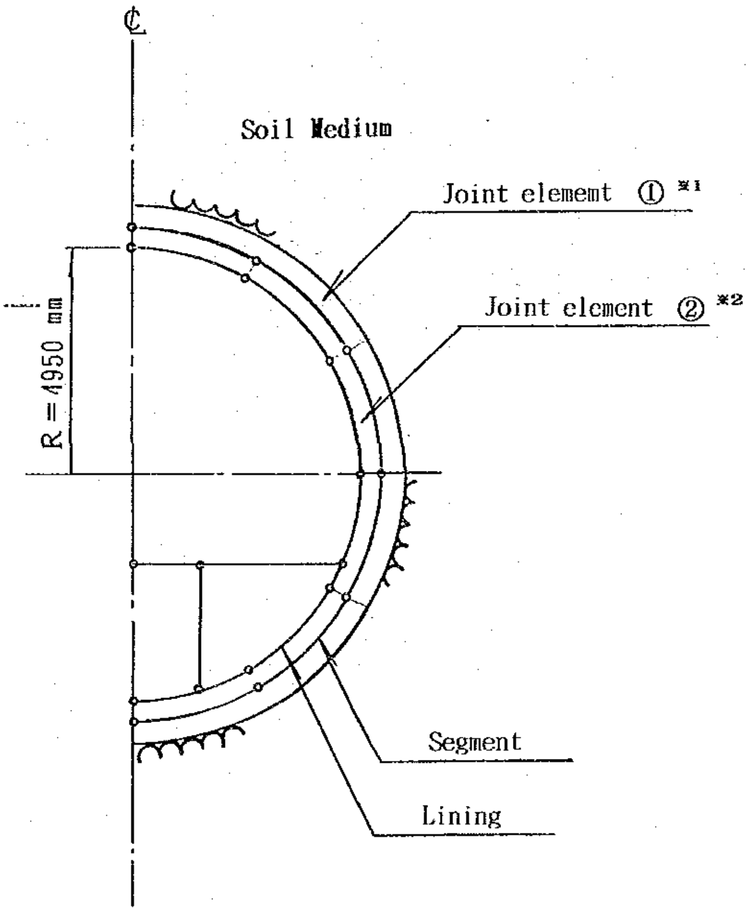
The condition used for studying is shown in Table 3.1.1.

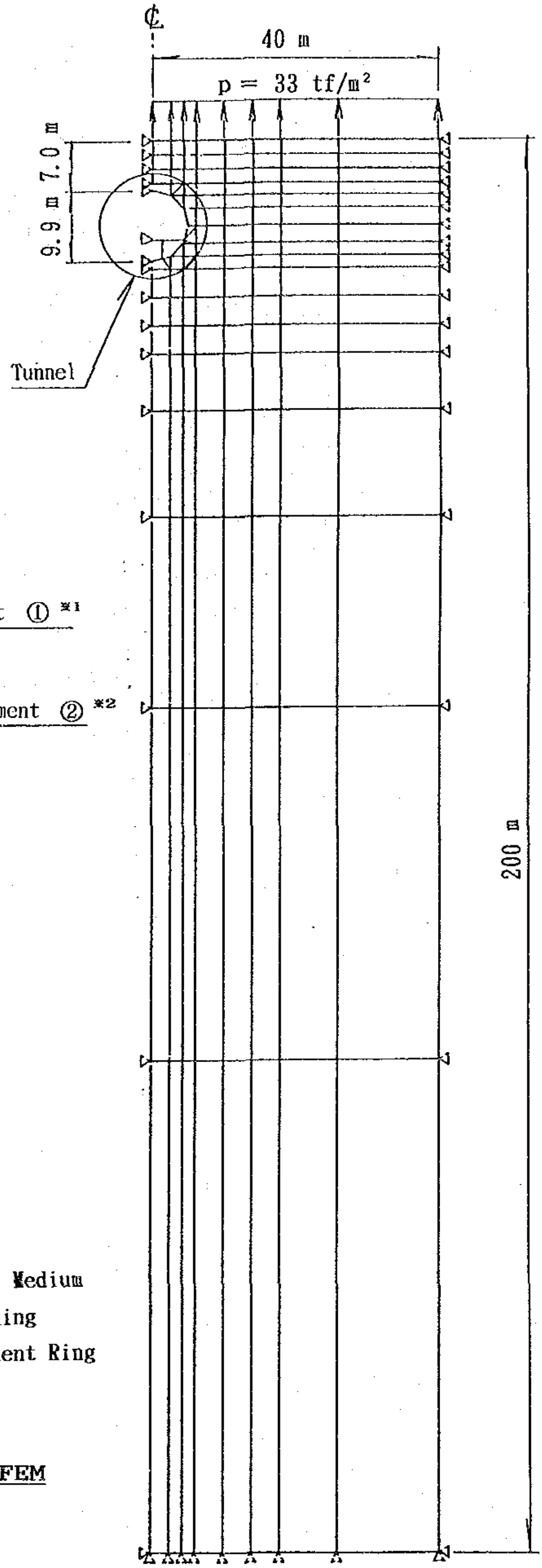
Table 3.1.1 Condition for FEM

Item	Condition
Earth Condition	
Elastic Modulus	15,000.0 tf/m^2
Poisson's Ratio	0.4
Tunnel Condition	
Segment	
Elastic Modulus	1,750,000.0 tf/m^2
Cross Sectional Area	0.3859 m^2
Moment of Inertia	0.0113 m^4
Lining	
Elastic Modulus	2,650,000.0 tf/m^2
Cross Sectional Area	0.4 m^2
Moment of Inertia	0.0053 m^4
Road Deck	
Elastic Modulus	2,650,000.0 tf/m^2
Cross Sectional Area	0.35 m^2
Moment of Inertia	0.0036 m^4
Support Wall of Road Deck	
Elastic Modulus	2,650,000.0 tf/m^2
Cross Sectional Area	0.3 m^2
Moment of Inertia	0.0023 m^4
Joint Element - 1	
Elastic Modulus	15,000.0 tf/m^2
Shear Modulus	6,250.0 tf/m^2
Joint Element - 2	
Elastic Modulus	12,500.0 tf/m^2
Shear Modulus	0.0 tf/m^2

A joint element (1) was provided between the soil medium and the segment to evaluate the disconnection between the soil medium and segment. Likewise, a joint element (2) was provided between the segment and the Lining. A fleece such as non-woven drain material was considered. The dimensions of these joint elements are as shown in Table 3.1.1.

Detail of tunnel element





※ 1 Interaction between Soil Medium and Segment Ring and Lining

※ 2 Interaction between Segment Ring and Lining

Fig.3.1.1 Analytical Model of FEM

3.1.2. Analytical Result

The amount of rebound produced by the Canal excavation and the Lining sectional force are as shown in Fig.3.1.2, 3.1.3, 3.1.4 and Table 3.1.2.

Table 3.1.2 Result of FEM Analysis

Item	Result
Rebound Amount Bottom of the Canal Tunnel Crown Tunnel Invert	 δ = 210.6 mm δ = 210.8 mm δ = 171.9 mm
Cross Sectional Force of Lining Axial Force Bending Moment Shear Force	 N = 18.0 tf M = 6.5 tf·m S = 12.7 tf

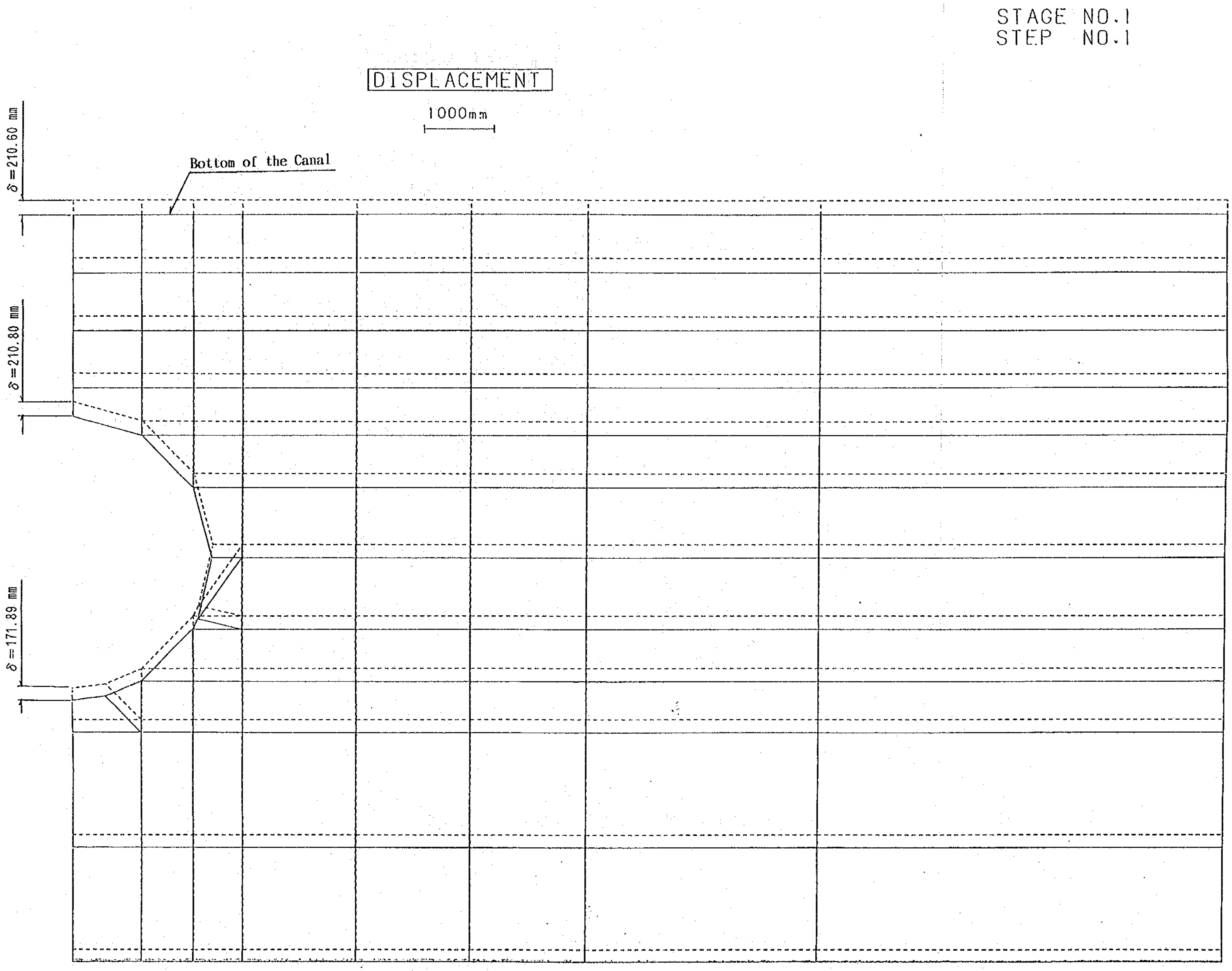
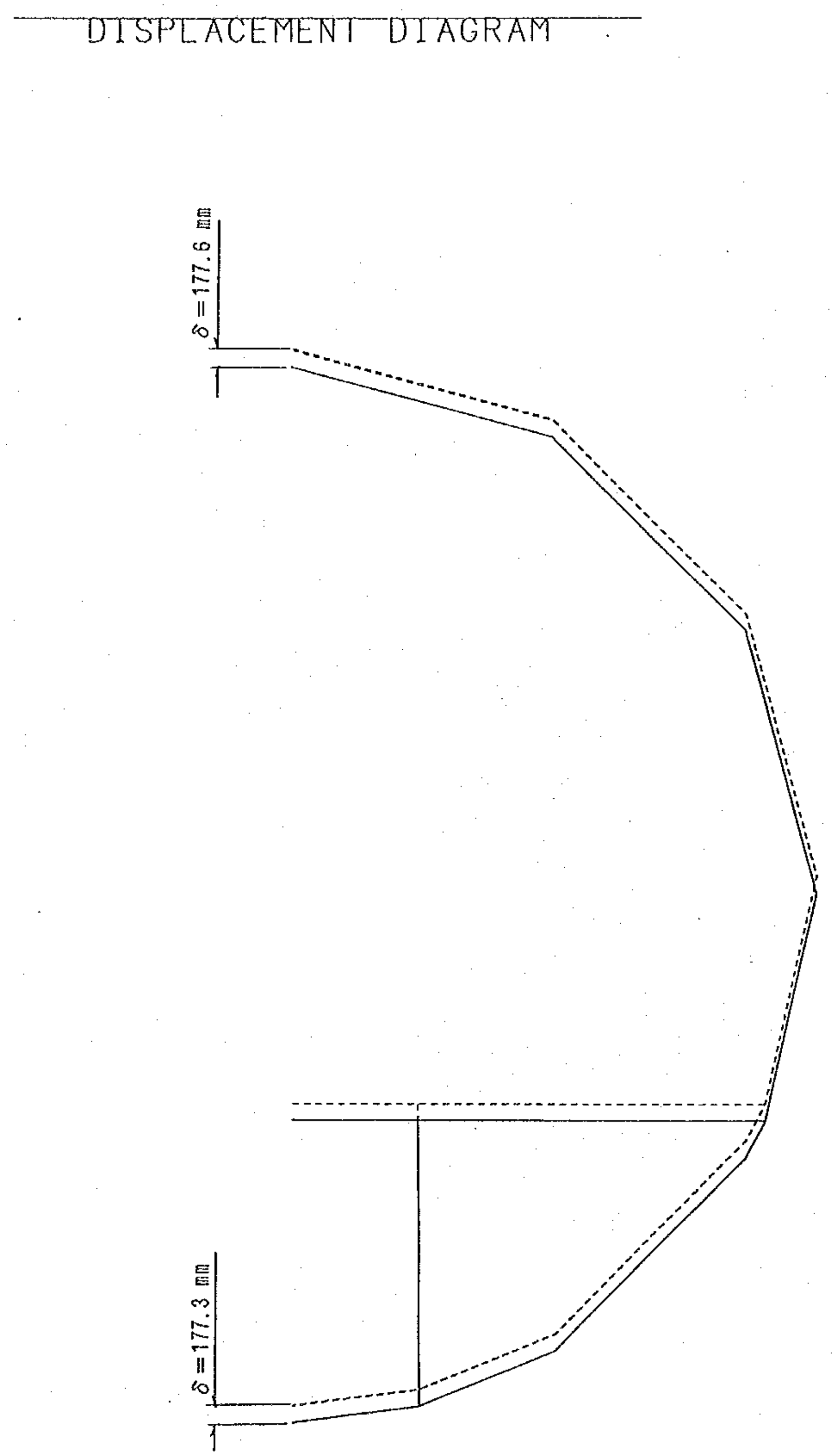
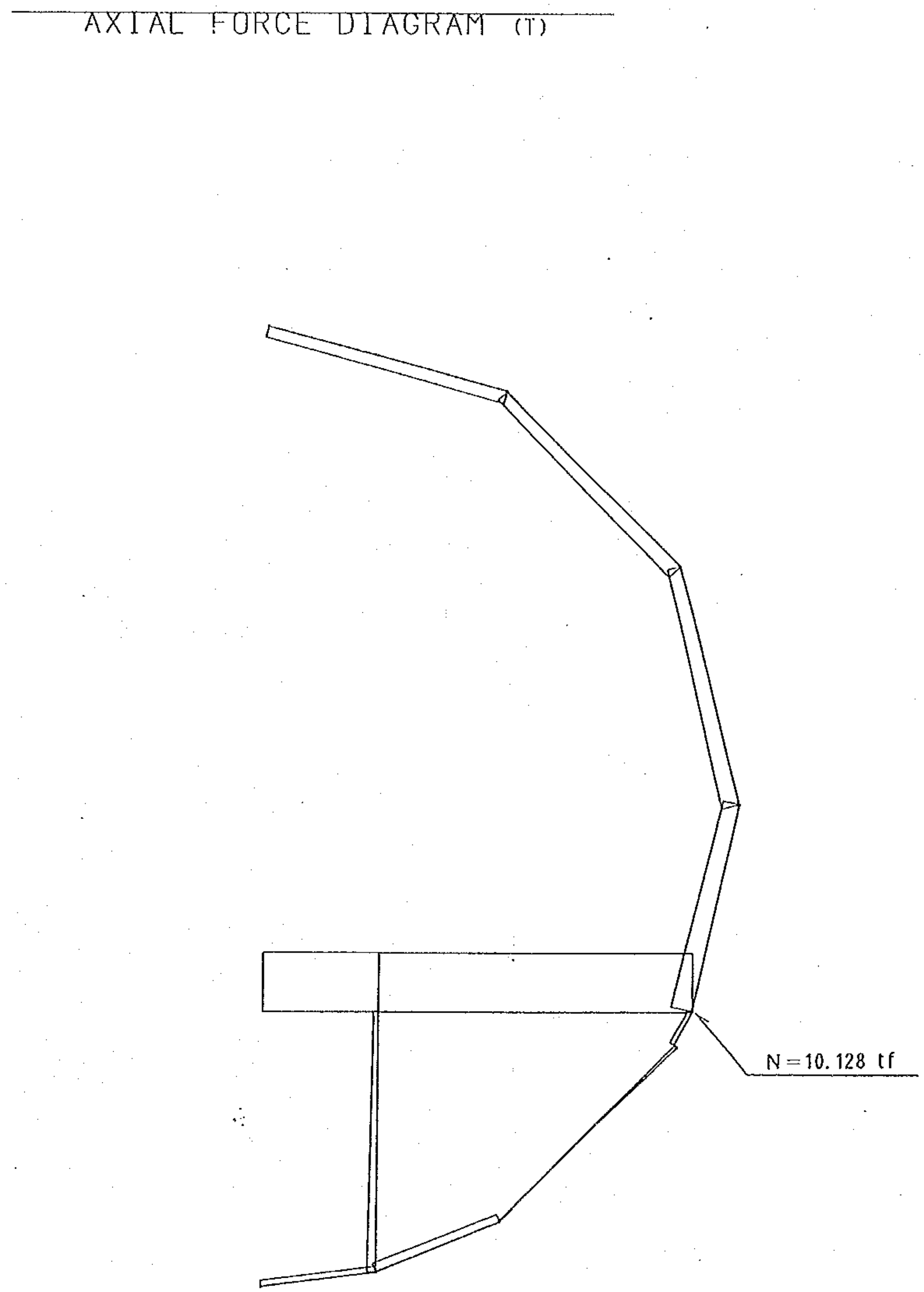


Fig.3.1.2 **Diagram of Ground Displacement**

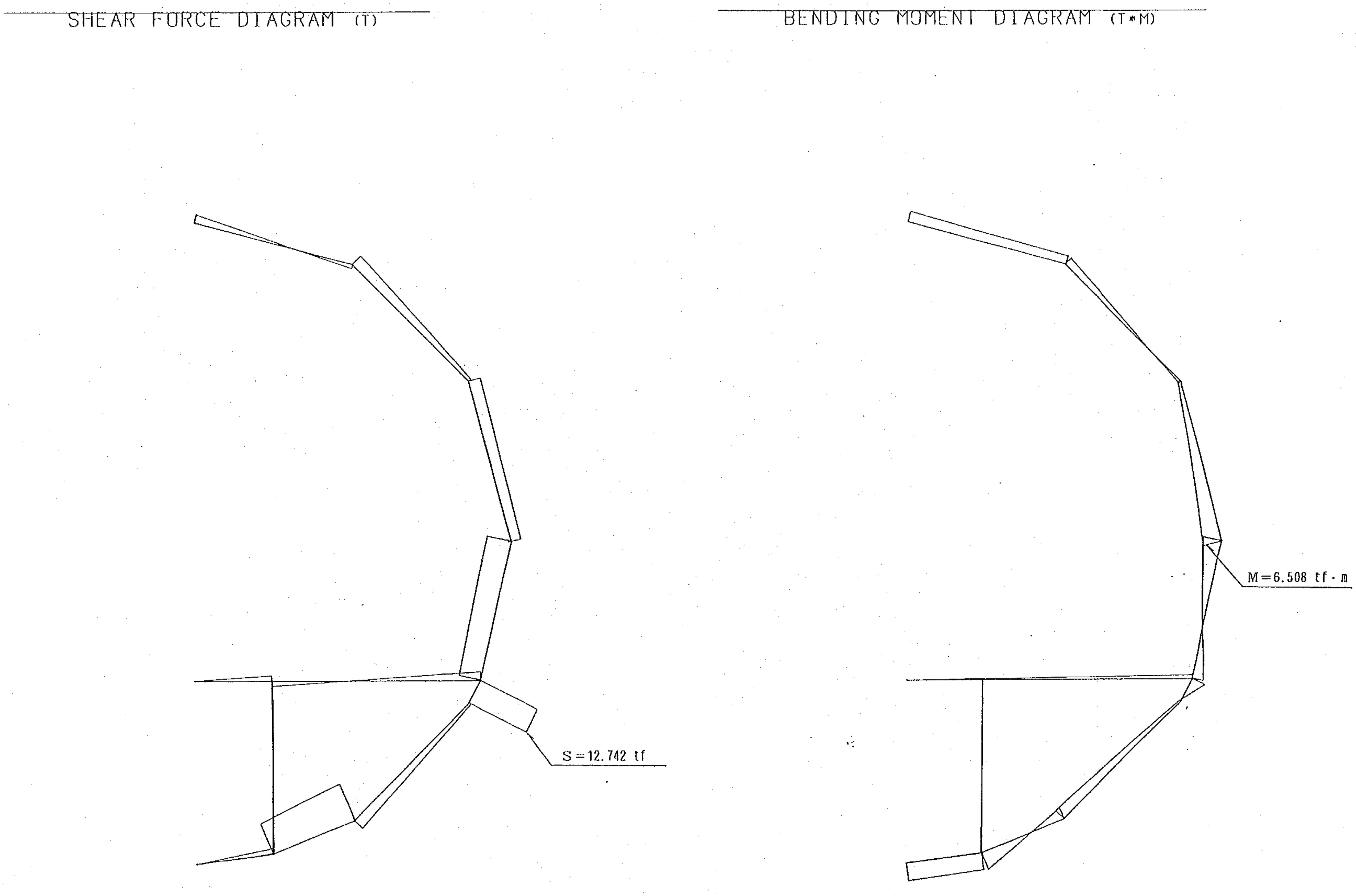


(a)



(b)

Fig.3.1.3 **Diagram of Tunnel Displacement and Axial Force of Lining**



(a) (b)

Fig.3.1.4 Diagram of Shearing Force and Bending Moment

3.1.3. Evaluation of safety

The Canal will be excavated over a long period of time after the lining is executed. Therefore, it is considered that the current earth pressure will be applied to the lining until the Canal excavation is finished.

Accordingly the safety of the lining is specified below.

- Ordinary Condition: Sectional force due to the current earth pressure + Sectional force produced by excavation
- Critical Condition: Only sectional force produced by excavation

The table below shows the result of calculation. Each result is within the stress limit.

Table 3.1.3. Summary of Cross Sectional Force

	Ordinary* (a)	Cross Sectional Force due to the Deepening(b)	Total (a)+(b)
Ordinary Condition			
N(tf)	49.183	18.012	57.983
M(tf·m)	0.486	6.508	6.994
S(tf)	—	12.742	12.742
Critical Condition			
N(tf)	—	18.012	18.012
M(tf·m)	—	6.508	6.508
S(tf)	—	12.742	12.742

* : Refer to the result obtained by the conventional method

Table 3.1.4 The Result of Stress Intensity

(unit:kgf/cm^2)

	σc	σca	σs	σsa
Ordinary Condition	46.0	90.0	104.4	2100.0
Critical Condition	46.6	135.0	1017.6	3150.0

3.2.Design using beam on the elastic foundation mode

3.2.1. Estimation of Rebound

The rebound of that may occur during the excavation of the Canal was estinated by using (assumed) swelling index, C.

- Bedrock Model

The depth to bedrock will be assumed to be 50 meters below the excavated bottom of the Canal, based on the Ahmed Hamdi Tunnel As-Built Drawings. The Bedrock will also be assumed to be uniform through the whole bottom.

- Estimation of Existing Voids Ratio, eo

Void ratio, eo, can be calculated with the following formula:

$$eo = (Gs \cdot rw)/rd - 1, \quad rd = rt/(1 + w)$$

Where,

w = moisture content (30%)

rt = unit volumetric weight of soil (1.95 tf/m3)

rw = unit volumetric weight of water (1.05 tf/m3)

The above three factors were obtained from the drawings.

Gs = specific gravity (2.5) (assumed)

$$rd = 1.9/(1+0.3) = 1.46 \text{ tf/m3}$$

$$eo = (2.5 \times 1.05)/1.46 - 1 = 0.797$$

- Estimation of Compression Index, Cc

Compression Index, Cc, can be obtained from the liquid limit, WL, using the following formula:

$$Cc = 0.009\ (WL - 10) \qquad \text{(Skempton's law)}$$

Liquid limit WL = 60 to 90% ≒ 70% (from the As-Built Drawings)

$$Cc = 0.009 \times (70 - 10) = 0.54$$

- Estimation of Swelling Index, Cr

It is believed that Swelling Index, Cr is generally between 1/10 and 1/30 of the Compression Index, Cc. Therefore, the Swelling Index, Cr can be assumed to be within the following range.

$$Cr = 0.54 \times (1/10 \text{ to } 1/30) = 0.054 \text{ to } 0.018$$

- Estimation of Rebound

Rebound can be estimated by the following consolidation settlement formula:

$$\rho = \frac{H \cdot Cr}{1+eo} \log\left(\frac{po + p}{po}\right)$$

Where,

$H = 50\ m$

$P_0 = (25 + 33) \times 0.9 = 52.2$ tf/m3 (at center of earth)

$\Delta P = 33 \times 0.90 = -29.7$

$$\rho_1 = \left(\frac{50.0 \times 0.018}{1 + 0.797}\right) \cdot \log\left(\frac{52.2 - 29.7}{52.2}\right)$$
$$= -0.183 \text{ m} = -183 \text{ mm}$$
$$\rho_2 = \left(\frac{50.0 \times 0.054}{1 + 0.797}\right) \cdot \log\left(\frac{52.2 - 29.7}{52.2}\right)$$
$$= -0.549 \text{ m} = -549 \text{ mm}$$

Therefore, the estimated rebound of the excavated surface of the Canal will be approximately as follows:

$$\rho = 183 \text{ to } 549 \text{ mm}$$

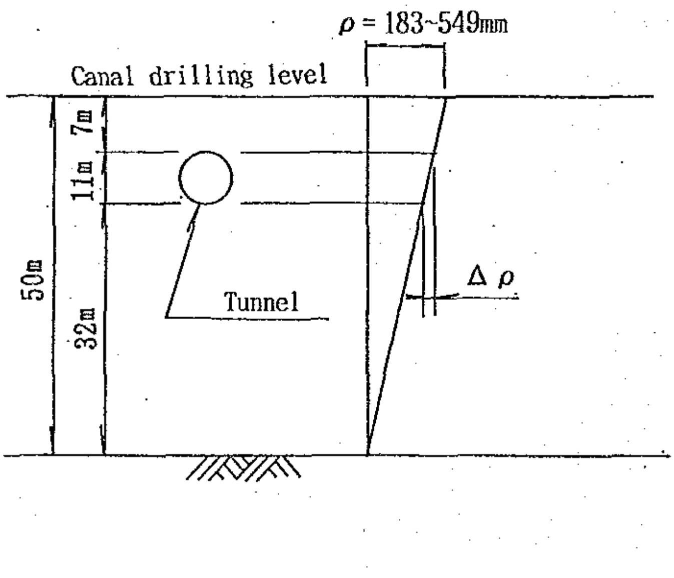
• Relative displacement at the Tunnel's position

The relative displacement at the Tunnel's position has been calculated as below by using an approximate estimation for the amount of rebound on the Canal excavation surface.

The relative displacement at the Tunnel's position is an approximate estimation by the past references, etc., and has considerable latitude.

According to the original "Ahmed Hamdi Tunnel Design Documents", on the other hand, the amount of rebound on the Canal excavation surface and the relative displacement at the tunnel position are 350 mm and 50 mm respectively.

Judging the above synthetically, the relative displacement at the tunnel position will be 50 mm in the study of the lining.

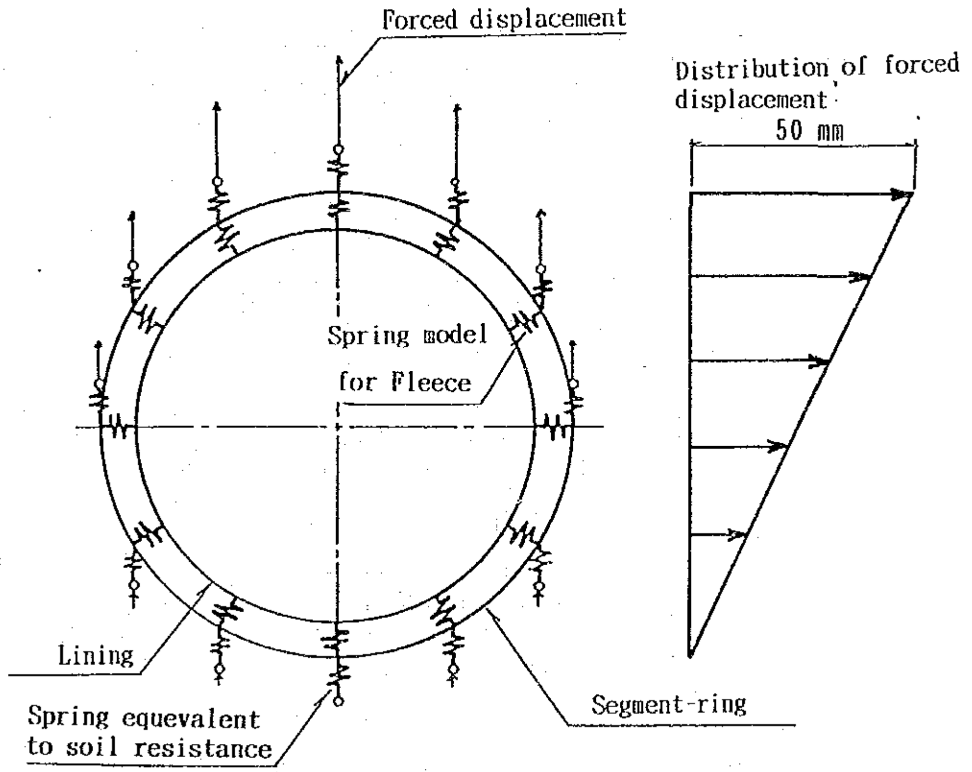


	Rebound at canal drilling level ρ (mm)	Relative displacement Δ ρ (mm)	Remarks
Minmum	183	40.3	[Document on A.H.Tunnel] ρ = 350 mm Δ ρ = 50 mm
Maxmum	549	120.3	

3.2.2. Design condition

The rebound of the ground affected by the Canal expansion work was calculated by applying it on a double-deck ring model, in which the segment ring and lining were evaluated by girder, as shown in Fig.3.2.1.

Since the fleece material to be provided between the segment and the lining together with waterproofing sheets at this time is compressible as shown in Fig.3.2.2, the fleece material was evaluated as a compressive spring between the segment ring and lining. Since, however, the nonwoven material is considered to be unable to resist the tensile force, no tensile spring was chosen.



Segment-ring,

$E = 3500000\ tf/m^2$

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

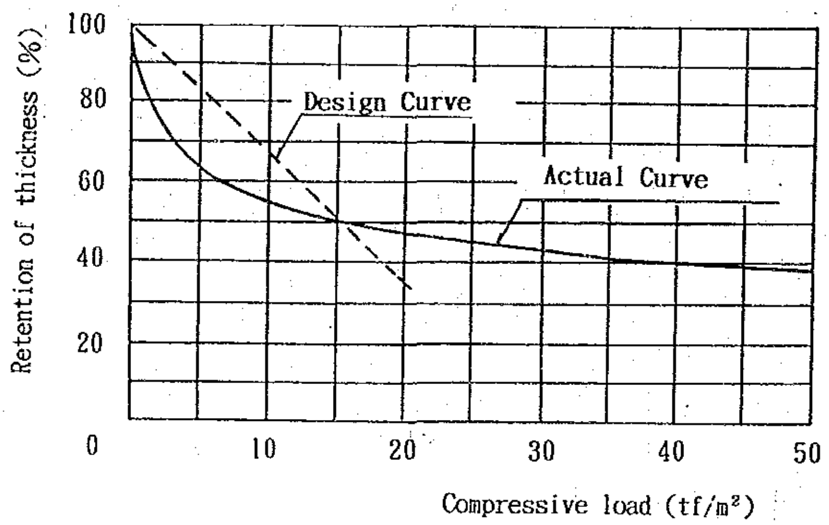
$A = 0.3859\ m^2$

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

Fig.3.2.1. Analytical Model

For the fleece compressive spring, the following values were used because of an relationship between a load with 50% in compressibility and the displacement as shown in Fig.3.2.2.

CASE-1: $k=2{,}500\ tf/cm^2$	When a sheet of fleece material of 7 mm is used.
CASE-2: $k=1{,}250\ tf/cm^2$	When two sheets of fleece material of 7 mm are used.



$$k = \frac{\text{Thickness of fleece} \times (100 - \text{Retention of thickness})\ \%}{\text{Compressive load}}$$

Fig.3.2.2 <u>Compressive Characteristic Curve of Fleece</u>

On calculating the sectional force, we have decided to use the calculation result for sectional force of lining before the Canal expansion because the lining is considered to have already received earth pressure before the Canal expansion work.

The calculation result of sectional force is shown in Table 3.2.1.

Also the displacements for the segment ring and lining at this time are shown in Table 3.2.2.

Table 3.2.1 Result of the Sectional Force

		Max. Bending Moment tf·m		Axial Force tf	
		Segment	Lining	Segment	Lining
Cross-Sectional Force before the Expansion of Canal		——	0.382	——	48.845
Case-1	Influence of Expansion of Canal	-80.435	33.644	-0.402	18.318
	Combined Effect	——	34.026	——	67.163
Case-2	Influence of Expansion of Canal	-77.346	22.144	6.322	12.794
	Combined Effect	——	22.526	——	61.639

Table 3.2.2 Result of the Lining Displacement

		Segment	Lining
Case-1	Displacement at Crown	48.44 mm	29.77 mm
	Displacement at Bottom	2.56 mm	15.10 mm
	Relative Displacement	45.88 mm	14.67 mm
Case-2	Displacement at Crown	48.76 mm	19.14 mm
	Displacement at Bottom	2.14 mm	9.88 mm
	Relative Displacement	46.62 mm	9.25 mm

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 in Case-1 and is within the ultimate strength in Case-2 as shown in Fig.3.2.3.

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

Although the segment ring causes local sectional failure, a hinge is formed because the peripheral area of the Tunnel is restricted by the ground and lining exists inside, and it is judged that the structure of segment ring will not be broken.

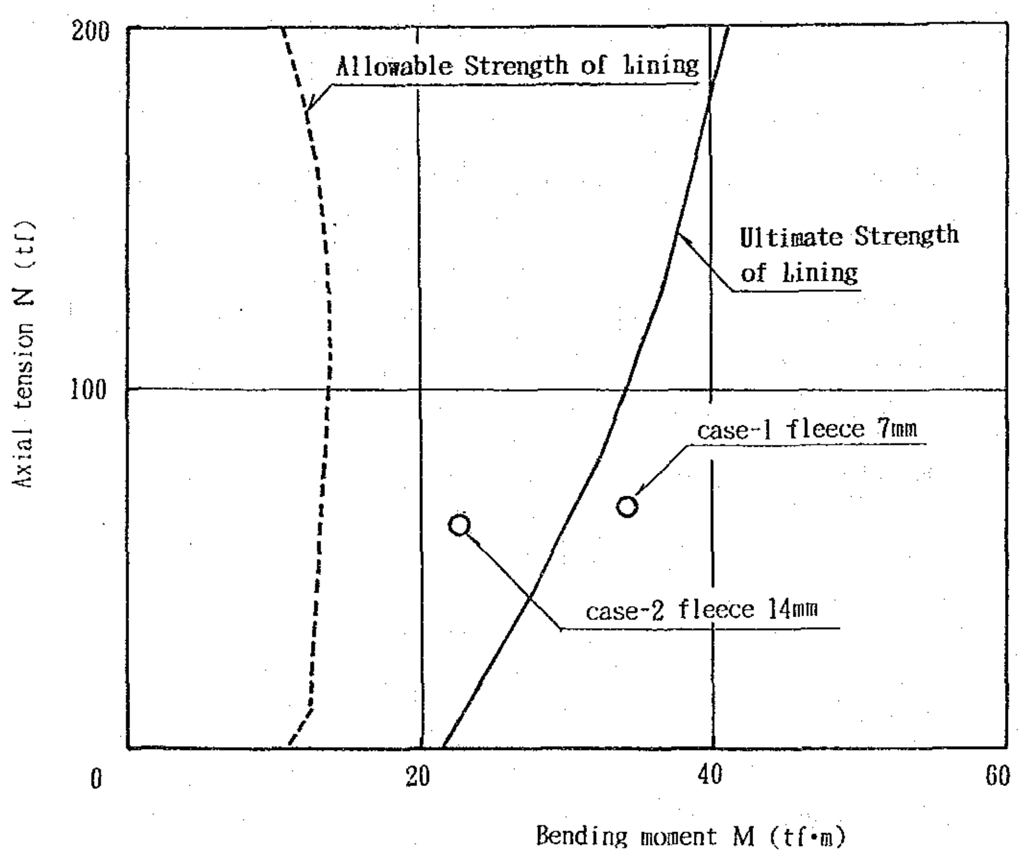


Fig.3.2.3 **Strength and Sectional Force of Lining**

When affected by the Canal expansion work, the safety was confirmed by superimposing the sectional force due to the traffic load on that due to the Canal expansion work because a great bending moment occurs as shown in Table 3.2.3.

Table 3.2.3 Road Deck Safety Verification

		Point A	Point B	Point C	Point D
M (Traffic Load)	tf·m	-5.50	3.37	-5.50	3.46
M (Canal Expansion)	tf·m	-1.38	-1.38	4.33	1.61
M (Total)	tf·m	-6.88	1.99	-1.17	5.07
N (Canal Expansion)	tf	81.14	81.14	80.10	80.10
Beam Height h	cm	53.00	35.00	43.00	31.00
Covered with Reinforcement d	cm	10.00	10.00	10.00	10.00
Amount of Reinforcement As=As'	cm^2	D19ctc200	D19ctc200	D19ctc200	D19ctc200
σc	kgf/cm^2	29.7	29.8	21.5	52.8
σs	kgf/cm^2	———	———	———	———

As a result, it can be confirmed that there are no problems on the Road Deck caused by fluctuations of the sectional force due to the Canal expansion work.

4. Design relative to the influence of the Canal expansion work (Longitudinal direction)

4.1. Design using beam on the elastic foundation + Loading method

4.1.1. Design condition

In this Tunnel, the earth covering will be greatly changed under the influence of the expansion work of the Canal to be carried out in the future.

Since about 40% of the Tunnel length will be affected by the Canal expansion work, design in the Tunnel's longitudinal direction was made.

The conditions used for the design are as shown in Tables 4.1.1 and 4.1.2.

These soil conditions are basically the same as the design conditions of the Tunnel's cross-section.

Regarding the coefficient of ground reaction of 3,000tf/m^3 among them, comparative design is also performed using K = 2,000 tf/m^3.

This is to confirm that the Tunnel will not be broken even if the coefficient of ground reaction of 3,000tf/m^3 which is estimated from the geological survey result is to be reduced to almost 2/3.

Table 4.1.1 <u>Soil Condition</u>

Item	Condition
Density of Soil	1.95 tf/m^2
Density of Water	1.05 tf/m^2
Density of Soil in Water	0.90 tf/m^2
Coefficient of Ground Reaction	3,000 (2,000)tf/m^3

Table 4.1.2 Structural Condition of Lining

Item	Condition
Tunnel Outer Diameter (Segment)	Do = 11.60 m
Tunnel Inside Diameter	Di = 9.50 m
Centroid Radius	Rc = 4.95 m
Lining Thickness	t = 0.40 m
Elastic Coefficient	E = 2650000.0 tf/m^2
Cross-Sectional Area	Al = 12.44 m^2/m
Moment of Inertia	Il = 152.66 m^4/m
Design Strength of Concrete	σck = 270 kgf/cm^2
Allowable Compressive Stress of Concrete	σca = 90 kgf/cm^2
Tensile Strength of Reinforcement Bar	σsy = 4000 kgf/cm^2
Allowable Tensile Strength of Reinforcement Bar	σsa = 2100 kgf/cm^2

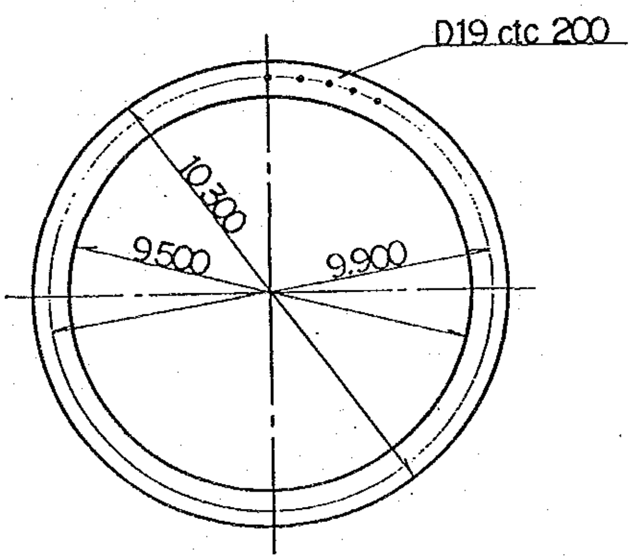


Fig.4.1.1 Cross Section of Tunnel Lining

4.1.2. Calculation of the load

The vertical load, which changes in the Tunnel's longitudinal direction due to the Canal expansion work, was calculated to find out difference between the current overburden and the overburden after the Canal excavation as shown in Fig.4.1.3.

The calculation results of the load are given in Table 4.1.3 below and in Fig.4.1.2.

Table 4.1.3 Calculation Results of Load

Distance (m)	Current Overburden (m)	Current Earth Pressure (tf/m^2)	Earth Overburden after Expansion (m)	Earth Pressure after Expansion (kgf/cm^2)
0.0	17.0	15.3	17.0	15.3
300.0	26.0	23.4	26.0	23.4
360.0	27.0	24.3	7.0	6.3
580.0	34.0	30.6	14.0	12.6
640.0	35.0	31.5	35.0	31.5
720.0	35.0	31.5	35.0	31.5
780.0	18.0	16.2	18.0	16.2
820.0	23.0	20.7	12.0	10.8
920.0	21.0	18.9	10.0	9.0
980.0	20.0	18.0	9.0	8.1
1030.0	40.0	36.0	10.0	9.0
1050.0	40.0	36.0	10.0	9.0
1160.0	34.0	30.6	34.0	30.6
1310.0	40.0	36.0	40.0	36.0
1647.0	23.0	20.7	23.0	20.7

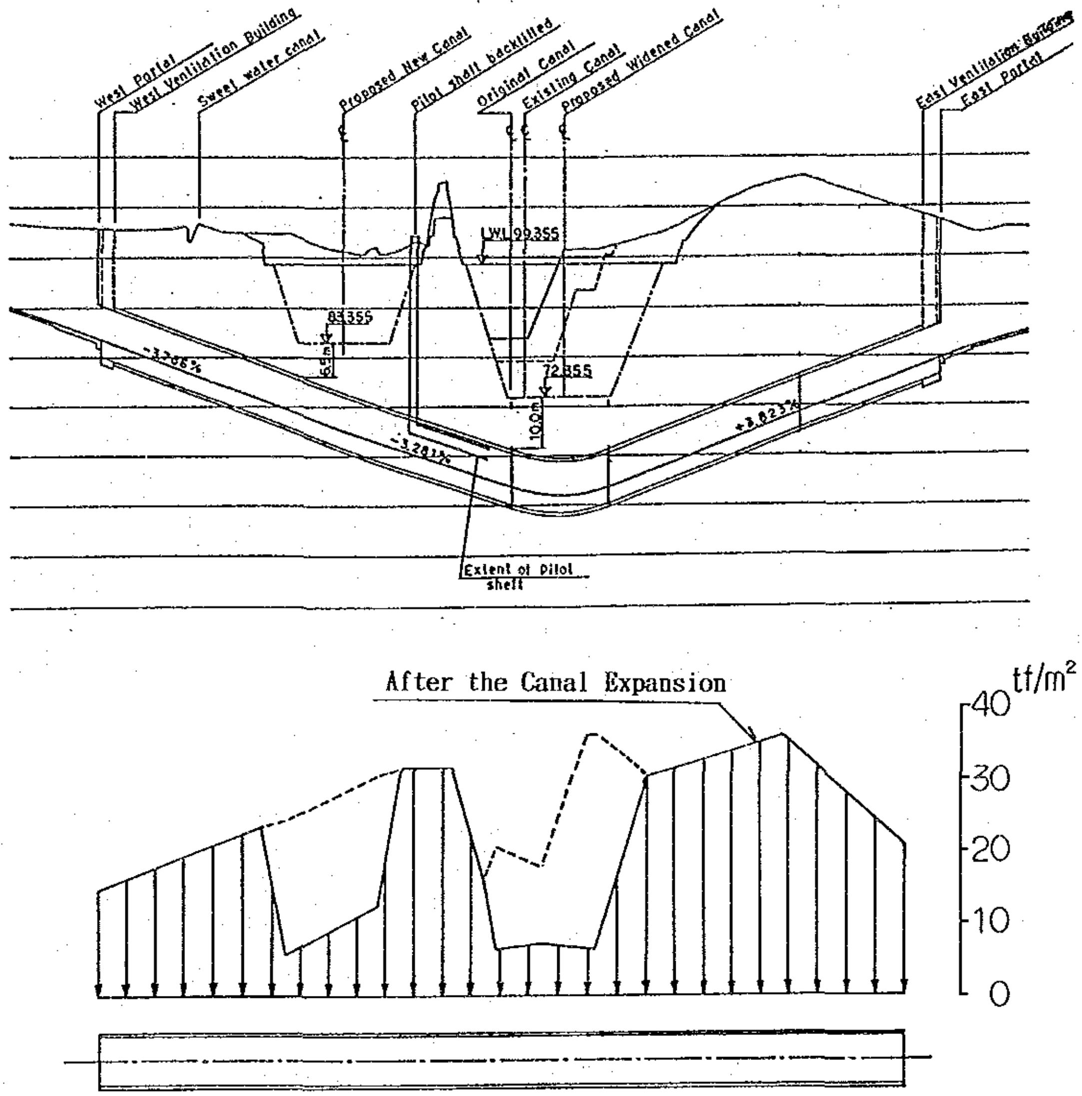


Fig.4.1.2 <u>Distribution of Vertical Load in the Longitudinal Direction of Tunnel</u>

4.1.3. Calculation of sectional force

To calculate the sectional force, the difference between the earth pressure due to the present earth covering and the earth pressure due to earth covering after the Canal expansion was calculated by applying a predetermined load to a structural model in which the lining was evaluated by beam having uniform rigidity in the longitudinal direction and the ground reaction was evaluated by elasticity as shown in Fig.4.1.3.

It has been assumed that the ground reaction can be always expected irrespective of the deformation of the Tunnel Lining. No shearing force transmits since there exist waterproofing sheets around the Lining, and the ground reaction is applied only in the perpendicular direction to the Tunnel axis.

This method evaluates variations in earth pressure produced when the ground is excavated and is most generally used in Japan. Table 4.1.4 and Figs.4.1.4-7 show some results of our calculation.

Table 4.1.4 Calculation on the Cross-Sectional Force

		Max. Bending Moment	Axial Force
CASE-1	K=3000 tf/m2	1994	8.8
CASE-2	K=2000 tf/m2	2835	13.1

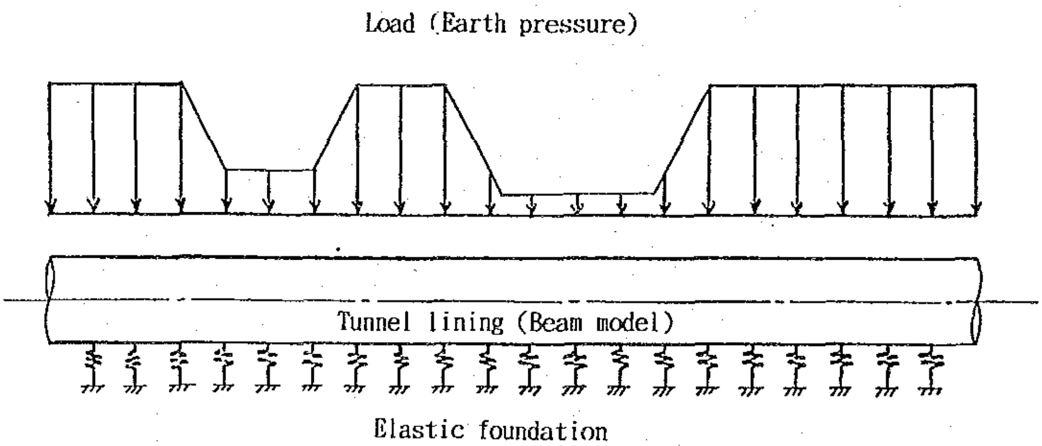
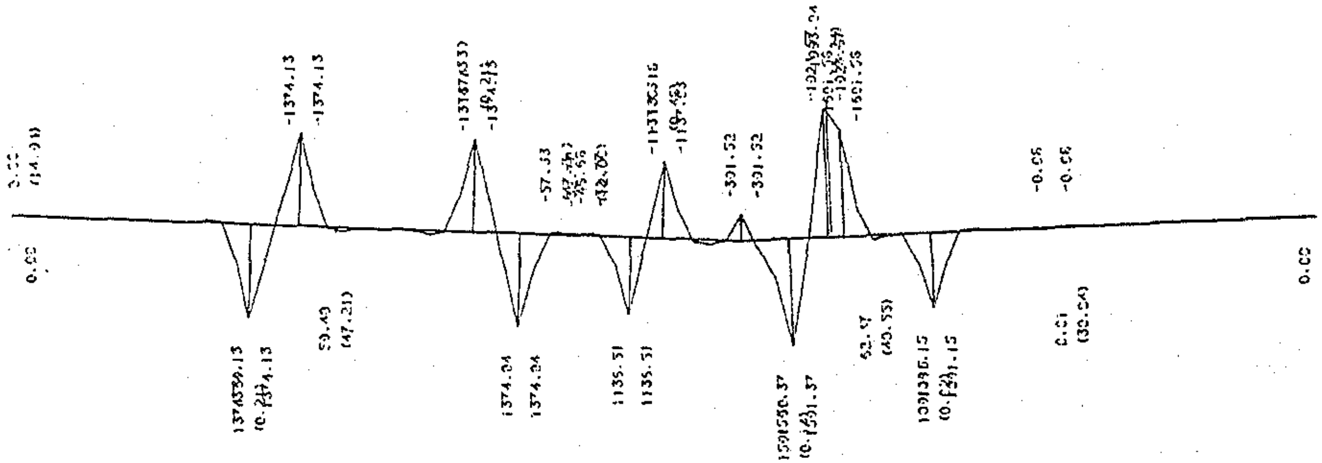


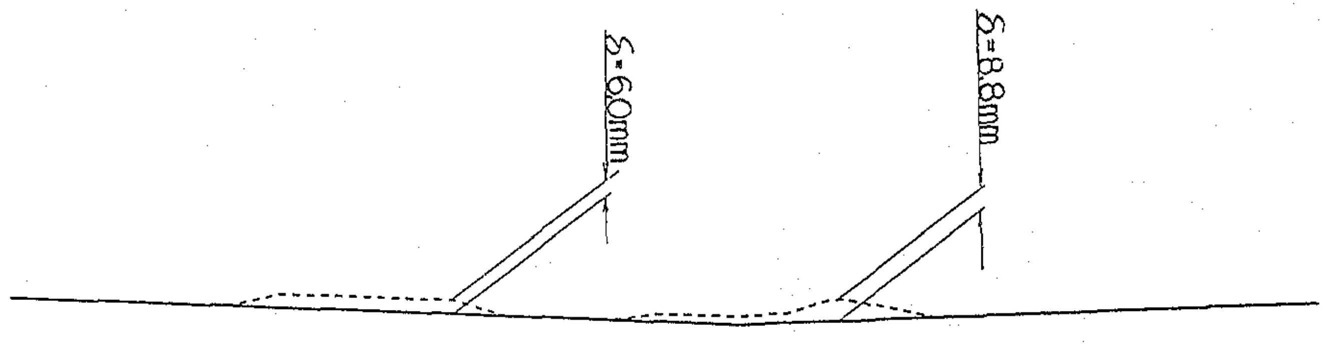
Fig.4.1.3 Analytical Beam Model for Canal Expansion

Mmax = 1994 tf·m



K=3000 tf/m²

Fig.4.1.4 Diagram of Bending Moment



K=2000 tf/m²

Fig.4.1.5 Diagram of Displacement

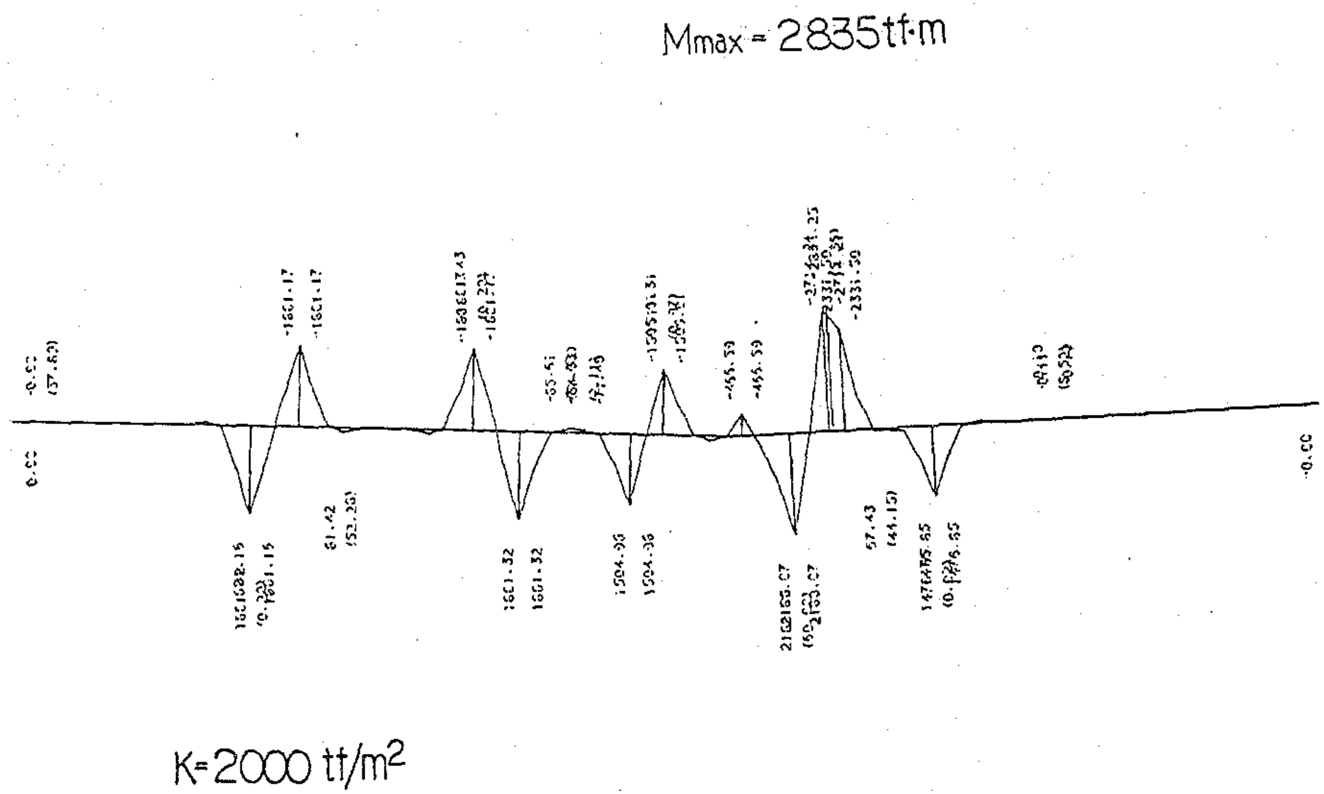


Fig.4.1.6 Diagram of Bending Moment

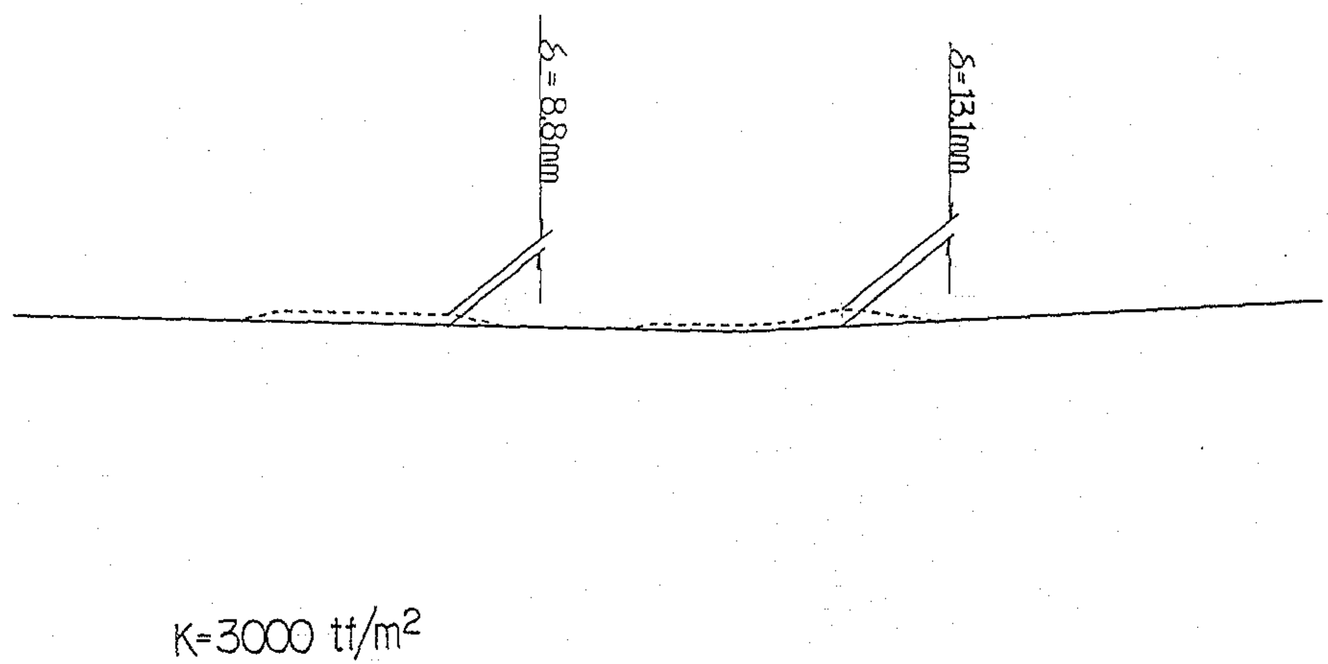


Fig.4.1.7 Diagram of Displacement

4.1.4. Evaluating of safety

From the calculation result of the sectional force, the stress intensity was verified by using the allowable unit stress method.

The stress intensity was calculated for the cross-section shown in Fig.4.1.8.

As the result of these verifications, both cases were below the allowable stress intensity.

Table 4.1.5 Result of Calculation on the Stress Intensity

		σ c	σca	σ s	σsa
CASE-1	K=3000 tf/m2	14.8	90.0	1310.7	2100.0
CASE-2	K=2000 tf/m2	21.2	90.0	1863.8	2100.0

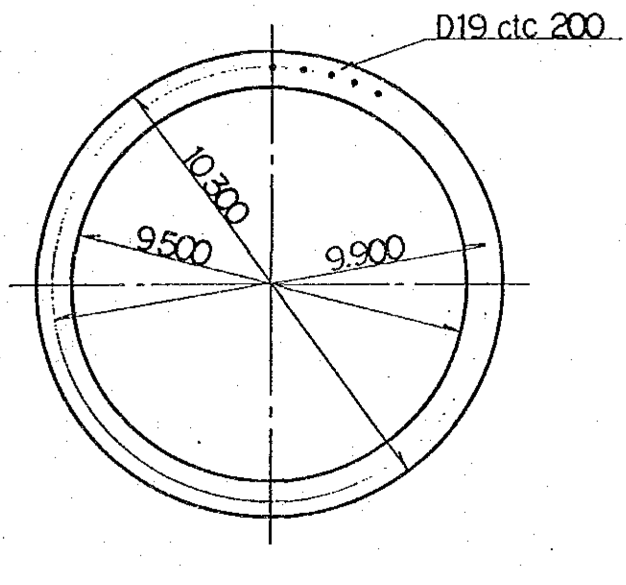


Fig.4.1.8 Cross Section of Tunnel Lining

4.2. Design using FEM

4.2.1. Design condition

Design is performed 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.4.2.1.

The analysis condition is as shown in Tables 4.2.1 and 4.2.2.

For the Tunnel, only lining will be taken into consideration, and it is considered as a beam member.

Table 4.2.1 Soil Condition

Item	Condition
Elastic Modulus (tf/m2)	15,000
Poisson's Ratio	0.4
Density of Soil (tf/m3)	0.9
Cohesion (tf/m2)	190
Internal Friction Angle (rad.)	20

Table 4.2.2 Structural Condition of Tunnel

Item	Condition
Elastic Modulus (tf/m2)	2,650,000
Cross-Sectional Area (m2)	12.44
Moment of Inertia (m4)	153.00

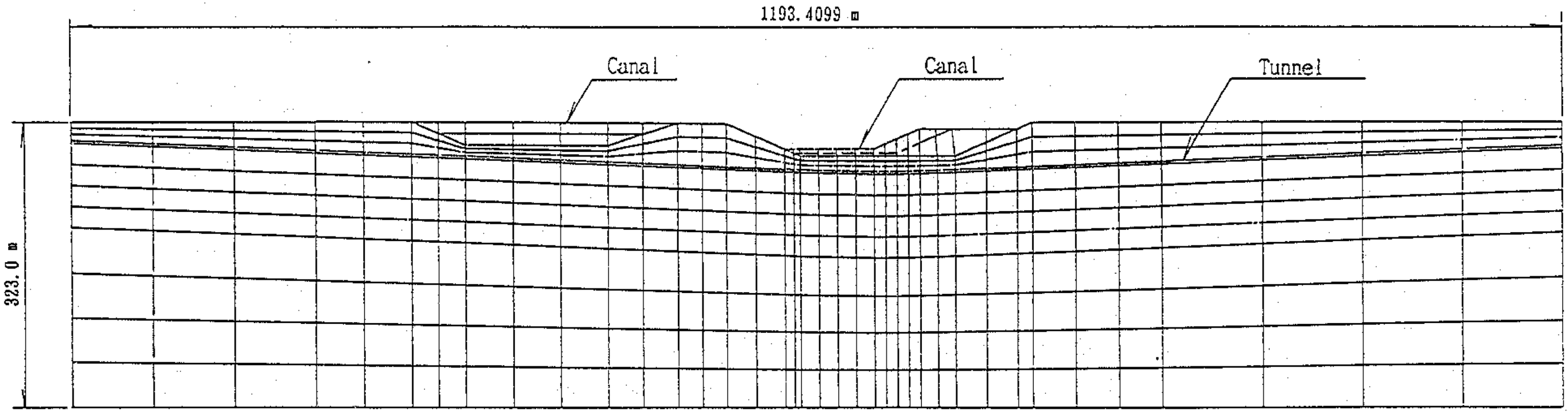


Fig.4.2.1 Analytical Model by FEM

4.2.2. Calculation result of sectional force and Displacement

The displacement and sectional force produced by the Canal expansion work are as shown in Table 4.2.3 and Figs.4.2.2 to 6.

Table 4.2.3 Calculation Result of Sectional Force and Displacement

Item	
Axial Force tf	1235.7
Bending Moment tf·m	-7910.0
Shearing Force tf	158.9
Max. Ground Displacement at the Bottom of Canal mm	239.0
Max. Displacement of the Tunnel mm	226.4

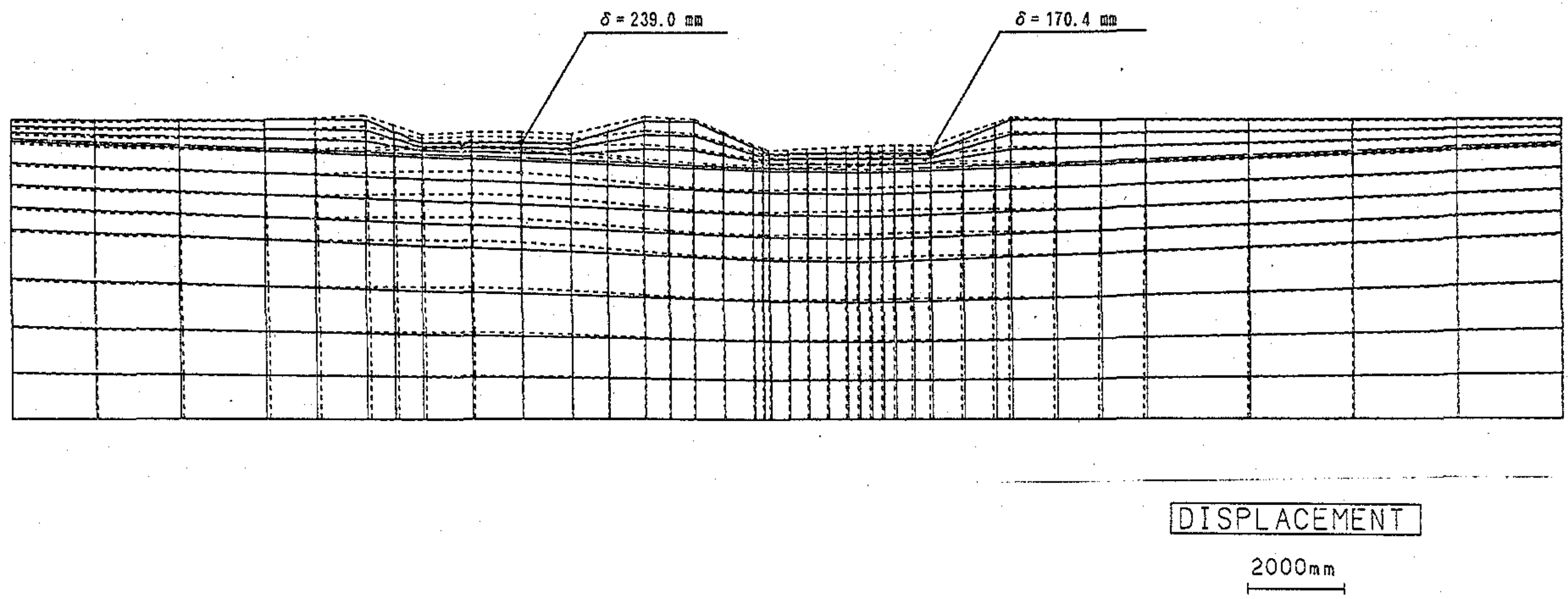


Fig.4.2.2 Ground Displacement Diagram

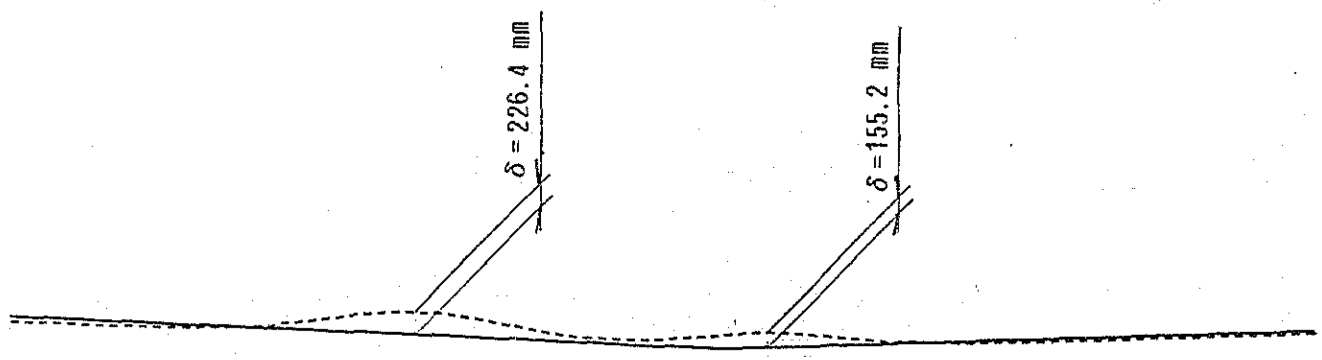


Fig.4.2.3 Tunnel Displacement Diagram

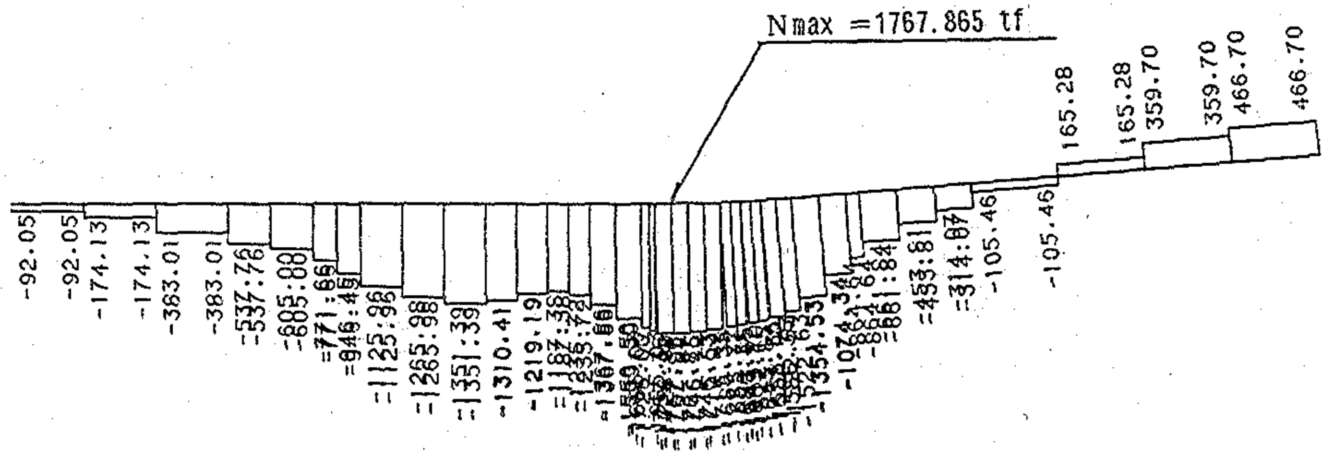


Fig.4.2.4 Axial Force Diagram

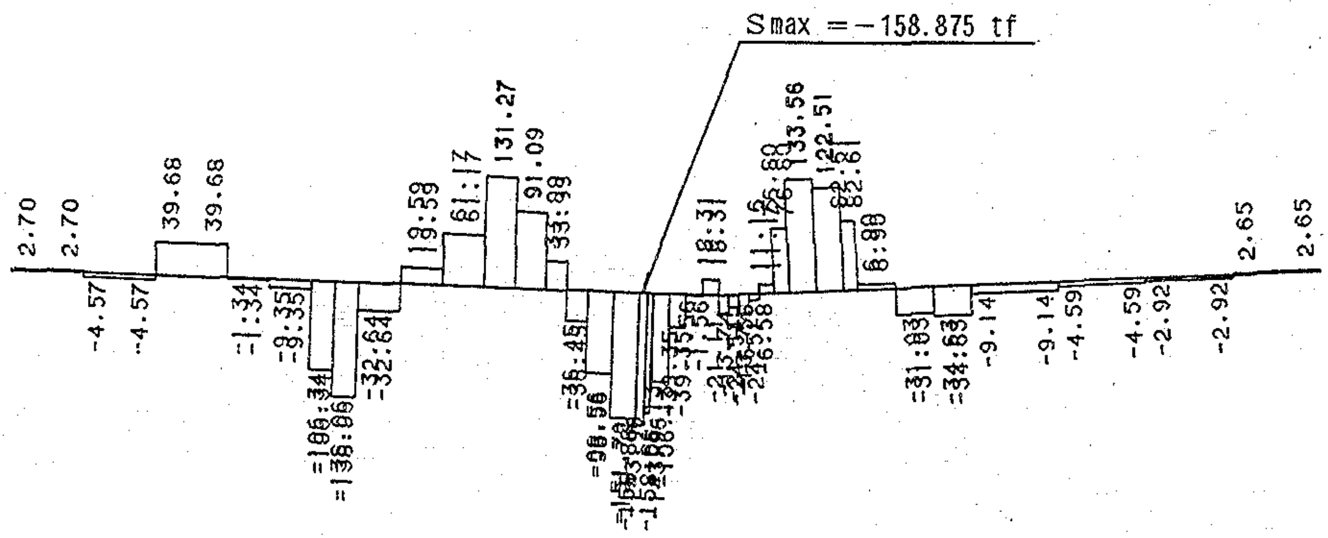


Fig.4.2.5 Shearing Force Diagram

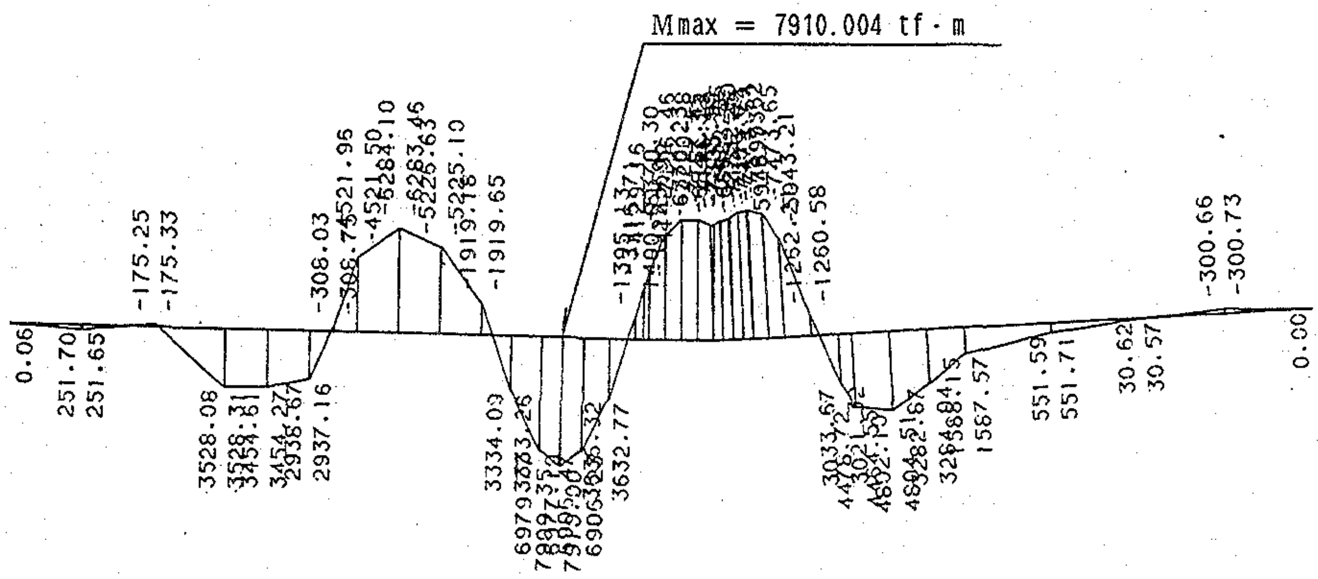
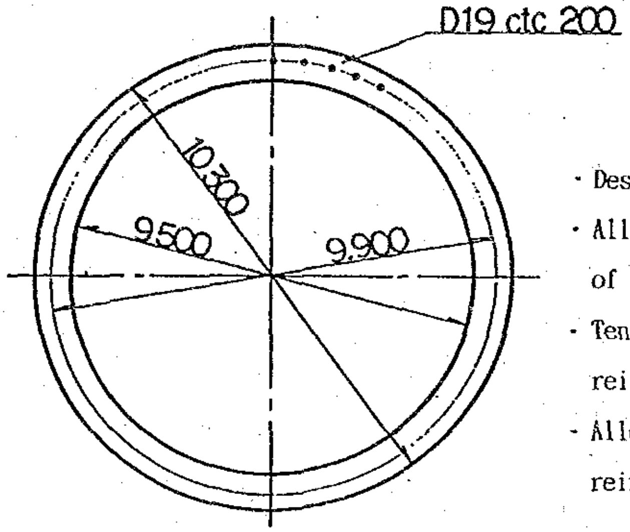


Fig.4.2.6 Bending Moment Diagram

4.2.3. Evaluating of safety

Safety at 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 result of which is as follows:



- Design strength of concrete: $\sigma ck = 270$ kgf/cm²
- Allowable compressive stress of concrete: $\sigma ca = 90$ kgf/cm²
- Tensile strength of reinforcement bar: $\sigma sy = 4000$ kgf/cm²
- Allowable tensile stress of reinforcement bar: $\sigma sa = 2100$ kgf/cm²

Fig.4.2.7 Cross Section of the Tunnel

In case of single reinforcement: $M_{ud} = 7615.9$ tf・m
In case of double reinforcement: $M_{ud} = 14908.9$ tf・m

$M = -7910.0$ tf・m $> M_{ud} = 7615.9$ tf・m , NO
In case of single reinforcement

$< M_{ud} = 14908.9$ tf・m , OK
In case of double reinforcement

In case of single reinforcement, ultimate strength is beyond the safety margin specified, 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.

5. Seismic Design of tunnel structure

5.1. Design Analyses by Dynamic FEM Program

Seismic design analyses for longitudinal movement of the Tunnel will be done using a dynamic FEM program (program title: FLUSH).

5.1.1. Determination of Input Earthquake Ground Motion

The input earthquake ground motion for seismic design analyses is generally determined based on the magnitude of earthquakes, epicentral distance, and dynamic characteristics of the ground, assumed according to past records of strong earthquakes that 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.5.1.1 and Table 5.1.1. 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.

It is understood from the distribution of earthquakes shown in Fig.5.1.1 and Table 5.1.1 that strong earthquakes, M = 5.0 or 5.4 occurred in the area within 100 kilometers (km) in radius from the structure, and their focus depths were relatively shallow, being 24 and 33 km in depth. Earthquakes with the intensities of M = 5.0 to 7.1 occurred eight times in only three years within an area 330 km southeast from the structure, and the record of the biggest earthquake, M = 7.1, is especially noticeable. It should also be noted that an earthquake of intensity M = 5 occurred between the place where the biggest earthquake occurred and the Tunnel.

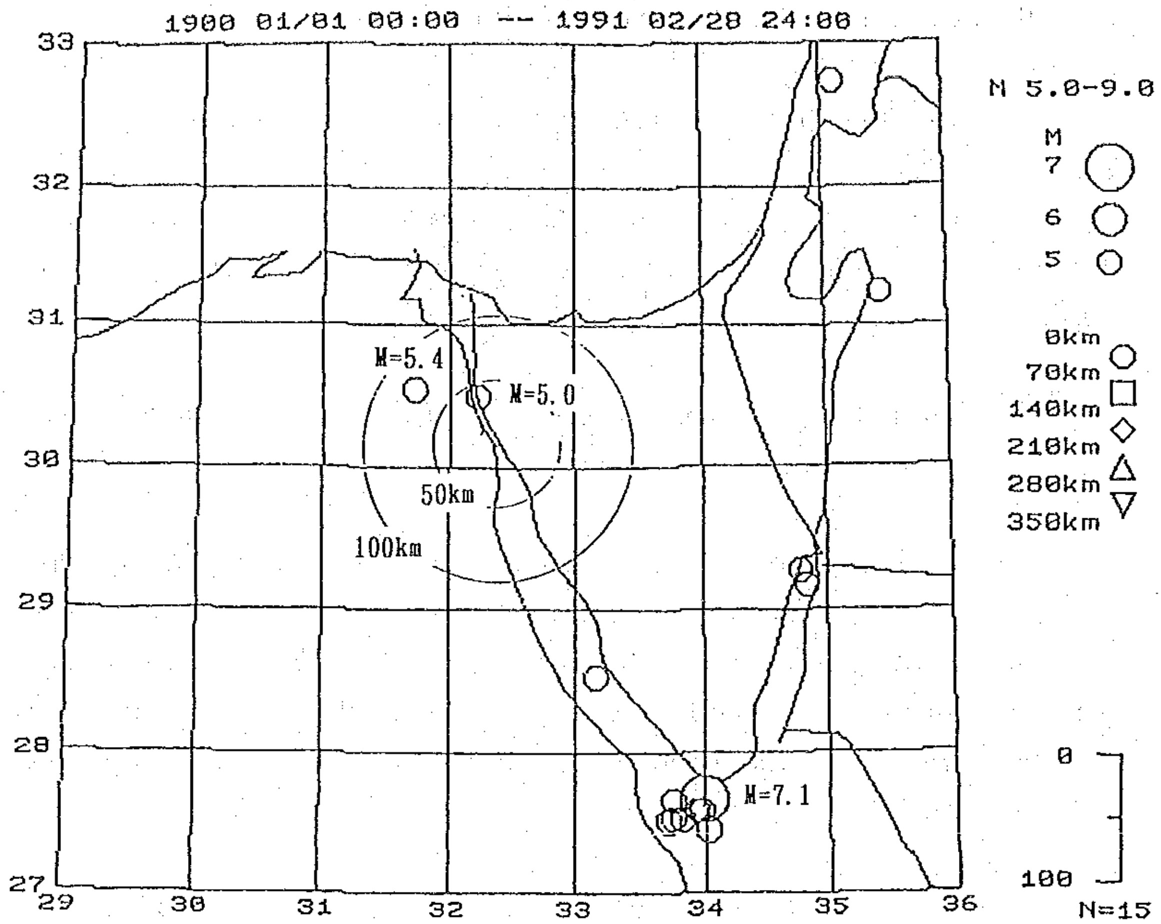


Fig.5.1.1 Past strong Earthquakes in Areas Adjacent to the Project Site

Table 5.1.1 Past Strong Earthquakes in Areas Adjacent to the Project Site

(Source: Focus catalogue of the Meteorological Agency)

1990. 1. 1 00:00 ~ 1991. 2. 28. 24:00

Magnitude 5.0 — 9.0 Depth: 0 — 350 m Area 1

N	Y	M D H MIN	Latitude	Longitude	Depth	M
1	1969	3/24 11:54	27° 31.7'	33° 49.7'	21	5.2
2	1969	3/31 7:15	27° 40.1'	33° 59.2'	33	7.1
3	1969	3/31 21:44	27° 27.7'	34° 1.7'	6	5.0
4	1969	4/ 8 10:31	27° 30.2'	33° 43.3'	15	5.2
5	1969	4/16 8:12	27° 35.2'	33° 58.1'	33	5.0
6	1969	4/23 13:37	27° 35.6'	33° 56.5'	28	5.0
7	1972	1/12 8:15	27° 31.6'	33° 44.9'	54	5.1
8	1972	6/28 9:49	27° 38.9'	33° 45.6'	15	5.6
9	1974	4/29 20: 4	30° 31.7'	31° 43.3'	33	5.4
10	1979	4/23 13: 1	31° 14.6'	35° 27.7'	33	5.1
11	1983	2/ 3 13:46	29° 9.9'	34° 49.9'	10	5.1
12	1983	2/ 3 23:30	29° 16.3'	34° 46.0'	10	5.1
13	1983	6/12 12: 0	28° 30.5'	33° 10.3'	10	5.0
14	1984	8/24 6: 2	32° 44.3'	35° 5.6'	24	5.0
15	1987	1/ 2 10:14	30° 28.8'	32° 13.3'	24	5.0

From these records, it is assumed that crustal alterations may occur in areas facing the Red Sea. The magnitude of alterations may be assumed as:

Epicentral distance	: within a radius of 100 km from the structure
Focus depth	: approximately 20 to 40 km
Magnitude	: M = 5 min

In this analysis, the following earthquake magnitude is assumed taking into consideration the importance of the structure.

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, which means earthquake-resisting bed rock here, based on the data from strong earthquakes recorded at ports and harbors in Japan, are shown in Fig.5.1.2.

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

αmax = 215, for which M = 7 and effective distance = 30 km

It is desirable that acceleration waveforms be determined based on the records of earthquakes similar in magnitude to the assumed earthquake or the actual record in an adjacent area of the subject structure. Since the desired records of earthquakes of an assumed magnitude at a specific site are rarely available, the past records of strong earthquakes equivalent in magnitude and with similar ground characteristics are normally used.

It is understood from the drilling log shown in Fig. 5.1.4 that the ground at the site of this design project is covered with silty clay to depths from 6.5 to 20.0 meters, with a substratum consisting of mudstone below the depth of 20.0 meters. The ground is estimated to be very solid, being 290 m/sec for clay soil and 300 m/sec for mudstone measured in wave velocity (Vs) per second, which was obtained by PS logging.

The observed waveform at El Centro (officially called "Imperial Valley") is a representative seismic wave and is widely used in seismic designs, as is the assumption of such solid ground and seismic magnitude M = 7.

The El Centro seismic waves will be used as an input waveform in this project, since ground conditions and seismic magnitudes are similar in this assumption. The actual El Centro waveform is shown in Fig. 5.1.3.

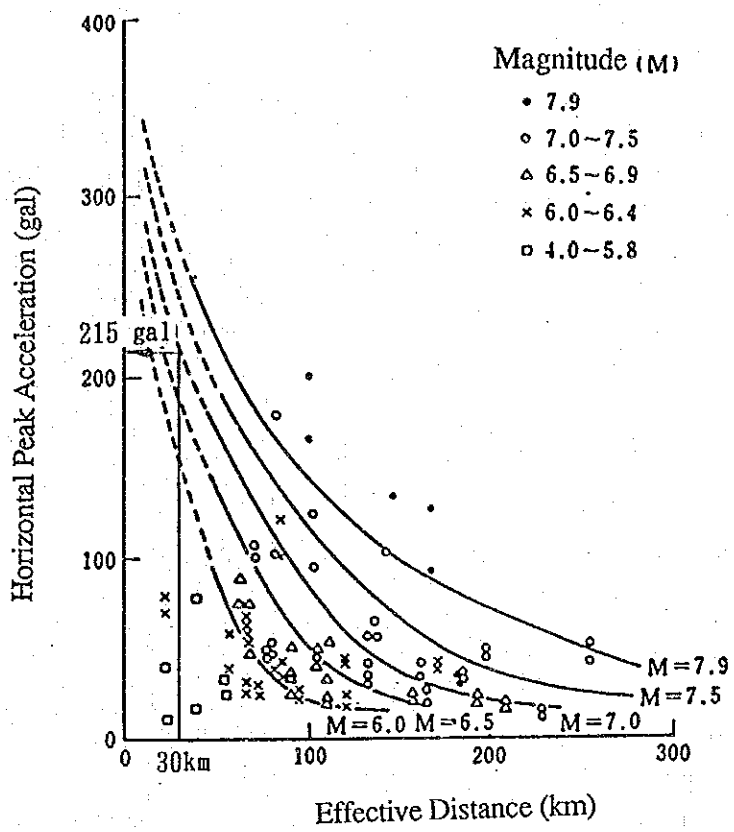


Fig.5.1.2 Dsistance Damping Curves at Peak Acceleration of Bed rock

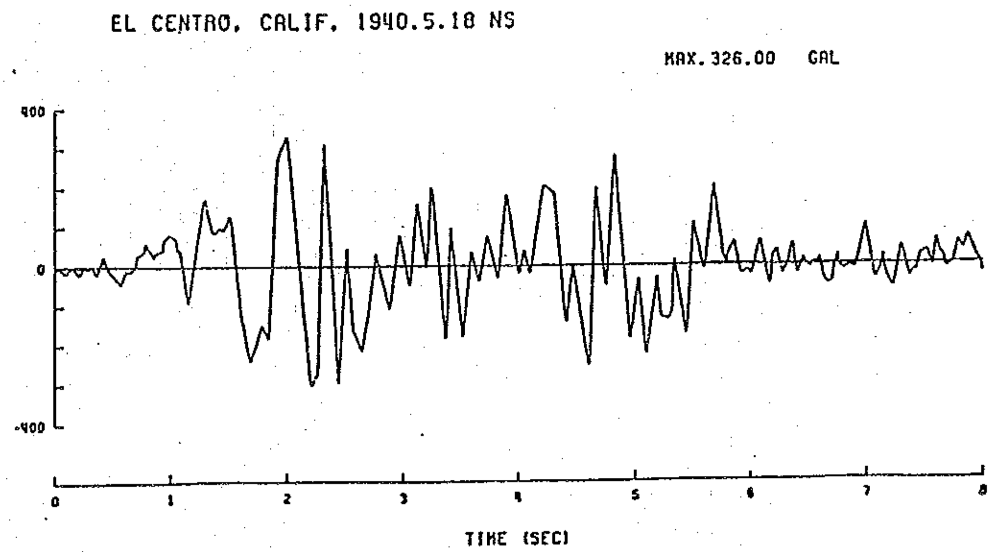


Fig.5.1.3 Seismic Waves of El Centro

DRILL LOG

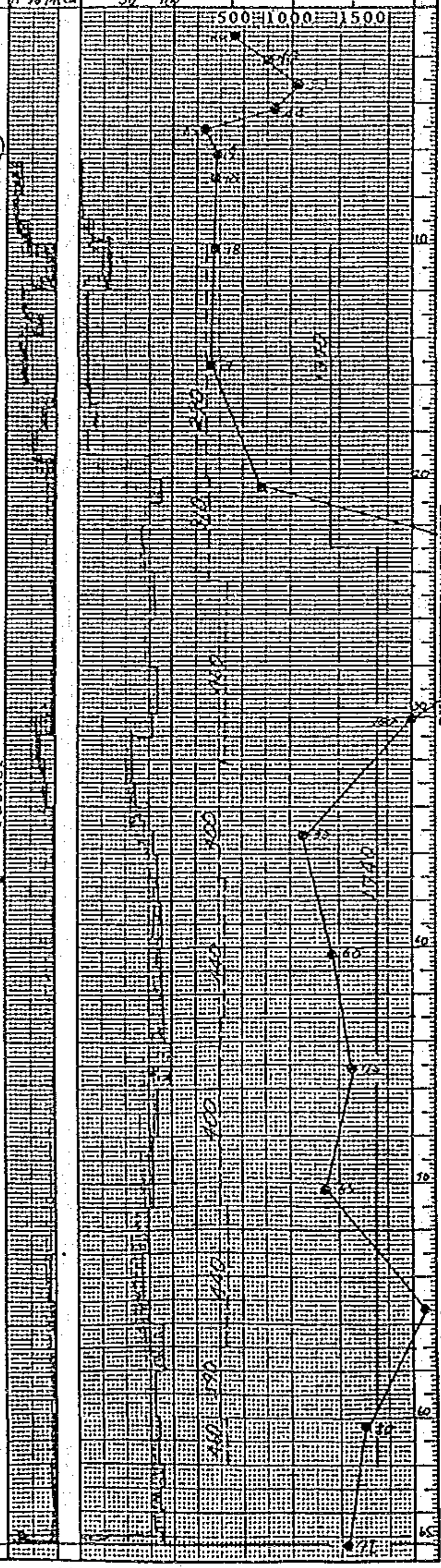
HOLE NO. BH-2 SHEET NO. 1 OF 1

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
1/9	4.50	+0.40	TAILINGor 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
	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.
	24.00	-19.10	Mode-rately 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
12/9	65.00	-60.60	Mode-rately Cemented MUDSTONE (39.1 – 50.7 Tunnel Formation)	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.

GROUNDWATER LEVEL: Stable during drilling; Equilibrium Table; Packer; Test, Dewatering

CORE RECOVERY | R Q D.% | S P T (N-Value) 25 50 75 | PS LOGGING (m/sec.) --S wave — P wave



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

NIPPON KOEI CO., LTD.

LOG FORM-3

Fig.5.1.4 DRILL LOG OF BH-2

5.1.2. Soil Parameters

Soil parameters used in seismic design are shown in Table 5.1.2.

Table 5.1.2 Soil Parameters

*1 Layer	Static Secondary Wave Velocity Vs (m/s)	Dynamic Secondary Wave Velocity Vs (m/s)	Shear Rigidity G *2	Unit Volumetric Weight γ (tf/m^3)	Poisson's Ratio ν
Ob	200	100	1990	1.95	0.48
Ch	290 - 310	150	4477	1.95	0.47
Mdm & Mdw	444 (460,400,440, 390,460) *2	222	9807	1.95	0.465

*1 : Ob - Surface Soil
Ch - Hard Clay
Mdm - Moderately Cemented Mudstone
Mdw - Wall Cemented Mudstone

*2 : $G = (\nu / g) Vs^2$ $G = 9.8\ m/s^2$

*3 : Weighted mean 444 m/sec. has been obtained by multiplying each layer by the height of layer

5.1.3. Structural Design Parameters

Structural design parameters are shown in Table 5.1.3.

Table 5.1.3 Structural Design Parameters

Item	Condition
Elastic coefficient	E = 350,000 (tf/m^2)
Sectional area	A = 12.44 (m^2)
Moment of inertia of area	I = 153 (m^4)

The earthquake resisting bed rock is defined as the basement which will not be affected by subsurface layers and can behave elastically deep in the ground, different from the definition used in geotechnics. It is generally limited to bed rock, or to dilluvial deposits having a bearing strength "N" of 50 or more. (Ref. Fig.4.1.4 or Page 4-15)

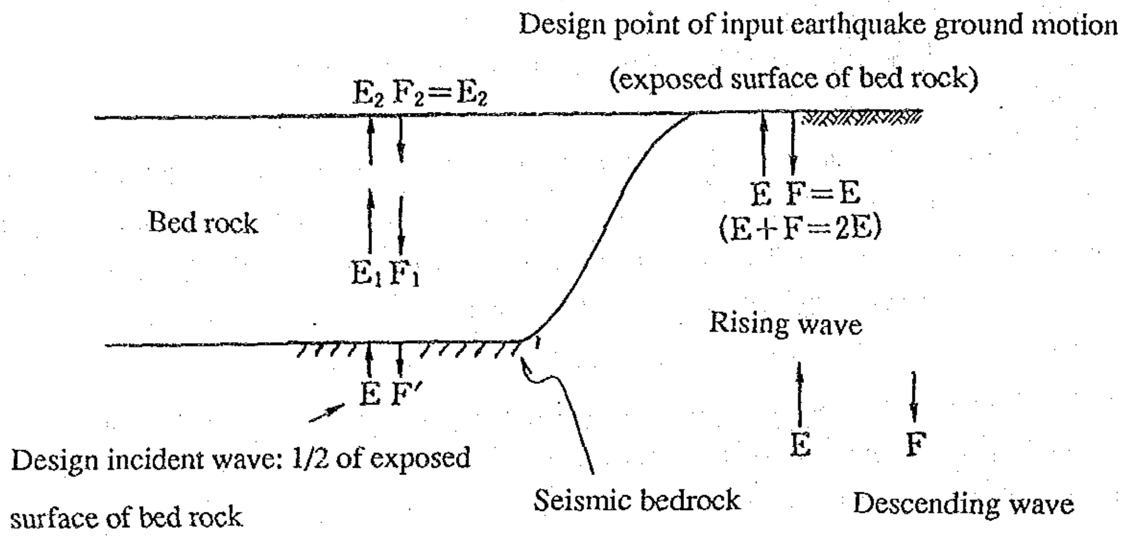
In this design project, the ground surface at a depth of 100 meters, where the "N" value is stabilized, will be assumed as bed rock as determined by the results of soil exploration.

The El Centro seismic waves will be considered as an earthquake ground motion at the exposed surface of bed rock. The earthquake ground motion, divided by two, will be given as an input earthquake ground motion (one half of the acceleration waveform and the maximum amplitude: α max = 215 gal/2 = 107.5 gal) for bed rock.

5.1.4. Method of Analysis

Seismic response analyses will be made by the dynamic FEM method using a program named "FLUSH" on a three-dimensional pseudo model. This three-dimensional pseudo modeling procedure has been adopted because a principal section together with a sub-section that reproduces natural ground in depth can be established, while a tunnel model with a depth is made by normal two-dimensional modeling procedures. The three-dimensional pseudo model has also a characteristic to make it possible to evaluate dissipating effects of seismic waves on the sub-section (out-of-plane direction).

Fig.5.1.5 is a sketch showing the concept of a three-dimensional pseudo model. Fig.5.1.6 shows the analytic model (mesh system data) used in this analysis.



Conceptual Sketch of Input Earthquake Ground Motion at Bed Rock

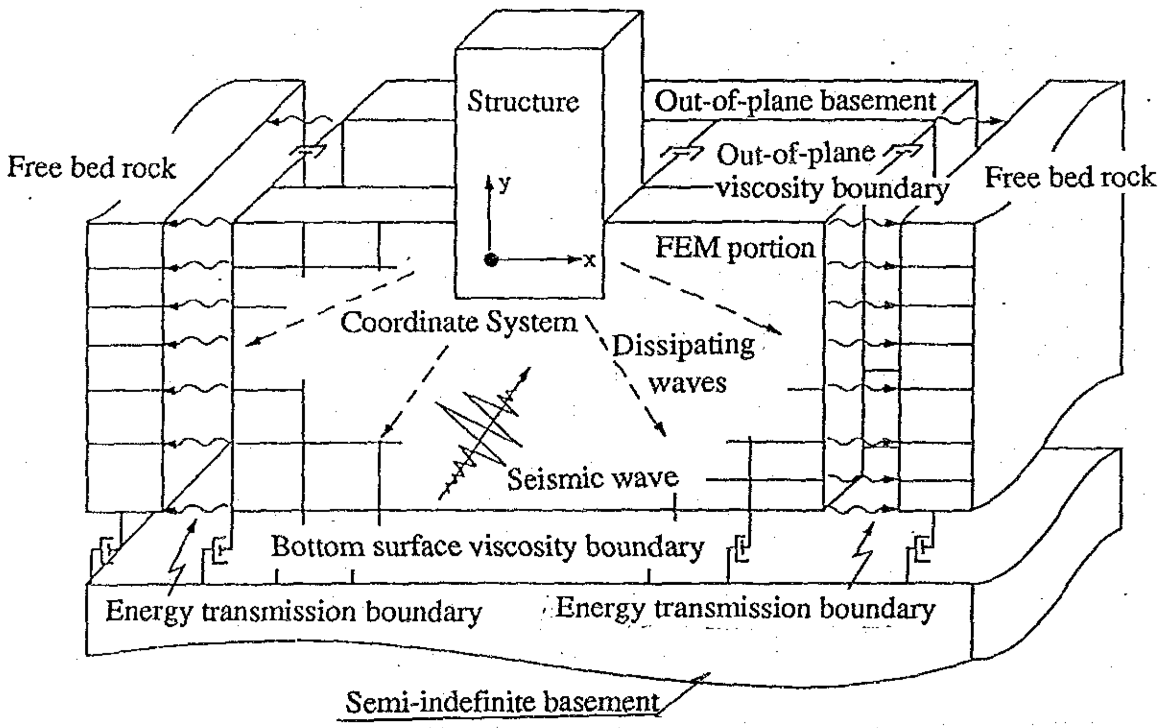


Fig.5.1.5 Conceptual Sketch of Three-Dimensional Pseudo Model

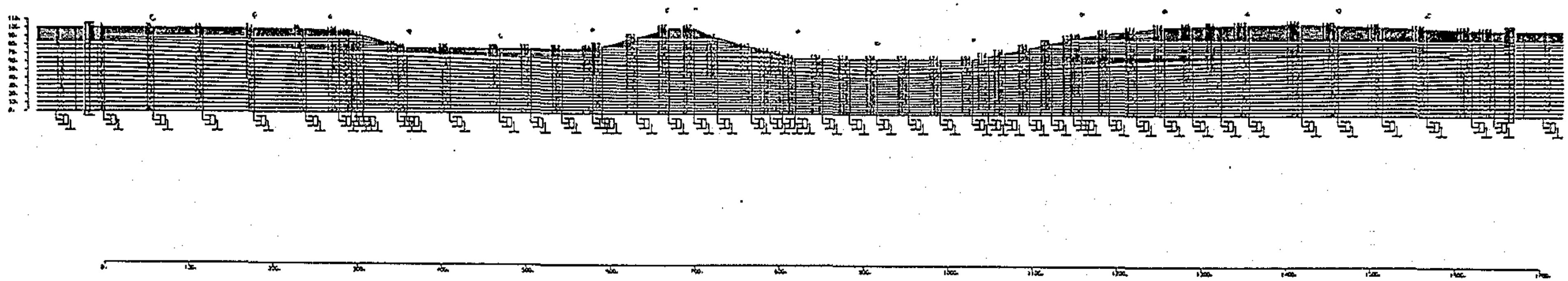


Fig.5.1.6 Three-Dimensional Pseudo Model Used in Analyses

5.1.5. Results of Analyses

The largest strains occurring between joints in the center of the Tunnel are shown in Table 5.1.2 from the results of analyses. These values have been obtained by dividing the largest relative displacement occurring between joints in the center of the Tunnel due to the distance in between. The negative symbols show tensile strain.

It is understood from Table 5.1.4 that the largest strain is occurring at locations shown in Fig.5.1.7, and the value of strain (g_{max}) is between 4,400 and 4,500μ. All other areas indicate a value under g_{max} = 4,000μ.

Fig.5.1.8 shows the distribution of horizontal displacement at the time of the largest strain, g_{max} = 4,400 to 4,500μ, for reference purpose.

Table 5.1.4 Maximum Strain at the Tunnel Center

NO.	JOINT NO.	Max. Strain ($x10^{-6}$)
1	9 - 33	423
2	33 - 57	-476
3	57 - 82	-1018
4	82 - 108	-205
5	108 - 134	1289
6	134 - 160	2423
7	160 - 187	2806
8	187 - 210	2396
9	210 - 232	2495
10	232 - 254	1771
11	254 - 276	1291
12	276 - 307	185
13	307 - 329	-1507
14	329 - 351	-3265
15	351 - 364	-4533
16	364 - 386	-4475
17	386 - 411	-3311
18	411 - 441	-1070
19	441 - 471	818
20	471 - 496	2153
21	496 - 518	3391
22	518 - 538	3281
23	538 - 558	2595
24	558 - 578	2168
25	578 - 598	2409
26	598 - 618	2399
27	618 - 638	1244
28	638 - 658	-792
29	658 - 678	-2120
30	678 - 698	-1651
31	698 - 718	-1080
32	718 - 739	-1497
33	739 - 762	-2835
34	762 - 786	-3280
35	786 - 812	-2322
36	812 - 840	-1404
37	840 - 868	-301
38	868 - 898	1459
39	898 - 932	2750
40	932 - 964	3183
41	964 - 997	2879
42	997 - 1028	2434
43	1028 - 1059	1412
44	1059 - 1089	-69
45	1089 - 1118	-226
46	1118 - 1147	146
47	1147 - 1175	-743
48	1175 - 1203	-565

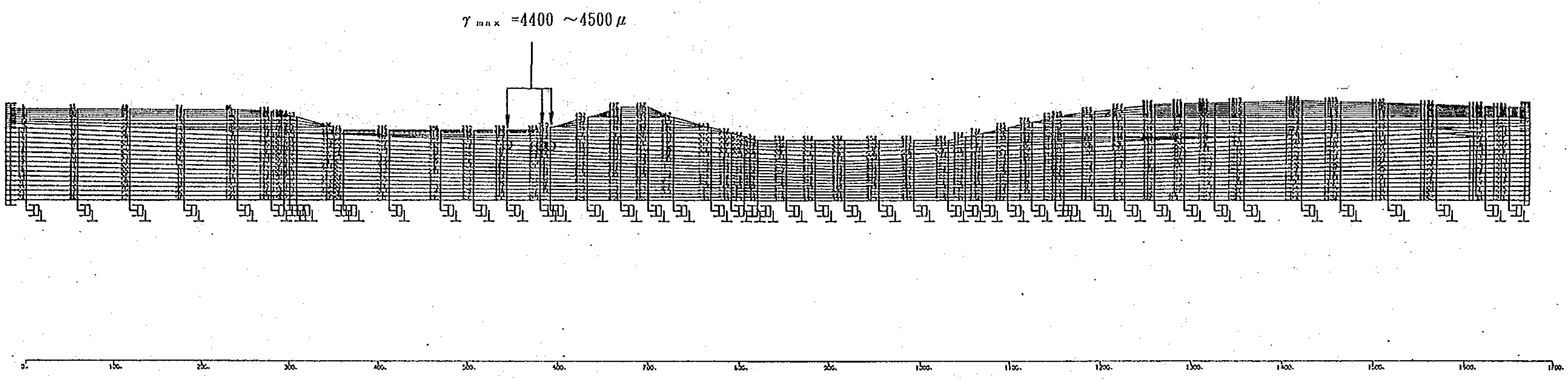


Fig.5.1.7 Location Where Maximum Strains Occurred

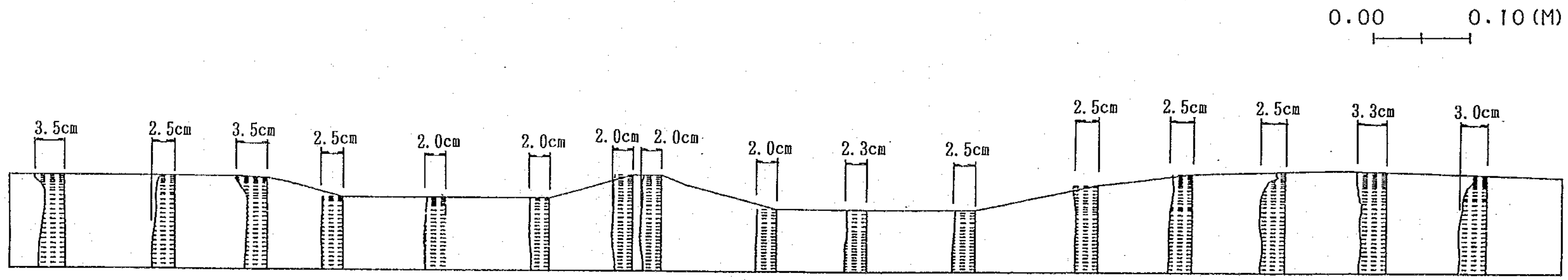


FIg.5.1.8 Distribution of Horizontal Displacment at TIme of Maximum Strain

5.1.6. Safety Evaluation

*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 (ϵ_{sy} = 2000µ), and will cause the residual strain. However, this (max) is smaller than the restoring limit strain (ϵ_{sh} = 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.

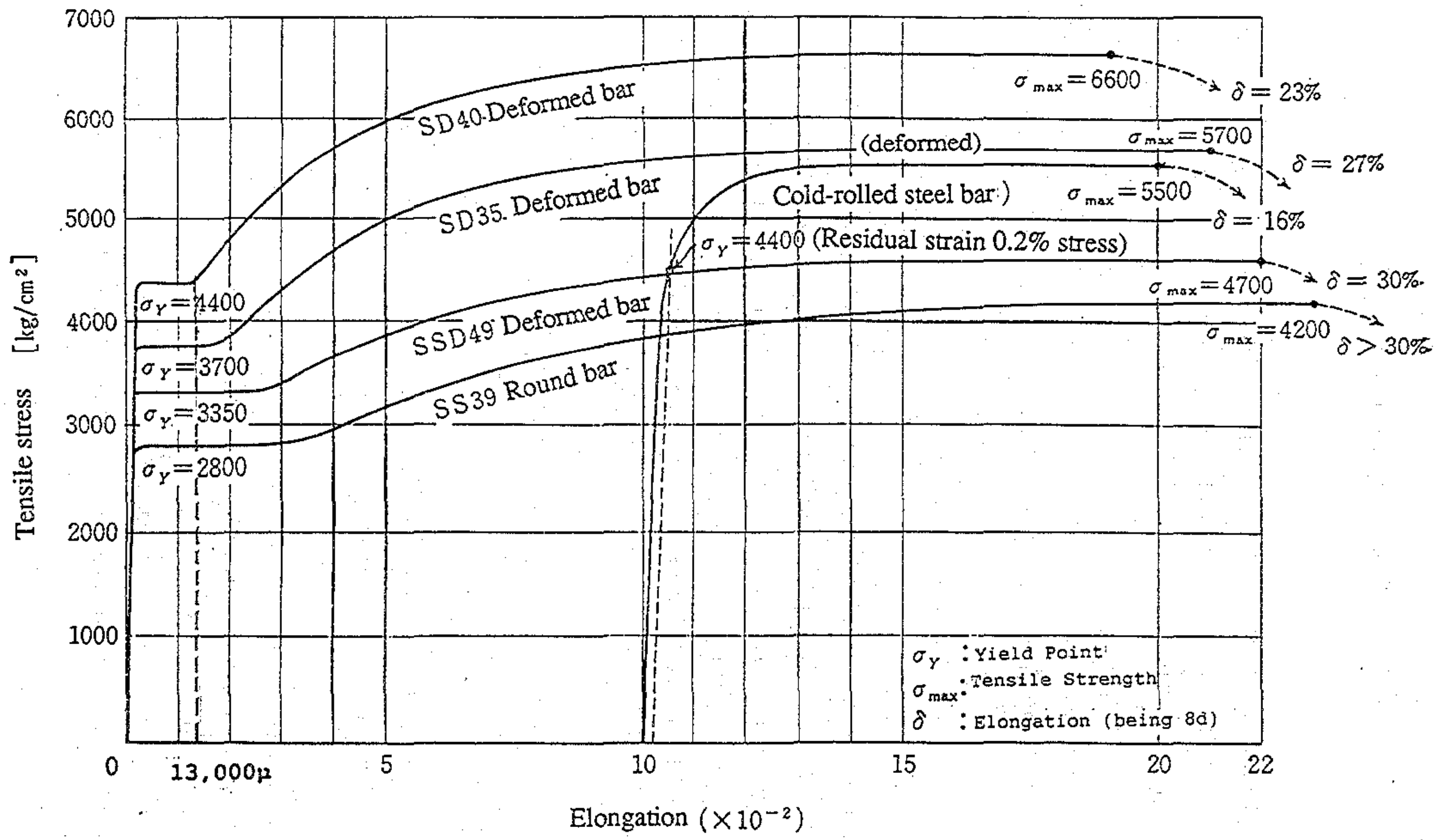


Fig.5.1.9 <u>Stress Strain Diagram of Various Steel Bars</u>

5.2. Design by Response Displacement Method

The earthquake resisting design of the Tunnel will be provided by using the response displacement method based on the " Design Guidelines for Underground Multi-Purpose Lifeline Duct" of the Japan Road Association.

This analysis method is for Tunnel and soil medium moving in the same manner when the Tunnel is under the earthquake.

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

5.2.1. Design Conditions

The location of the Tunnel is shown in Fig.5.2.1. The design bed rock surface is assumed to be 70 meters below the bottom of the Canal (approximately 50 meters below the bottom of the Tunnel).

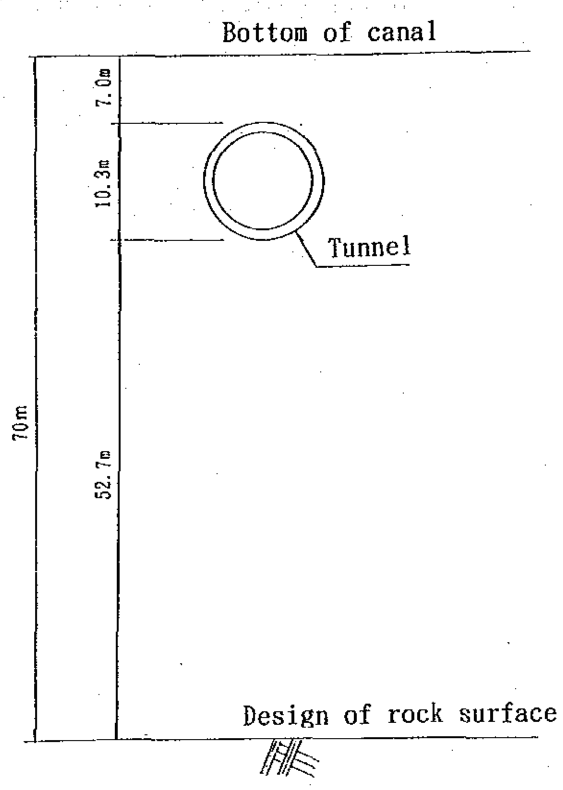


Fig.5.2.1 Bed Rock Model for Seismic design Analyses

The design conditions are shown in the followwing two tables.

Table 5.2.1 Calculation Condition

Thickness of Subsurface Layer	(m)	70.00
Velocity of Secondary Wave	(m/sec)	1000.00
Depth of Tunnel Center	(m)	12.80
Elastic Modulus of Tunnel Lining	(tf/m^2)	2650000.00
Cross-Sectional Area	(m^2)	12.44
Moment Inertia	(m^4)	152.66

Table 5.2.2 Condition of Ground

Thickness of Subsurface Layer	(m)	70.00
Average Density of Soil	(tf/m^2)	1.95
Average Velocity of Second Wave	(m/sec)	300.00

5.2.2. Determination of Sectional Force by Response Displacement Method

The displacement of subsurface layers occurring during earthquakes is assumed, by the response displacement method, to be caused by propagating sine seismic waves shown in Fig.5.2.2. Section force will be determined using the following equations based on the beam theory for elastic foundation.

$$\left.\begin{aligned}
P_h &= \alpha_1 \cdot \xi_1 \frac{\pi E \cdot A}{L} \cdot U_h \\
P_v &= \alpha_1 \cdot \xi_1 \frac{\pi E \cdot A}{L} \cdot \frac{U_h + U_v}{2} \\
M_h &= \alpha_2 \cdot \xi_2 \frac{4\pi^2 E \cdot I_h}{L^2} \cdot U_h \\
M_v &= \alpha_3 \cdot \xi_3 \frac{4\pi^2 E \cdot I_v}{L^2} \cdot U_v
\end{aligned}\right\} \cdots\cdots\cdots\cdots \text{(Equation 1)}$$

Where,

Ph, Pv : Axial force from seismic shocks within horizontal and vertical planes (t)

Mh, Mv : Bending moment from seismic shocks within horizontal and vertical planes (t.m)

E · A : Axial rigidity of Tunnel (t)

E · Ih : Bending rigidity within horizontal plane in tunnel ($t \cdot m^4$)

E·Iv : Bending rigidity within vertical plane in tunnel ($t.m^4$)

Uh, Uv : Horizontal and vertical displacement amplitude caused by seismic shocks at the depth of center of gravity of multi-purpose lifeline duct (m)

However, only the horizontal displacement amplitude is considered in this analysis. Therefore, Uv = 0.

Uh can be determined by the following equation.

$$U_h(z) = \frac{2}{\pi^2} S_v \cdot T_s \cdot \cos\frac{\pi z}{2H} \cdots\cdots\cdots\cdots \text{(Equation 2)}$$

Where,

Uh (z) : Horizontal displacement amplitude at the depth of (z) from ground surface (m)

Sv : *Design response rate (m/sec) for the proper* period (Ts) of subsurface layers and seismicity zoning can be obtained from Fig.5.2.2 below

Ts : Proper period of subsurface layers (s)

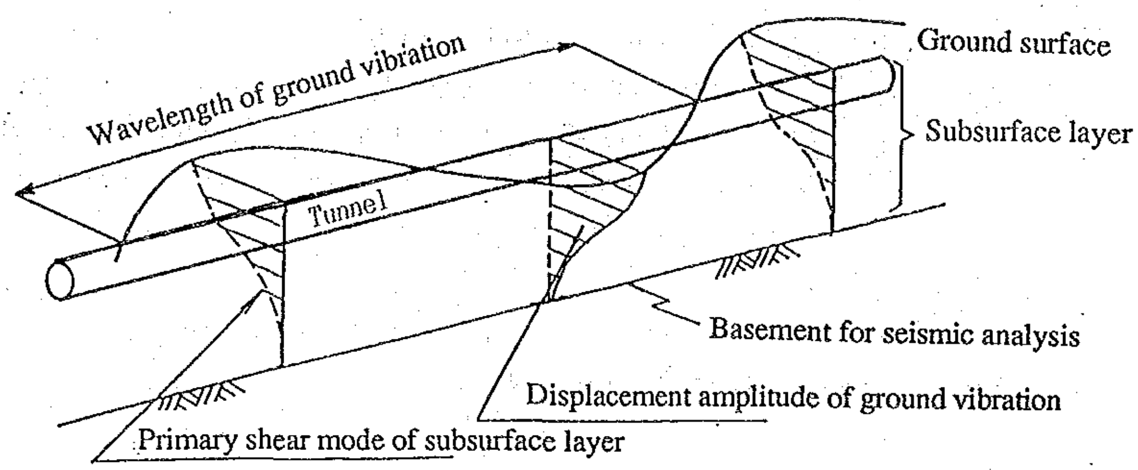
L is the wave length of ground vibration which can be obtained by the following equation.

$$\left.\begin{array}{l} L = \dfrac{2L_1 \cdot L_2}{L_1 + L_2} \\ L_1 = V_{DS} \cdot T_s \\ L_2 = V_{BS} \cdot T_s \end{array}\right\} \quad \cdots\cdots\cdots \quad \text{(Equation 3)}$$

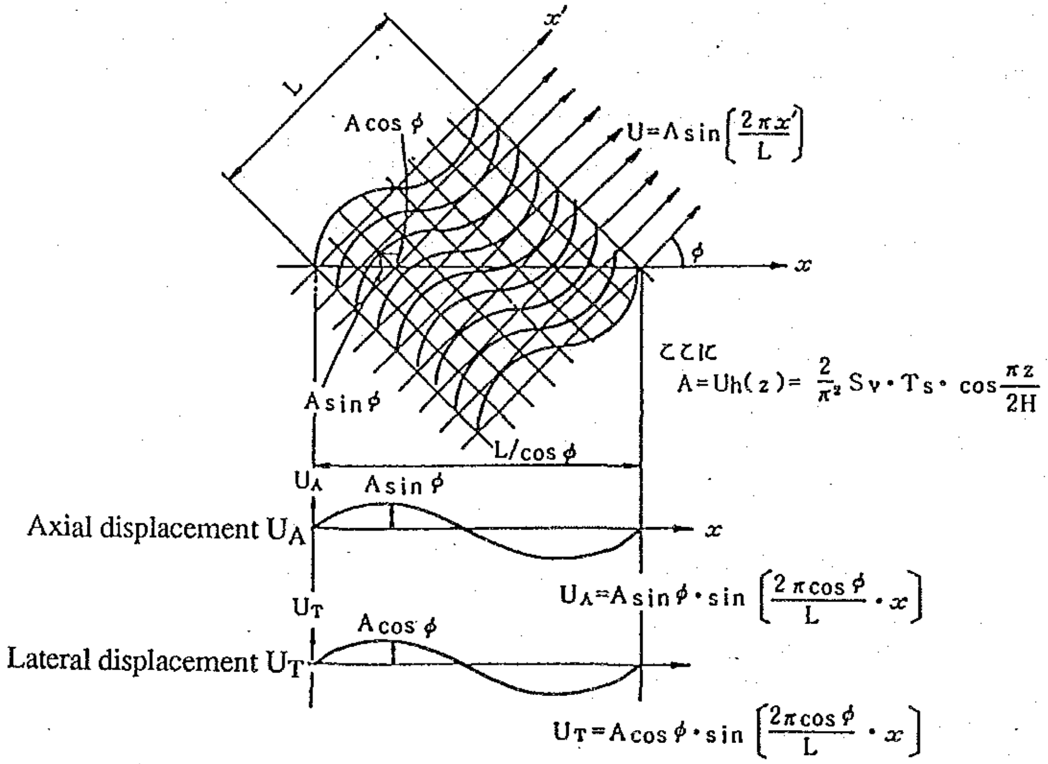
Where,

V_{BS} : shear elastic wave velocity of bed rock (m/sec)

V_{DS} : shear elastic wave velocity of subsurface layers (m/sec)



Wavelength of ground vibration and displacement amplitude



Displacement Caused by Earthquake Ground Motion

Fig.5.2.2 Conceptual Sketch of Displacement of Subsurface Layeres Shown by Response Displacement Method

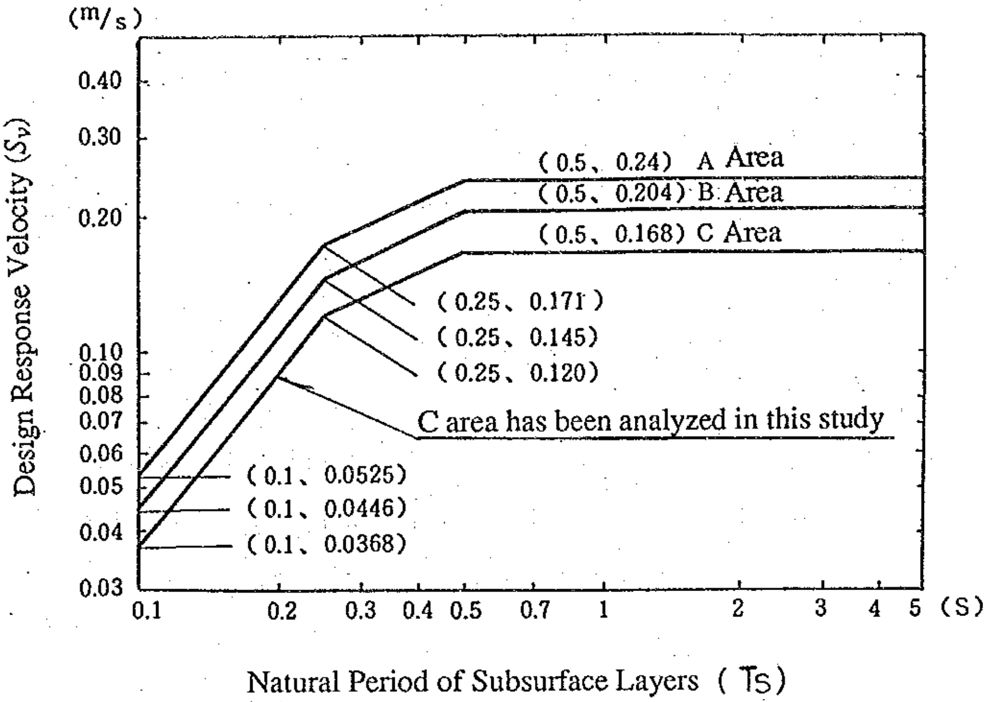


Fig.5.2.3 Design Response Velocity

1-67

5.2.3. Results of Analyses

The results of analyses are shown in Table 5.2.3 below.

Table 5.2.3. Results of Analyses

Axial force	P (tf)	8541.23
Bending moment within horizontal plane	M (tf·m)	2106.38
Bending moment within vertical plane	M (tf·m)	1054.12

5.2.4. Safety Evaluation

The safety at maximum ultimate strength will be verified here by the limit state design method. It is assumed that axial forces will not occur on the Lining because of the isolation effect of the waterproofing sheets.

The moment estimated to occur will be 2106 tf·m. The structure is estimated to be safe, since the maximum ultimate strength (Mud) is 7615.9 tf·m when the axial force of a single reinforcing bar is zero.

M = 2106 tf·m > Mud = 7615.9 tf·m ...... OK

6. Design for Effects of Cruising Ships

6.1. Design for Wreck Load

An increase of load due to sinking of a ship varies, depending upon how the weight of the ship changes upon sinking and how the ship contacts the bottom of the Canal. Here, the increased load due to sinking is simply assumed to be a quotient of the displacement tonnage of the ship divided by its bottom area.

The allowable unit stress of the lining against the wreck load can be assumed to be 50% more of the allowable unit stress against the permanent load because the wrecked ship is likely to be left on the canal bottom constructed above the Tunnel only for a short period of time.

$$\sigma_{ca} = 1.5 \cdot 90\ \text{kgf/cm}^2 = 135\ \text{kgf/cm}^2$$
$$\sigma_{sa} = 1.5 \cdot 2{,}100\ \text{kgf/cm}^2 = 3{,}150\ \text{kgf/cm}^2$$

The Tunnel lining is designed to withstand an unit stress of maximum σ_c = 86.3 kgf/cm^2 even after the expansion work of the Canal has been completed. Regarding the wreck load as a downward bearing load, the allowable unit stress, σ_{ca} = 135 kgf/cm^2 indicates an allowance of 48.7 kg/cm2.

The following is the tolerable load strength obtained from the above result:

$$dP_{max} = 21.3\text{tf/m}^2$$

If a tanker size should be obtained by associating a tolerable load strength to a corresponding tanker size, this tolerable load strength is roughly associated with a 250 thousand-ton class tanker as Fig.6.1.1 indicates.

This conclusion suggests that the Tunnel lining would safely withstand a ship of 250 thousand tons or less when it has sunk directly above the Tunnel.

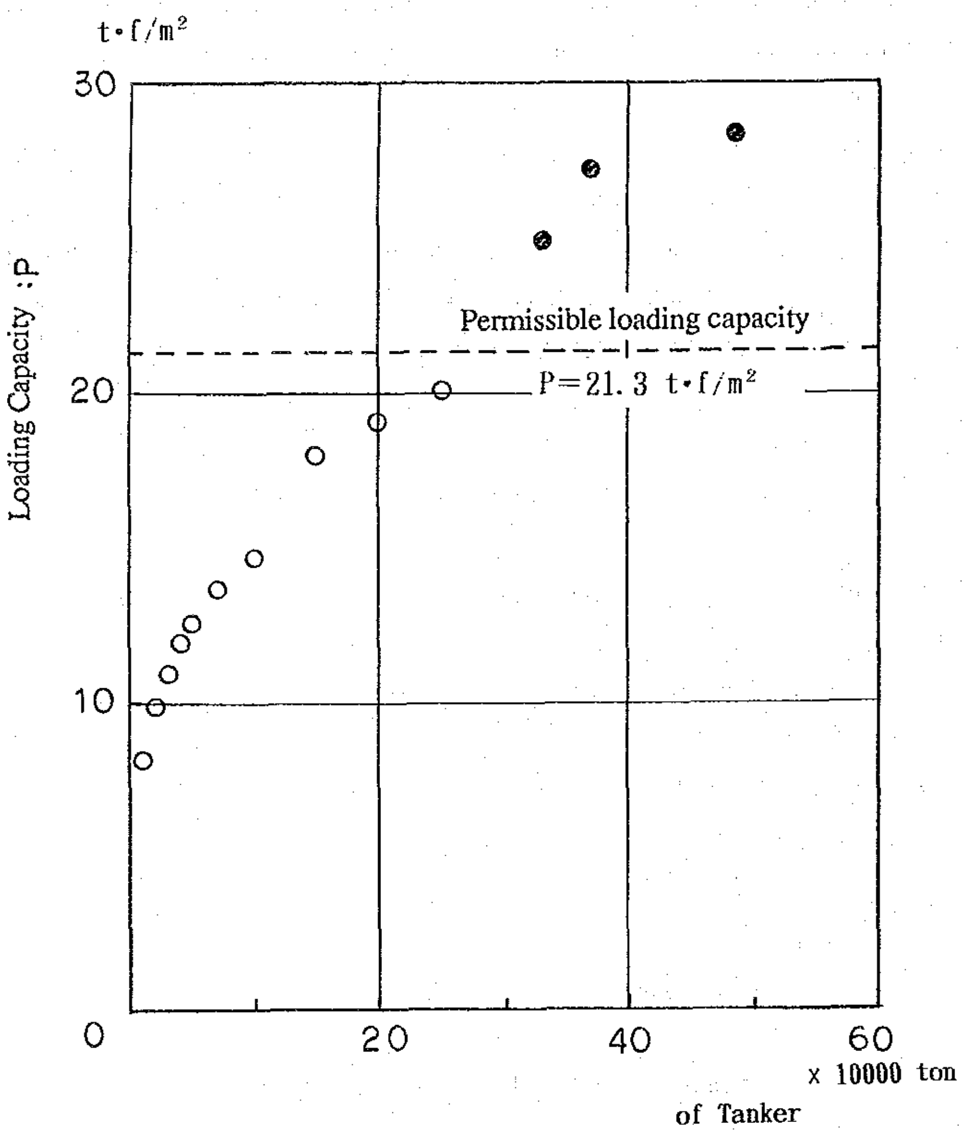


Fig.6.1.1 Size of Tankers vs Loading Capacity

6.2. Consideration on Impact Load Caused by Anchoring

We should like to quote the "Technical Standards for Port Facilities, and Explanation on the Standards", (corporation) Japan Port and Harbor Association, September 1980.

"2.2.9 Impact Load by Anchoring"

$$Wa = \frac{WD}{(L + 2 \cdot h \cdot \tan\theta_1)(B + 2 \cdot h \cdot \tan\theta_2)} (1 + i)$$

W : anchor weight in the water

D : external diameter of the conduit

L : length of the anchor bottom

B : width of the anchor bottom

h : depth of the conduit (tunnel) installation

θ_1, θ_2 : distribution angle

i : additional factor by impact

According to this Code, h = 3 to 4 m and i = 15 to 25. It is recommended to use such appropriate values based on experiments.

* It may be different from automobile load or train load.

After the final expansion work of the Canal has been completed, ships cruising in the Canal will include tankers as big as 500,000 tons.

According to "Port and Harbor Engineering Laboratory, Literature No.215, Amount of Anchor Penetration Obtained by Anchoring Tests", an anchor used by a 500 thousand ton class tanker is assumed to be 30 tf.

Similarly, the length and width of the anchor bottom were estimated based on the following expressions by referring to the specifications of the JIS type stockless anchor.

$$L = 0.60 \cdot \sqrt{W} + 0.56$$
$$B = 0.23 \cdot \sqrt{W} + 0.22$$

If $W = 30$ tf, $L = 3.85$ m, and $B = 1.48$ m.

The distribution angle was obtained by the following expression:

$$\theta_1 = \theta_2 = \frac{\pi}{2} - \phi_u = 45° - 15° = 30°$$

W, the weight of the anchor in the water, was obtained as follows, assuming the unit volume weight of iron being 7.85 t/m3, and the specific gravity of the sea water in the Canal being 1.05 t/m^3:

$$W' = W \cdot \frac{(ri - rw)}{ri} = 30 \cdot \frac{(7.85 - 1.05)}{7.85}$$
$$= 26 \text{ tf}$$

The impact load, Wa is expressed as follows:

$$Wa = \frac{26 \cdot 10.4}{(3.85 + 2 \cdot 7 \cdot \tan 30°)(1.48 + 2 \cdot 7 \cdot \tan 30°)} (1 + 15)$$
$$= 37.9 \text{ tf/m}^2$$

Where the load length of the impact load is either of the following two:

$$3.85 + 2 \cdot 7 \cdot \tan 30° = 11.9\text{m} \longrightarrow 10.4\text{m}$$
$$1.48 + 2 \cdot 7 \cdot \tan 30° = 9.6\text{m}$$

These load length can be regarded as larger than the external diameter of the Tunnel or very close to it. It is treated as an additional bearing load.

The anchoring impact load(Wa), 37.9 tf/m^2, is far greater than the tolerable load strength, 21.3 tf/m^2, leading to the conclusion that the safety of the lining of the Tunnel will be threatened when anchoring is conducted.

Based on the above discussion, it should be concluded that the particular care must be taken of cruising in the water area above the Tunnel. In particular, sinking or anchoring must be avoided in this water area.

7. 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.7.1.1)
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.7.1.1)

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 reinforce ment steel bar (See Fig.7.1.2 and Refer to Table 7.1.1), 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 7.1.1 is a summary of our protection plan.

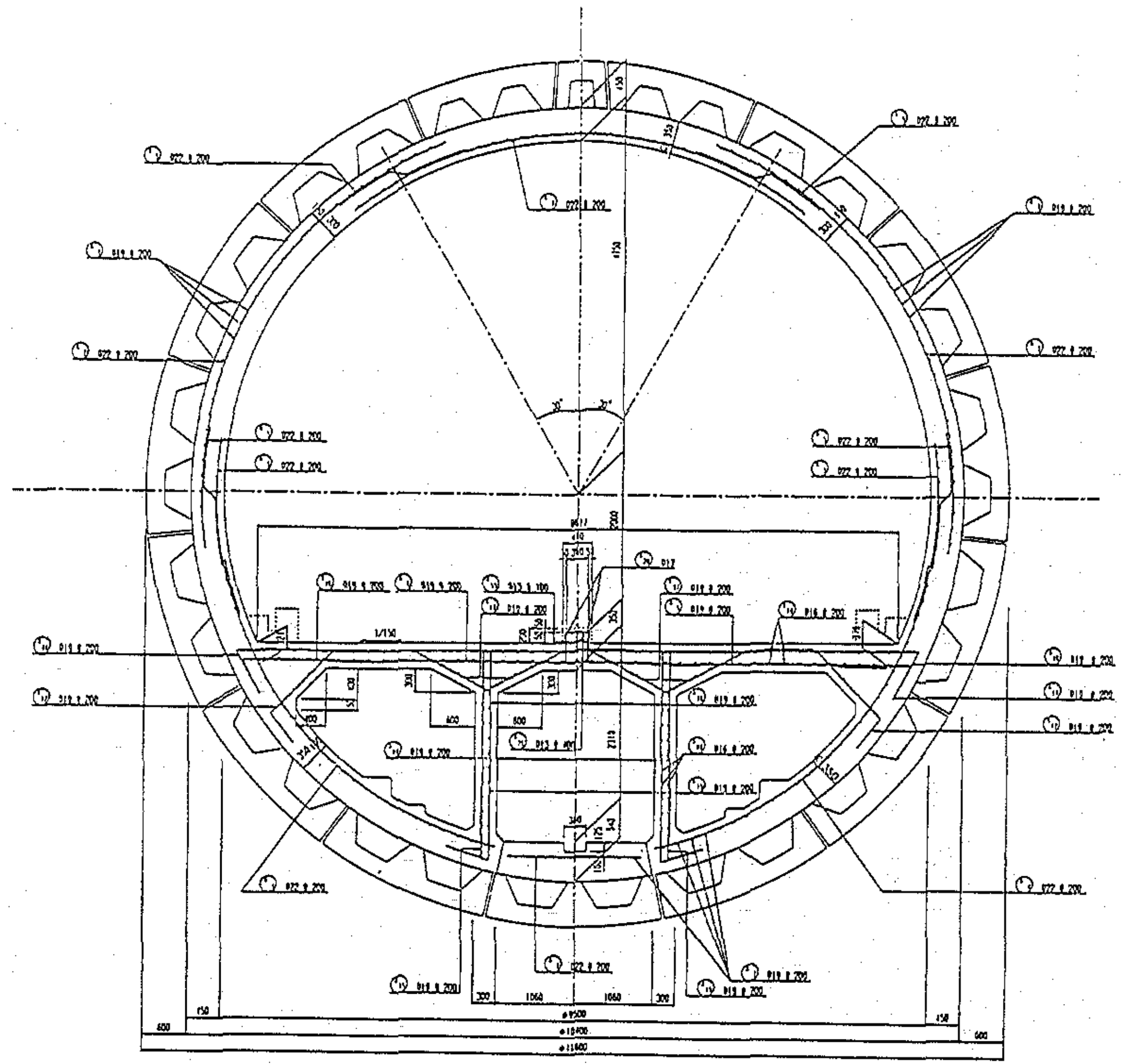


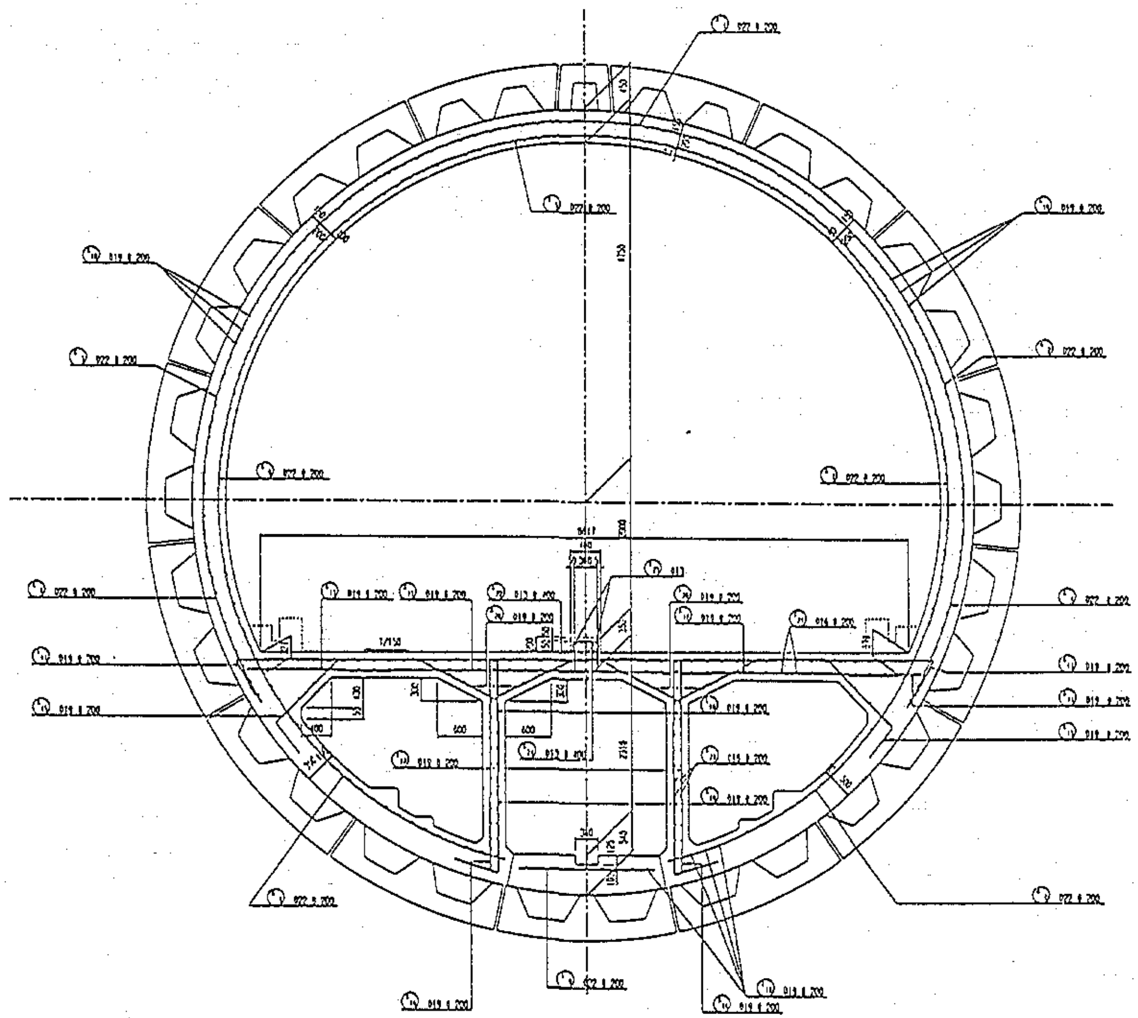
Fig.7.1.1 General Arrangement of Reinforcing Steel Bar

LIST OF REINFORCEMENT

MARK	DIA	LENGTH	EACH	[illegible]	[illegible]	WEIGHT	REMARKS
PER ONE SEGMENT (L = 10,500 m)							
1	Ø22	6 620	52	2.98	20.1	1 045	
2	"	6 820	104	"	20.8	2 163	"
3	"	2 160	104	"	6.41	778	"
4	"	5 570	104	"	14.8	1 718	"
5	"	1 820	208	"	5.53	1 150	
6	"	1 870	52	"	5.44	283	
7	Ø19	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	"	1 170	104	"	2.76	287	
13	"	2 510	104	"	5.65	588	
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	Ø16	10 200	142	1.56	15.9	2 258	
19	Ø13	400	56	0.995	0.398	22	
20	"	500	27	"	0.498	13	
					Ø22	7 187 kg	
					Ø19	9 482 "	
					Ø16	2 258 "	
					Ø13	36 "	
					TOTAL	18 963 kg	

AHAMED HAMDI TUNNEL		
GENERAL ARRANGEMENT OF REINFORCING STEEL BAR		
Scale	Date	Review
JICA		

33



LIST OF REINFORCEMENT

MARK	DIA	LENGTH	EACH	[illegible]	[illegible]	WEIGHT	REMARKS
		PER ONE SEGMENT RING (L = 48,500 m)					
1	D22	6 060	52	3.04	18.4	957	
2	"	7 000	52	"	21.3	1 110	"
3	"	6 310	52	"	19.2	998	"
4	"	2 650	104	"	8.06	838	"
5	"	5 850	52	"	17.8	925	"
6	"	7 560	52	"	23.0	1 196	"
7	"	6 780	52	"	20.6	1 071	"
8	"	3 590	104	"	10.9	1 132	"
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	9 355 kg	
					D19	10 887 "	
					D16	2 258 "	
					D13	54 "	
					TOTAL	22 554 kg	

Fig.7.1.2 Arrangement of Reinforcing Steel Bar Under the Canal

AHAMED HAMDI TUNNEL

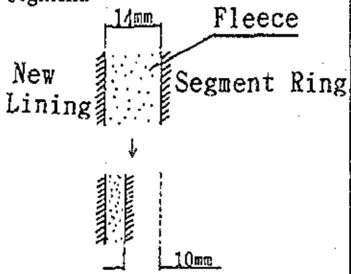
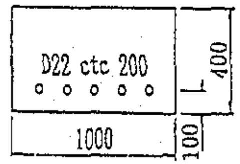
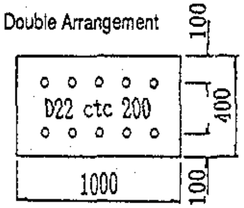
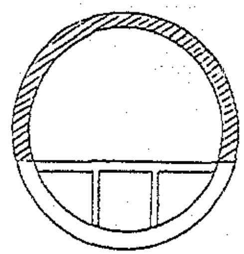
ARRANGEMENT OF REINFORCING STEEL BAR UNDER THE CANAL

Scale		Date		Review	

JICA

34

Table 7.1.1 Protection Plan of Tunnel Structure from 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 ease of construction. (maximum volume will be: D 22 ctc 200). Therefore, cracks cannot be prevented with this method. Single Arrangement  Double Arrangement 	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	△	-	○	○	○	△

Appendix-7

Data Provided by the Governor of Ismailia (from S.C.A.)

REPUBLIC OF EGYPT
CANAL AUTHORITY

August 21, 1991
of Works

: 064-220030/9 Ismailia

220784
220785

جمهورية مصر العربية
هيئة قناة السويس

١٩ ن

إدارة :
رقم القيد
رقم الملف :
عدد المرفقات

-1/3

Subject : - Data given from his Excellency the Governor of Ismailia, Professor Dr. Ahmed El-Gowely, in his office to the Japanese experts (JICA) Mr. T. OHTSUKA and Mr. T. TSUNASHIMA on Sunday 11/8/91 (from 10 to 11 a.m.)

Concerning: - The future Development plan of Sinai and the relationship between these projects and Ahmed Hamdy Tunnel

1) - The Canal Area devided into : -
North Sinai Governorat, South Sinai Governorat, Port-Said Governorate, Ismailia (and East of the Canal) Governorate, Suez Governorate and Sharkia Governorate.

2) - The Area of Ismailia Governorate: 1158929 feddans (x4200 m2).
This area including the lands from Al-Kantara to Sarabium (east and west the Canal).
* 488550feddans lands east the Canal in Sinai.
* 626122 feddans lands west the Canal.
* 52013 feddans the water area of the Canal and the lakes.

3) - Water sources in Sinai: -
South part: The syphon of Sarabium (km. 93.15)
North part: The syphon of km. 27 which is an extension of Salam Canal which is a branch of the River Nile (Damieta branch). They have terminated the first stage and they will start the second stage immediatly. That stage includes the operation of passing the syphon under the Canal water, and it is expected that this stage will be terminated within two years. That stage is financed by a Kuwait fund.

./..

-2/3

4) - Lands Reclamation: -
That project includes 400000 feddans out of which 75000 feddans belong to Ismailia Governorate and the rest belong to North Sinai Governorate.

5) - The labour hands required to reclame the land at least 2 persons per feddan, one for cultivation process and one for the service of the land, and the result is 800000 job opportunity.

Since the average number of the family of each worker is 5, the total volume of immegrants coming from the west of the Canal to Sinai (North Sinai and the easter Ismailia) becomes 8000 x 5 = 4000.000 people at least (between 4 and five million people).

6) - Prior projects: -
- Agriculture: cultivating fruits, vegetables and olives.
- Agroindustries such as storting, processing and packing agricultural products, that kind of industries that mainly depend on agriculture, and industries that depend of the existence of quarries as lime quarries, marble quarries .... etc.

7) 7) Tourism

Many tourist villages and hotels including playgrounds and green ares have been established on the beaches of the Bitter lakes and the Canal, in addition to many other projects that will be established during the development plan of the next stage 1992/1997. These areas are characterized by their being near from Cairo, the pure nature and the marvellous nature of the beaches.

8) 8) Infrastructure projects which are necessary for the immigrants and the merger of the east and west regions of the Canal including building primary, preparatory and secondaschools, agricultural and industrial training centers, hospitals, in addition to establishing internal transportation roads.

./...

31

9) - Marketting: -

Arranging markets inside and outside Egypt for the agricultural and industrial products of Sinai.

10) - The present number of inhabitants in Ismailia

200000 in Sinai

700000 in the area west the Canal .

In holidays and festivals, Ismailia receives about one million Egyptians guest / year.

11) - Means of connection between east and west the Canal

Ahmed Hamdy Tunnel is considered one of the most important means for connecting the Delta and the Nile valley to Sinai and the project of the rehabilitation of Ahmed Hamdy Tunnel is very important and vital to excute the development plans of Sinai the present time and the future,

We hope to increase the cooperation between us and the Government of Japan for building Tunnels and Bridges for the same purpose.

The Governor of Ismailia hopes to exchange cooperation between the Egyptian Government and Japanese Government in training farmers and agronomists in the field of agriculture development and Land reclamation.

Appendix-8

Expansion Plans of Suez Canal

1/2

ARAB REPUBLIC OF EGYPT
SUEZ CANAL AUTHORITY
Ismailia, July 21, 1991
Dept : of Works
No :
Re :
Telephone : 064-220030/9 Ismailia
Fax. | 064 / 220784 220785

جمهورية مصر العربية
هيئة قناة السويس

١٩ ــــ في ــــ
دارة ــــ
رقم القيد ــــ
رقم الملف ــــ
عدد المرفقات ــــ

FAX 81 - 3 - 3807 - 0866

Sent to : -

Name : Eng. Takayoshi OHTSUKA
Address: Japan Shield Engineering Co., Ltd.
FAX : 81 - 3 - 3807 - 0866

From : -

Name : Dr. Eng. Isis A. KAMEL
Number of pages to follow : 4 including this one

Sub.: Basic Design Study on the project for Rehabilitation of AHMED HAMDI TUNNEL

REF. YOUR FAX DATED JULY 9, 1991 WE HAVE THE PLEASURE TO SEND THE ANSWER OF YOUR QUESTION No. (4) . HOPPING THAT YOU HAVE RECEIVED SOME OTHER INFORMATION / DATA CONCERNING OTHER QUESTIONS. STILL PREPARING FOR THE REST.

BEST REGARDS.

Deputy Director of Works Dept.
Dr. Eng.
(Isis A. KAMEL)
٢١.٧.١٩٩١

RECEIVED JUL 22 1991

33

6

-3/4

4 .. <u>The relation between the tunnel and the expansion plans of the Suez Canal</u> : -

In alignment with the tunnel in 1976, the far future expansion and doubling plans of the Suez Canal were taken into consideration. These plans include : -

Case 1 - Widening and deepening the Canal cross-section (existing at 1976) to allow the transit of loaded ships of 150,000 D.W.T. with 35 feet draught .

That plan- known as " The First Stage of Development " - was implemented by the end of 1980 .

Case 2 - Widening and deepening the Canal cross-section to allow the transit of loaded ships of 260,000 D.W.T. with 68 feet draught, i.e, " The Second Stage of Development ",

the First phase of this 2nd stage of Development aims at the widening of the Canal cross-section only to allow the transit of loaded ships up to 180,000 D.W.T. with 56 feet draught . This phase has already been implemented all through the Suez Canal except for that southward part (km 123 - 162) .

The Second Phase of this 2nd Stage of Development is under feasibility study, which is expected to be ready by the end of 1991 .

Case 3 - Widening and deepening the Canal in far future to allow the transit of loaded ships up to 500,000 D.W.T. with 75 feet draught .

Case - 4 - Doubling the Canal to allow the transit of ships in two directions at the same time . The Western branch will be assigned for Southbound convoys, the majority of which are in ballast . The Eastern branch will be assigned for Northbound convoys which are relatively of bigger draught .

<u>Restrictions On Executing Expansion Plans (Imposed By The Tunnel)</u>

As stated above, only the first stage of Development was implemented (by the end of 1980) in the presence of the tunnel . Recommendations were given by the Tunnel Designer that utmost care should be taken when executing any dredging works required for expansion plans on the top of the tunnel .

336
7

These recommendations make some difficulties for S.C. expansion plans .

Since the Japanese Consultant will carry-out the repairing plan for the tunnel; S.C.A. requests the Consultant to consider- in repair design- the dredging works on the top of the tunnel, to enable S.C.A. to carry out safely all future plans as previously explained .

In this respect, S.C.A. suggests to carry-out from now on - before beginning the tunnel repair - the dredging works needed for the 2nd Stage of Development, on the top of the tunnel following Consultant's proposed method of dredging . Thus, any deffects, that may happen in the tunnel, resulting from this dredging will be included in the tunnel repairing process .

As an example of a method of dredging, dredging should be carried into layers of .... ? meter, with time intervals between dredging layer .... ? days ; with a cutter of not more or not less than.. ? 1

The Consultantis requested to study the above proposed method of dredging and give the limitations .... or he could propose any other method .

By S.C.A. suggestion, we can avoid any future defects that may occur in the tunnel body due to dredging works after repair .

Consultant's decision is requested to be delivered to S.C.A. as soon as possible, to be able to begin executing the mentioned operation (2nd Stage) before the beginning of the tunnel repair operations .

20/7/1991

337

4/4

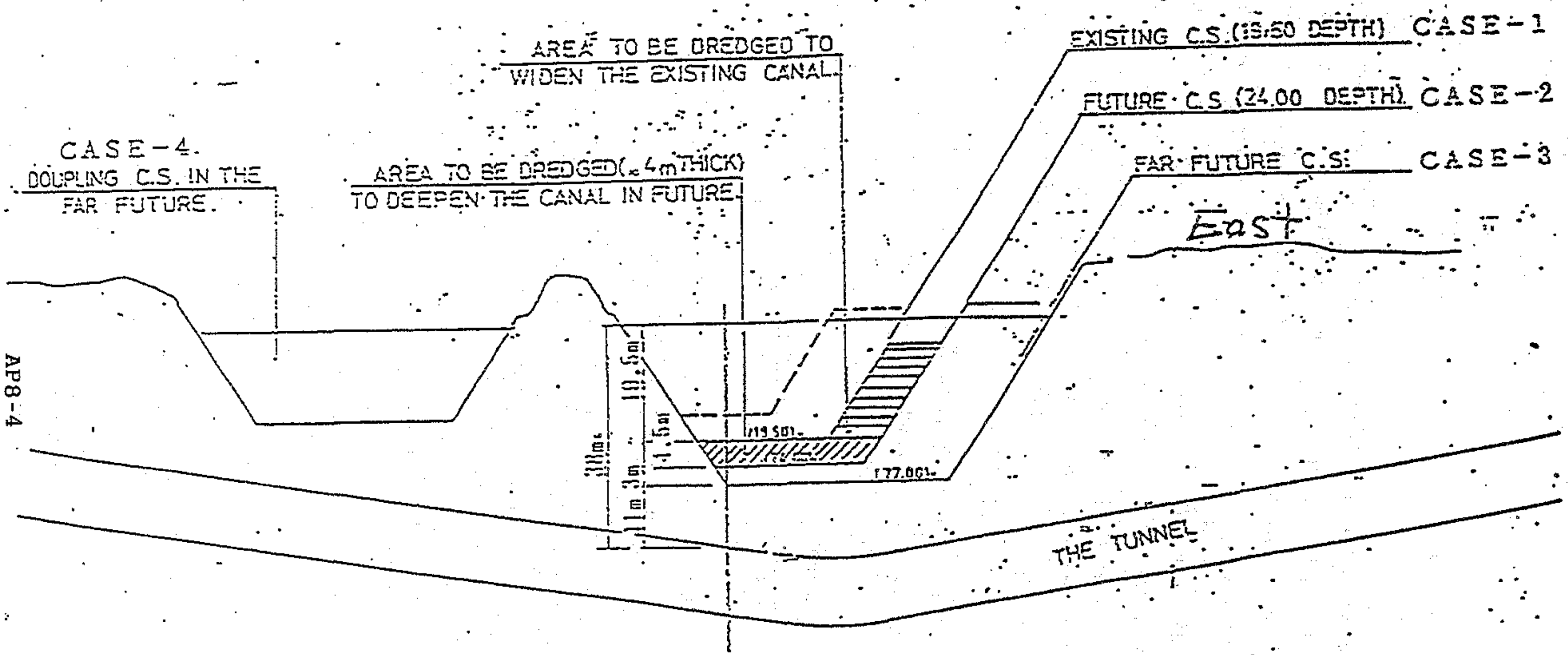


Fig. 3.2.1 LONGITUDITIONAL SECTION OF THE TUNNEL

7

Aug 14, '91
TSU

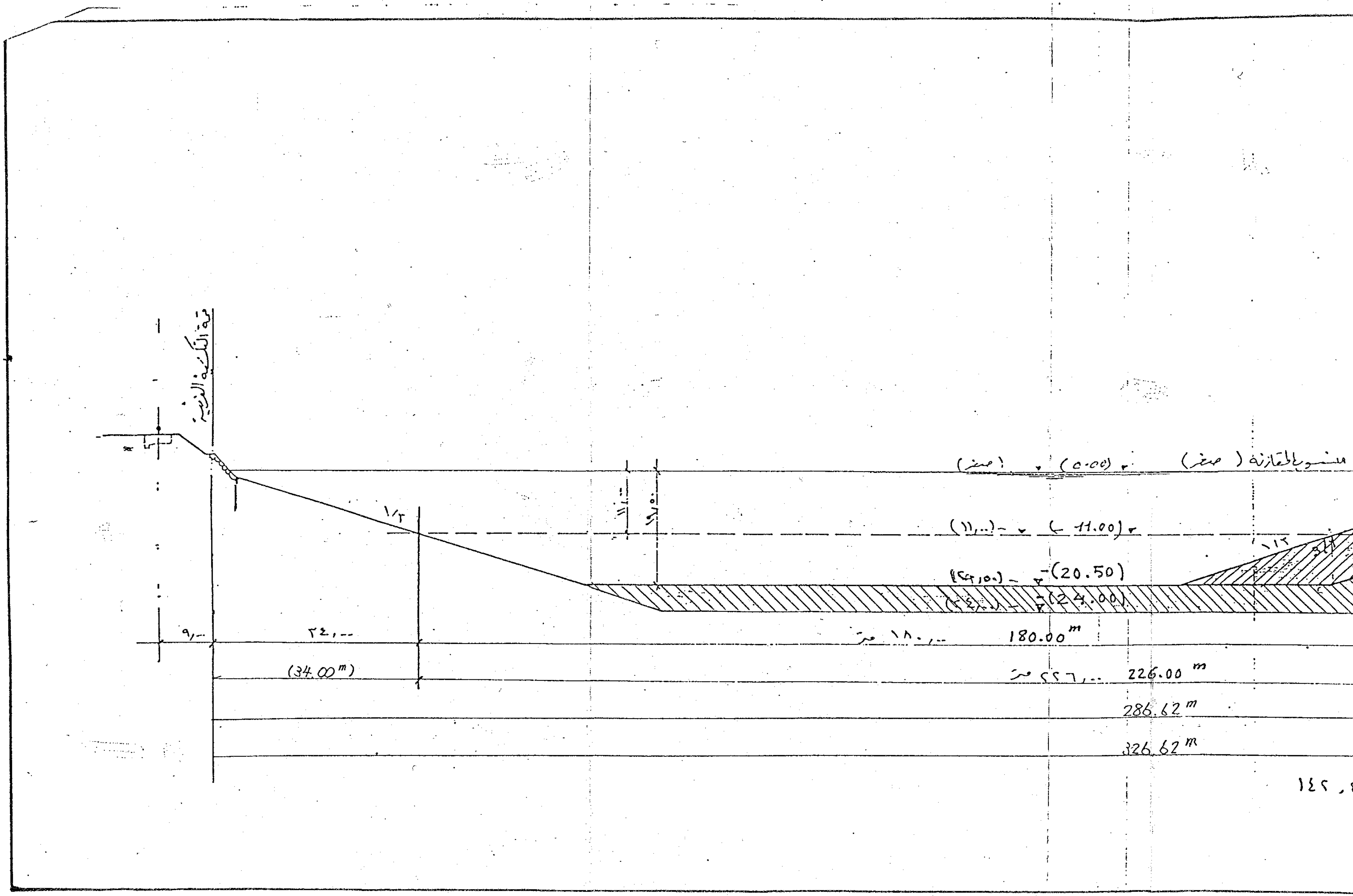
Suez Canal Expansion Plan vs TUNNEL

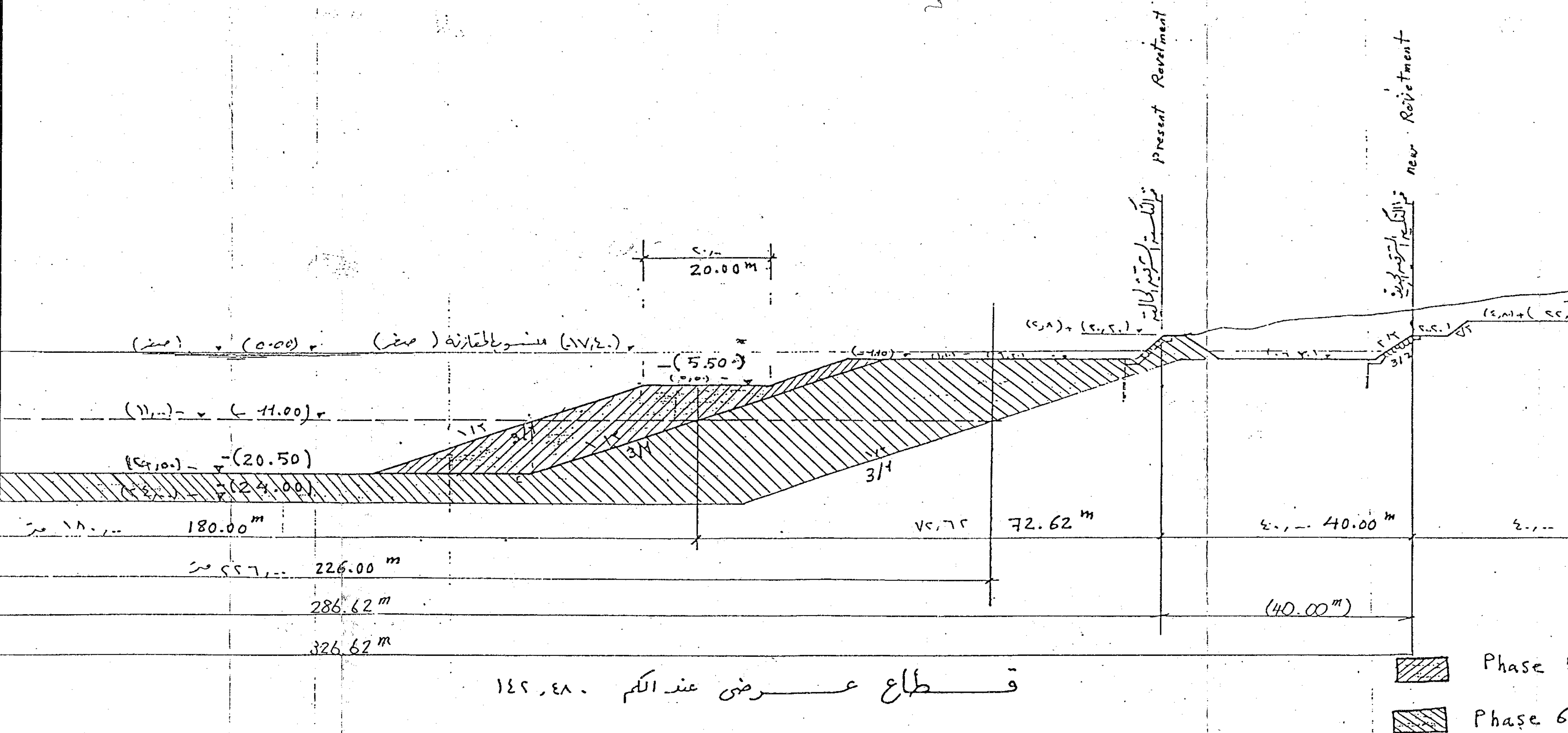
= "4. The relation between the tunnel and the expansion plans of the Suez Canal" in Dr ISIS FAX of July 21, 1991 =

The subject will be tablized as below:

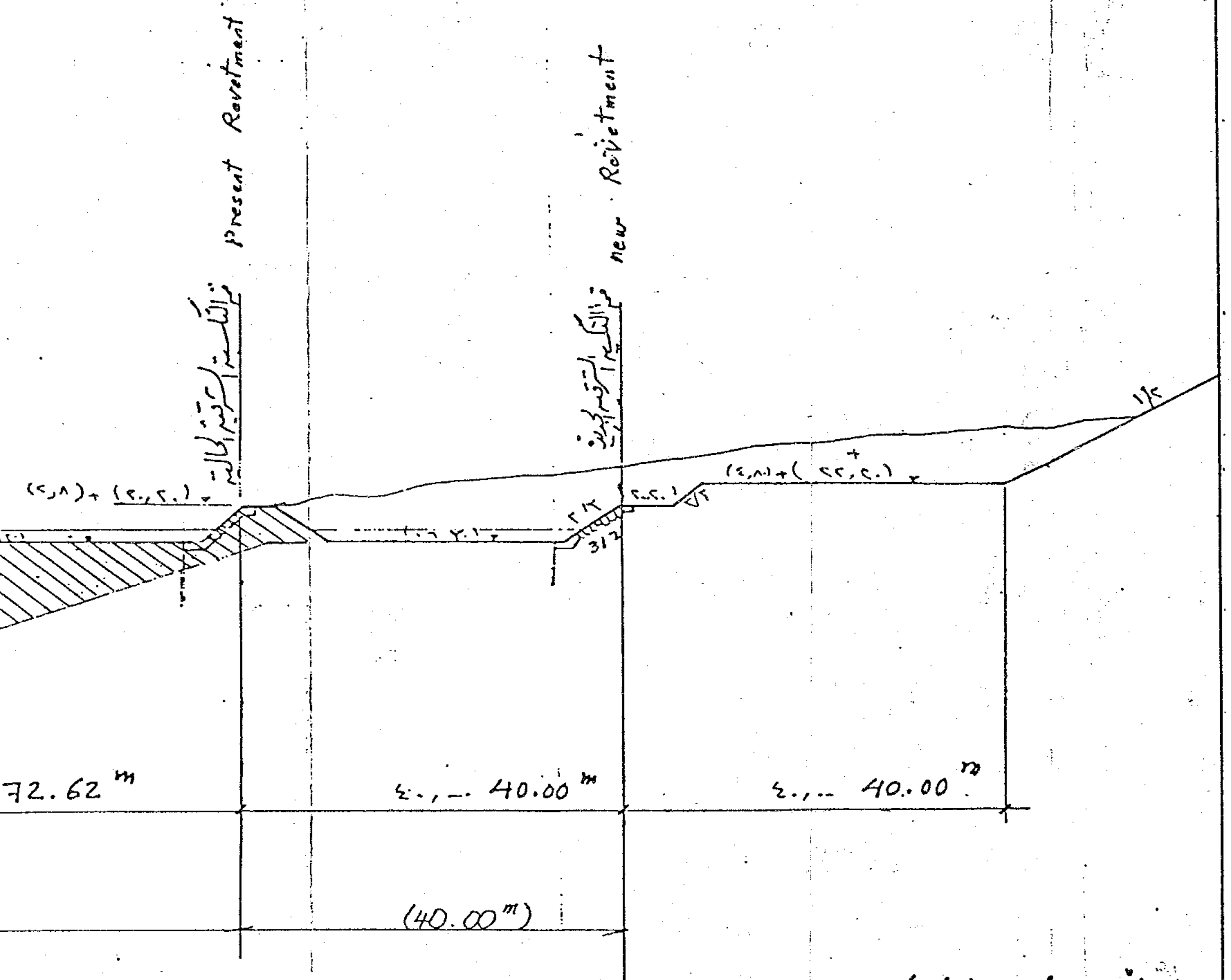
	Ship Capacity (DWT/draught ft)	Depth (m) Width (m)	1976 ~ 1980	'91	'92	'93	'94	'95	'96	'97
Case 1 First Stage	150,000 (10.67m) 35	19.50								
Case 2 Second Stage (First Phase)	180,000.- (17.07m) 56	(24.00)				*1	(Except for Km 123-162)			
Case 2' Second Stage (Second Phase)	260,000 (20.73m) 68	+4.5 24.00				*1	(Under F.S.)			
Case 3	500,000 (22.86m) 75	+3.0 27.00					(FAR FUTURE)			
Case 4	Doubling { East for North, West for South						(FAR FUTURE)			
Tunnel Rehabilitation	BD / DD WORK							*2		

Notes : { (1) *1 : by Dr ISIS on Aug. 11, 1991.
(2) *2 : by JICA Report of May 1990.


(0.00)
(11.00)
(20.50)
(24.00)
180.00 m
226.00 m
286.62 m
326.62 m
(34.00 m)



قطاع عرض عند الكم ١٤٢،٤٨٠



Phase 56′ (depth -20.50)

Phase 68′ (depth - 24.00)

أعمال التطهير للمرحلة ب

موقع نفق الشهيد أحمد حمدي

Widening + Deepening SCA at Tunnel Site CROSS - Section

مقياس الرسم

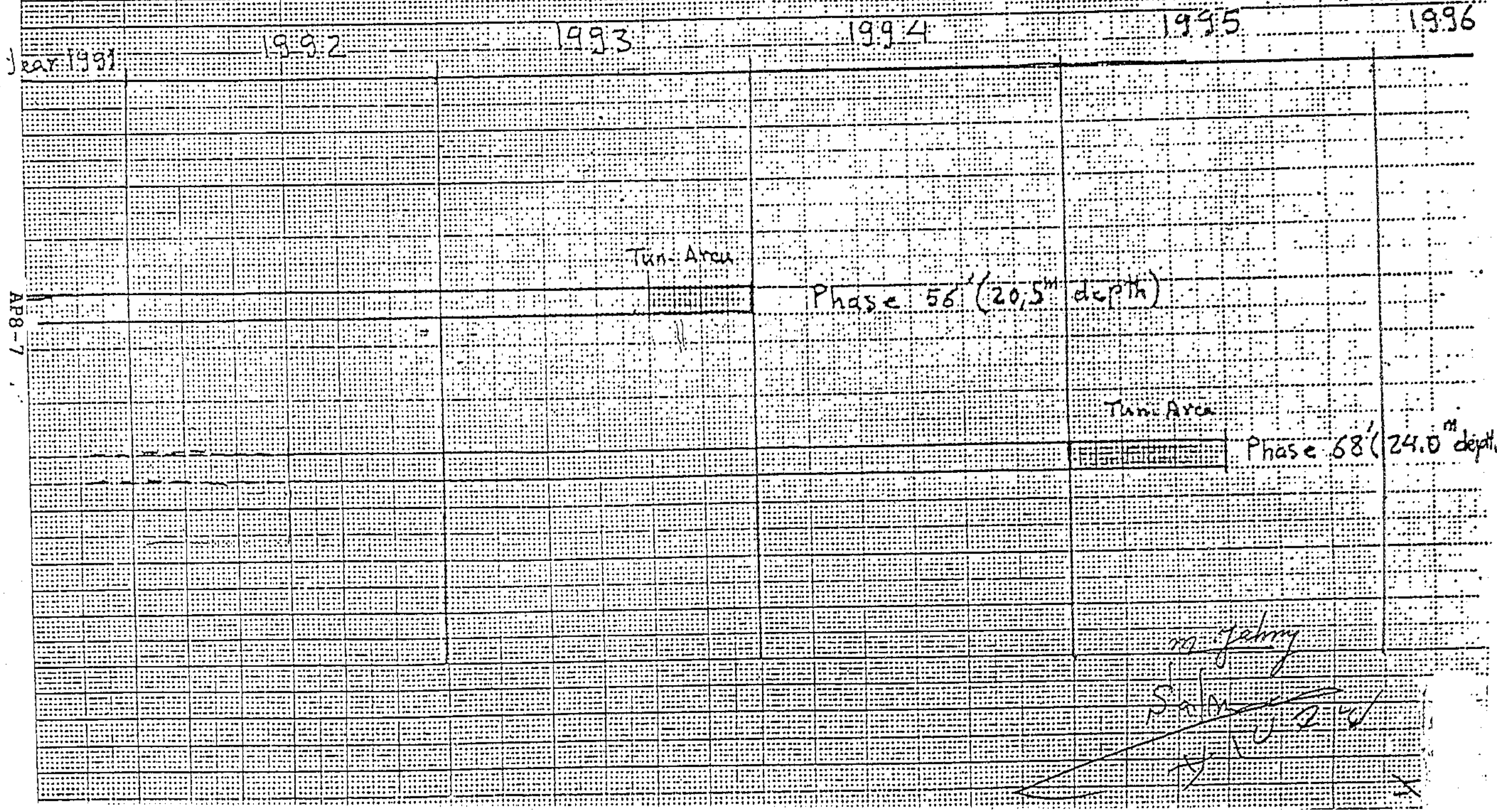
١/٥٠٠

	m. Yahya	رسم

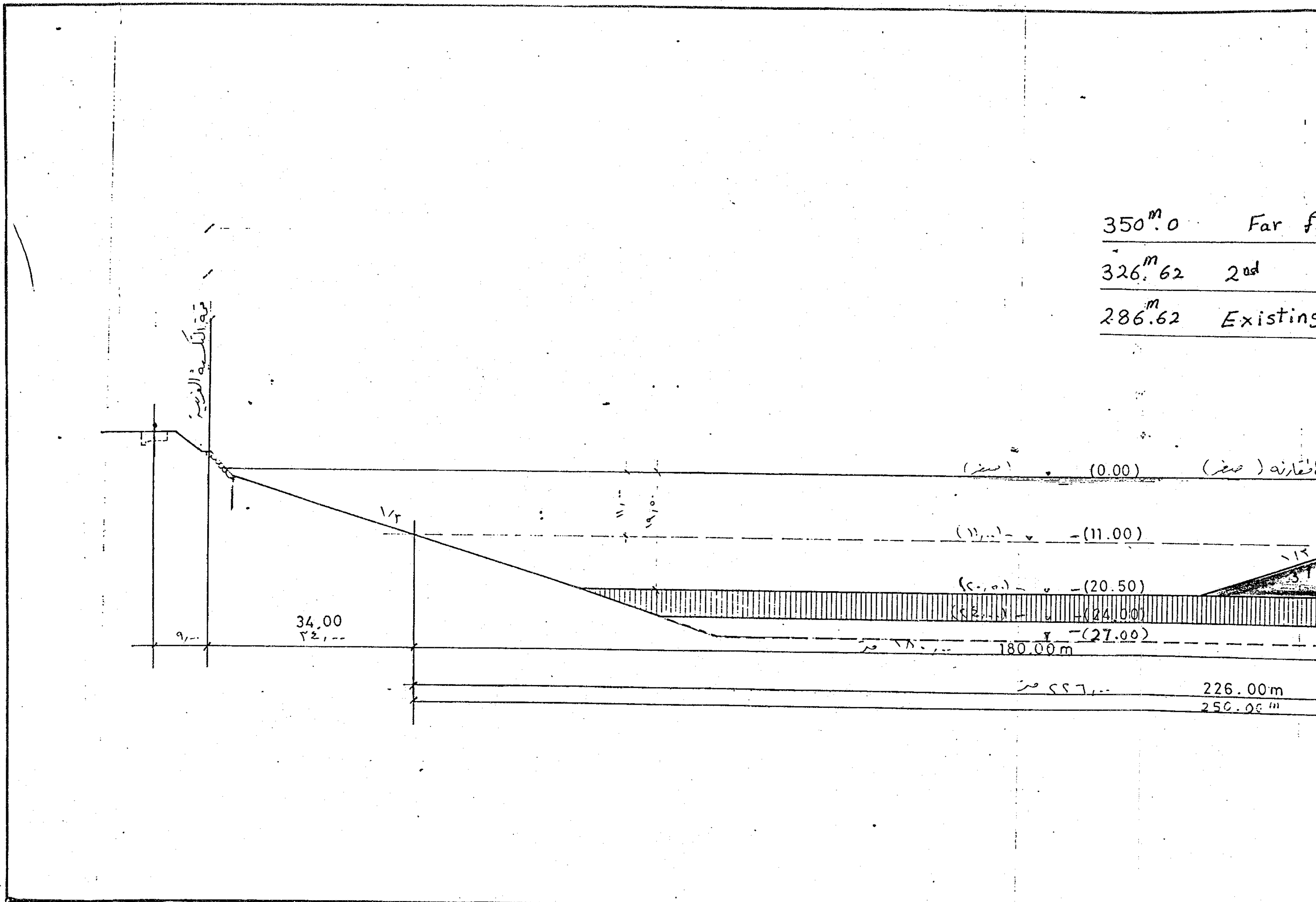
V.10 سبتمبر ١٩٩١

Schedule for

SCA Projects (Widening + Deepening)

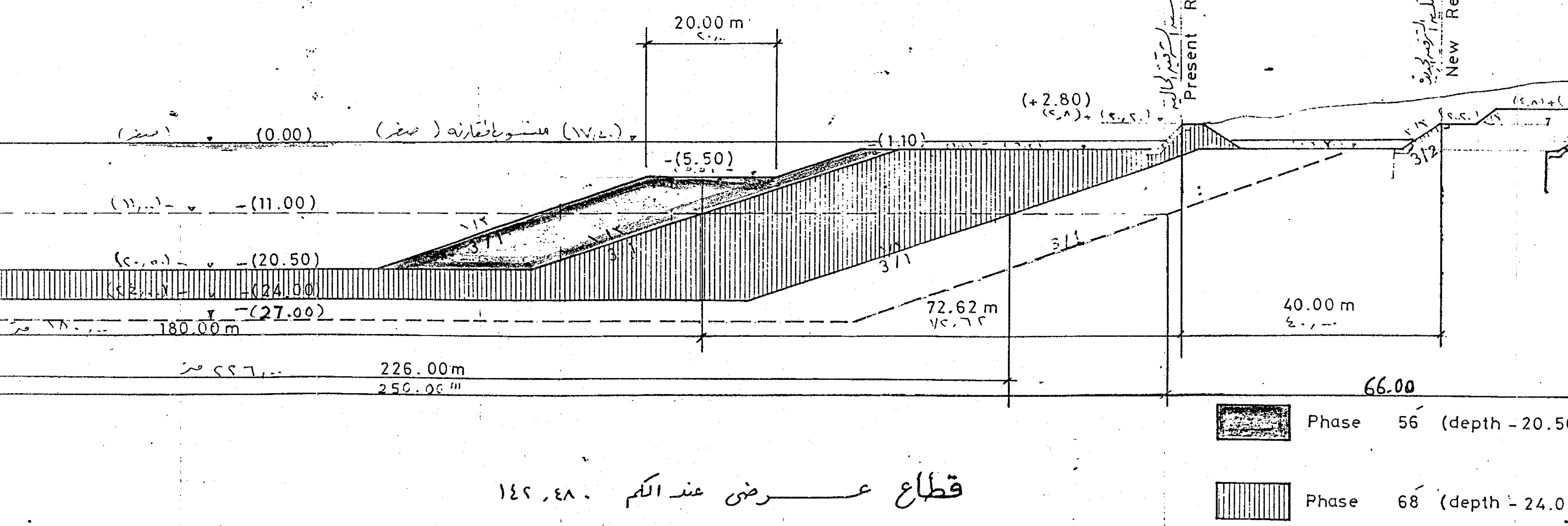


m. Fahmy


350m.0 Far f
326m.62 2nd
286m.62 Existing
(0.00)
(11.00)
(20.50)
(24.00)
(27.00)
1/٢
34.00
٣٤,--
٩,--
180.00m
226.00m
250.00m

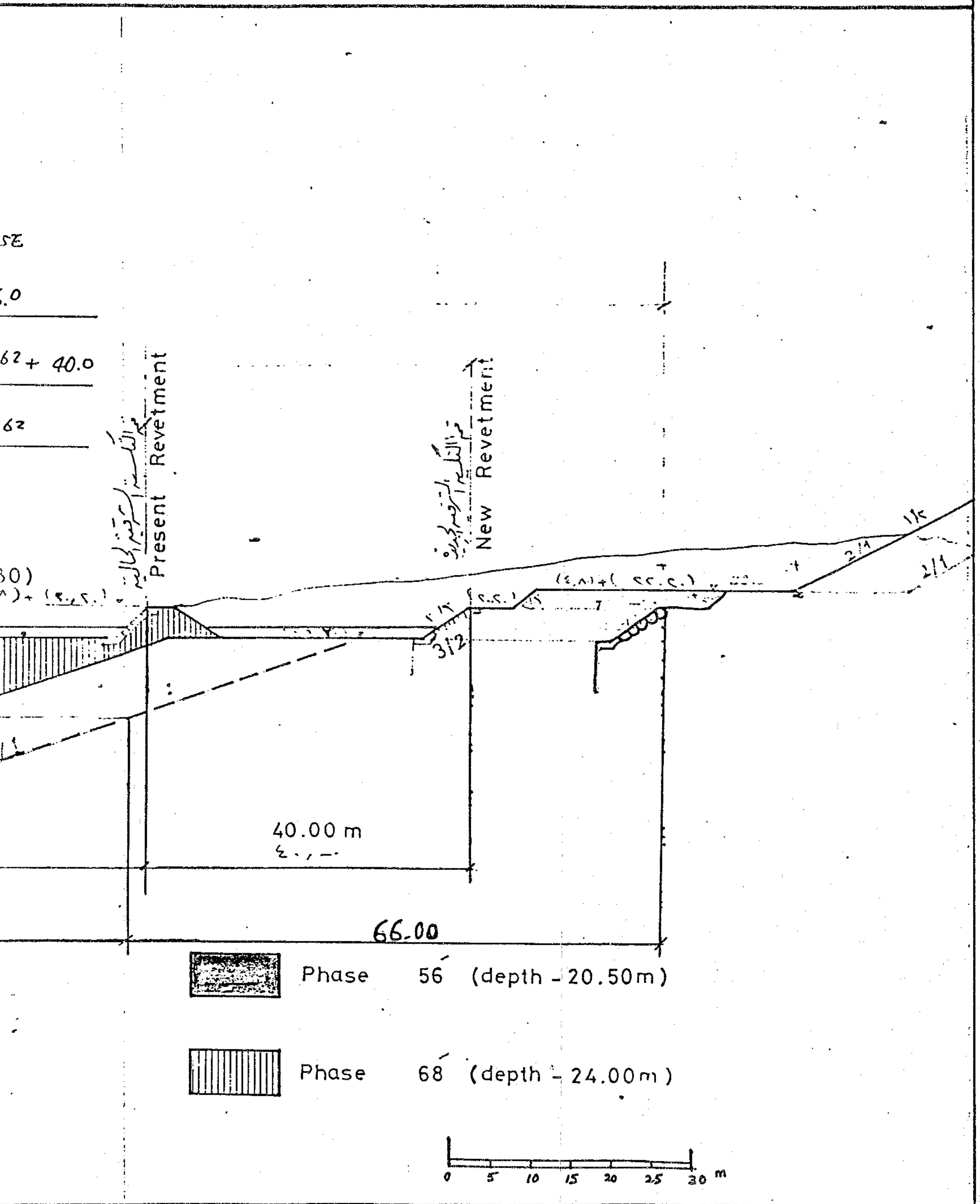
Analysis by TSU/JICA·JSE

350.0 m	Far future Phase	(72')	350.0 = 34.0 + 250.0 + 66.0
326.62 m	2nd Phase	(68')	326.62 = 34.0 + 180.0 + 72.62 + 40.0
286.62 m	Existing S.C.		286.62 = 34.0 + 180.0 + 72.62



قطاع عرضى عند الكم ١٤٢،٤٨٠

Cross Section at Km 142.480 m



Widening & Deepening

of Seuz Canal

at

Tunnel Site Km. 142 .480

أعمال التطهير للمرحلة الثانية

بموقع نفق الشهيد أحمد حمدي

(قطاع عرضي)

Cross Section

مقياس الرسم

١/٥٠٠

JICA رجاء تسليمه لـ

يسلم هذا الرسم لمندوب الجايكا اليابانى

مدير الادارة	وكيل مدير:	الرسام :
	نائب مدير محمد حسنين	المهندس :
		وكيل قسم زينب

٧٠١٥ / رسم رقم

AP8-8

الاسماعيلية فى سبتمبر ١٩٩١