CHAPTER II

STAGE OF CONSTRUCTION

Soft Ground Tunnelling Methods

Elements of the soft ground tunnelling operation can be classified into subsystems according to their functions in the tunnelling process.

1. The Route Selection

To avoid the easements under private property, the tunnel route is located in public street alignment. It is essential to obtain knowledge of ground conditions expected to be encountered along the tunnel's path. To find the optimum tunnel path, the geological investigation is necessary.

The items of the investigation are as follows:

a) In-Situ Exploration

Ground water level.

Coefficient of permeability.

b) Soil Testing Laboratory

Strength parameters.

Compressibility index.

2. Excavation and Erection of the Primary Lining

The excavation of soft ground tunnels will be carried out from bull eyes of the working shafts by shield tunnelling machines. Each working shaft will be sunk to the design depth below ground surface. The tunnel driving machine will drive towards another working shaft by keeping their routes in the correct alignments.

The use of compressed air techniques may be required for ground water control, when the excavation is anticipated to be in problematically ground conditions.

The excavation of soft ground tunnel is a cyclic operation consisting of 3 main activities.

1. Excavation of ground by the tunnel driving machine as it is advanced. The tunnelling machine operator will try to keep the tunnel route in direct grade and alignment.

2. The erection of primary lining under the protection of the tail skin of the tunnel driving machine Segments erection is carried out by using a hydraulic erector arm attached to the main body of the tunnel driving machine. A ring of the lining consists of 5 precast concrete segments.

3. Filling the annular void between the lining and the surrounding ground by grouting material.

3. Secondary Lining

When the section of tunnel between shafts has been driven and tunnelling machine removed, a travelling steel form is installed and the tunnel will be lined down to its final diameter with a secondary lining of reinforced concrete.

Detailed Description

1. Shaft

The shaft is a structure, which is sunk through ground or water for the purpose of excavating and placing the foundation at the prescribed depth and which subsequently becomes an integral part of the permanent work.

The vertical shafts required for tunnel construction are from shaft and access one. After the outbreak at the required level, a horizontal transverse adit of tunnel will be driven by the tunnel driving machine.

1.1 Lateral Load at Side Wall of the Shaft

Considering that a part of the earth pressure at the bottom of shaft is conveyed downward by shearing resistance in the equilibrium condition under compressed density of clay, Tscheboratioff[19] suggested the distributed of earth pressure loads as follows:

At the upper part, the coefficient of lateral earth pressure of equilibrium under compressed density of clay is K .

At the lower part, it shall be lessened in consideration of downward pressure conveyance, and the earth pressure in Fig. 1 shall be available from his investigation. Tschebotarioff[19] represented the lateral earth pressure of both case 1 and 2 by ABD, however Nishimatsu's opinion[12] is that it shall be ABED.

U_A is the required strength to resist the design load.

1.2 Formulae in Relation to Shaft Sinking

The occurrence of sinking of shaft is expressed by the following formula.

$$W_c + W = U + R \qquad \ldots\ldots\ldots\ldots\ldots\ldots \quad 2.1$$

where W_c = weight of shaft

W = load added on shaft causing sinking of shaft

U = Uplift pressure

R = resistant force (= skin friction)

Since the shaft will be constructed by the open caisson method, the above U is neglected. The skin friction working on the surface of side walls is assumed for each type of clay as follows:

Soft clay	1.2 tsm.
Medium clay, loose sand	2.5 tsm.
Stiff clay, compact sand	4.0 tsm.

Unit weight of materials for shaft designing.

Reinforced Concrete	2.4 tcm.
Soft clay	1.65 tcm.
Medium clay	1.68 tcm.
Stiff clay	1.89-1.99 tcm.
Sand	1.92 tcm.

1.3 Components of The Shaft

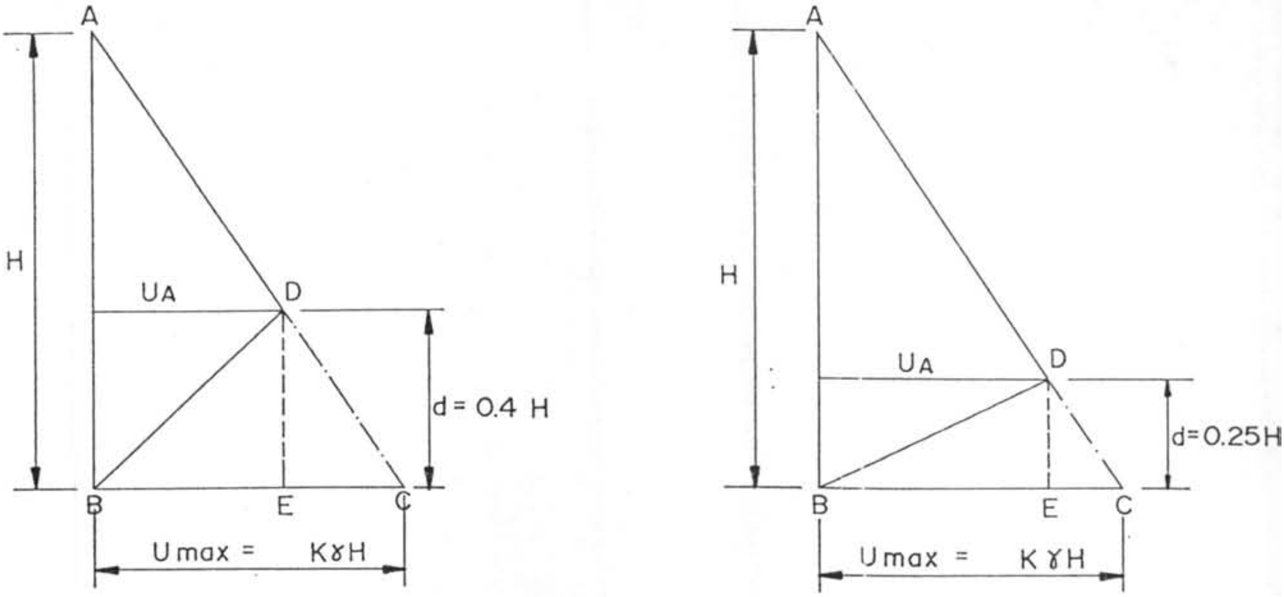
The main components of shaft are shown in Fig. 2

1. Base slab.
2. Cutting edge.
3. Side wall or steining.
4. Bull eye.

1.4 Study on Base Failure in Soft Clay

While excavating in soft clay, base failure sometimes occurs accompanied by a consequent sinking of surrounding ground.

The procedure for estimating the danger of base failure in a strutted excavation in clay was given by Terzaghi [18] but is limited to shallow excavations. For deep excavations and shafts, experience has shown that a conventional analysis will lead to unreliable estimates.



Case I Stiff clay

Case 2. Medium clay

Fig I Apparent earth pressure to side wall of the shaft

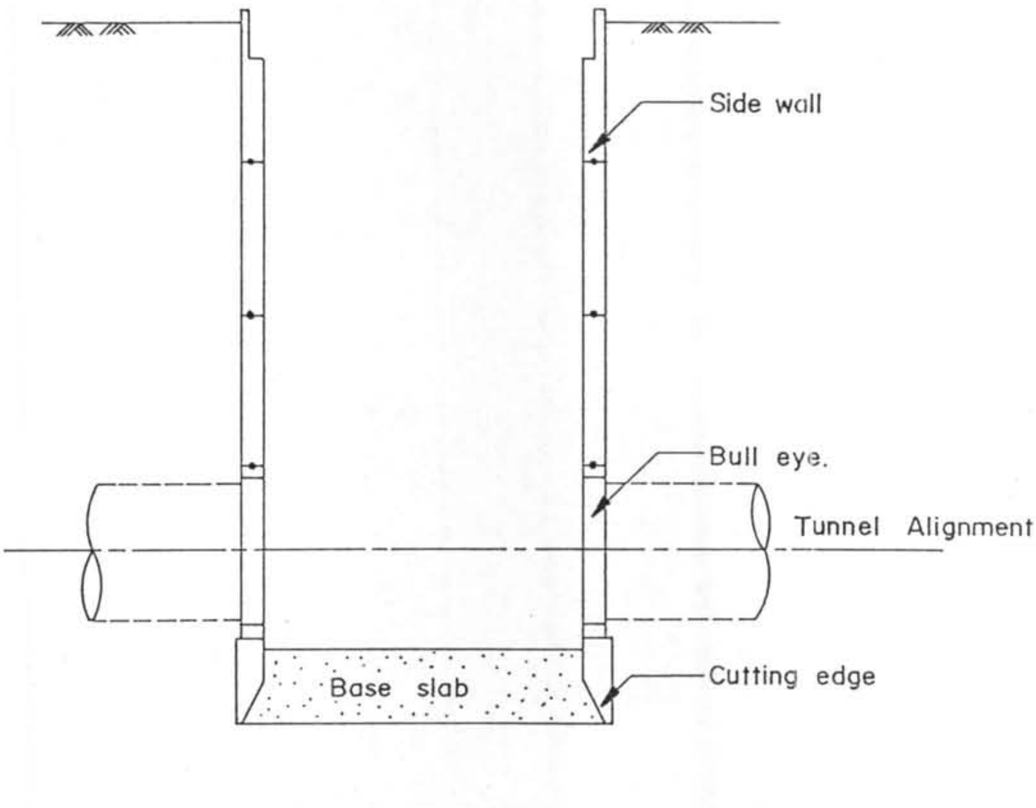


Fig 2. Components of the shaft

On the basis of the bearing capacity calculations for foundations on clay a new approach to this problem is developed by Bjerrum and Eide[4] in which the critical depth, D_c of an excavation is given by

$$D_c = N_c \frac{c}{\gamma} \qquad \text{.................} \quad 2.2$$

Where c and are cohesion and density of clay respectively, N_c is a dimensionless coefficient depending on the form of the excavation i.e. the width-length ratio in Fig. 4.

The values of N_c are identical to those given by Bjerrum and Eide[4] for bearing capacity formula.

From the above equation the safety factor against base failure is expressed by the equation.

$$F_s = \frac{D_c}{D} = \frac{N_c c}{\gamma D}$$

Considering the surface surcharge, we have

$$F_s = \frac{N_c c}{\gamma D+q-q_w} \qquad \text{.................} \quad 2.3$$

where F_s = the safety factor against base failure.

D = the depth of the excavation.

q = the surface surcharge.

q_w = the water surcharge in shaft.

A minimum of 1.25 is required for the safety factor F_s.

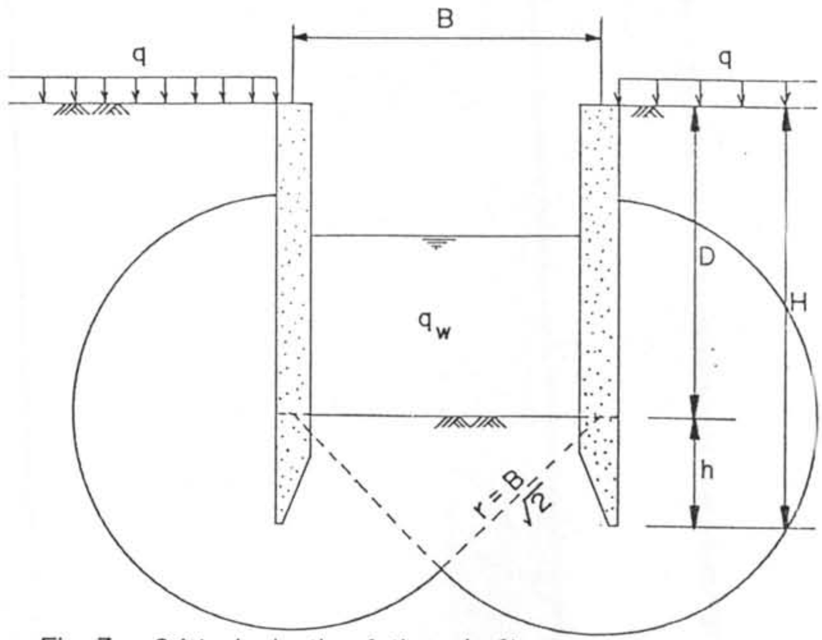


Fig. 3 Critical depth of the shaft.

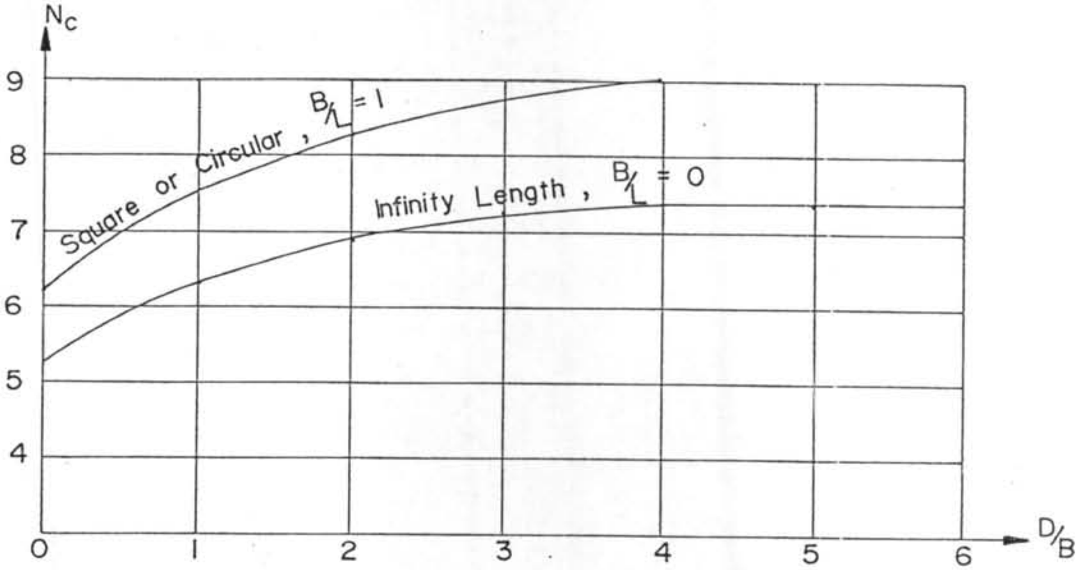


Fig. 4 Values of N_c from Bjerrum and Eide (4).

The calculation is done every 2.0 m. depth to the layer of clay or sand for each shaft.

where, B = the width of excavation

L = the length of excavation

1.5 Design of The Shaft

a) Design of Base Slab

The base slab is the plain concrete that will be casted after the main body of shaft has been lowered.

The base slab shall be designed as a two way slab simply supported at its perimeter by the side walls. The span shall be equal to AB as shown in Fig. 5.

The load acts on the base slab is the subgrade reaction due to the dead weight of working shaft structure after assuming the base slab thickness.

$$\text{Subgrade reaction, } w = \frac{\text{dead weight of working shaft}}{\text{base area of working shaft}}$$

$$\ell_x = L_x - 2D \cot 60^\circ \qquad \text{.................} \quad 2.4$$

$$\ell_y = L_y - 2D \cot 60^\circ \qquad \text{.................} \quad 2.5$$

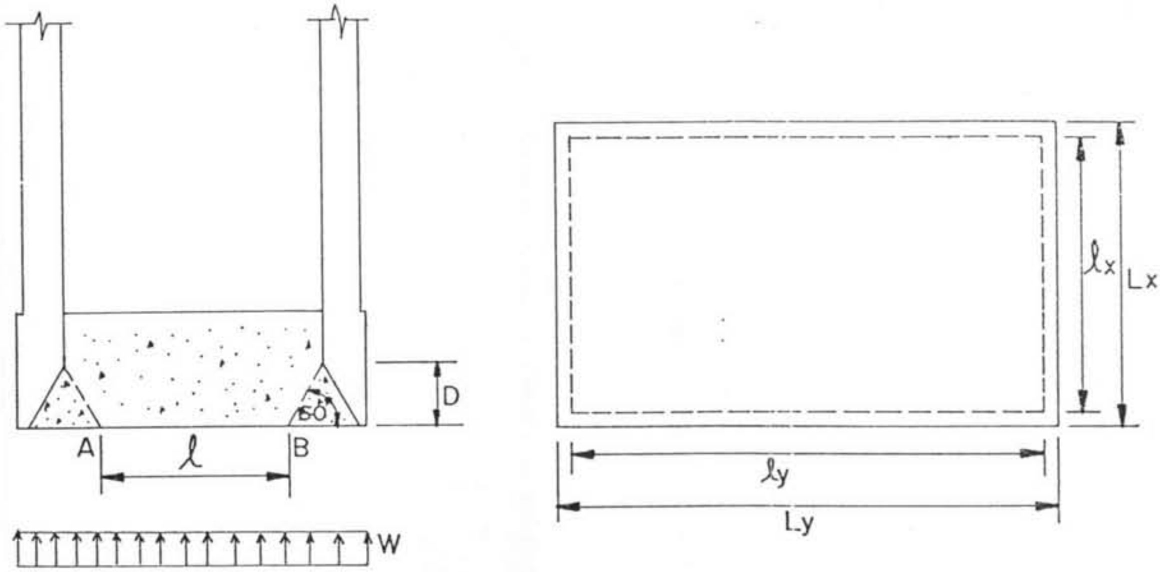


Fig. 5 Base slab of the shaft.

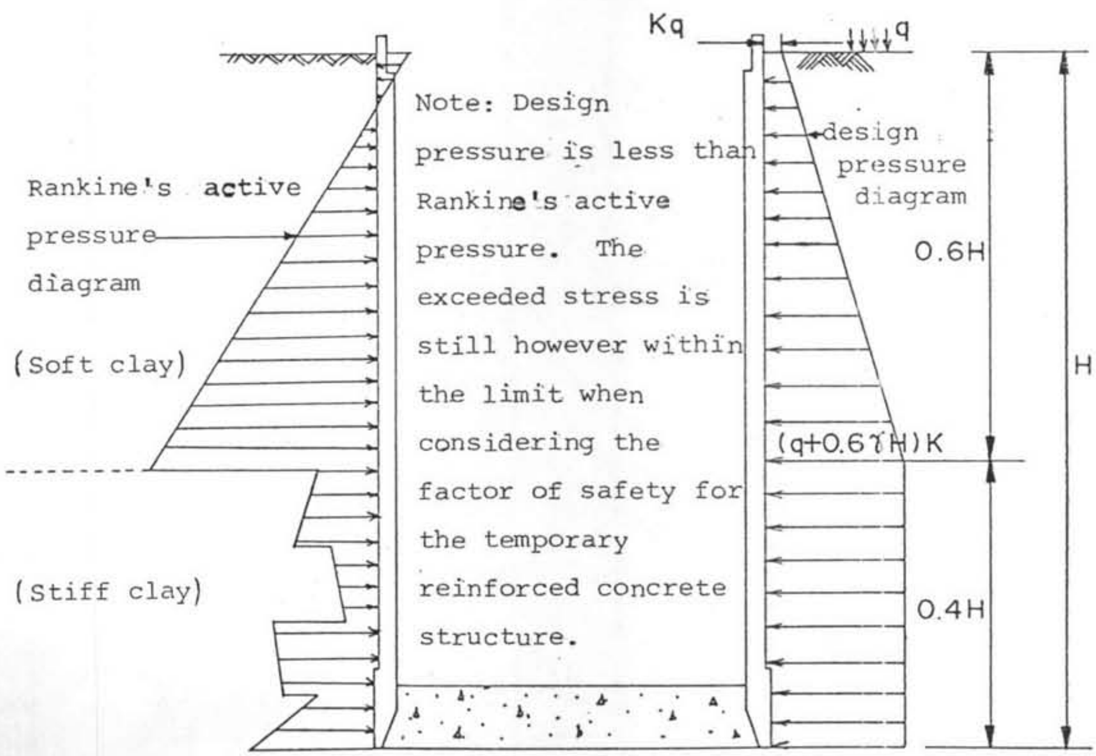


Fig. 6 Design earth pressure to the shaft.

If M_x = the moment at mid span on a strip of unit width spanning, ℓ_x.

M_y = the moment at mid span on a strip of unit width spanning, ℓ_y.

C_x, C_y = the bending moment coefficient for rectangular panel supported on four sides.

$$M_x = C_s w \ell_x^2 \quad \text{.................} \; 2.6$$

$$M_y = C_y w \ell_y^2 \quad \text{.................} \; 2.7$$

Therefore, the maximum tensile stress of plain concrete is

$$f_t = \frac{M_{max}}{Z} \quad \text{.................} \; 2.8$$

$$Z = \frac{bh^3}{6} \quad \text{.................} \; 2.9$$

where h is the thickness of assumptive base slab.

d is the width of assumptive base slab.

b) <u>Design of the First Lot</u>

The first lot is attached to the cutting edge. The shaft wall is considered as a beam of which lateral earth pressure is reaction force and its both ends are assumed to be fixed by the other walls.

The cutting edge shall be considered as the cantilever slab, of which reaction is assumed to be carried by the above side wall as shown in Fig. 6.

Load carried by the side wall of the first lot as shown in Fig. 7.

$$P = P_3 + \frac{\ell_1}{\ell_2} P_3 \qquad \ldots\ldots\ldots\ldots\ldots\ 2.10$$

where $P_3 = (q + 0.6\ H)K$ (Fig. 6)

c) Cutting Edge

The cutting edge is the terminating rim of shaft which makes contact with the soil in direct bearing. The cutting edge should have as sharp an angle as practicable for penetrating into the soil. It is therefore making less vulnerable to damage from the obstruction, if the cutting edge is wrapped with steel plate (Fig. 8).

In the design of cutting edge, it shall be calculated as the cantilever as follows (Fig. 7).

where

w = lateral earth pressure at cutting edge.

ℓ = length of cutting edge.

M = maximum moment for design cutting edge.

Q = maximum shear for design cutting edge.

$$M = \frac{1}{2} w\ell^2 \qquad \ldots\ldots\ldots\ldots\ldots\ 2.11$$

$$Q = w\ell \qquad \ldots\ldots\ldots\ldots\ldots\ 2.12$$

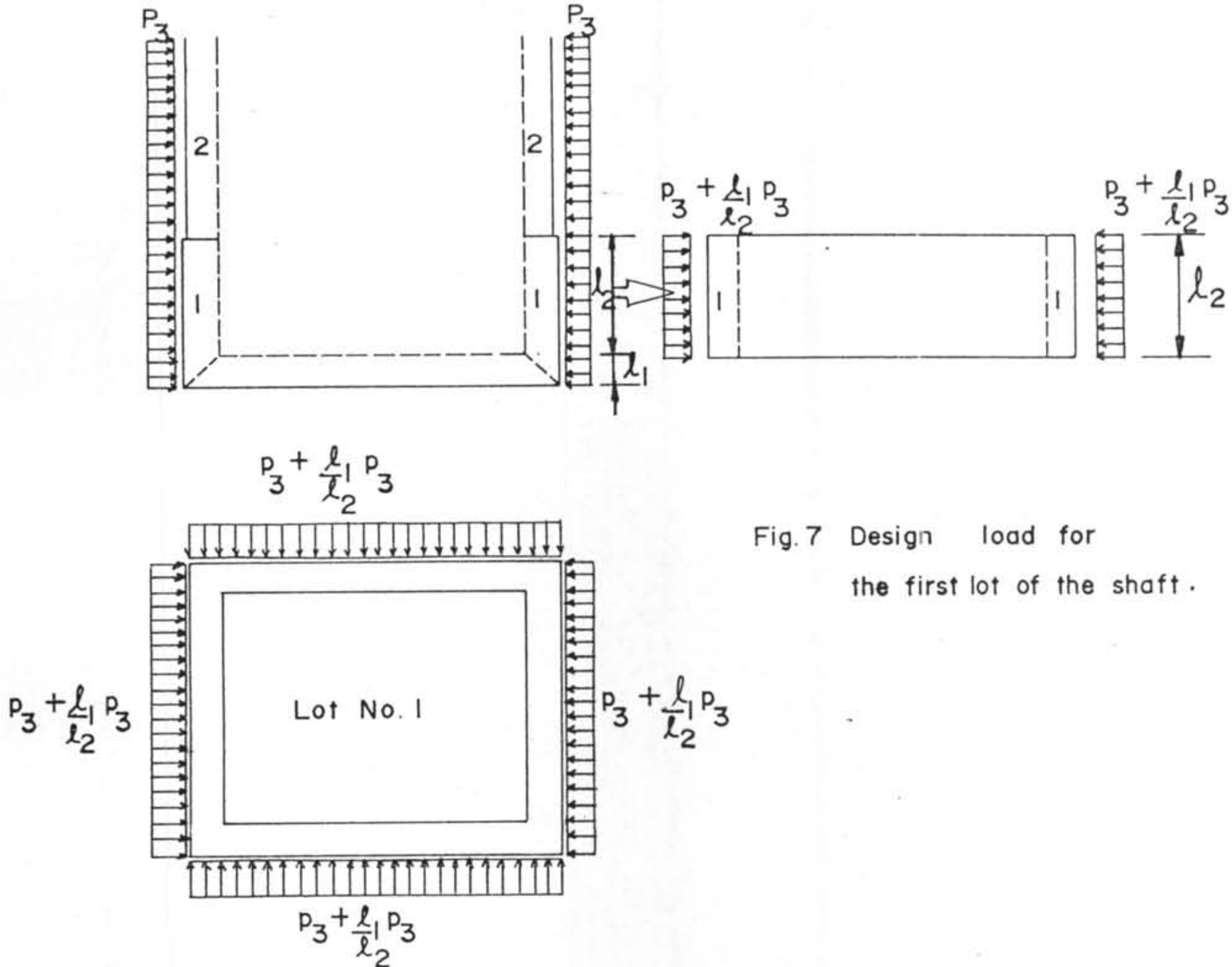


Fig. 7 Design load for the first lot of the shaft.

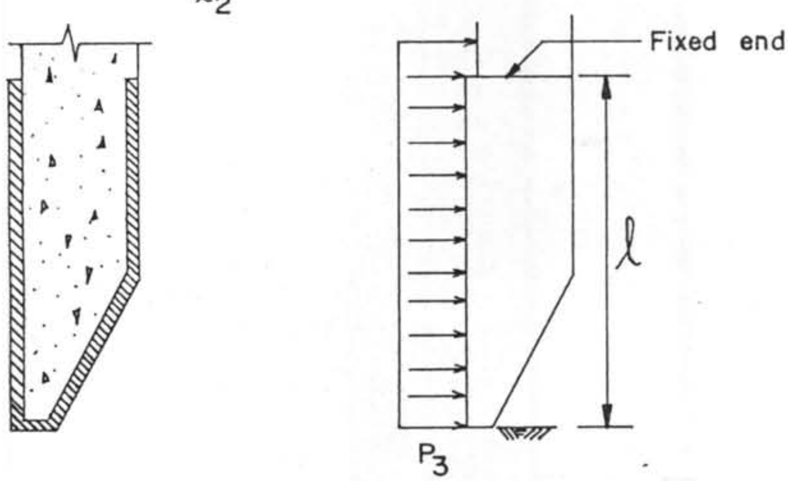


Fig. 8 Cutting edge and pressure to the cutting edge.

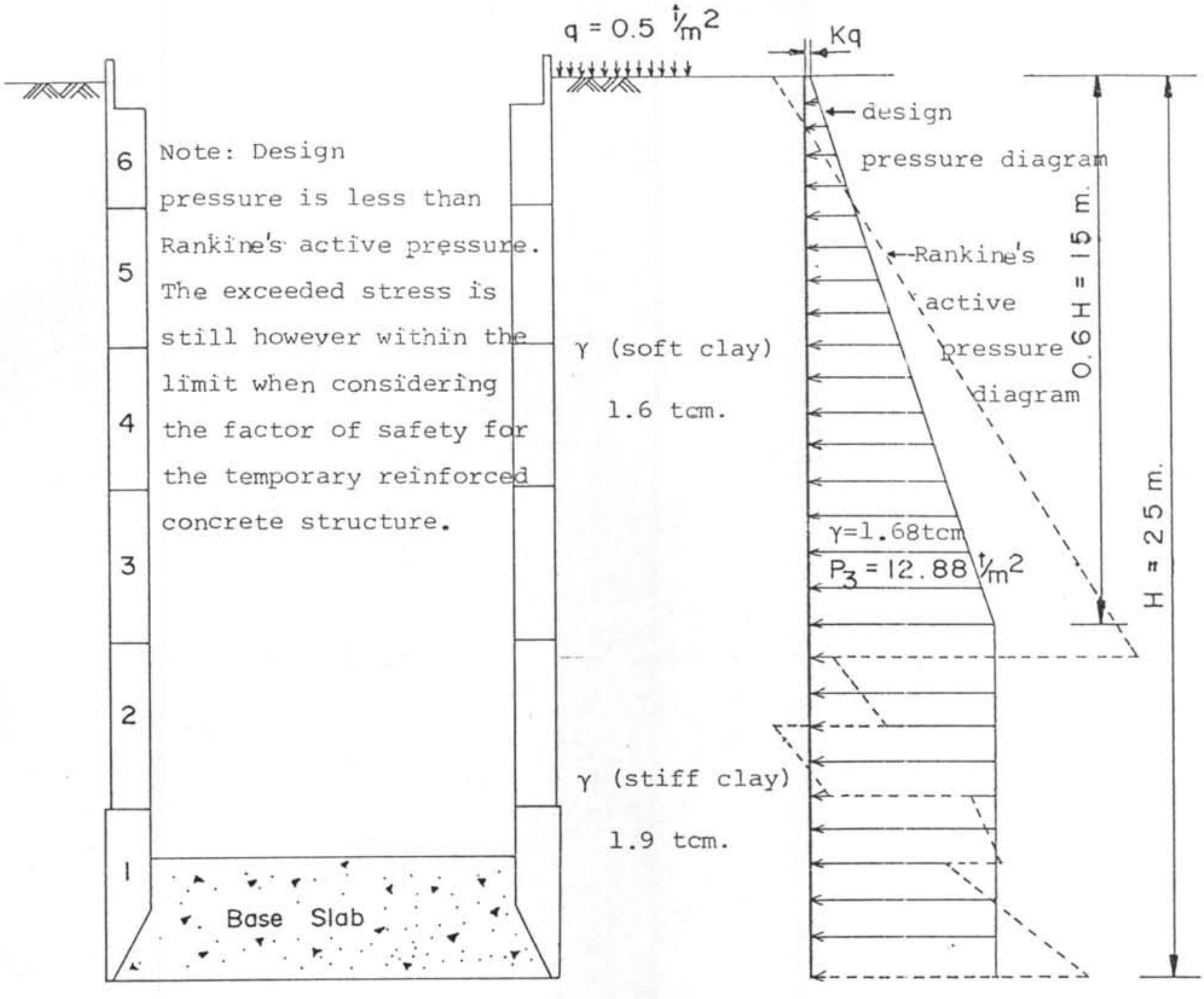


Fig. 9 Horizontal pressure to the shaft walls.

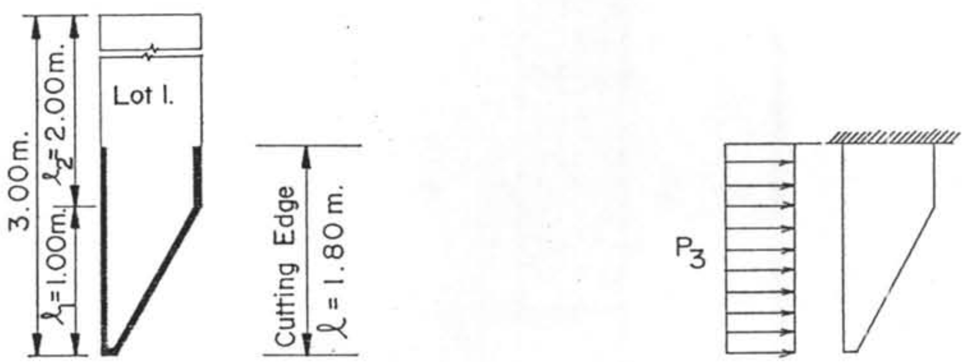


Fig. 10 Pressure to the cutting edge.

1.6 Design Example for Lot No. 1 of The Shaft.

a) Design of The Cutting Edge

From Fig. 9-11

Pressure at 0.6 H depth ; $P_3 = (q + 0.6H\gamma)K$

$= (0.5 + 15 \times 1.684)0.5$

$= 12.88$ tsm.

Max. moment of cutting edge;

$$M = \frac{1}{2} P_3 \ell^2$$

$$= \frac{1}{2} \times 12.88 \times 1.80^2$$

$$= 20.86 \quad \frac{t\text{-}m}{m}$$

Max. shear of cutting edge;

$$Q = P_3 \ell$$

$$= 12.88 \times 1.80$$

$$= 23.18 \quad t/m$$

b) Design of Lot No. 1 of The Shaft Wall

Lateral pressure to side wall;

$$w = p_3 + \frac{\ell_1}{\ell_2} p_3$$

$$= 12.88 + \frac{1.00}{2.00} \times 12.88$$

$$= 19.32 \quad \text{tsm.}$$

Relative stiffness $(\frac{I}{L})$ of the side wall.

$$K_{AB}, K_{CD} = \frac{I}{9.0} \times 63.0 \text{ and } K_{AC}, K_{BD} = \frac{I}{7.0} \times 63.0$$

$$= 7.0\ I \qquad\qquad = 9.0\ I$$

Distribution factors of the long sides : Short sides

$$= \frac{7I}{9I + 7I} \quad : \quad \frac{9I}{0I + 7I}$$

$$= 0.44 \quad : \quad 0.56$$

Fixed end moment $(\frac{w\ell^2}{12})$ of the side walls.

$$AB = \frac{19.32 \times 9.0^2}{12} \quad ; \quad AC = -\frac{19.32 \times 7.0^2}{12}$$

$$= 130.41 \text{ t-m} \qquad\qquad = -78.89 \text{ t-m.}$$

Shear force of side walls

AB, CD

at A, B, C, D

$$= 19.32 \times \frac{9}{2}$$

$$= 86.94 \text{ t.}$$

AC, BD

at A, B, C, D

$$= 19.32 \times \frac{7}{2}$$

$$= 67.62 \text{ t.}$$

Table 1

Bending moment of lot No. 1 of the shaft wall by moment distribution method

Joint	A		B		C		D	
Member	AB	AC	BA	BD	CA	CD	DC	DB
Distribution factor	0.44	0.56	0.44	0.56	0.56	0.44	0.44	0.56
Fixed end moment; $\frac{w\ell^2}{12}$	+130.41	- 78.89	-130.41	+ 78.89	+ 78.89	-130.41	+130.41	- 78.89
Distributed moment	- 22.67	- 28.85	+ 22.67	+ 28.85	+ 28.85	+ 22.67	- 22.67	- 28.85
Carry over moment	+11.335	+14.425	-11.335	-14.425	-14.425	-11.335	+11.335	+14.425
Distributed moment	-11.335	-14.425	+11.335	+14.425	+14.425	+11.335	-11.335	-14.425
Carry over moment	+ 5.667	+ 7.213	- 5.667	- 7.213	- 7.213	- 5.667	+ 5.667	+ 7.213
Distributed moment	- 5.667	- 7.213	+ 5.667	+ 7.213	+ 7.213	+ 5.667	- 5.667	- 7.213
Sum. of moment	+107.74	-107.74	-107.74	+107.74	+107.74	-107.74	+107.74	-107.74

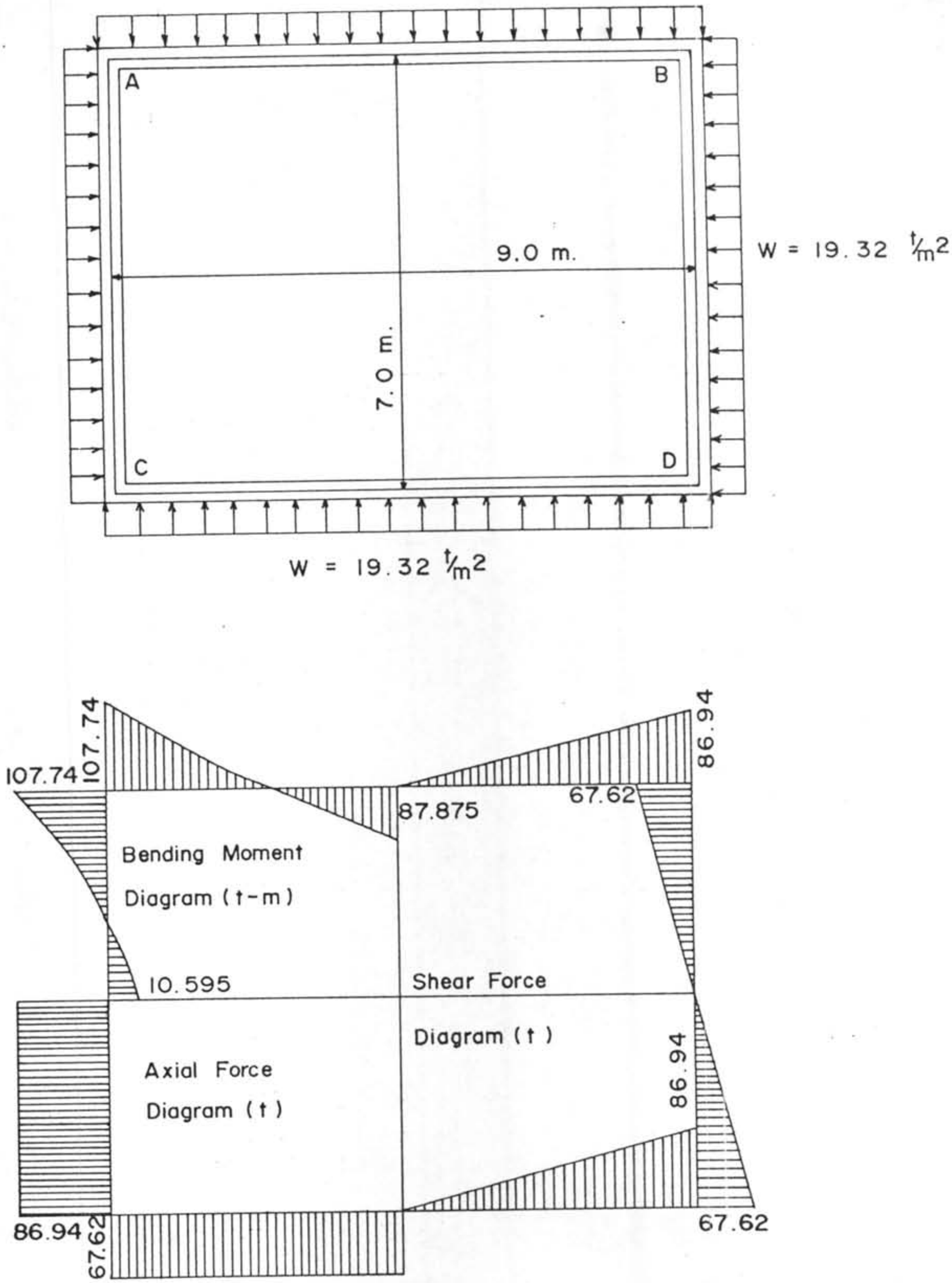


Fig 11. Axial force, shear force and bending moment diagram of lot no.1 of the shaft wall.

Axial force of side walls

AB, CD	AC, BD
$= 19.32 \times \frac{7}{2}$	$= 19.32 \times \frac{9}{2}$
$= 67.62$ t.	$= 86.94$ t.

Maximum positive moment at middle of the side walls

AB, CD	AC, BD
$= - 107.74 + \frac{w\ell^2}{8}$	$= - 107.74 + \frac{w\ell^2}{8}$
$= - 107.74 + \frac{19.32 \times 9.0^2}{8}$	$= - 107.74 + \frac{19.32 \times 7.0^2}{8}$
$= + 87.875$ t-m	$= 10.595$ t-m

1.7 Shaft Construction

Shafts can be circular or rectangular in shape as shown in Fig. 12 and 13. They are sunk by their own weight while the soil is being excavated from the dredging well.

The primary aim in shafts sinking is to sink them straight and at the correct position.

The factors of a disastrous tipping in shaft sinking are

1. Digging unevenly within the shafts.

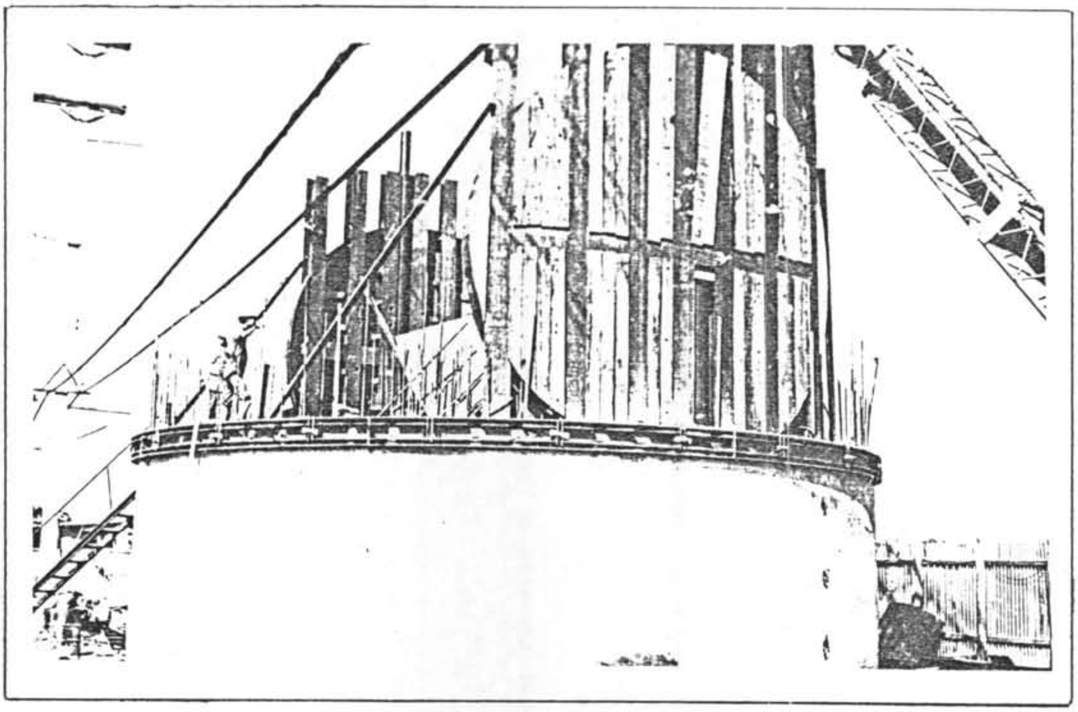


Fig. 12 Circular shaft construction.

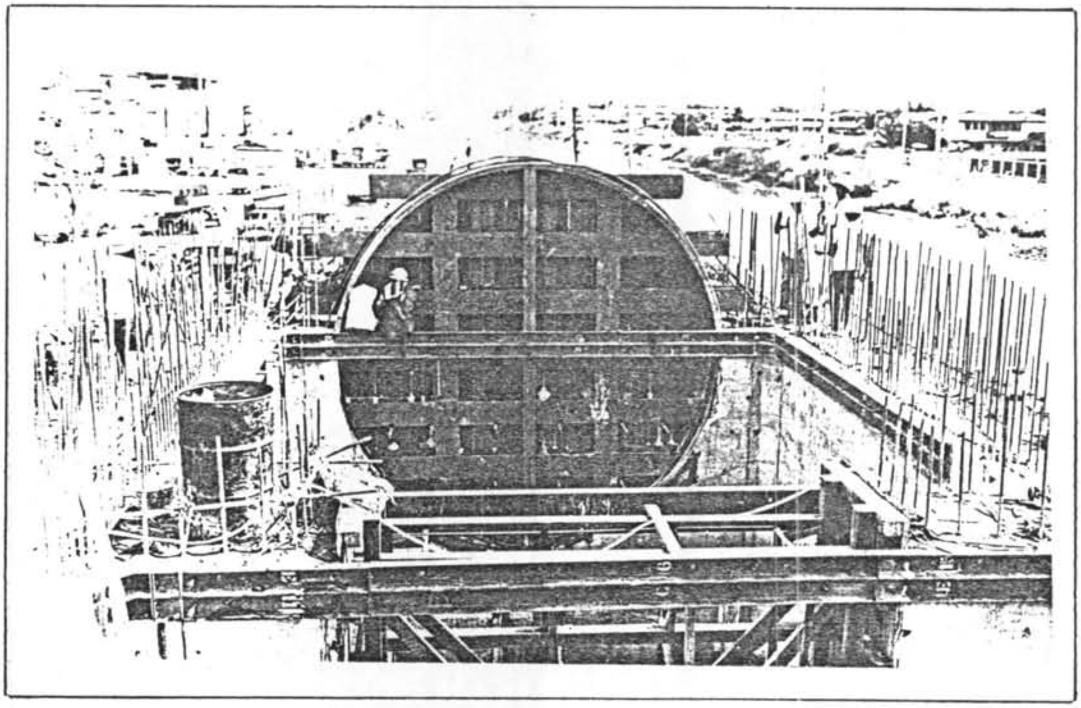


Fig. 13 Rectangular shaft construction.

2. The presence of obstructions and varying soft and hard strata which will cause uneven sinking.

Suitable precautions are taken to avoid tilts and shifts as follows:

1. Outside shaft walls should be as regular and smooth as possible.

2. The cutting edge of the curb should be uniform in thickness and sharpness since the sharper edge has greater tendency of sinking than blunt edge.

3. Construct the support structure around the outer shaft wall to control the sinking of shaft in straight line. Support structures are used for maintaining position and verticality of shafts during sinking.

1.8 Method of Shaft Sinking

a) Shaft Sinking by Conventional Method

Shafts are built up in section by precast concrete segments erection. They are sunk by their own weight, while the internal soil is being excavated. To control the sink in vertical alignment, ring beams are constructed around the perimeter of the shafts wall. If their diameters are more than 9 m, ring beam must be built on 15 cm. diameter, 8 m. long wooden piles.

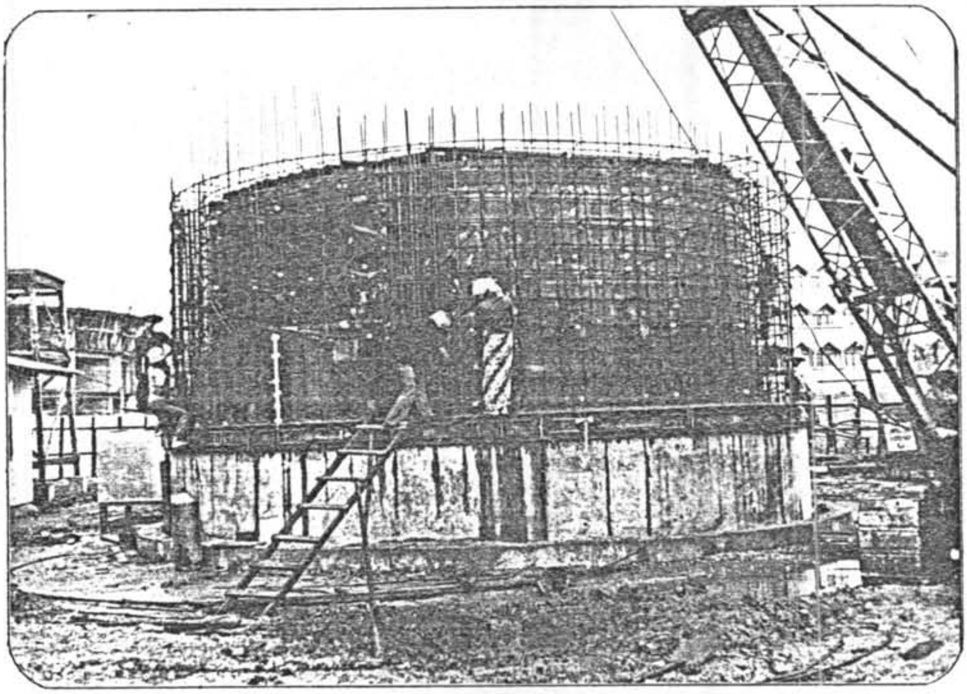


Fig. 14a

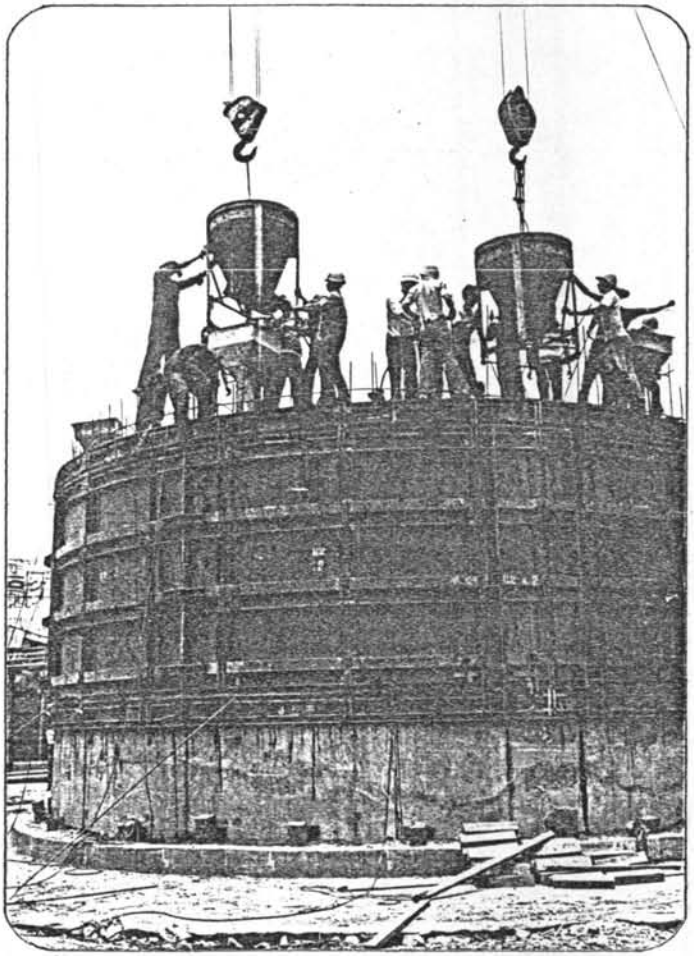


Fig. 14b

Fig. 14 Shaft construction by excavation and concrete placing.

To assist the shafts sinking, weighting on their top to promote downward movement is performed.

The skin friction between shafts wall and surrounding soil can be reduced by bentonite slurry and by keeping the shafts moving down uninterruptedly.

After sinking to the full depth of the section, further segment rings were added and the sinking process can be continued.

b) Excavating and Concrete Placing Alternation Method

The cycle of this method is sinking process and concreting the additional section (Fig. 14). However, the sinking of the shaft must be interubed during the concreting of the additional section, which is a disadvantage.

Bentonite slurry is used as lubricant during the sinking process.

c) OWS SOLETANCHE Method

By dividing the side wall of the shaft in segment on ground, the segment is constructed after the guide trench is built, in which slurry is then filled to complete preparation work for excavation that as shown in Fig. 15.

The shaft segments are excavated by KELLY excavator, by the powerful hydraulic grab on the tip of the KELLY bar and the own weight of the KELLY bar which pushes the grab into the soil.

During the excavation work, Bentonite slurry is constantly filled inside the segment. The slurry forms a waterproof layer on the surrounding part of excavated holes and this prevents the hole wall collapse.

After completion of excavation, slime is removed and the prefabricated reinforcement cage is inserted into the excavated holes. Concrete is poured through the tremie and slurry may be gradually pushed upwards from the bottom of the hole. The tip end of the tremie is carefully adjusted in such a manner that it will be constantly sustained more than 2 m. below concrete upper surface.

After completion of one segment of the wall, the adjacent segment construction is carried out.

Consequential construction of the wall segment will finally be completed to form the continuous reinforced concrete walls.

The OWS-SOLETANCHE walls thus constructed will form a water-tight and solid structure in which the wall segments are strongly held together.

After finishing all segment the soil inside the shaft is excavated, the segment acts like sheet piles.

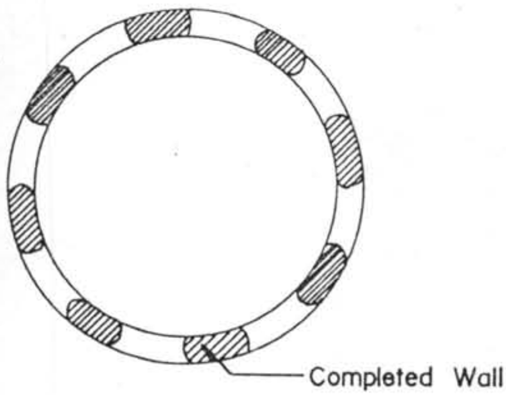


Fig. 15 a Shaft segments.

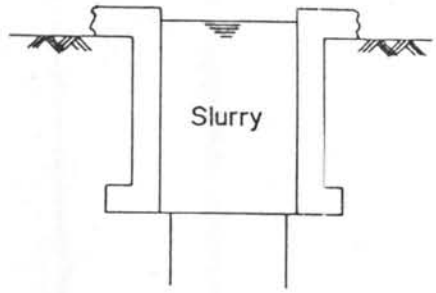


Fig. 15 b Guide trench

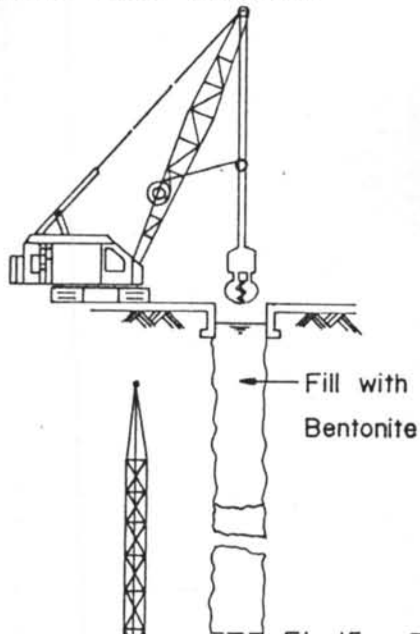


Fig. 15 c Excavation

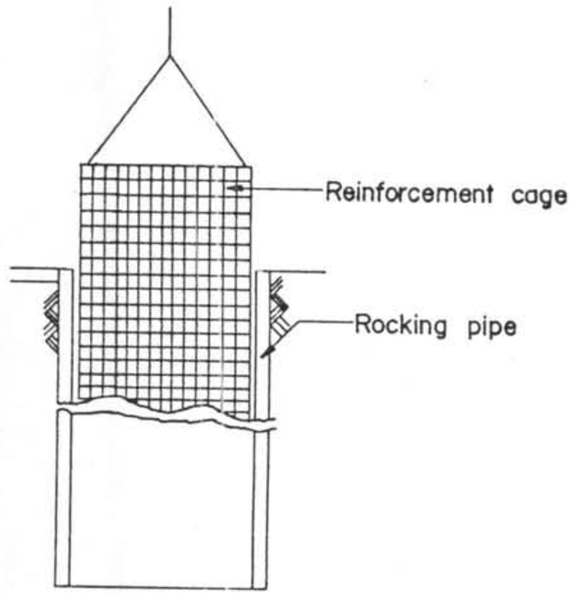


Fig. 15 d. Steel reinforcement cage insertion.

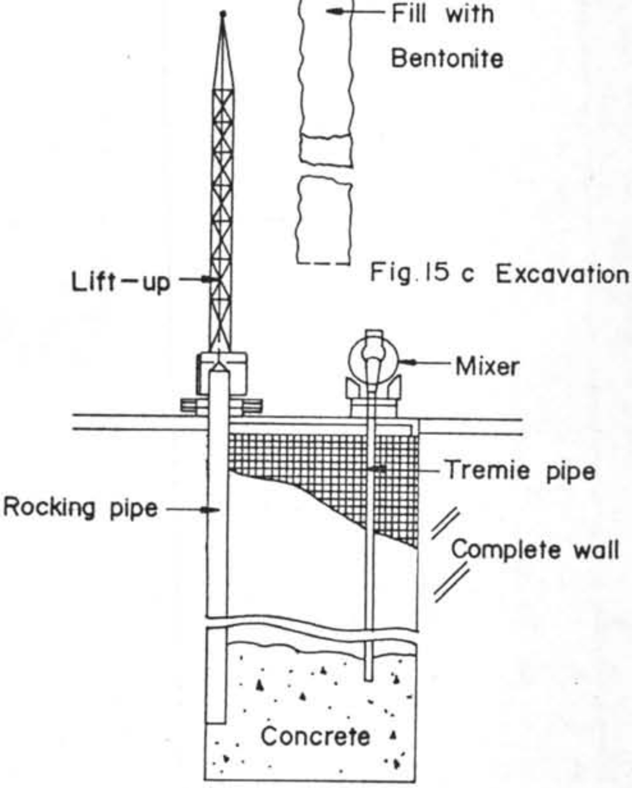


Fig. 15 e Concrete placing

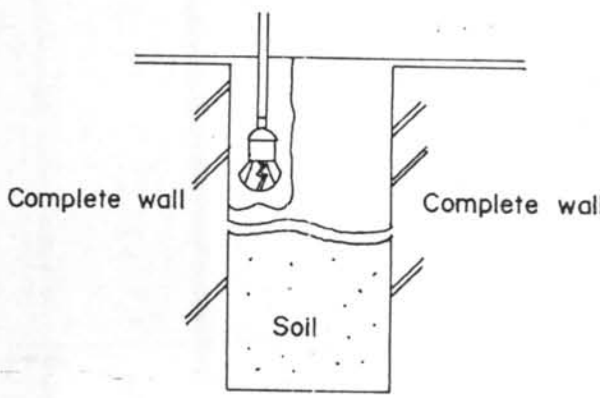


Fig 15 f Excavation after wall is completed.

Fig 15 Shaft construction by OWS – SOLETANCHE method.

2. Surveying for Alignment

2.1 Scope of Survey Work for Tunnel

a) Preliminary Survey

A preliminary horizontal and vertical control survey is required to obtain general site data for route selection and for structural design. The preliminary survey relies on the existing survey records and monuments.

b) Geological Survey and Exploration

The exploration of geological conditions is the most important phase of preliminary work in tunnelling. It affected on both the loads acting on the tunnel lining and the choice of the preferable tunnelling method.

The general location of the tunnel route is governed by existing traffic, while the exact location is controlled by geological condition.

c) Primary Control during Final Design and Construction

After the route selection is complete, a horizontal and vertical survey of a high order of accuracy is conducted permanent monument and bench marks are secured in concrete over the tunnel alignment to serve as primary control during construction.

d) Survey Work during Construction

During the construction of tunnel, the following survey work is performed.

1. Transfer of tunnel center line, and grade from the primary control monument at the ground surface to the tunnel.

2. Underground control monuments are established in the tunnel crown by transferring from the primary control monument at the ground surface.

3. Control of a tunnel construction system is developed to assure the driving of tunnels within the allowable tolerance.

4. Observation holes are installed to check the tunnel alignment, before the tunnelling machine is driven to the accessible shaft.

5. Measure the ground surface movement over the tunnel alignment.

2.2 Controlling of Tunnel Alignment

The tunnel is like a continously extending cylinder along a given axis. If the tunnel is in any deviation the tunnel cross sectional plane will depart from the true axis and the correction must immediately be taken to return the tunnel to its correct axis.

The following tunnel terms are used for the variations of the plane to the horizontal or vertical axis.

a) Vertical Axis

If the vertical cross-sectional plane of the tunnel is perpendicular to the horizontal tunnel axis, the tunnel is plumb (Fig. 16a).

The plumb is measured by hanging a plumb bob from the crown of the tunnel of the leading edge of a ring and measuring the difference at invert of the tunnel (Fig. 16b). If the tunnel axis is going up, the tunnel plane at the invert of the tunnel will be deviated forward from the tunnel theoretical plane, the tunnel is called look up (Fig. 16b). If the tunnel axis is going down, the tunnel plane at the invert of the tunnel will be deviated backwrad from the tunnel theoretical plane, the tunnel is called overhang.

b) Horizontal Axis

If the horizontal cross-sectional plane of the tunnel is perpendicular to the horizontal tunnel axis, the tunnel is called square (Fig. 17a). For curved tunnel, the tunnel is square when the cross-sectional plane is radial to the curve. If one side of tunnel is deviated from the theoretical square, the tunnel is called lead

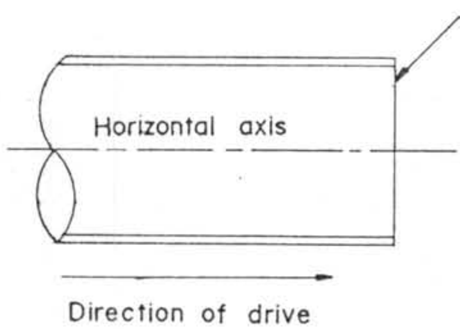


Fig 16 a Horizontal tunnel

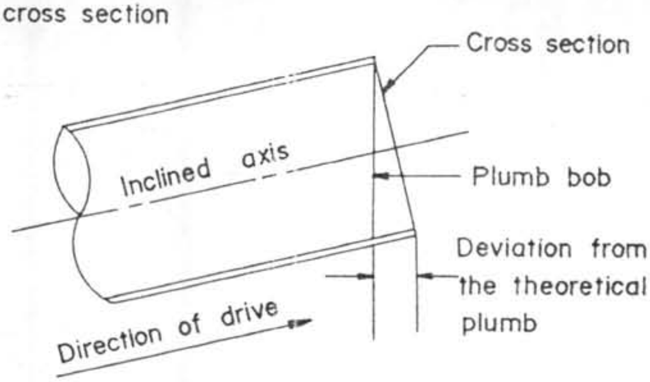


Fig 16 b Inclined tunnel

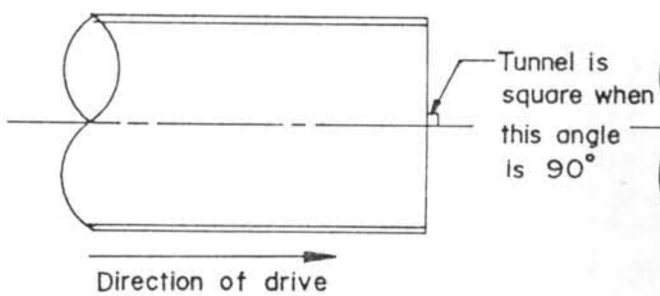


Fig. 17 a Plan of tunnel square

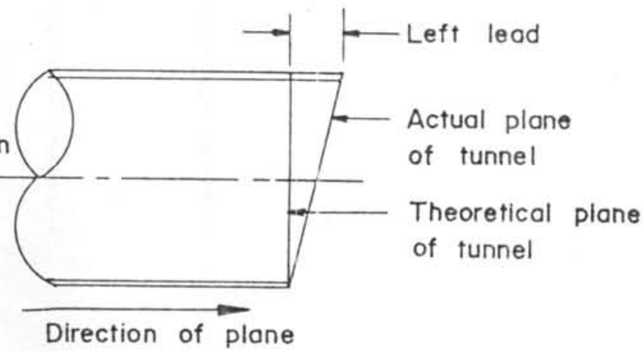


Fig 17 b Plan of tunnel with left lead

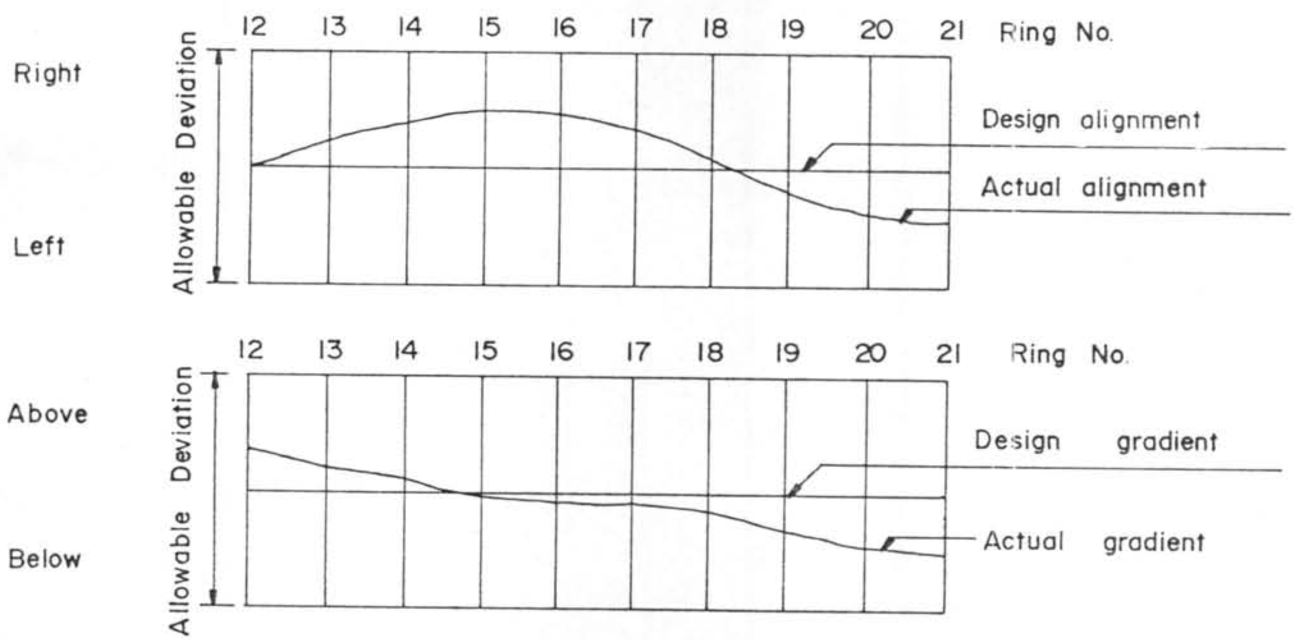


Fig 18 Wriggle diagram

(Fig. 17b). Left lead causes the tunned to deviate to the right. Right lead causes the tunnel to deviate to the left.

All referances to the left or right side of the tunnel assuming that the observer is looking toward the tunnel face.

2.3 Wriggle Survey

It is necessary to prepare a wriggle diagram as soon as possible after the driving is complete. The position of primary lining rings are shown relative to the design tunnel alignment.

Measurements are taken from the offset line to each side of the lining and level taken at the crown and invert while the driving was in progress. The actual tunnel axis is measured at the middle point of horizontal and vertical diameter of each ring.

The differences in the form of left or right and above or below of tunnel true position and design position of each primary lining ring are plotted to a distorted scale on a diagram (Fig. 18)

A small rectangular on the graph in the longitudinal represents one ring and chainage present the allowable intervals, along the tract centerline given on the diagram Ring numbers, design alignment and gradients are also shown in Fig. 18. The straight heavy line in each graph represents the intended position. If the deviation of the true tunnel position and design tunnel position is more than the allowable tolerance, the construction of that ring is not approved.

2.4 Monument Transferred to the Tunnel

A tunnel base line is the line whose bearing and end point co-ordinates are determined directly from the surface.

One end of the base line is arranged to be used as the tunnel monuments when the "wiesbach wire" method is used.

a) The Principles of Wiesbach Wire Method

The wiesbach wire method is designed to eliminate the constant error in angles and transfer the tunnel base line from the surface without any loss of accuracy.

1. Estimate the surface and underground wire points w_1, w_2 whose positions are assumed to be triangular bases. The base angle must not exceed 30" ($\theta = \sin\theta$, if θ is a small angle).

2. Ensure that the wire anchorage points w_1, w_2 at the top of the shaft are completly stable and have no effect by vibration, wind or any interference.

3. The 15 kgs. plumb bobs are suspended to the bottom of the shaft and oil in the suitable size containers to ensure their free movement. The plumb bob should have vanes to resist rotation.

4. The No. 4 piano wires for the plumb bob hanging wire must be free from kink and rust.

b) Calculation Principles

From Fig. 19.

If W_1, W_2 are the wire points.

R_1, R_2 are the reference objects on surface whose co-ordinates and azimuth of line R_1 to R_2 are known.

M_1, M_2 are the underground monuments whose co-ordinates and azimuth of line M_1 to M_2 must be found.

1. On ground surface

$$\frac{W_1W_2}{\sin \delta_1} = \frac{W_1R_1}{\sin \delta_2} \qquad \text{(sine law)}$$

W_1W_2 is the distance from wire to wire

W_1R_1 is the distance from wire point to the nearest reference object.

If δ_1 and δ_2 are the small angles.

$$\delta_1 = \left(\frac{W_1W_2}{W_1R_1}\right) \delta_2 \qquad \text{............... 2.13}$$

From δ_1, the bearing and azimuth of W_1W_2 are known.

2. In the tunnel

$$\theta_1 = \left(\frac{W_2M_1}{W_1W_2}\right) \theta_2 \qquad \text{............... 2.14}$$

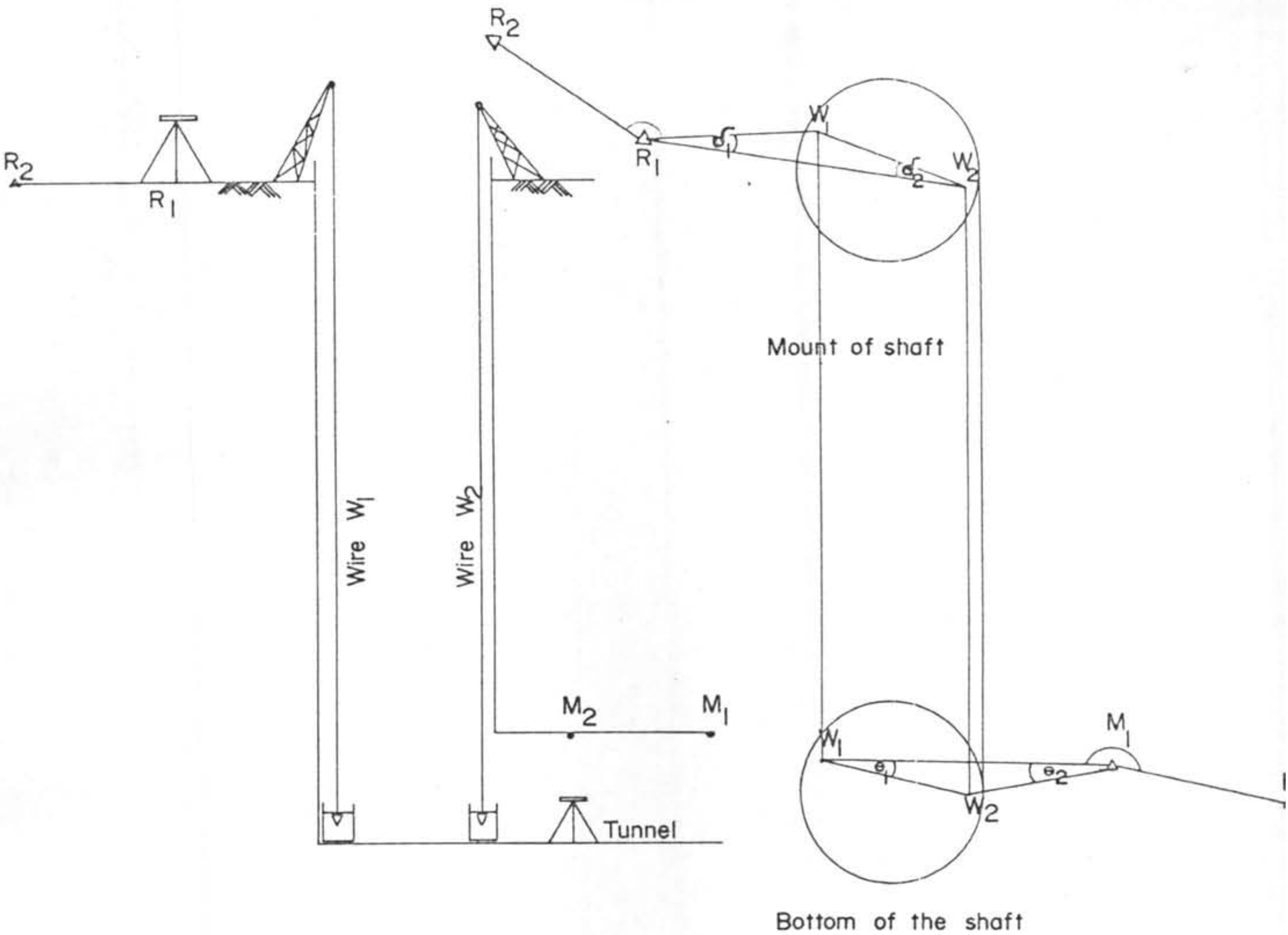


Fig. 19 Monument transferred to the tunnel.

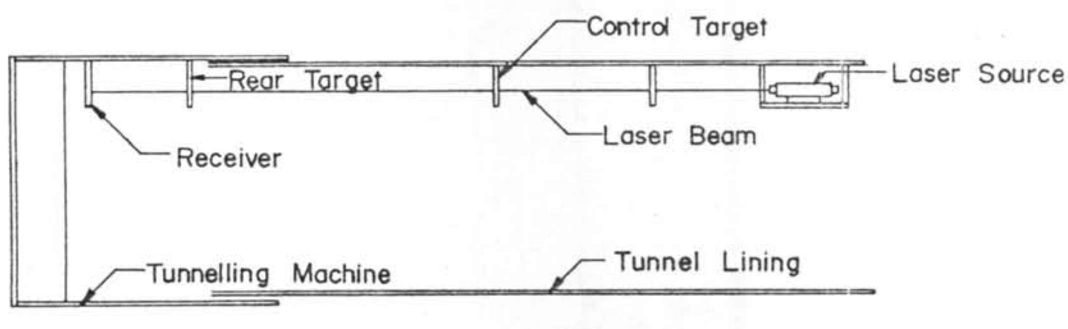


Fig. 20 Control of tunnelling machine by laser system.

W_2M_1 is the distance from wire to the nearest monument. From θ_1, the bearing, azimuth of M_1M_2 and co-ordinates of M_1 and M_2 are known.

Because of the disturbance of any interference in the tunnel, the tunnel monuments should occasionally be checked.

2.5 Tunnel Guidance System

Laser (Light of amplification by stimulated emission of rediation), the device for producing a beam of light is basically a gas filled tube with laser action to stimulated emission, it is the most efficiency tunnel guidance equipment.

The stability of alignment is important when the beam is used for visual guidance.

a) The Composition of Alignment Control Laser

The composition of Alignment control laser (in Fig. 20) are as follows:

1. Laser Source

Establish a line of sight which is parallel to the tunnel center line by laser source and provides an uninterrupted view of the reciever on the shield. Ensure that the source is stable and free from any interference.

2. Control Target

Control target is a piece of metal with a hole just large enough for the laser beam to pass through. The control targets are set on laser beam line between the laser source and receiver at tunnelling machine (Fig. 20).

A good method of laser alignment checking is three control targets. If the source moves, the disturbance is quickly noticed.

3. Receiver

The attitude of tunnelling machine is shown by the the suitable receiver. The receiver is intersected by the laser beam which produces a bright red spot. The shield is guided by attempting to maintain coincidence of actual line intersection point with the predetermined intersection point in the receiver.

The receiver screen is calibrated in both horizontal and vertical directions with the position of each jack shown to true scale (Fig. 21).

b) The Precision of the Beam Deflection

When laser alignment is used for tunnelling, it is limited to line of sight operations. If the tunnel alignment is in curve or incline, the laser beam will be off axis. The precision beam

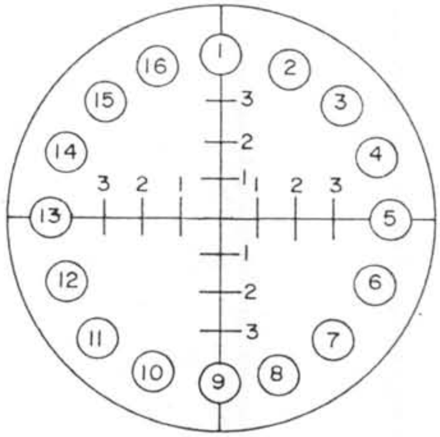


Fig. 21 The Laser Receiver.

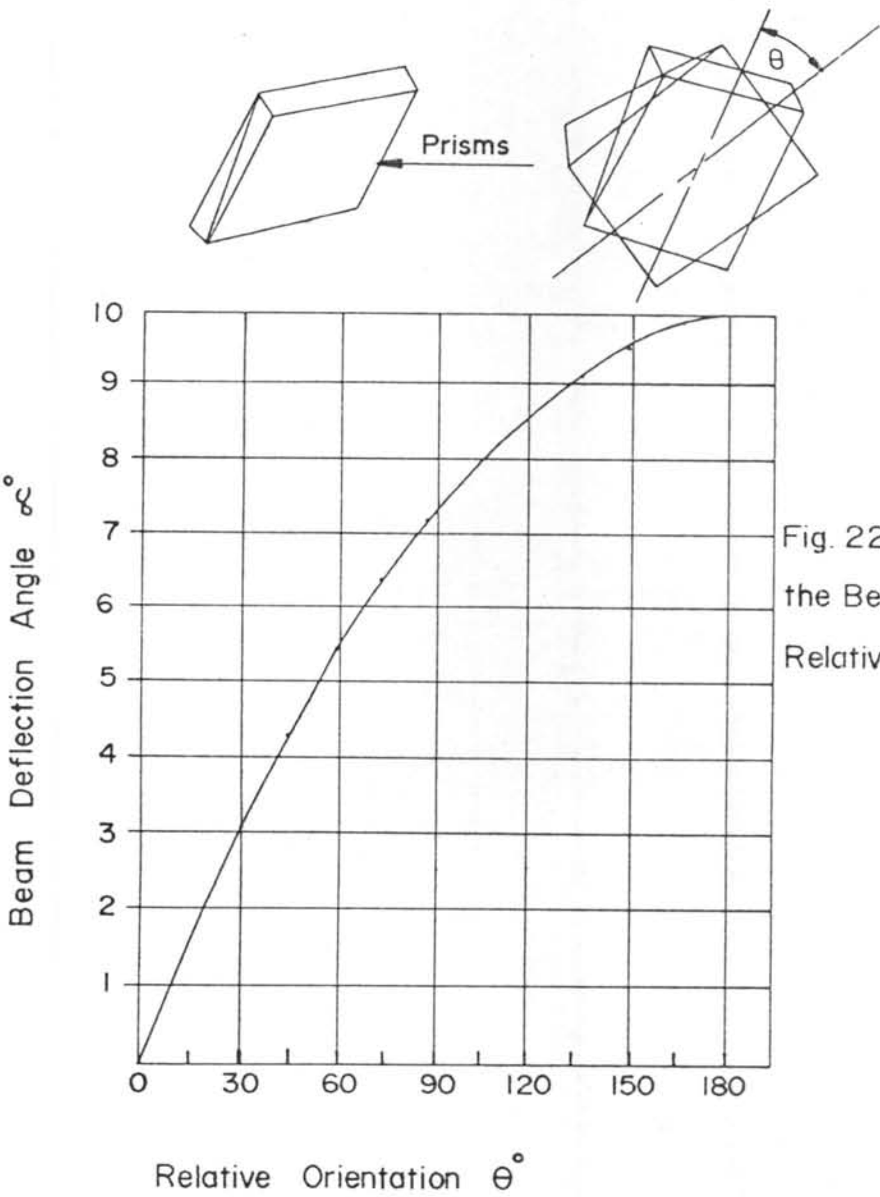


Fig. 22 Relationship between the Beam Deflection Angle α an[d] Relative Orientation $\theta°$

deflection is used to deflect the laser beam along the chords of curves and inclines (Fig. 22).

The use of this deflection system allows the laser source to be situated in a convenient position at an increased distance from the working zone.

1. Technical Description

The beam deflection technique is based on the fact that a pair of similar optical prisms when placed together and rotated relative to each other will give the same effects as a single prism angle varies from zero to twice of the prism angle of the components.

Fig. 22 shows the relationship between the beam deflection and the relative orientation of the prisms which can be expressed as

$$\alpha = \alpha_{max} \sin \frac{\theta}{2} \qquad \text{.................} \; 2.15$$

where

α_{max} = The maximum deflection angle of the precision beam deflection.

α = The off-axis deflection of the laser beam.

θ = Relative orientation of the prisms.

3. Tunnel Driving Machine

Tunnel driving machine or shield is a moving steel cylinder which is driven in advance of the permanent tunnel lining, to support the loose ground surrounding the tunnel-bore and afford protection for construction of the permanent lining.

3.1 Characteristic of Shield Tunnelling Method

a) Advantages and Disadvantage

The shield tunnelling method has both advantages and disadvantages in comparison with the conventional cut and cover method.

Advantages

1. Less trouble to the structures on the ground surface.
2. Construction under deep may be possible.
3. Less removal of the underground installation.
4. Less noise and vibration.
5. Leakage of water may be prevented by the use of compressed air.
6. Reduction of construction period may be possible.
7. More economical construction costs.

Disadvantages

1. The surface settlement may occur.

2. Careful geological survey may be neccessary.

3. Safety treatments are necessary in compressed air section.

b) Machanized Shield

Shield tunnelling in the past had been proceeded by man power, but this method caused many risks and troubles for work. Machanizing methods of tunnelling become available after long times required for the improvement.

Excavating machinery for the soft ground excavation are called machinized shield, which is required when .

1. The tunnel is long enough.

2. There is sufficient supply of electric power.

3. The project site is well-organized to proceed high efficiency works.

Advantages and disadvantages of machanized shields are as follows:

Advantages

1. Man power can be reduced.

2. Construction schedule can be shortened and project cost can be cut down.

3. Collapse of the tunnel face can be prevented.

4. Working conditions (for labours) are more favorable.

Disadvantages

1. Machinery costs are high.

2. Repair of machinery takes much times and difficulties.

3. A partial trouble of machinery sometimes causes stop of the whole work.

4. The design and manufacture of the shield takes much times.

3.2 Structure and Machanism of The Shield (Fig. 23)

a) The Shield Shell

The shield shell (Fig. 24) is the steel plate, bent to the shape of the tunnel section but slightly larger, it is composed of the following three main parts, in accordance with their purposes.

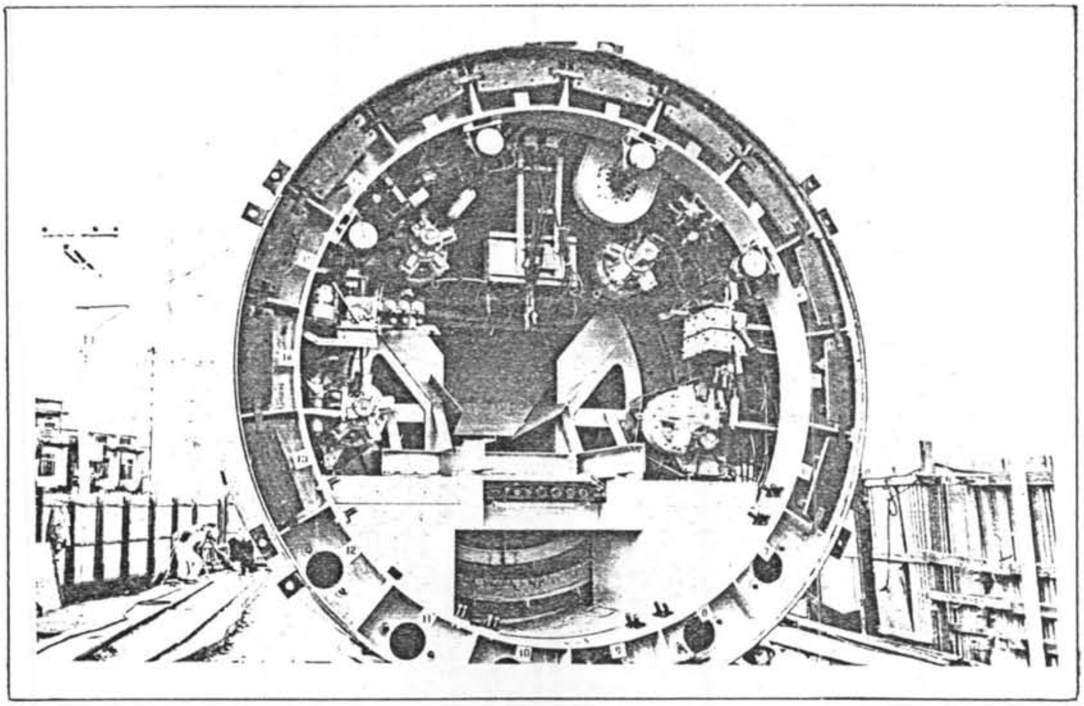


Fig. 23 Structure of tunnelling machine

Fig. 24 The shield shell

1. The Front Part

The front part of the shield shell (Fig. 25), where excavation is effected, is heavily reinforced generally with steel to form the "Cutting edge".

Its principal purpose is to facilitate possible smooth advanced and steerability of the shield shell by cutting the face, and to provide for as uniform as possible distribution of the pressures induced by its being forced ahead. The purpose of the front part is to give adequate shelter to workmen engaged in the excavation.

2. The Intermediate Part

The intermediate part (Fig. 25) consist of ring girder which links "front part" and "tail part" of the shield. This part is designed for the room of hydraulic (hydraulic jacks, high pressure pump installation).

3. The Tail Part

The tail part of the shield (Fig. 25) is designed for the erection of the lining segments. The length of the tail part (L) is determined by Kawai, Takahashi and Sasaki[9] as shown below.

$$L = t + L_s + X \qquad \text{.................... 2.16}$$

where t = width of lining segment

L_s = thickness of spreader

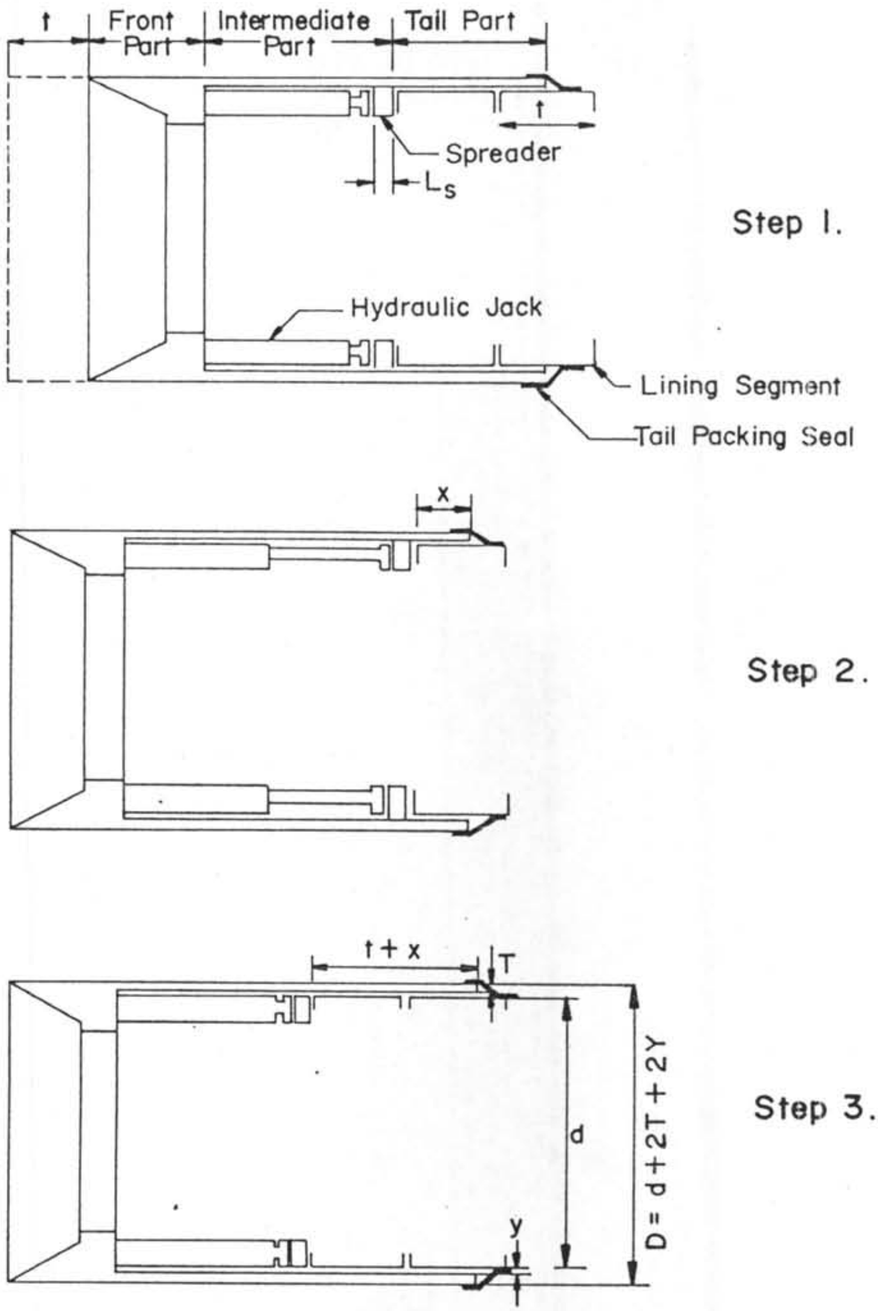


Fig. 25 Diagrammatic section of shield shell .

X = the provisional space between spreader face and segment in erection.

Tail packing seal is provided between the tail skin and the external face of segments in order to prevent the leakage of the back fill material (grouting) as shown in Fig. 25.

Natural or synthetic rubber is used for the packing materials in double or triple layer.

4. Structural Design of the Shield Shell

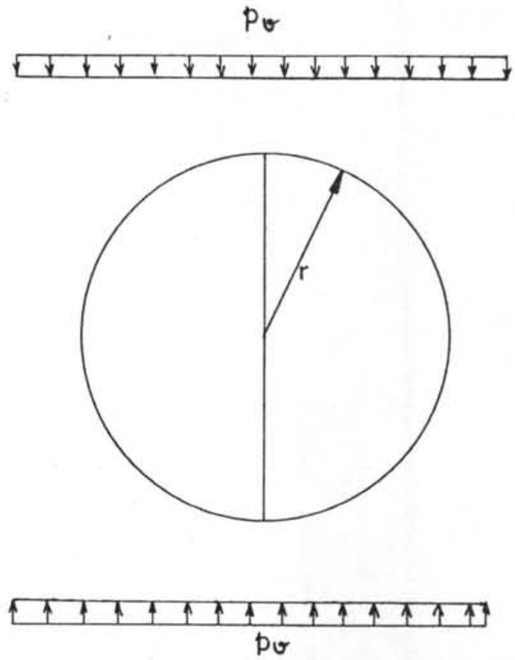
In the design of the shield shell, accurate estimation of loosened earth pressure should be required in order to calculate thickness of the shell (Fig. 26).

The design load is represented by the action of full overburden weight from above and counteracted by an equal bottom reaction, and no lateral support owing to an excessive deformation on the sides of the shell.

From Szechy[15]

Bending moment of the shield shell.

$$M = \frac{P_v r^2}{4} \cos 2\phi \qquad \text{.................... 2.17}$$



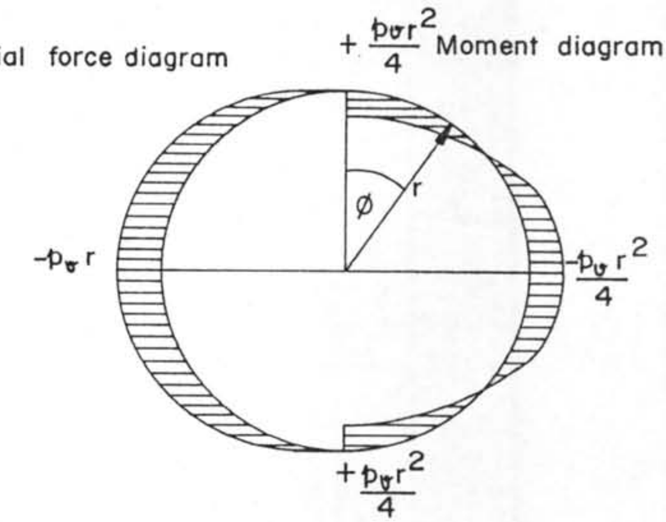


Fig. 26 Moment and axial force diagram of shield shell.

and axial force

$$N = p_v r \sin^2 \phi \qquad \ldots\ldots\ldots\ldots\ldots 2.18$$

Maximum bending moment at crown, invert and both sides

$$M_{max} = \pm \frac{p_v r^2}{4}$$

Maximum axial force at the both sides.

$$N_{max} = - p_v f$$

Minimum axial force at crown and invert.

$$N_{min} = 0$$

when p_v = the vertical earth pressure.

5. Dimension of the Shield Shell

It is specified that the external diameter of the shell is as short as possible, however the erection of segments should be smoothly taken in the shield interior.

From Szechy[15], the external diameter of shield, D will be as shown in Fig. 25 and following formula.

$$D = d + 2T + 2y \qquad \ldots\ldots\ldots\ldots\ldots 2.19$$

where

y = tail clearance between inner face of skin plate and external face of segment.

T = thickness of skin plate.

d = external diameter of segment ring.

b) Shoving Equipment

Powerful hydraulic shoving equipment or jacks are placed near inside the shell skin in order to bring their axes close to the center of gravity of the lining segments. The location of jacks should be selected to provide the safe steerability to the shield and necessary space for the erection of the lining segments.

Resistant forces against propulsion of a shield (W) are as follows:

$$W = W_1 + W_2 + W_3 \quad \text{(Szechy}^{(15)}\text{)} \quad \ldots\ldots\ldots\ldots \; 2.20$$

W_1 = friction between the external surface of shield and surrounding ground.

W_2 = friction between the internal surface of tail skin and the lining segments.

W_3 = passive earth pressure of soil against the intruding surface of the cutting edge.

$$W_1 = \left(\frac{p_v + p_h}{2} L\pi D + Q\right) f_1 \quad \ldots\ldots\ldots\ldots\ldots \; 2.21$$

where

p_v = the vertical earth pressure.

p_h = the horizontal earth pressure.

L = the length of shield.

D = the diameter of shield.

f_1 = friction coefficient between the shield shell and soil.

Q = the weight of the shield

$$W_2 = G f_2 \qquad \text{...............2.22}$$

where

f_2 = friction coefficient between the tail skin and segments.

G = the weight of the lining.

$$W_3 = \pi D_1 b p_v K_p \qquad \text{...............2.23}$$

where

D_1 = diameter of shield at center line of cutting edge

b = thickness of cutting edge.

p_v = the vertical earth pressure.

K_p = the coefficient of passive earth pressure.

c) Excavating Equipment

The excavating equipment of the shield is designed to cope with the situation of the surrounding ground. The open-face cutter head is used in stiff clay. In case of loose surrounding ground, blind faced cutter head with rotary cutter plate is performed.

The most important part of the cutter head is to support mechanism which endures large thrust and radial load. The cutter plate is provided with the cutter bits and slits at the cutter frames (Fig. 27).

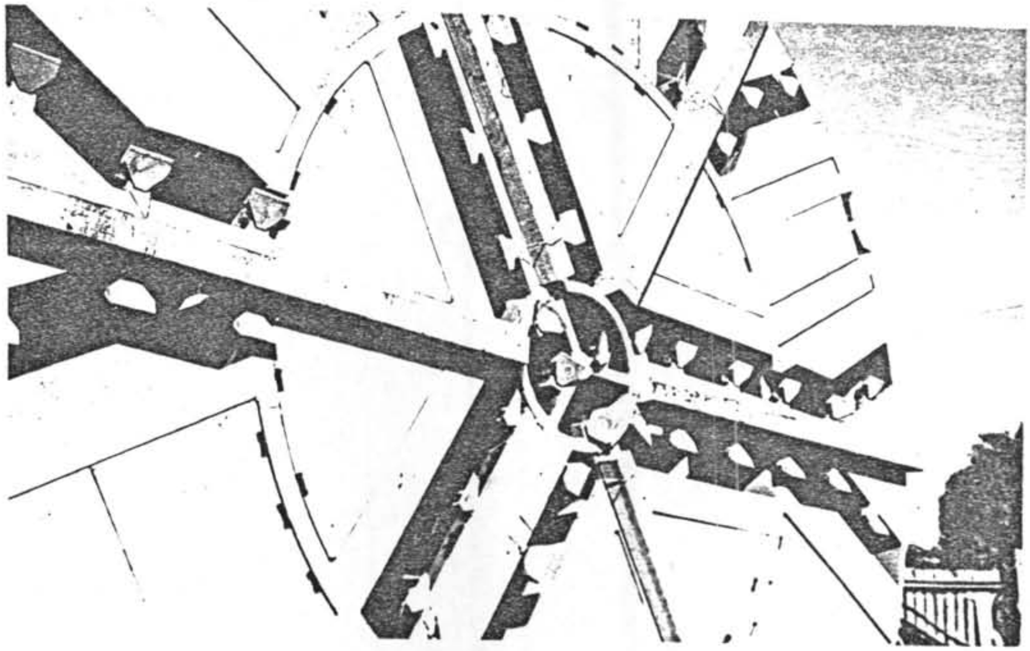


Fig. 27 Blind face cutter head of tunnelling machine

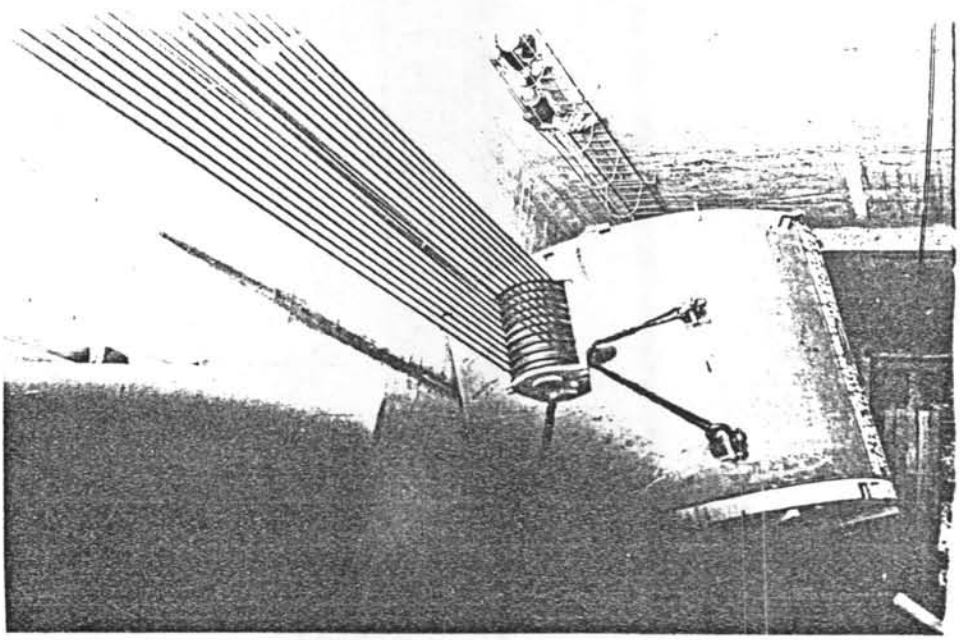


Fig. 28 The transportation of the tunnelling machine

When the cutter plate rotates, the front face attached to the soil is cut into small pieces and enter into the shield rhrough the slits.

The cutter frame not only transmits the cutter torque but also resists loads and earth pressure.

The rotaring torque of cutter head is calculated by Kawaî, Takahashi and Sasaki[9] in two ways. When the face stands by itself, the rotating torque of-cutter head (T) will be obtained as follows:

$$T = T_e + T_f + T_c + T_n \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.24$$

where

T_e = torque which overcomes cutting resistances of soil.

T_f = torque which overcomes frictional resistances.

T_c = torque which overcomes cohesive resistances.

T_n = torque required for speed up the cutter head

If the face does not stand by itself, the formula is

$$T = T_a + T_n \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.25$$

where

T_a = torque which overcomes friction resistance of loosened earth pressure.

d) Spreader

The shield is forced ahead by propulsion of shoving jacks with reaction against permanent lining segments, spreader is used between jacks and segments in order that the propulsion may not destroy segments and it may be distributed uniformly to the facr of segments. Spreader is usually made from hard rubber or wooden material.

e) Segment Erector

Segments of primary lining are erected with a hydraulically operated erector arm, which is mounted directly on the axis of the shield tail.

The erector arm can be rotated around the center axis of shield to any required position by hydraulic jacks. Rotating torque of the erector and propulsion of the jack are determined by segment weight. The erector is operated through electro-magnetic valves under the control of shield operator.

3.3 Elements in the Design of Shield

The following items are considered in the design of shield.

1. Over-burden of ground.

2. Ground water pressure.

3. Results of geological survey.

4. Shape of tunnel section.

5. Form of tunnel alignment.

3.4 The Shape of Shield

Tunnelling machines or shields are usually circular cross-section. There are several reasons for this.

1. The circular shape is the most ideally suited to resist the semifluid pressures of soft ground and provided the greatest cross-sectional area with a minimum of perimeter.

2. The circular shape gives the rotation of the shield without effecting the primary lining, which is being erected within the tail.

The length of shield should be sufficient to provide protection for the men and allow space for support erection within the tail of the shield.

A shield should be long enough so that it will not hvae tendency to change its direction rapidly, when it is pushed forward.

3.5 Specialty of Shield Machinery

a. Semi mechanized shield.

b. Mechanized shield.

c. Slurry shield.

The first and second type of shield are suitability in ground, under compressed air condition. The sufficient pressure of compressed air is maintained in the tunnel to balance the water pressure and restrain the inflow of water and muck.

An unbalanced pressure exists on the face of the tunnel because of the difference in hydraulic head at the crown and invert of the tunnel. In low permeability soil, hydrostatic pressure is reduced by compressed air, and the effect of hydraulic head difference between the crown and invert is neglilible.

a) Semi Mechanized Shield

The semi mechanized shield or open face is suitable in stiff to medium clay. It is provided with such excavating equipments as hydraulic shover, digger and excavator.

b) Mechanized Shield

The mechanized shield is suitable in soft clay. The excavating equipment is a full face rotating cutter head that have been designed with the object of letting through just the right amount of spoil to avoid settlement.

c) Slurry Shield

A tunnelling machine is required to drive through non cohesive ground above or below the water table without exposing men to work in high compressed air condition.

In non cohesive soil such as water bearing sand, it is quite pervious. The air can penetrate into the sand for a distance, until a point of equilibrium is reached.

If compressed air pressure at the center of tunnel is equal to the pressure head, the effect of pressure difference at the crown and the invert of tunnel will be as shown in Fig. 29. At the invert, the deficiency of air pressure will allow a flow of water into the tunnel, while the water at crown is expelled by the excess air pressure. Hence the sand above the crown is unstable and moves down to the tunnel.

The slurry shield is a full face excavator with a bulk-head. The chamber forward of bulk-head is filled with slurry which is pressurised to resist ground movement and water inflow during excavation.

The choice of slurry depends on the ground to be excavated. In non-cohesive soils a thixotropic slurry (bentonite) is suitable. In cohesive soils with low permeability bentonite is not necessary.

The slurry was circulated so that slurry plus spoil travelled to a separation plant where the spoil was removed and the purified slurry returned to working chamber. The separation plant could be in the tunnel, or at reclamation site on ground surface.

The separation plant contains a series of vibrating screens that sift and separate the slurry so that the heavier stones are discarded down on chute and sand particles down another. Clean bentonite slurry returns to its own storage tank for recycling (Fig. 30).

The main problems of slurry shield are as follows.

1. Control of slurry pressure.

2. Selection of slurry.

3. Performance of tail seals.

3.6 The First Stage of Shield Advancement

After the shaft was sunk to the required depth, a shield machine is brought into bottom of the shaft as shown in Fig. 28.

When a reinforced concrete shaft is sunk, it is often provided with bull-eye (the opening through which the shield is forced ahead).

The shield starts to advance at bull-eye with the erection against the shaft well, which should be rigid enough to endure the propulsion. For this purpose, steel forms are used as transmitting the propulsion between the temporary erected segments and the shaft wall (Fig. 31).

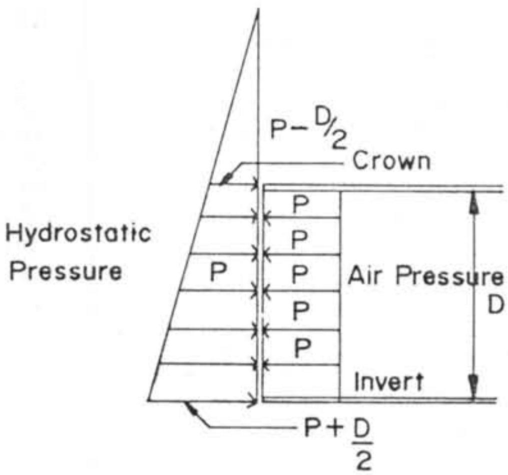


Fig. 29 − a

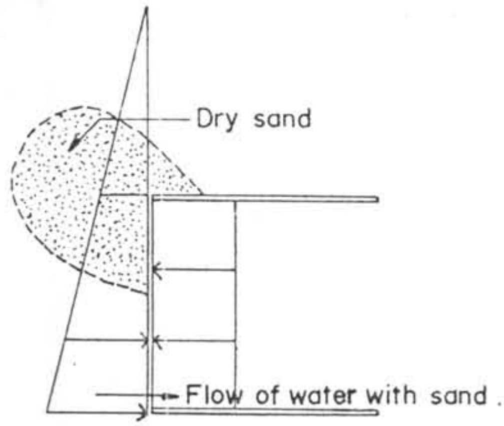


Fig. 29 − b

Fig. 29 Effect of compressed air to surrounding ground

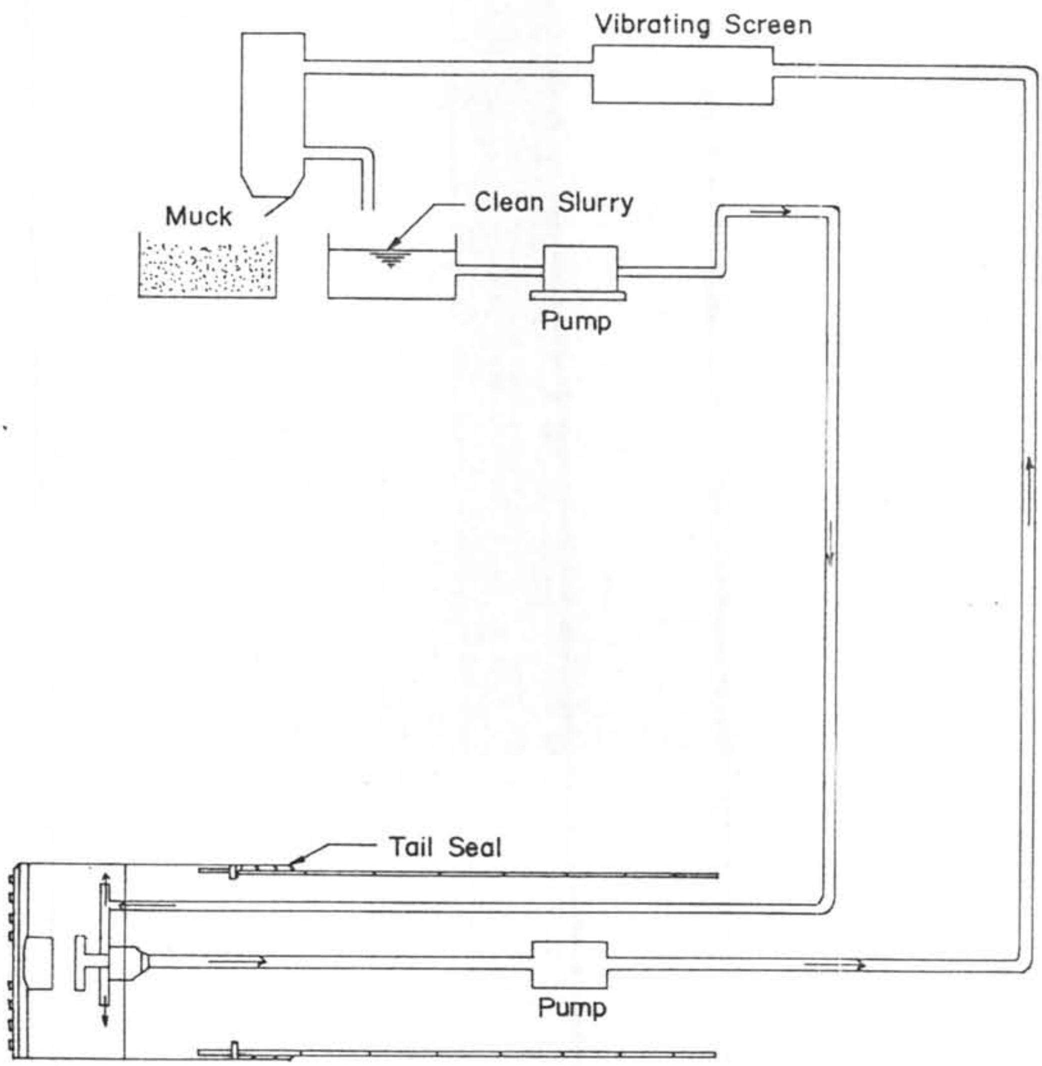


Fig. 30 Slurry Shield cycle.

3.7 Mucking

Effective mucking is one of the major works for the efficient tunnel driving.

In case of shield tunnelling (except slurry shield) it is performed in two steps. The first is the removal of soil from the shield body and the second is its conveyance from the working area. In the first step the rapid removal from the shield body is the most important, therefore the use of efficient belt conveyors in necessary (Fig. 32).

The second step is the transportation of spoil through the completed tunnel section to the access shaft by transporting locomotive.

In the slurry shield the spoil and slurry are mixed together and flow up to a separation plant by out-let pipe. After separating the spil from slurry, clean bentonite slurry returns to its own stage tank for using again.

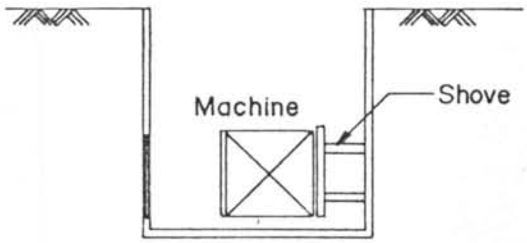
3.8 Cycle of Shield Tunnelling

A full cycle of shield tunnelling consists of three main steps.

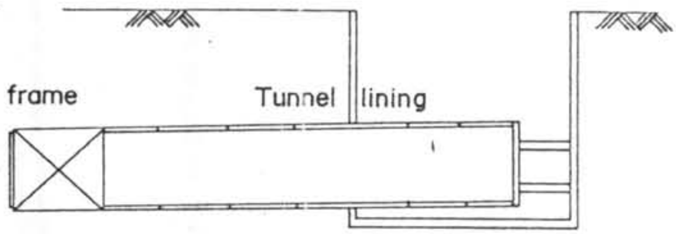
1. The excavation of ground by the shield tunnelling machine as it advances.

2. The erection of primary lining under the protection of the tail of the shield, and

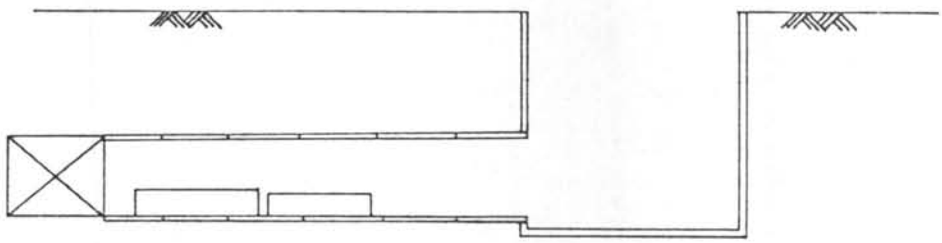
a) Shaft sinking



b) Shield installation



c) Shield driving



d) Machine installation

Fig 31 Shield advancement

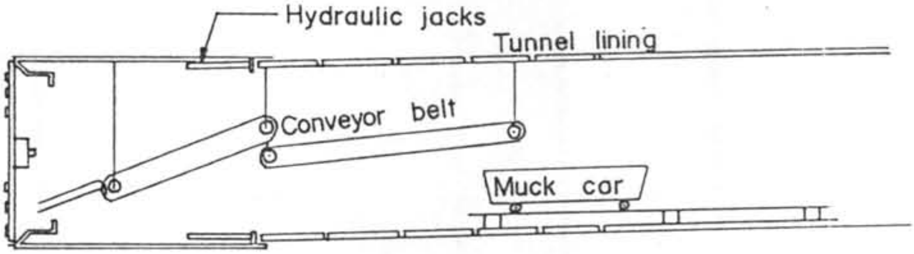


Fig.32 Removal of soil

3. The fill of the annular void between the lining and the surrounding ground.

4. Primary Lining

The primary lining is the prefabricated reinforced concrete segments that can be erected with-in the tunnel as soon as the tunnel driving machine advances. It is shown in Fig. 33.

4.1 Active Load on Primary Lining

a) Permanent Load

It is the full soil pressure immediately as the shield is advanced to the next ring.

b) Temporary Load

The tunnelling machine is advanced by thrusting on the completed tunnel lining, the erection of each ring has to be proceeded rapidly within the protection of tail piece of tunnel driving machine. The jack stress is greatest on the first or second ring after erection, after that the jack pressure passes out to the ground. The jack pressue passing through a ring is quite small after the machine gets 15-20 rings ahead.

4.2 Active Pressure and Reactions

For all conditions of tunnel loading, active pressures and reactions must be in static equilibrium. The hydrostatic uplift must be safely counteracted by the overburden and the weight of tunnel lining.

The tunnel footing loads should not exceed the safe bearing capacity of the supporting material.

4.3 Stress Analysis

Stresses and deflections in tunnel lining seldom agree with the derived equations based on the assumed loading and accepted methods of analysis.

The reasons are as follows:

1. The assumed loading and distribution of reactions do not conform with actual loading conditions.

2. The materials and construction of the lining do not conform with basic assumptions as to elasticity, stiffness and other properties.

3. The application of the theories used to solve for stresses involves approximation and assumptions that may not be strictly correct.

4. There may be initial stresses resulting from erection or construction methods, which are difficult to evaluate.

The design and investigation of a tunnel lining have to be made to cover all reasonable loading conditions.

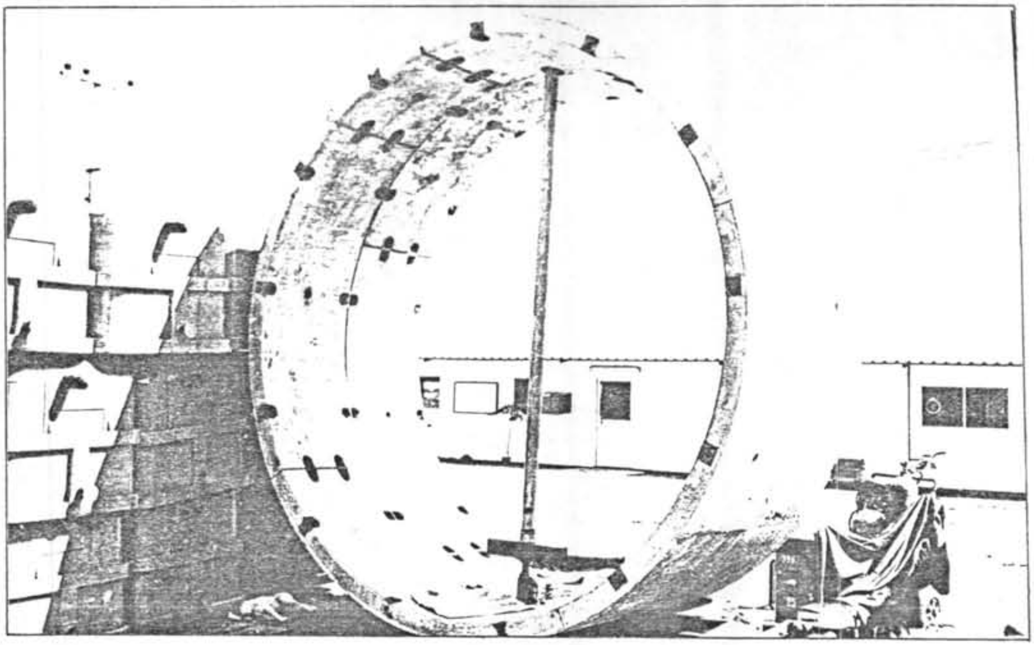


Fig. 33 Prefabricated reinforced concrete primary lining

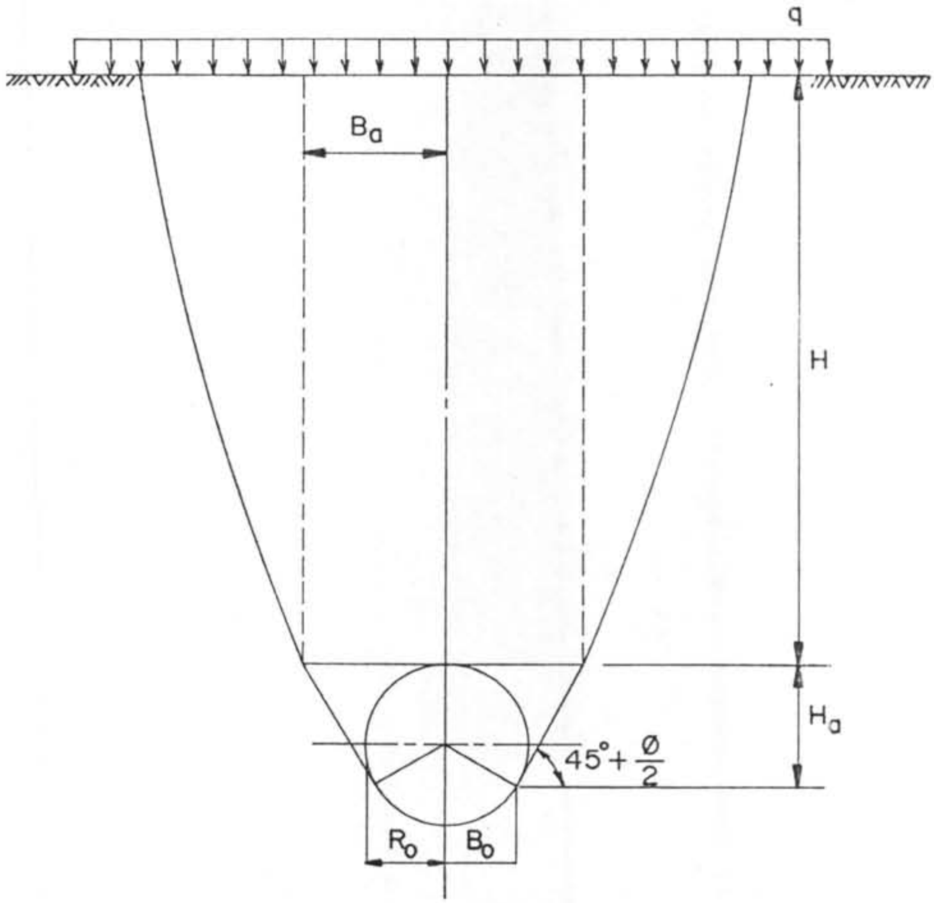


Fig. 34 Loosen zone of soil above tunnel lining

Tunnel linings are generally in the class of statically indeterminate structures and analytical methods for computing thrusts, bending moments and shear are available which permit stresses to be determined accurately with respect to assumptions.

4.4 <u>Design of Circular Tunnel Lining</u>

The methods of design are as follows:

a) Approximate method.

1. Design by dividing the section into segments.
2. Design by monolithic ring section method.
3. The Hewett-Johannesson's method.

b) <u>By considering sections elastically embedded in subsoil</u>

1. Bodrov-Gorelik's method.
2. The polygonal method.
3. Bougaveva's method.
4. Davidov's method.
5. Varga's method.
6. Meissner's method.
7. Duddeck and Schulze's method.
8. Orlov and Rozsa's method.

4.5 Design by a Monolithic Ring Section Method

This method has been considered in primary lining design in water transmission tunnel in Bangkok (Nishimatsu[13]).

4.5.1 Cohesion of Soil around The Lining

For cohesive soil

$$S = \frac{Q_u}{Q_r} \qquad \text{.................... 2.26}$$

where

Q_u = Unconfined compressive strength of undisturbed sample.

Q_r = Unconfined compressive strength of remolded sample.

S = Degree of sensitivity.

Assuming that the turbulence of clay around the shield excavation be about 1/3 of the turbulence of remolded sample them the unconfined compressive strength of clay around shield, Q_s

$$Q_s = \frac{3Q_u}{S} \qquad \text{.................... 2.27}$$

If c, cohesion of saturated clay, is about 1/2 of Q_s, then

$$c = \frac{Q_s}{2} \qquad \text{.................... 2.28}$$

4.5.2 Analysis of Earth Pressure due to Loosening

The earth pressure due to loosening was shown in "Theoretical Soil Mechanics" by Terzaghi[18]. It corresponds to the vertical load P_v acting against the top of segment ring as shown in Fig. 34.

$$B_o = R_o \cos (45^o - \frac{\phi}{2}) \qquad \text{.................. 2.29}$$

$$H_a = R_o \left[1 + \sin (45^o - \frac{\phi}{2}) \right] \qquad \text{.............. 2.30}$$

$$B_a = B_o + H_a \tan (45^o - \frac{\phi}{2}) \qquad \text{............. 2.31}$$

$$p_v = \frac{(\gamma . B_a - C)(1 - e^{\beta})}{K . \tan \phi} + qe^{\beta} \qquad \text{............ 2.32}$$

$$\beta = \frac{-K.H \tan \phi}{B_a} \qquad \text{................ 2.33}$$

where

p_v = vertical load acts against the top of the segment ring.

K = the ratio between the horizontal and vertical earth pressure.

H = Depth of the top of segment ring.

γ = Unit weight of soil.

γ_w = Unit weight of water.

R_o = Excavation radius.

ϕ = Angle of internal friction of soil.

c = Cohesion of soil.

q = The surface surcharge.

K_a = Coefficient of active earth pressure.

If the tunnel lining is under water table.

$$W_a = U_a + p_v \qquad \text{.................. } 2.34$$

W_a = Total vertical load at the top of segment ring.

U_a = Water pressure at the top of segment ring.

W_b = Horizontal load at the top of segment ring.

$$W_b = K_a(W_a - U_a) + U_a \qquad \text{.................. } 2.35$$

U_e = Water pressure at the bottom of segment ring.

Considering that the thickness of the lining is very small in comparison with the diameter of tunnel.

$$U_e = U_a + 2.\gamma_w.R_g \qquad \text{.................. } 2.36$$

R_g = Design radius of the lining

$W_a + W_c$ = Horizontal load at bottom of segment ring.

$$W_a + W_c = K_a(W_a + 2.\gamma.R_g - U_e) + U_e \qquad \text{............. } 2.37$$

Dead load of segment and its reaction force.

w = Dead load of segment ring per 1 m. width.

G = Dead load of segment ring per 1 m. of curve length per 1 m. width.

$$G = w/2\P.R_g \qquad \ldots\ldots\ldots\ldots\ldots\ 2.38$$

W_e = Subgrade reaction due to the dead load of segment ring per 1 m. width.

$$W_e = \frac{w}{2.R_g} = \P G \qquad \ldots\ldots\ldots\ldots\ldots\ 2.39$$

4.5.3 Horizontal Displacement of Primary Lining

From Szechy[15]

If

δ = Horizontal displacement of lining (final stable condition).

δ_a = Horizontal displacement by external force (without W_d).

δ_b = Horizontal displacement by horizontal subgrade reaction.

q = Coefficient of horizontal subgrade reaction.

W_d = Horizontal subgrade reaction.

E = Modulus of elasticity of the lining.

I = Moment of inertia around the centroid of section.

In Fig. 35

$$\delta_a = \frac{(2.W_a - 2W_b - W_c).R_g^4}{24.E.I} \qquad \ldots\ldots\ldots\ldots\ldots\ 2.40$$

$$\delta_b = \frac{0.0454.q.\delta.R_g^4}{EI} \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 2.41$$

$$\delta = \delta_a - \delta_b \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 2.42$$

$$= \frac{(2.W_a - 2.W_b - W_c).R_g^4}{24.E.I} - \frac{(0.0454.q.\delta.R_g^4)}{EI}$$

$$\delta = \frac{(2W_a - 2W_b - W_c)\ R_g^4}{24(EI + 0.0454q\ R_g^4)} \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 2.43$$

$$W_d = q.\delta \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 2.44$$

4.5.4 Effect of Design Loads

The design loads of tunnel lining taking from active load of earth pressure are developed by Szechy[15].

If M,N,Q are bending moment, axial force and shear force of the lining in Fig. 36.

a) Formulae of M_a, N_a, Q_a by vertical load W_a are

$$M_a = \frac{R_g^2.W_a}{4} \cos 2\,\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 2.45$$

$$N_a = R_g.W_a \sin^2 \theta \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 2.46$$

$$Q_a = \frac{-R_g.W_a}{2} \sin 2\,\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 2.47$$

b) Formulae of M_b, N_b, Q_b by horizontal load W_b are

$$M_b = \frac{-R_g^2.W_b}{4} \cos 2\,\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 2.48$$

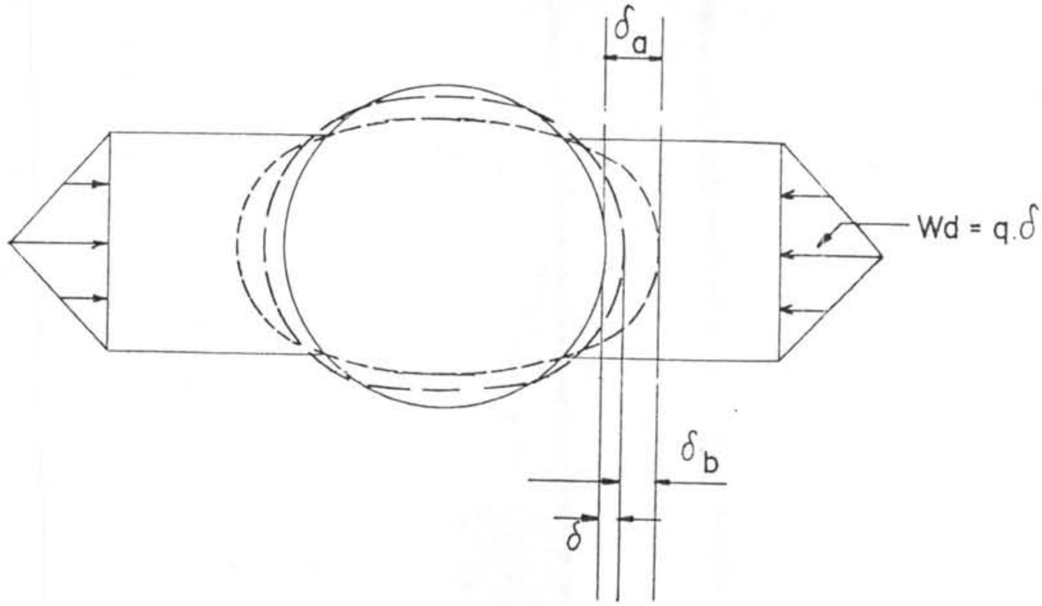


Fig. 35 Horizontal displacement of tunnel lining.

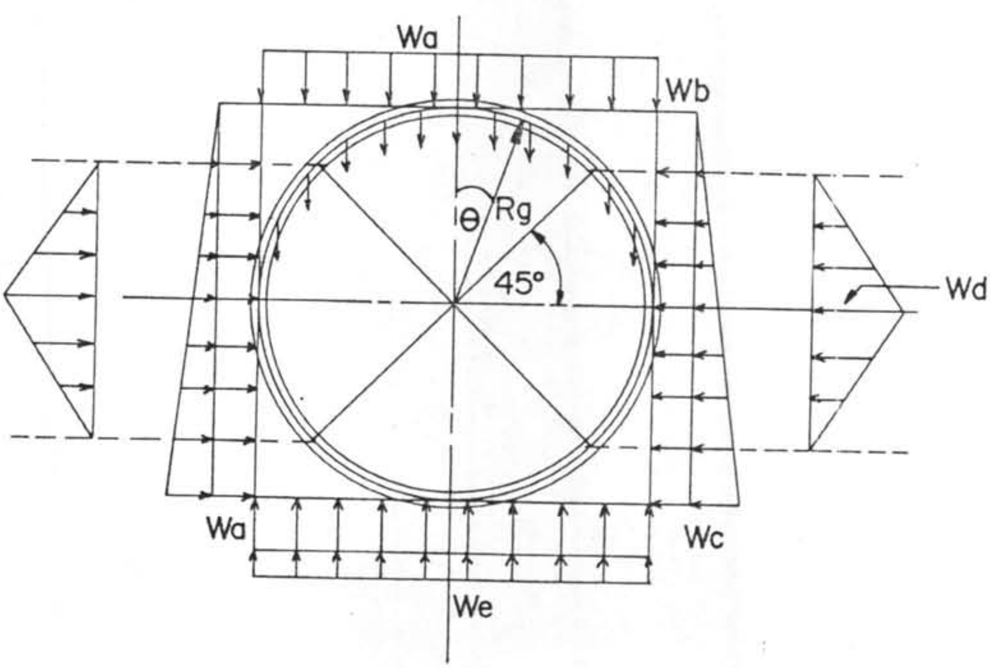


Fig. 36 Load diagram to tunnel lining.

$$N_b = R_g . W_b . \cos^2\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 2.49$$

$$Q_b = \frac{R_g . W_b}{2} . \sin 2\theta \quad \ldots\ldots\ldots\ldots\ldots\ldots \; 2.50$$

c) Formulae of M_c, N_c, Q_c by horizontal load W_c are

$$M_c = \frac{R_g^2 . W_c}{48} (6 - 3\cos\theta - 12\cos^2\theta + 4\cos^3\theta) . \; 2.51$$

$$N_c = \frac{R_g . W_c}{16} (\cos\theta + 8\cos^2\theta - 4\cos^3\theta) \; \ldots \; 2.52$$

$$Q_c = \frac{R_g . W_c}{16} (\sin\theta + 8\sin\theta . \cos\theta - 4\sin\theta . \cos^2\theta) . \; 2.53$$

d) Formulae of M_d, N_d and Q_d by horizontal subgrade reaction force W_d are

(1) $0 \leqslant \theta \leqslant \frac{\pi}{4}$

$$M_d = W_d . R_g^2 (0.2346 - 0.3536 . \cos\theta) \quad \ldots\ldots \; 2.54$$

$$N_d = 0.3536 . W_d . R_g \cos\theta \quad \ldots\ldots\ldots\ldots\ldots \; 2.55$$

$$Q_d = 0.3536 . W_d . R_g . \sin\theta \quad \ldots\ldots\ldots\ldots\ldots \; 2.56$$

(2) $\frac{\pi}{4} \leqslant \theta \leqslant \frac{\pi}{2}$

$$M_d = W_d . R_g^2 (-0.3487 + 0.5000\sin^2\theta + 0.2357\cos^3\theta) . \; 2.57$$

$$N_d \;=\; W_d.R_g(-0.7071\cos\theta + \cos^2\theta + 0.7071.\cos\theta.\sin^2\theta).\; 2.58$$

$$Q_d \;=\; W_d.R_g(\sin\theta.\cos\theta - 0.7071.\cos^2\theta.\sin\theta) \;\ldots\ldots\; 2.59$$

e) Formulae of M_e, N_e and Q_e by dead weight G of lining are

(1) $0 \leqslant \theta \leqslant \frac{\pi}{4}$

$$M_e \;=\; G.R_g^2\left(\frac{3\pi}{8} - \theta\sin\theta - \frac{5}{6}.\cos\theta\right) \;\ldots\ldots\ldots\ldots\; 2.60$$

$$N_e \;=\; G.R_g\left(\theta\sin\theta - \frac{\cos\theta}{6}\right) \;\ldots\ldots\ldots\ldots\; 2.61$$

$$Q_e \;=\; -G.R_g\left(\theta.\cos\theta + \frac{\sin\theta}{6}\right) \;\ldots\ldots\ldots\ldots\; 2.62$$

(2) $\frac{\pi}{2} \leqslant \theta \leqslant \pi$

$$M_e \;=\; G.R_g^2\left[-\frac{\pi}{8} + (\pi - \theta)\sin\theta - \frac{5}{6}.\cos\theta - \frac{\pi}{2}\sin^2\theta\right].2.63$$

$$N_e \;=\; G.R_g\left[-\pi.\sin + \theta.\sin\theta + \pi\sin^2\theta - \frac{\cos\theta}{6}\right] \;\ldots\ldots 2.64$$

$$Q_e \;=\; G.R_g\left[(\pi - \theta)\cos\theta - \pi\sin\theta.\cos\theta - \frac{\sin\theta}{6}\right] \;..\; 2.65$$

4.5.5 Example of Load Calculation

a) Soil properties.

Effective overburden stress, $\bar{\sigma}_v$	16.0	tsm.
Unit weight ; $\gamma = \frac{\bar{\sigma}_v}{H}$	1.06	tcm.
Angle of internal friction of soil; ϕ	8^o	
Cohesion of soil; c	2.2	tsm.

The ratio between horizontal and vertical pressure; K = 1

Coefficient of horizontal subgrade reaction; q	4.8	kg/cm^3.
Coefficient of active earth pressure; K_a	0.4	

b) Detail of tunnel lining.

Depth of the top of tunnel Lining; H	15	m.
Surface load; q	0.5	tsm.
Outside diameter, D_o	2.980	m.
Excavation radius, $R_o = \frac{D_o}{2}$	1.45	m.
Inside diameter; D_1	2.680	m.
Designed radius of tunnel lining; R_g	1.415	m.
Thickness of tunnel lining; h	0.15	m.
Width of tunnel lining; b	1.00	m.

Moment of intertia around centroid of section; I

$$I = \frac{bh^3}{12} = \frac{100 \times (15)^3}{12} = 28125 \quad cm^4/\text{Ring}$$

$$= 281.25 \quad cm^4/cm.$$

Assume modulus of elasticity of the lining; E = modulus of elasticity of concrete, E_c

$= 2.9 \times 10^5$ ksc.

Centroid perimeter; $\ell = 2.\P.R_g$ = 8.89 m.

Segment weight per ring = $\ell h \times 2.4 = W$ = 3.20 t/m

From Eq. 2.38

Dead load of lining; $G = \frac{W}{2 R_g} = \frac{3.20}{8.89}$

$= 0.36$ tsm.

c) Calculation of earth pressure due to loosening.

From Eq. 2.29, $B_o = R_o \cos\left(45^\circ - \frac{\phi}{2}\right)$

$= 1.45 \cos\left(45^\circ - \frac{8^\circ}{2}\right)$

$= 1.09$ m.

From Eq. 2.30 $H_a = R_o\left[1 + \sin\left(45^\circ - \frac{\phi}{2}\right)\right]$

$= 1.49 \quad 1 + \sin\left(45^\circ - \frac{8^\circ}{2}\right)$

$= 2.40$ m.

From Eq. 2.31 $B_a = B_o + H_a \tan\left(45^\circ - \frac{\phi}{2}\right)$

$= 1.45 + 2.40 \tan\left(45^\circ - \frac{8^\circ}{2}\right)$

$= 3.17$ m.

From El. 2.33, $\beta = \frac{-K.\tan\phi.H}{B_a}$

$= \frac{-1.\tan 8^\circ \times 15}{3.17}$

$= -0.66$

From Eq. 2.32 $$p_v = \frac{(\gamma . B_a - c)(1 - e^{\beta})}{K \tan \phi} + qe^{\beta}$$

$$= \frac{(1.06 \times 3.17 - 2.2)(1 - e^{-0.66})}{1 \tan 8^{o}} + 0.5e^{-0.66}$$

$$= 4.19 \quad \text{tsm.}$$

d) Design load calculation

Water table level is 6.50 m. below the ground surface

From Eq. 2.34 $$W_a = p_v + U_a$$

$$= 4.19 + (20.00 - 6.50)$$

$$= 17.69 \quad \text{tsm.}$$

From Eq. 2.35 $$W_b = K_a(W_a - U_a) + U_a$$

$$= 0.4(17.69 - 13.50) + 13.50$$

$$= 15.18 \quad \text{tsm.}$$

From Eq. 2.36 $$U_e = U_a + 2\gamma_w R_g$$

$$= 16.48 \quad \text{tsm.}$$

From Eq. 2.37 $$W_b + W_c = K_a(W_a + 2.\gamma.R_g - U_e) + U_e$$

$$= 0.4(17.69 + 2 \times 1.684 \times 1.415 - 16.48) + 16.48$$

$$= 19.87$$

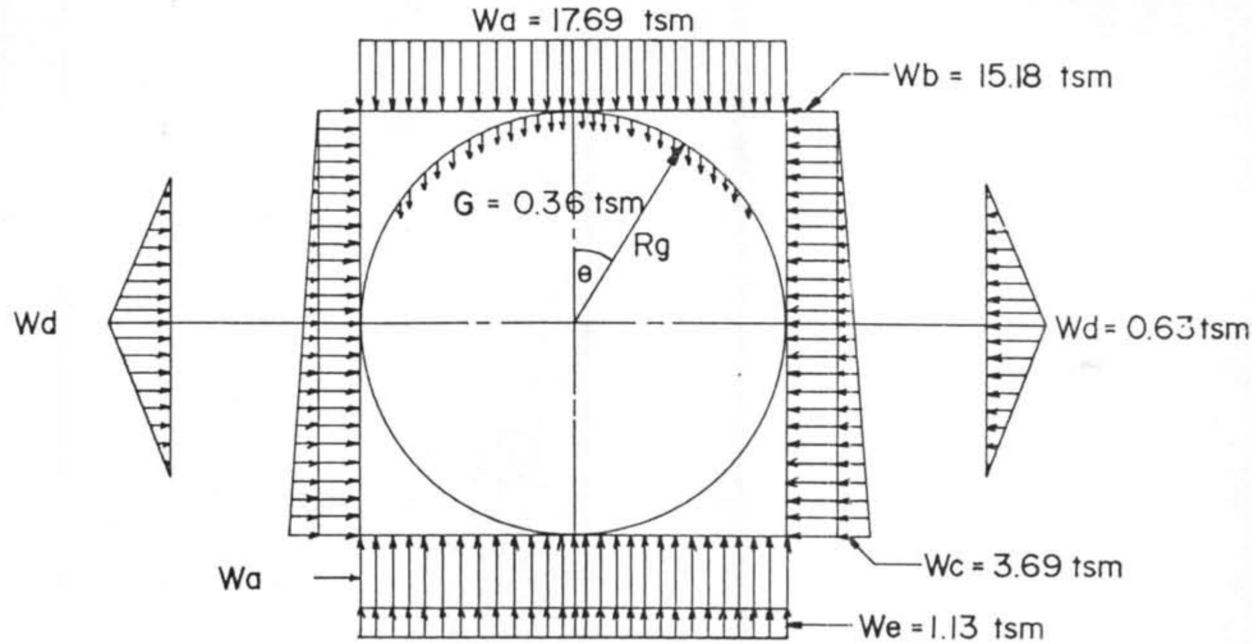


Fig. 37 Design loads diagram

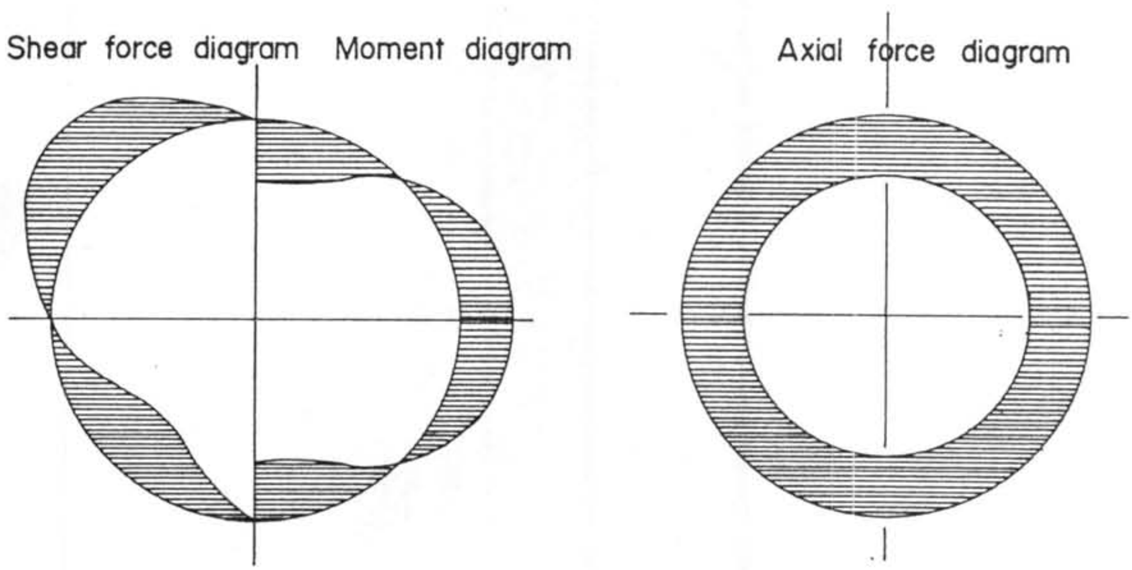


Fig. 38 Bending moment, shear force and axial force diagrams of primary lining under design loads.

For 1 m. of ring

Bending moment T-m	Axial force T	Shear force T
$M_a = W_a \frac{R_g^4}{4} \cos 2\theta$ $= 17.69 \times \frac{1.415^2}{4} \cos 0^\circ$ $= 8.85$	$N_a = W_a . R_g . \sin^2\theta$ $= 17.69 \times 1.415 \sin^2 0^\circ$ $= 0$	$Q_a = -W_a \frac{R_g}{2} \sin 2\theta$ $= -17.69 \times \frac{1.415}{2} \sin 0^\circ$ $= 0$
$M_b = W_b \frac{R_g^2}{4} \cos 2\theta$ $= -15.18 \times \frac{1.415^2}{4} \cos 0^\circ$ $= -7.60$	$N_b = W_b . R_g \cos^2\theta$ $= 15.18 \times 1.415 \cos^2 0^\circ$ $= 21.48$	$Q_b = W_b . \frac{R_g}{2} \sin 2\theta$ $= 15.18 \times \frac{1.415}{2} \sin 0^\circ$ $= 0$
$M_c = \frac{-(6-3\cos\theta-12\cos^2\theta+4\cos^3\theta) W_c R_g^2}{48}$ $= \frac{-5 \times 3.69 \times (1.415)^2}{48}$ $= -0.77$	$N_c = \frac{(\cos\theta+8\cos^2\theta-4\cos^3\theta) W_c . R_g}{16}$ $= \frac{5 \times 3.69 \times 1.415}{16}$ $= 1.63$	$Q_c = (\sin\theta+8\sin\theta-4\sin\theta\cos^2\theta) \times \frac{W_c . R_g}{16}$ $= 0$
$M_d = (0.2346-0.3536\cos\theta) W_d . R_g^2$ $= (0.2346-0.3536\cos\theta^\circ) 0.63 \times 1.415^2$ $= -0.15$	$N_d = 0.3536 \cos\theta W_d R_g$ $= 0.3536 \times 0.63 \times 1.415$ $= 0.31$	$Q_c = 0.3536 \sin\theta W_d . R_g$ $= 0$
$M_e = (\frac{3\pi}{8} - \theta\sin\theta - \frac{5}{6}\cos\theta) . G . R_g^2$ $= (\frac{3\pi}{8} - \frac{5}{6}) 0.36 (1.415)^2$ $= 0.25$	$N_e = (\theta\sin\theta - \frac{\cos\theta}{6}) G . R_g$ $= -\frac{1}{6} \times 0.36 \times 1.415$ $= -0.08$	$Q_e = -\left(\theta\cos\theta + \frac{\sin\theta}{6}\right) . G . R_g$ $= 0$
Total $M = 0.58$	Total $N = 23.34$	Total $Q = 0$

Table 2 Bending moment, axial force and shear force at crown ($\theta = 0^\circ$) due to design loads (From Eq. 2.45 - 2.62)

$$W_c = 19.87 - W_b \quad \text{tsm.}$$

$$= 3.69$$

From Eq. 2.44 $\quad W_d = q\,\delta$

Eq. 2.43 $\quad \delta = \dfrac{(2W_a - 2W_b - W_c) \cdot R_g^4}{24(EI + 0.0454 \times q.R_g^4)}$

$$W_d = \frac{4.8 \times (100)^3}{1000} \times \frac{(2 \times 17.69 - 2 \times 15.18 - 3.69) \times (1.415)^4}{24\left[\dfrac{281.25 \times 2.9 \times 10^5}{1000 \times 100} + \dfrac{0.0454 \times 4.80 \times (1.415 \times 100)^4}{1000 \times 100}\right]}$$

$$= 4800 \times 1.315 \times 10^{-4} \quad \text{tcm. x m.}$$

$$= 0.6312 \quad \text{tsm.}$$

From Eq. 2.39 $\quad W_e = \P.G$

$$= \P \times 0.36$$

$$= 1.13 \quad \text{tsm.}$$

Diagram of design loads and force diagram are shown in Fig. 37 and 38 respectively.

4.6 Stress of Main Section of Primary Lining

The stress of reinforced concrete primary lining is calculated by the method specified in ACI-318-77[2] using the straight-line theory of stress and strain in flexure.

From Fig. 39, the maximum concrete stress becomes

$$f_c = \frac{N}{A_i} + \frac{NeU}{I_t} \qquad \text{.................} 2.66$$

and the stress in the tension steel

$$f_s = n\left[\frac{N}{A_i} + \frac{Ne\ (d-U)}{I_t}\right] \qquad \text{.................} 2.67$$

A_i is the equivalent sectional area

$$A_i = bh + nA_s + (2n - 1)A'_s \qquad \text{.................} 2.68$$

The distance from extreme compression fiber to G; the center of equivalent sectional area is U.

$$U = \frac{(\frac{bh^2}{2} + nA_s d + (2n - 1)\ A'_s d')}{A_i} \qquad \text{..............} 2.69$$

Moment of inertia of equivalent sectional area around gravity axis

$$I_t = \left[\frac{u^3 + (h - u)^3}{3}\right] b + nA_s (d - u)^2 + (2n - 1)\ A'_s\ (u - d')^2 \quad 2.70$$

$$\text{..................} 2.70$$

and e is the eccentricity of equivalent sectional area

$$e = \frac{M}{N} + (u - \frac{h}{2}) \qquad \text{................} 2.71$$

4.7 Design of Segment Joint

a) Allowable Bending Moment of Segment Joint

Segment joints are regarded to have more tnedencies of 'hinges' than rigid circular ring. The tunnel lining segments are

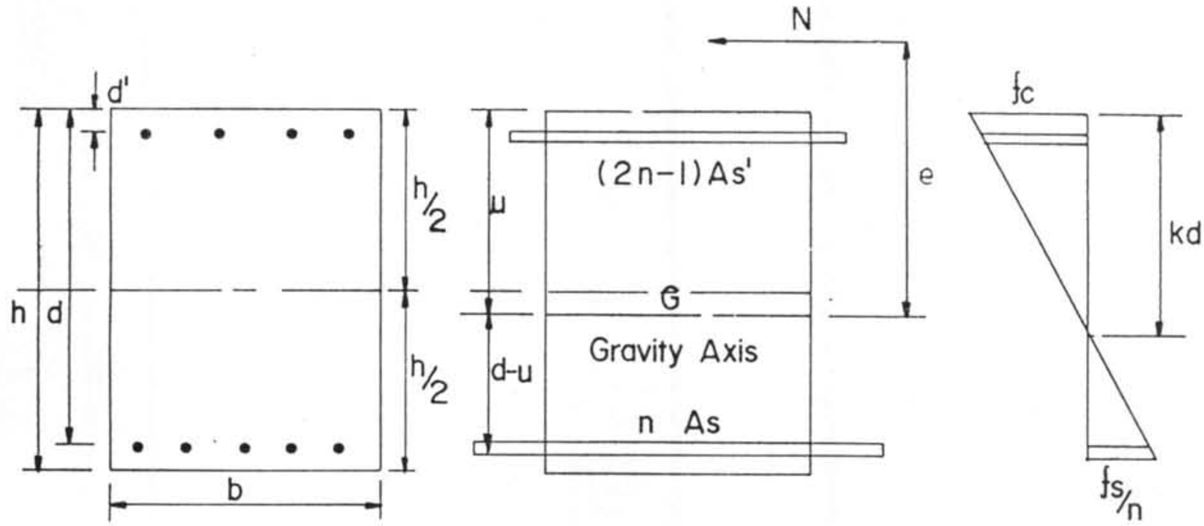


Fig. 39 Double reinforced concrete lining segment

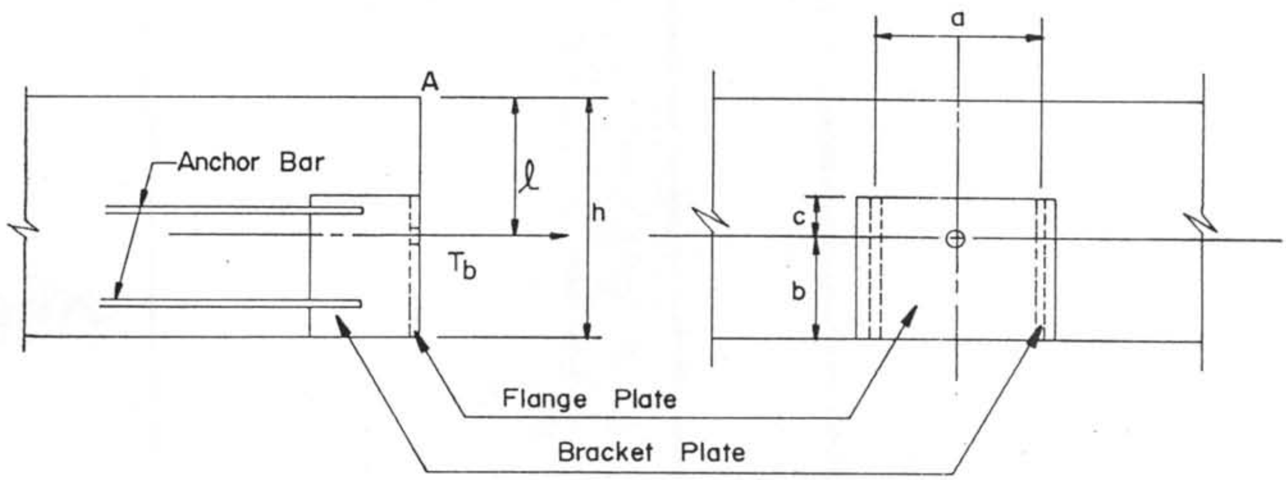


Fig. 40 Joint of lining segment

fabricated zigzag in order to prevent themselves from deterioration of rigidty. In this case, all of bending moment is not delivered through segment joint but a part of it is delivered through adjacent segment.

Therefore the tendencies of segment joints have too many complicated and unknown factors. By the above reason, allowable bending moment of a segment joint is taken to be 60 per cent of a single segment proper in accordance with the results of its strength test in the past (from the standard segments for the shield tunnel construction by the Japan Society of Civil Engineers).

b) Resisting Moment of Segment Proper

The allowable resisting moment of main section, M_a should be the less one of M_c and M_s.

where

M_c = bending moment when the compressive stress of concrete reaches allowable one.

M_s = bending moment when the tensile stress of bar reaches allowable one.

The safety factor against destruction of segment being determined to be 2.5, the ultimate resisting moment of main section, M_u is

$$M_u = 2.5\, M_a \qquad \text{.....................2.72}$$

M_c and M_s are calculated as double reinforced rectangular section, the neutral axis position (from the compressive fiber), kd is obtained from the following equation. (Fig. 39)

$$\frac{b(kd)^2}{2} + (2n-1)\,A'_s\,(kd - d') = nA_s\,(d - kd) \quad \ldots\ldots\ldots\ldots 2.73$$

Then M_c and M_s are

$$M_c = f_{ca}\left[\frac{b.kd}{2}\left(d - \frac{kd}{3}\right) + (2n-1)\,A_s\,\frac{kd - d'}{kd}\,(d - d')\right] \ldots 2.74$$

$$M_s = f_{sa}\cdot\frac{kd}{n(d-kd)}\left[\frac{b.dk}{2}\left(d - \frac{kd}{3}\right) + (2n-1)\,A_s\,\frac{kd - d'}{kd}\,(d - d')\right] \quad \ldots . 2.75$$

where

f_{ca} = Allowable compressive stress due to bending of concrete

f_{sa} = Allowable tensile stress of reinforced steel

Therefore M_a is;

when $M_c < M_s$, $M_a = M_c$

$M_s < M_c$, $M_a = M_s$

b) <u>Resisting Moment of Segment Joint</u>

If M_d is the resisting moment of segment joint from section 4.7 (a)

$$M_d = 0.6\ M_u$$

$$= 0.6 \times 2.5\ M_a \qquad \text{.....................2.76}$$

4.8 Design of Bolt and Flange Plate

If the segments are perfectly jointed, the destruction of a segment joint will occur in the following order:

1. Breaking of bolt.
2. Plastic collapse of flange plate.
3. Yielding of bracket.

Fig. 39 shows the component of segment joint

a) Study of Bolt

Assuming that only tension of bolt is available for the resistance to the bending moment, considering the equilibrium condition at Point A in Fig. 40.

$$M_d = f_b . A\ell \qquad \text{.....................2.77}$$

where

M_d = design resisting moment of a segment joint.

f_b = breaking stress of bolt.

A = stress area of bolt.

b) Design of Flange Plate

The inner dimension of flange plate surrounded by the brackets is indicated in Fig. 40.

Considering that the yielding or collapse of flange plate occurs when the concentrated load is given to the bolt.

The required thickness of flange plate 't_f' is obtained from the formula.

$$t_f = \sqrt{\frac{f_b \cdot A}{\gamma \cdot f_y}} \qquad \text{.................. 2.78}$$

where

f_y = Yield stress of flange plate.

γ = coefficient which is determined by a,b,c in Fig. 40 (As shown in Appendix)

4.9 Example of Primary Lining Segments Calculation

If	Bending moment, M	=	0.58	$\frac{t\text{-}m}{m.}$
	Axial force, N	=	23.34	t/m.
	Assume thickness of segment, h	=	15	cm.
	d	=	15 - 3.5 = 11.5	cm.
	d'	=	3.5	cm.
	A_s	=	6 - DB 10 = 4.68	cm^2.
	A'_s	=	6 - DB 10 = 4.68	cm^2.

Allowable compressive stress due to bending

(assume 37.5 % of Cylindrical compressive stress) = 130 ksc.

Allowable tensile stress of f_{sa} = 1400 ksc.

$$\frac{E_s}{E_c} = n = 7$$

According to eq. 2.66 - 2.70

$$A_i = (100 \times 15) + (7 \times 4.68) + (2 \times 7 - 1)\ 4.68$$

$$= 1593.60 \ \text{cm}^2.$$

$$u = \frac{\left[\frac{100 \times 15^2}{2} + (7 \times 4.68 \times 11.50) + (2 \times 7 - 1)4.68 \times 3.5\right]}{1593.6}$$

$$= 7.43 \ \text{cm}.$$

$$I_t = \frac{\left[(7.43)^3) + (15 - 7.43)^3\right]}{3} 100 + 7 \times 4.68 \times (4.07)^2$$

$$+ 13 \times 4.68 \times (3.93)^2$$

$$= 29614.69 \ \text{cm}^4.$$

$$e = \frac{0.58}{23.34} \times 100 + (7.43 - \frac{15}{2})$$

$$= 2.42 \ \text{cm}.$$

$$f_c = \frac{23.34}{1593.6} + \frac{23.34 \times 2.42 \times 7.43}{29614.69} = 0.02882 \quad \text{T/cm}^2.$$

$$= 28.82 \ \text{ksc}.$$

$$f_s = 7\left[\frac{23.34}{1593.6} + \frac{23.34 \times 2.42(11.50 - 7.43)}{29614.69}\right] = 0.15686 \ \text{T/cm}^2.$$

$$= 156.86 \ \text{ksc}.$$

$f_c = 28.82$ ksc $< f_{ca} = 130$ ksc.

$f_s = 156.86$ ksc $< f_{sa} = 1400$ ksc.

a) <u>Calculation of Moment at Segment Joint</u>

According to formulae 2.73 - 2.75

$$\frac{100\ (kd)^2}{2} + 13.\times 4.63 \times (kd - 3.5) = 7 \times 4.68\ (11.5 - kd)$$

$$kd = 2.58 \text{ cm.}$$

$$M_c = 130\left(\frac{100 \times 2.58}{2}(11.5 - 0.86) + \frac{13 \times 4.68(-0.92) \times 8}{2.58}\right)$$

$$= 1.559 \times 10^5 \text{ kg-cm.}$$

$$M_s = 1400.\ \frac{2.58}{7(11.5 - 2.58)}\left[\frac{100 \times 2.58}{2}(11.5 - 0.86) - \frac{13 \times 4.68 \times 0.92 \times 8}{2.58}\right]$$

$$= 6.94 \times 10^4 \text{ kg-cm.}$$

$$M_s < M_c$$

Allowable resisting moment $M_a = M_s = 0.694$ T-m.

b) <u>Design of Segment Joint</u>

1. <u>Design of Bolt</u>

Resisting moment of segment joint ; M_d

From Eq. 2.76

$$M_d = 0.6 \times 2.5 \; M_a$$

$$= 0.6 \times 2.5 \times 0.694$$

$$= 1.041 \text{ T-m.}$$

From Eq. 2.77

$$M_d = f_b . A . \ell$$

Assume 2 bolts 2 cm. dimeter (Allowable tensile stress = 4000 ksc)

Stress area of 2 cm. diameter bolt = 2.448 cm^2.

All stress area = 2 x 2.448 = 4.896 cm^2.

From F_y. 38, if

ℓ = 9 cm.

b = 6 cm.

c = 5 cm.

a = 12.5 cm.

$$f_b = \frac{M_d}{A.\ell} = \frac{1.041 \times 10^5}{4.896 \times 9} = 2363 \text{ ksc} < 4000 \text{ ksc.}$$

2. <u>Design of Flange Plate</u>

According to formulae 2.78

$$t_f = \sqrt{\frac{f_b . A}{\gamma . f_y}}$$

If $f_y = 2500$ ksc.

$$\gamma = \frac{2(b + c)}{a} = \frac{2(6 + 5)}{12.5} = 1.76$$

$$t_f = \sqrt{\frac{2362 \times 2.448}{2.76 \times 2500}} = 1.314 \text{ cm.}$$

Thickness of flange plate $= 1.314$ cm.

5. Compressed Air System

5.1 Air Pressure required

To prevent the water and material from running into the working area, the heading is kept under air pressure sufficient to balance the hydrostatic pressure of the ground water.

The effect of the compressed air depend upon conditions of the surrounding ground, it has the greatest effect in clay to provide watertight to the ground.

In silt and sand the compressed air displaces the greater part of the pore water and causes cohesion between grains of the soil by surface tension. Another side effect of compressed air in the ground is to reduce its permeability to the flow of water.

However, silt and sand are pervious soil. Any excess of air pressure at the crown drives the water back into the voids, drying the soil until it cracks and sloughs off. At the same time a deficiency of pressure at the bottom allows water to percolate through the ground.

The problem of balancing the external water pressure increases with the height and the size of the tunnel. The depth of ground water level and the texture of the ground will determine the quantity of air pressure.

5.2 Application of Compressed Air

a. The face-stabilizing effect

1. The face can be prevented from fault by dewatering effect of compressed air,

2. The face can be supported by pressure of the air.

3. Compressed air can expel water from the void of the front face soils and increase their strength, resulting stabilization of the face.

b. Dewatering the working area by pressed back or rather retained in the pores of the surrounding soil by increased air pressure.

5.3 Supplying Compressed Air

1. The consideration was done on the basis of the calculation of stability ratio.

$$\text{Stability ratio} = \frac{p_z - p_a}{\tau} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots 2.79$$

Where

p_z = full overburden pressure at depth of tunnel axis.

p_a = air pressure in tunnel above atmospheric pressure

τ = undrained shear strength of soil.

It is desirable to have a stability ratio not exceeding 4.

$$\tau = c + \sigma \tan \phi \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ 2.80$$

σ = normal stress.

c = cohesion of soil.

ϕ = angle of shearing resistance.

$$p_a = p_z - 4\tau \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ 2.81$$

2. The required quantity of compressed air in the tunnel is proportional to the volume of the tunnel. It is in terms of the formulae given by the Japanese Society of Civil Engineers (JSCE).

The factors of air consumption to be considered are natural leakage from the face and the wall, necessity for locking operation including man-material lock (Fig. 41, Fig. 42) and for ventilation and leakage along pipe line.

Accordingly, the compressed air quantities are obtained in terms of JSCE formulae.

$$V_1 = \alpha . \pi . (\frac{D}{2})^2 . \frac{H_o}{10.33} \quad m^3/min. \qquad \ldots\ldots\ldots\ldots\ldots\ 2.82$$

where

V_1 = leakage from the face.

D = diameter of shield (m).

H_o = theoretical water head of compressed air (m).

α = Coefficient

$= \frac{6}{10}$ for case A, sand condition

$= \frac{4}{10}$ for case B, sand beneath clay layer condition.

$= \frac{3}{10}$ for case C, clay condition.

$$V_2 = \frac{1}{2}.\gamma.2\pi r.\ell.\frac{H_o}{10.33} \quad m^3/min. \quad \ldots\ldots\ldots\ldots 2.83$$

where

V_2 = leakage from the wall.

γ = Coefficient

$= \frac{1}{90}$ for case - B

$= \frac{1}{80}$ for case - B

$= \frac{1}{120}$ for case - C

r = inner radius of tunnel lining (m)

ℓ = length of lining (m)

$$V_3 = \left(\frac{r_1 N_1}{60} + \frac{r_2 N_2}{60}\right) \times \frac{H_o}{10.33} \quad m^3/min. \quad \ldots\ldots\ldots\ldots 2.84$$

where

V_3 = leakage from the locks

r_1 = Capacity of material-lock (m^3)

r_2 = Capacity of man-lock (m^3)

N_1 = times of operation at material-lock. (times/hour)

N_2 = times of operation at man-lock (times/hour)

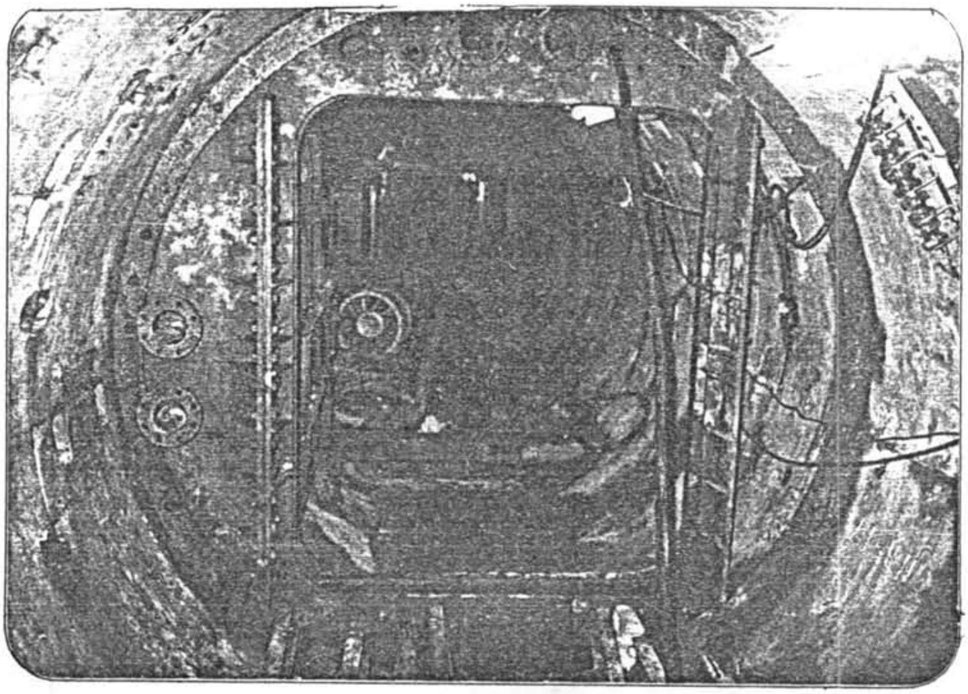


Fig. 41 Man – material air lock.

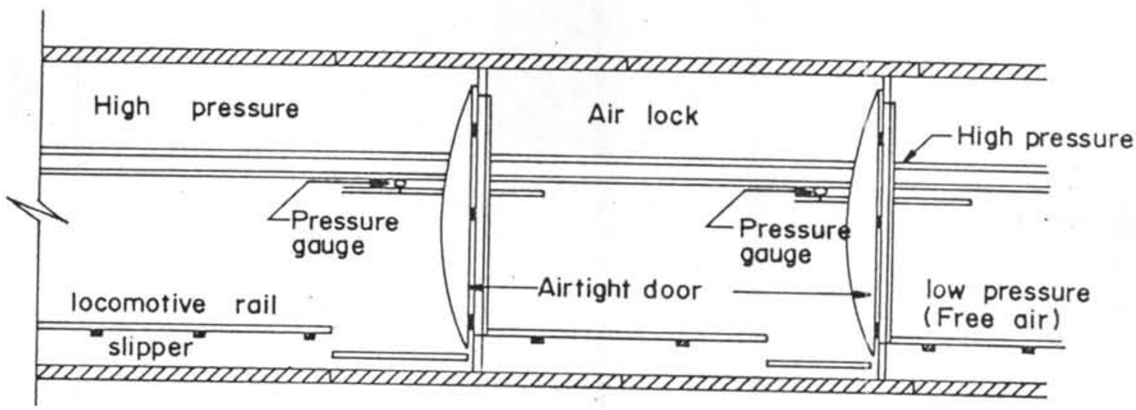


Fig. 42 Air – lock arrangement.

$$V = V_1 + V_2 + V_3 \qquad m^3/\text{min.} \quad \ldots\ldots\ldots\ldots\ 2.85$$

V is the total volume of leakage air.

3. A quantity of ventilation of 0.7 m^3/ min. is required for one worker. The capacity of the compressed air supply can be obtained from the three cases above.

The quantity of supplied air to a tunnel can be assumed from the total sum of the following items:

1. The quantity of escaped air from the front face. This depends upon perviousness of the ground, the air pressure and earth cover depth.

2. The quantity of escaped from the shield tail and primary lining. This depends upon the total area of lined tunnel and whether it was grouted or not.

3. The quantity of escaped air by passing of workman through the air lock and used air for drainage.

The necessary investigation prior to the application of compressed air are

1. Geological survey including under ground water investigation.

2. Finding of the old wells and the boring holes on the tunnel line.

3. Investigation for building in tunnelling area both existing and in construction, soil conditions under existing buildings and underground structures, so that "blow out" of the compressed air may not take place.

5.4 Supplying Compressed Air Calculation

a) Soil properties and detail of tunnel

Depth of tunnel centerline	20	m.
Inner diameter of tunnelling machine	3.42	m.
Inner diameter of tunnel lining	3.10	m.
Angle of internal friction of soil; ϕ	3^{o}	
Cohesion of soil; c	0.375	ksc.
Normal stress ; σ	5	ksc.
Full overberden pressure at 20 m. depth, p_z	33.68	tsm.

From Eq. 2.80

Shear strength, τ $= 0.375 + 5 \tan 3^{o}$

$= 0.637$ ksc.

b) Air pressure designed

From Eq. 2.81

Air pressure, P_a $= P_z - 4\tau$

Full overberden pressure, $P_z = 3.368$ ksc.

$P_a = 3.368 - 4 \times 0.637$

$= 0.819$ ksc.

$= 8.19$ m. of theoretical water head (H_o)

c) Required quantity of compressed air.

From Eq. 2.82 - 2.85

$$\text{Leakage from the face, } V_1 = \alpha . \P . \left(\frac{D}{2}\right)^2 . \frac{H_o}{10.33}$$

$$= \frac{6}{10} . \P . (1.71)^2 . \frac{8.19}{10.33}$$

$$= 4.37 \quad m^3/\text{min.}$$

$$\text{Leakage from the wall, } V_2 = \frac{1}{2} . \gamma . 2\P r . \ell . \frac{H_o}{10.33}$$

$$= \frac{1}{2} . \frac{1}{90} . 2\P \times 1.55 \times 2100 \times \frac{8.19}{10.33}$$

$$= 90.08 \quad m^3/\text{min.}$$

$$\text{Consumption at the lock, } V_3 = \left(\frac{r_1 N_1}{60} + \frac{r_2 N_2}{60}\right) \frac{H_o}{10.33}$$

If $r_1 = 116.7\ m^3$, $N_1 = 2.4$ times/hour

$r_2 = 7.6\ m^3$, $N_2 = 3$ times/hour

$$V_3 = \left(\frac{116.7 \times 2.4}{60} + \frac{7.6 \times 3}{60}\right) \frac{8.19}{10.33}$$

$$= 4.00 \quad m^3/\text{min.}$$

$$\text{Total quantity, } \quad V = V_1 + V_2 + V_3$$

$$= 4.37 + 90.08 + 4.00$$

$$= 98.45 \quad m^3/\text{min.}$$

Quantity for ventilation for 30 worker

$$= 0.7 \times 30$$

$$= 21.0 \ m^3/min. < 98.45 \ m^3/min.$$

5.5 Decompression Calculation

In tunnelling work the pressure of air in which men have to work is increased until it is sufficient to prevent water seeping into the working area.

Compressed air workers sometimes get pains in their joints soon after leaving the working area. These pains are called "the bends","caisson sickness" or "pains". They may occur in muscles and other part of the body.

This condition is due to gas, in particular nitrogen, which has been dissolved in the blood and body fluids while working under increased pressure, being liberated in the form of bubbles if decompression is too rapid. This liberated gas causes the gas embolism resulting in pain in joints or chronic bone disease.

According to the medical code of practice for work in compressed air[6], the following rules are applied whenever the persons concerned have been exposed to pressure of 1 bar or above.

a) Decompression Procedures

Reduce the pressure at the rate of about 0.4 bar per min, at the first stage according to Table 3. Retain that pressure for the prescribed number of minutes, then reduce the pressure at the same rate as before.

b) Decompression Table Calculation

This is based on two principle assumptions.

Assumption 1

If a man is exposed to an absolute pressure of P_1 for some time t. After a long period at P_1 the man is to be decompressed rapidly to some lower pressure P_2, then according to the "Haldon concept" P_1/P_2 should be a constant.

$$\frac{P_1}{P_2} = r = \frac{27.5714}{P_1 + 12.407} \qquad \text{..................2.86}$$

P_1 must be expressed in bar.

Assumption 2

The onset of decompression sickness would be dependent upon the quantity of gas absorbed at pressure. The shape of pressure-time curve for the onset of decompression sickness seems to be the same as that of the curve for the uptake of nitrogen by the whole body.

The particular equation being used is

$$\text{Fractional saturation} = 1 - \frac{8}{\pi}(e^{-kt} + \frac{1}{9}e^{-9kt} + \frac{1}{25}e^{-25kt} + \ldots\ldots \qquad 2.87$$

where

$$k = \frac{D\pi^2}{4b^2}$$

D = diffusion coefficient.

b = thickness of the slab exposed on.

t = time in minutes.

This is a solution to Fick's law.

$$-\frac{dc}{dt} = D\frac{d^2c}{dX^2}$$

Where c is the gas concentration at some distance x inside the slab, for particular conditions of thickness b.

From the experiment, the value k = 0.007928 was chosen to represent the rate of removal of nitrogen from the body.

As an example let it be supposed that a man is exposed to a gauge pressure of 3.4 bar for 30 min, then in order to calculate the decompression one proceeds as follows.

From the assumptions, it is assumed that the rate of uptake of gas is 1.5 times as great as the rate of elimination.

The relationship between fractional saturation and exposure peroid when k = 0.007928 is shown in Table 3 and Fig 43

Table 3

Fractional Saturation	Exposure Perlod (min)
0.3503	30
0.4950	60
0.6027	90
0.6869	120
0.7532	150
0.8054	180
0.8466	210
0.8791	240
0.9047	270
0.9073	300

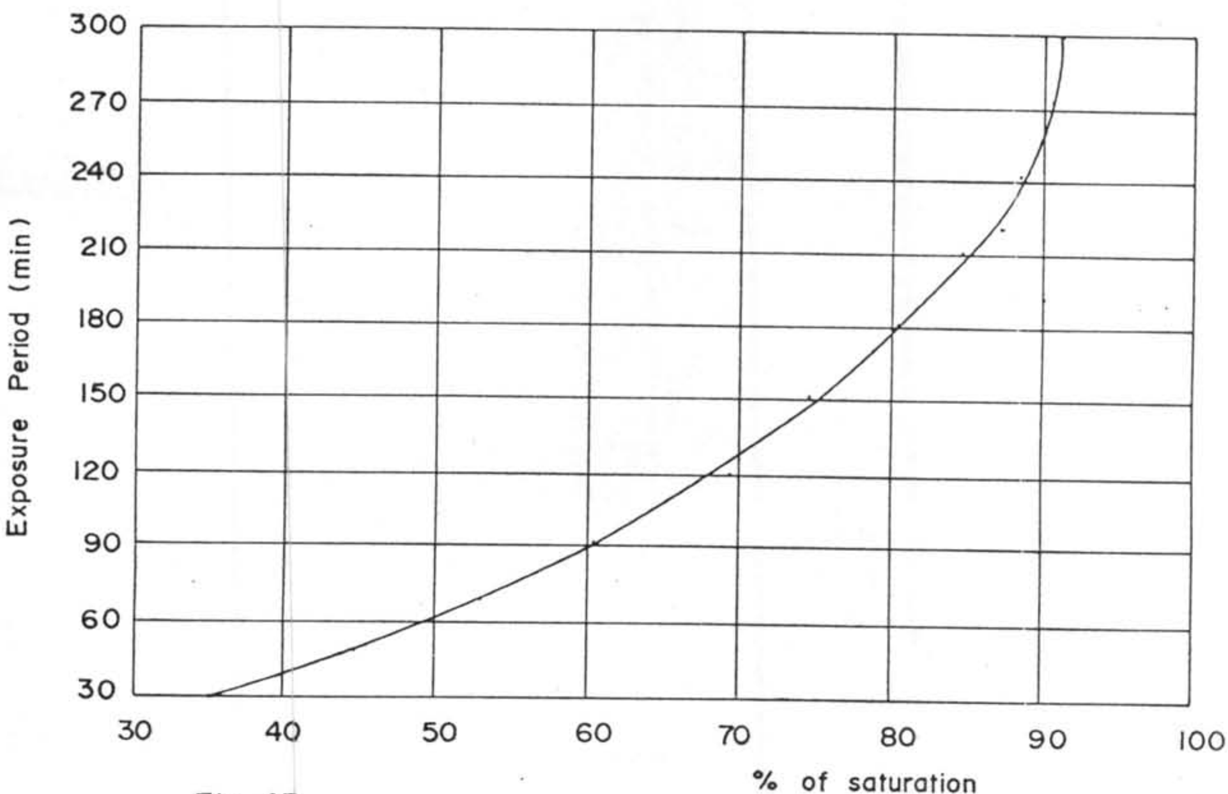


Fig 43

Decompression tables

Table 4.

Exposure peroid 0 to ½ hour.

Maximum working pressure (bar) (Note. 1)	Stage pressure (bar) (Note 2).								Total time (min) (Note. 3)
	1.6	1.4	1.2	1.0	0.8	0.6	0.4	0.2	
1.0 to 1.2									—
1.2 1.4									—
1.4 1.6									—
1.6 1.8									—
1.8 2.0									—
2.0 2.2								5	5
2.2 2.4								5	5
2.4 2.6								5	5
2.6 2.8								5	5
2.8 3.0							5	5	10
3.0 3.2							5	5	10
3.2 3.4							5	10	15

Exposure peroid ½ to 1. hour.

Maximum working pressure (bar) (Note. 1)	Stage pressure (bar) (Note 2)								Total time (min) (Note. 3)
	1.6	1.4	1.2	1.0	0.8	0.6	0.4	0.2	
1.0 to 1.2									—
1.2 1.4									—
1.4 1.6								5	5
1.6 1.8								5	5
1.8 2.0								10	10
2.0 2.2							5	15	20
2.2 2.4							5	20	25
2.4 2.6							10	25	35
2.6 2.8						5	10	35	50
2.8 3.0						5	15	40	60
3.0 3.2					5	5	20	40	70
3.2 3.4					5	10	25	40	80

Notes.

1. For borderline values of the maximum working pressure use the longer procedure
2. Decompression both to the first stage and between stages must be at rate not faster than 0.4 bar per minute.
3. Not including time between stages.

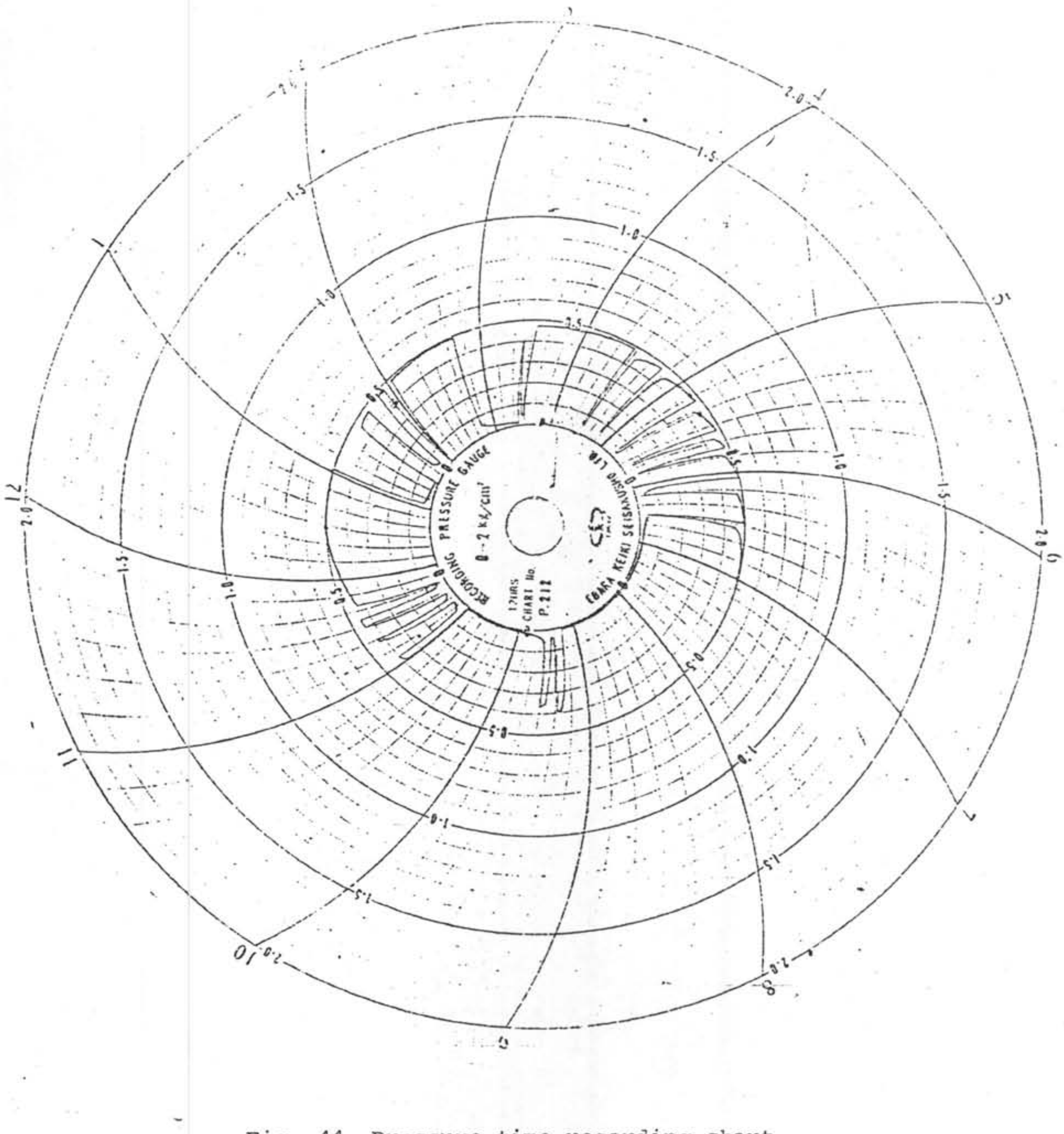


Fig. 44 Pressure-time recording chart

After 30 minutes of exposure period (= 45 minutes of decompression), he is 42.9 % saturated (from Fig. 43). Thus the quantity of gas in the body is 3.4 x 0.429 = 1.46 bar

$$r = \frac{27.5714}{P_1 + 12.407}$$

$$P_1 = 1.46 + 1 = 2.46 \text{ bar (absolute pressure)}$$

$$r = \frac{27.5714}{14.867} = 1.85$$

The first stage of decompression is at an absolute pressure of 2.46/1.85 = 1.3 bar or gauge pressure of 0.3 bar, but the working is in 0.2 bar increment would mean stopping the first stage of decompression at a gauge pressure of 0.4 bar.

Table 4. shows the medical code of Practice for work in compressed air[6] decompression tables.

Fig. 44 is the pressure-time recording chart used for recording pressure in air lock.

6. Grouting

Grouting can be defined as the injection under pressure of suspensions, emulsions and solutions to improve the geotechnical characteristic of soils and rocks and to facilitate the filling of voids for structural purposes.

6.1 The Properties of Grouting

The ideal grouting material would incorporate all of the following properties:

1. Low viscosity.
2. Small particle size.
3. Controllable setting time.

and 4. It is water resistant, durable and impermeable.

There are two principal uses of grouting techniques.

1. To seal a porous soil or fissured rocks in order to stop a flow of water.

2. To stabilize a soft soil so as to increase its strength.

A general distinction is made between:

1. The primary grouting intended to fill the back space, it is carried out under low pressure.

2. The secondary grouting for sealing and stabilizing, it makes place under high pressure.

The materials commonly used for grouting may be divided into three general types called suspensions, solutions and emulsions.

1. Suspensions are mixtures of cement and water or sometimes cement, clay and water. The relative amounts of water and

solids that are used in the mixture can vary over a considerable range, although it is customary to use as high a percentage of solids as practicable. Suspensions of this type are useful for the grouting of rock fissures or the grouting of beds of coarse sand and gravel.

2. This solutions that are used for soil injections are chemical components either with water or with each other.

At low working pressures, solutions aim at permeation of the pores, driving out the trapped water or air until all voids are filled with injected material.

The form of chemical grouting most familiar to the civil engineer is silicate injection. The basic ingredient of nearly all the silicate processes is a solution of sodium silicate in water known as "water glass"

The solution of sodium silicate will form a stiff silica gel so that the soil is waterprooved as well as strengthened.

3. The emulsions that are used as grouting materials are emulsions of bitumen and water. Hot bitumen injections have been used very effectively in conditions where high water pressures and velocities have carried away alternative sealing materials.

6.2 Control of Grouting

Three main factors have to be taken into account considering grouting control on site are grouting pressure, speed of injection and control of the grouting material properties.

6.3 Cement Grout and Admixture

Cement grout is the most commonly used because of the lowest cost. A disadvantage of cement grout is its high viscosity at low water-cement ratio and its low penetration due to 20 micron mean size of the cement particles.

The application of bentonite-cement grouts proved very successful compared with the use of pure cement grouts, when penetrating into finer voids or fissures.

Bentonite may be used as a lubricant, and the dispersed bentonite in water acting as a suspending agent, prevents settling out of the cement particles.

The addition bentonite provides both better sealing and increases the setting time of grout.

6.4 Principles of Grouting Design

Any grouting process for the treatment of water bearing ground may be described as the saturation of an incoherent mass with a fluid which will produce one or both of the following effects:

1. Rendering the ground impermeable to ground water by depositing a substance in the voids within the ground.

2. Increasing shearing resistance, and hence the strength of the ground, by the introduction of a substance, which will adhere to the in situ materials and increase their cohesion.

6.5 Selection of Materials

In soil the ability of a grout to penetrate is dependent on the permeability of porosity of the strata, the pressure being used for injection, the time or period of injection and the viscosity of the grout together with the average particle size of its components. Since the permeability of the strata is a geological characteristics and cannot be varied, improved penetration can only be affected by altering the remaining variables..

As the tunnel shield advance, the completed lining is thrust out of the shield tail. The annular void is created behind the tunnel shield, because of the thickness of shield tail, segment erection clearance and over cutting.

The annular void left between the surfaces of the circular shield skin and the smaller diameter lining ring is sickle-shaped, its maximum height at crown being equal to the sum of the thickness of tail skin, overcutting and the erection clearance (Fig. 45).

In order to prevent surface settlement, filling of the annular void between the ground and the lining is made by grouting material, as the tunnel shield was forced ahead.

Synthetic rubber "Tail packing seal" is provided between the tail skin of the shield and the external face of the segment in order to prevent leakage of grouting material (Fig. 46).

The grouting material is forced in by grouting machine through plugs provided in the lining segments (Fig. 47). Injected volume of grouting material sometimes ranges between 130 % and 150 % of estimated volume.

6.6 Mixing Proportion

1. Cement-bentonite ratio 4:1

   Water-cement ratio 0.5

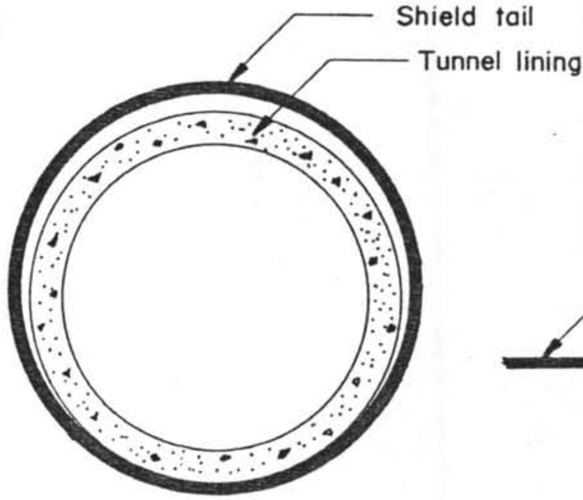


Fig. 45 The annular void behind the tail shield.

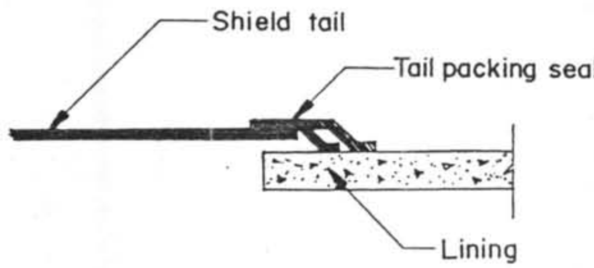


Fig. 46 Tail packing seal.

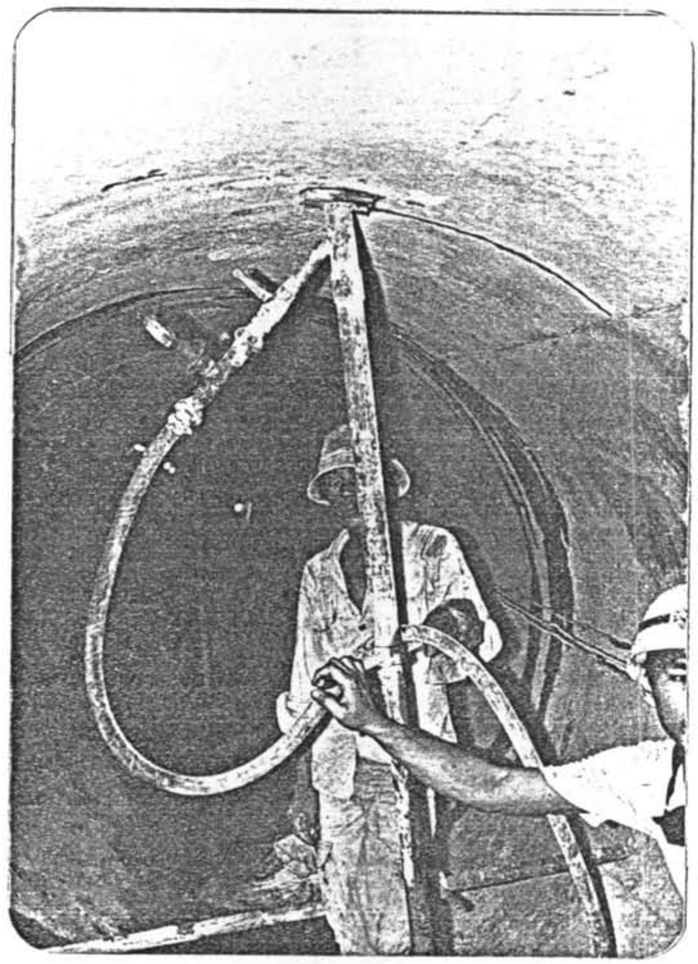


Fig. 47 Grouting in tunnel.

2.	Cement-bentonite ratio	3.6:1
	Water-cement ratio	0.8
3.	Cement-bentonite ratio	1:1
	Water-cement ratio	2.15-2.50

Fig. 48 shows the effect of bentonite additive upon compressive strength of cylinder sample.

6.7 Contact Grouting in Tunnel

Contact grouting shall be performed by starting at one end of the tunnel and advancing continuously towards the other end. The lower holes in each ring shall be treated first and the uppermost hole last. Holes shall be drilled through the concrete lining and a thick cement or fill cement grout shall be pumped in by pressure.

6.8 Secondary Grouting

Secondary grouting is carried out several months after primary grouting, usually for the sole purpose of sealing, when the ground settles into a condition of equilibrium.

6.9 Injection Procedure

Injection shall be started on a thin mix which shall be thickened progressively in accordance with the variation in the

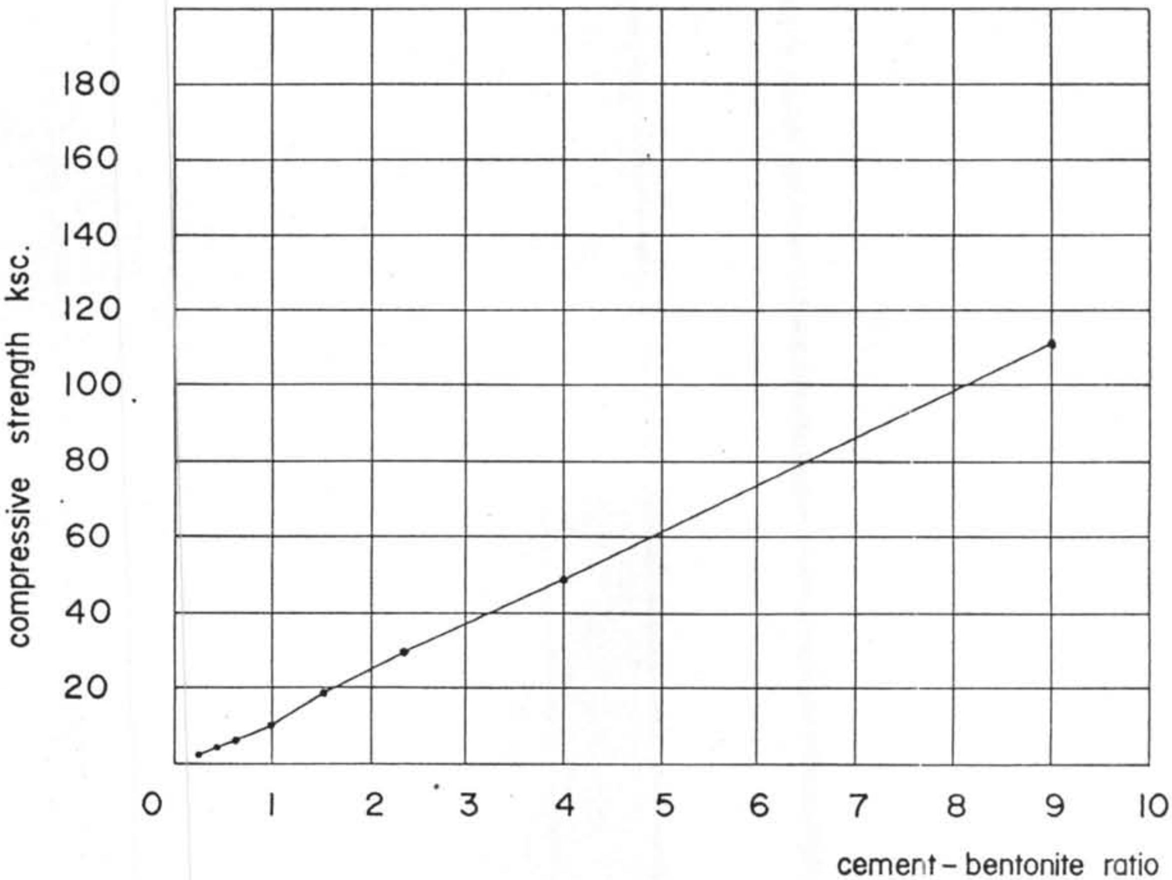


Fig. 48 Effect of cement - bentonite ratio to cylindrical compressive strength (The ratio are $2/8$, $3/7$, $4/6$, $5/5$, $6/4$, $7/3$, $8/2$ and $9/1$) water - cement ratio = 0.66, at 7 days age .

pumping rate and pressure during the course of the injection. The maximum injection pressure is 3.5 in back space filling.

Where leakages of grout occur at the setment joints, they shall be restricted by caulking with wooden wedges.

7. Distortion on Primary Lining and Settlement

In designing of tunnel lining it is necessary to relate the effects of stiffness of the lining to the characteristics of the ground since the strength of the lining is related to both factors.

Where a tunnel is driven in clay, long term effects of consolidation settlement have also to be considered and the relationship is established between the ultimate stress in a tunnel and the consolidation characteristics of the ground.

7.1 Deformation of Tunnel Primary Lining and Ground

Deformation and bending stress of tunnel lining are the result of surrounding load. Failure of the ring results by exceeding the ultimate strength of material of the lining. The critical pressure of the lining in clay is analysed by Morgan[10]

Fig. 49 shows the cartesian and polar coodinates for tunnel at depth H below the surface.

From loading the circular lining will be distorted to an ellipse as shown in Fig. 50.

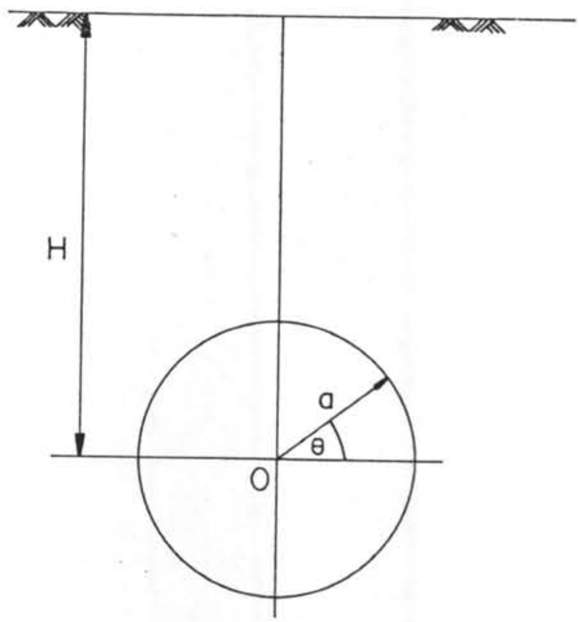


Fig. 49 Cartesian and polar co-ordinates for tunnel at depth H below surface.

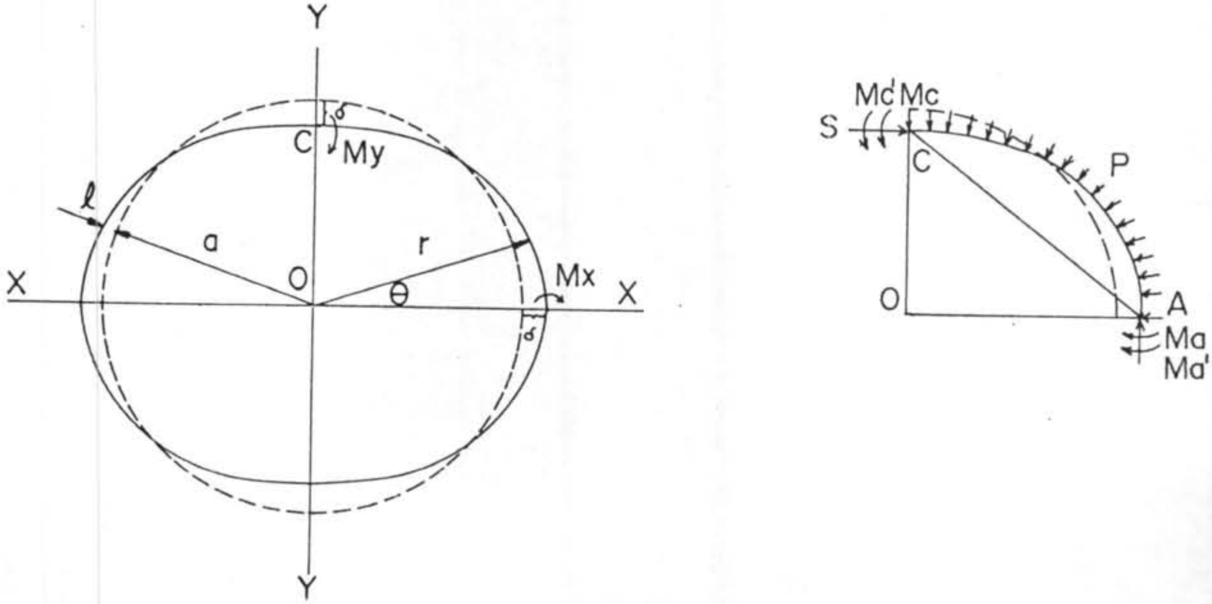


Fig. 50 Elliptical deformation of a circular tunnel lining.

In polar coordinates

$$r = a + \ell$$

where $\ell = \delta \cos 2\theta$

The maximum distortions $+\delta$ and $-\delta$ will occur at point on the x-x and y-y axes respectively.

From Timoshenko and Goodier[19] the bending moment at any point in the ring.

$$M = EI\left(\frac{1}{\rho} - \frac{1}{a}\right) \qquad \text{.................} \quad 2.88$$

where

E = modulus of elasticity of the lining

I = moment of inertia of the section of the lining per unit width of the ring.

a = original radius of curvature of the lining.

ρ = deformed radius of curvature of the lining.

$$\frac{1}{\rho} = \frac{a + \delta}{(a - \delta)^2} \text{ on x-x axis at point x.}$$

$$\frac{1}{\rho} = \frac{a - \delta}{(a + \delta)^2} \text{ on y-y axis at point y.}$$

$$\frac{M_x}{EI} = \frac{a + \delta}{(a - \delta)^2} - \frac{1}{a}$$

$$\frac{M_y}{EI} = \frac{a - \delta}{(a + \delta)^2} - \frac{1}{a}$$

if $a >> \delta$

$$M_x = - M_y \simeq \frac{3\delta EI}{a^2} \quad \text{................ 2.89}$$

If the tunnel is in the ground of mean elasticity modulus E_o and Poisson's ration ν. The symmetrical loading in the ground set up by this distortion of the tunnel lining may be represented by a stress function; ϕ (Timoshenko and Gere[20]).

$$\phi = (Ar^2 + Br^4 + \frac{C}{r^4} + D) \cos 2\theta \quad \text{.............. 2.90}$$

where A, B, C and D are constants.

ϕ = relates to the components of stress that vary respect to θ.

The radial stress in the ground; p_r

$$p_r = \frac{1}{r}\frac{\delta\phi}{\delta r} + \frac{1}{r^2}\frac{\delta\phi^2}{\delta r^2} \quad \text{................ 2.91}$$

$$= \frac{1}{r}(2Ar + 4Br^3 - \frac{2C}{r^3}) \cos 2\theta - \frac{4}{r^2}(Ar^2 + Br^4 + \frac{C}{r^2} + D) \cos 2\theta$$

$$= -(2A + \frac{6C}{r^4} + \frac{4D}{r^2}) \cos 2\theta$$

If $r \longrightarrow \alpha$; $p_r \longrightarrow 0$ (St. Venant's principle)

$$A = 0$$

$$\phi = (Br^4 + \frac{C}{r^2} + D) \cos 2\theta$$

The circumferential stress in the ground ; P_θ

$$P_\theta = \frac{\partial^2 \phi}{\partial r^2} \qquad \dots\dots\dots\dots\dots\dots\dots\dots 2.92$$

$$= (12Br^2 + \frac{6C}{r^4}) \cos 2\theta$$

If $r \rightarrow \alpha$; $P_\theta \longrightarrow 0$ (St. Venant's principle)

$$B = 0$$

$$\phi = (\frac{C}{r^2} + D) \cos 2\theta$$

$$P_r = -(\frac{6C}{r^4} + \frac{4D}{r^2}) \cos 2\theta$$

$$P_\theta = \frac{6C}{r^4} \cos 2\theta$$

In the case of the ground is continuous for a long distance in the direction of the z-z axis along the line of the tunnel and no strain occurs in this direction.

$$P_r + P_\theta = \text{constant} \quad (\text{Morgan}^{(11)})$$

$$D = 0$$

$$P_\theta = -P_r$$

$$= \frac{6C}{r^4} \cos 2\theta$$

If u is the radial displacement at any point.

$$\text{radial strain, } \varepsilon_r = \frac{\partial u}{\partial r}$$

$$\frac{\partial u}{\partial r} = \frac{1}{E_o}(P_r - \nu P_\theta) \quad \text{(from Hook's law)} \quad \ldots\ldots\ldots\ldots \quad 2.93$$

$$= -\frac{1}{E_o}\left[\frac{6C}{r^4}(1 + \nu)\right]\cos 2\theta$$

$$u = \frac{2C}{r^3 E_o}(1 + \nu)\cos 2\theta + \text{constant}$$

If $a >> u$

$$u = \ell = \frac{2C}{E_o r^3}(1 + \nu)\cos 2\theta + \text{constant}$$

If $r = a$

$$u = \delta \cos 2\theta = \frac{2C}{E_o a^3}(1 + \nu)\cos 2\theta + \text{constant}$$

$$\text{constant} = 0$$

$$C = \frac{\delta E_o a^3}{2(1+\nu)}$$

$$P_a = P_r$$

$$P_a = -\frac{3\delta E_o}{a(1+\nu)} \text{ at } \theta = 0, ¶$$

$$= \frac{3\delta E_o}{a(1+\nu)} \text{ at } \theta = \frac{¶}{2}, \frac{3¶}{2}$$

Considering the effect of loading P_a on stresses in the tunnel lining. On segment xy the change in horizontal load due to P_a for small values of δ is.

$$\int_0^{\frac{\pi}{2}} P_a a \cos\theta \, d\theta$$

and the change in vertical load due to P_a is

$$\int_0^{\frac{\pi}{2}} P_a a \sin\theta \, d\theta$$

Take moment about 0 for segment AC.

$$M'_A - M'_C = \int_0^{\frac{\pi}{2}} P_a a\,(a+\delta)\sin\theta\,d\theta - \int_0^{\frac{\pi}{2}} P_a a\,(a-\delta)\cos\theta\,d\theta$$

for small value of δ

$$M'_A - M'_C = \int_0^{\frac{\pi}{2}} P_a a^2 \sin\theta\,d\theta - \int_0^{\frac{\pi}{2}} P_a a^2 \cos\theta\,d\theta$$

$$= \frac{-3\delta E_o a}{(1+\nu)}\left[\int_0^{\frac{\pi}{2}} \cos 2\theta \sin\theta\,d\theta - \int_0^{\frac{\pi}{2}} \cos 2\theta \cos\theta\,d\theta\right]$$

$$= \frac{-3\delta E_o a}{(1+\nu)}\left[\int_0^{\frac{\pi}{2}} \frac{1}{2}(\sin 3\theta - \sin\theta)d\theta - \int_0^{\frac{\pi}{2}} \frac{1}{2}(\cos 3\theta + \cos\theta)d\theta\right]$$

$$= \frac{-3\delta E_o a}{2(1+\nu)}\left[\left(-\frac{\cos 3\theta}{3} + \cos\theta\right)_0^{\frac{\pi}{2}} - \left(\frac{\sin 3\theta}{3} + \sin\theta\right)_0^{\frac{\pi}{2}}\right]$$

$$= \frac{-3\delta E_o a}{2(1+\nu)}\left[-\left(-\frac{1}{3}+1\right) - \left(-\frac{1}{3}+1\right)\right]$$

$$= \frac{2\delta E_o a}{(1+\nu)}$$

7.2 Critical Pressure of the Lining

From Timoshenko and Gere[20]

In Fig. 50

$$S = p(CO)$$

$$= p(a - \delta)$$

where

S is the compressive force at A and C.

Take moment about A for AC.

$$M_c = M_A + S(CO) - p(\frac{AC}{2})^2 + (M'_A - M'_C)$$

$$= M_A + p(a - \delta)^2 - \frac{p}{2}(AO)^2 + (M'_A - M'_C)$$

$$= M_A + p(a - \delta)^2 - \frac{p}{2}(a + \delta)^2 - \frac{p}{2}(a - \delta)^2 + (M'_A - M'_C)$$

$$(M_A - M_C) + (M'_A - M'_C) + p(a - \delta)^2 - \frac{p}{2}(a + \delta)^2 - \frac{P}{2}(a - \delta)^2 = 0$$

$$\frac{6EI\delta}{a^2} + \frac{2\delta E_o a}{(1+\nu)} - 2\,pa\delta = 0$$

$$p = \frac{6EI}{a^3} + \frac{Eo}{(1+\nu)} = 0 \qquad \text{.......... } 2.94$$

7.3 Distortion Measurement Equipments

Very accurate measurement of movement and deformation of tunnel lining can be obtained by the extensometers developed by the Department of Mining Engineering, Newcastle upon tyne University. The instruments are calibrated to read in 0.01 mm. increments and accuracies have been obtained over distances up to 45 m.

The system operates by tensioning a tape to a pre-determined load, and recording the change of distance between a reference point on the wire in relation to a fixed point known as a station. Such a tape may be installed on each side of tunnel lining.

The quipment consists of six basic components: Extensometer, Stations, Anchors, Tapes or wires and Universal standardizer.

7.4 Extensometer and Accessories (Fig. 51 and 52)

a) Extensometer

The extensometer is designed for measuring displacement from 2-10 m. when movements are expected to be within 25 mm. It is equipped with dial gauge for reading the tape or wire tension, and micrometer for reading the movement of tunnel lining at the same tape tension.

b) Stations

The stations are fixed on concrete surface of tunnel lining.

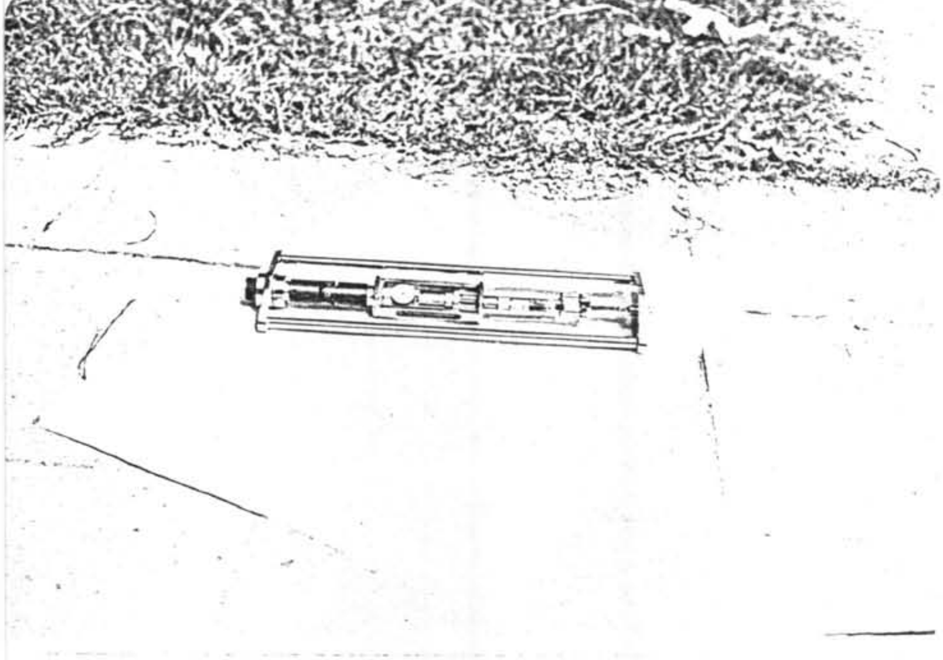


Fig. 51 Extensometer with universal standardizer

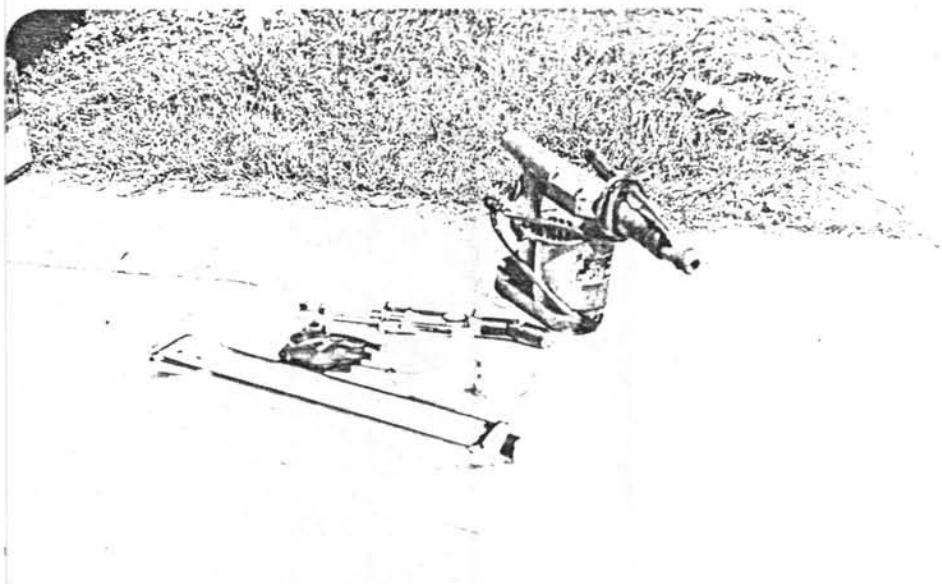


Fig. 52 Distortion measurement equipment

c) Anchors

Bolts with pin are used for anchoring wires or tapes at various points on tunnel lining.

d) Tape or Wire

Stainless steel tape normally used is 10 mm. wide by 0.5 mm. thick.

e) Universal Standardizer

The Universal standardizer is used for checking and standardizing the change of extensometer.

From Fig. 53 A,B,C,D,E,F,G and H are anchors which can be taken off from stations and the extensometers are fixed in the desired positions. The measurement lines are AB, CD, EF, EG, FH, EH, FG, AC, AD, AG and AH for known distorted tunnel shape.

The relationship between the distortion of primary lining and elapsed time in days is shown in Table 5 and Fig. 54.

7.4 Settlement

The settlements immediately occurred from construction (exclusive of long-time consolidation) are separated into the earth movement toward the working face, invasion of the surrounding soil into the annular space left by tail skin clearance and imperfections

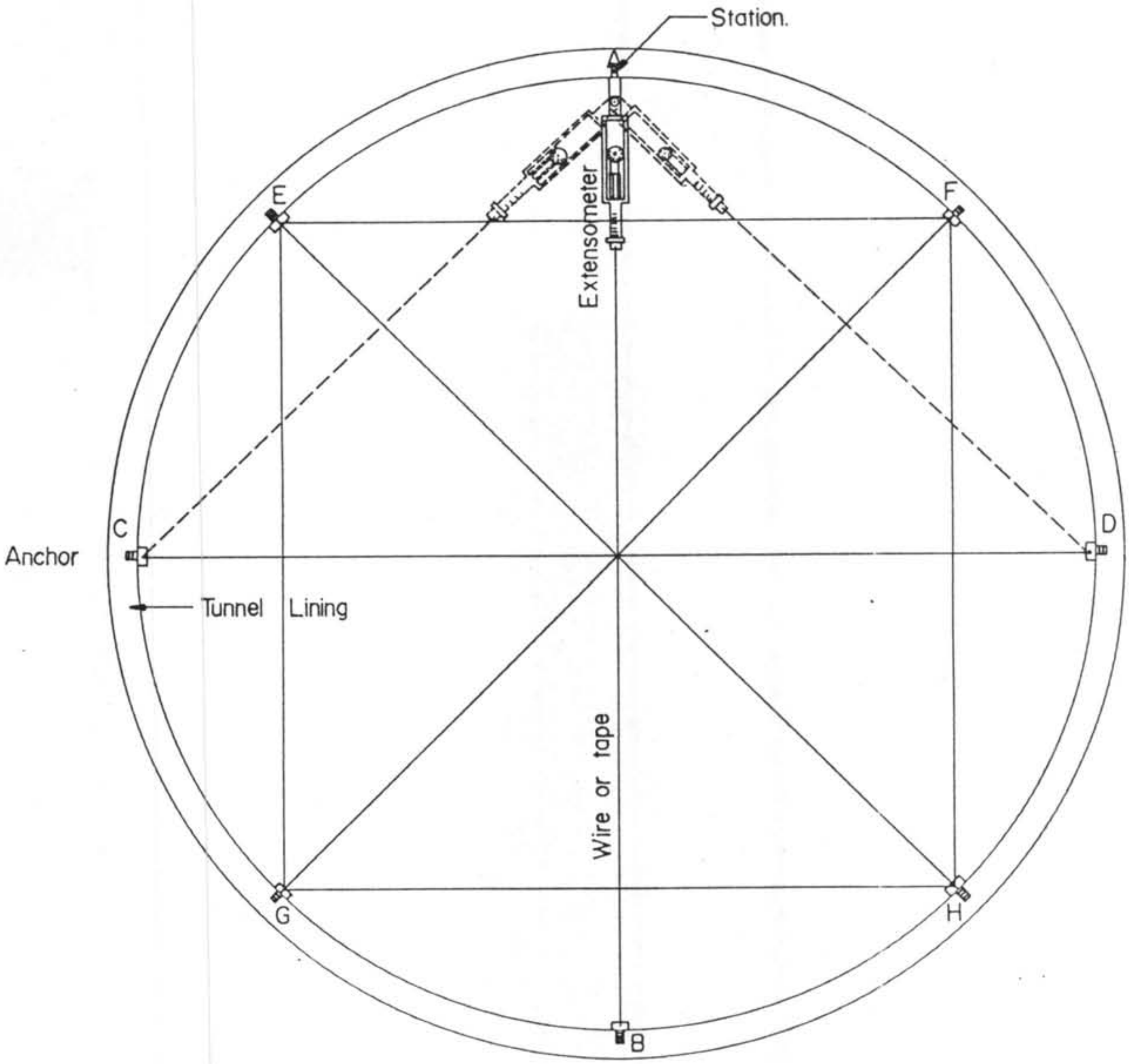


Fig. 53 Distortion measurement on tunnel lining by Extensometer.

BANGKOK WATER TRANSMISSION TUNNEL

DISTORTION VALUES IN TUNNEL LINING

SITE REFERENCE LAT YA RISTER TO THA PHRA RAT BURANA V.C.

RING NO. 64 SEGMENT TYPE NORMAL

LOCATION 44.5 m FROM ℄ OF LAT YA R.S.

SOIL CHARACTER STIFF CLAY

DATE OF ERECTION 26-12-78

DATE OF GROUTING 26-12-78

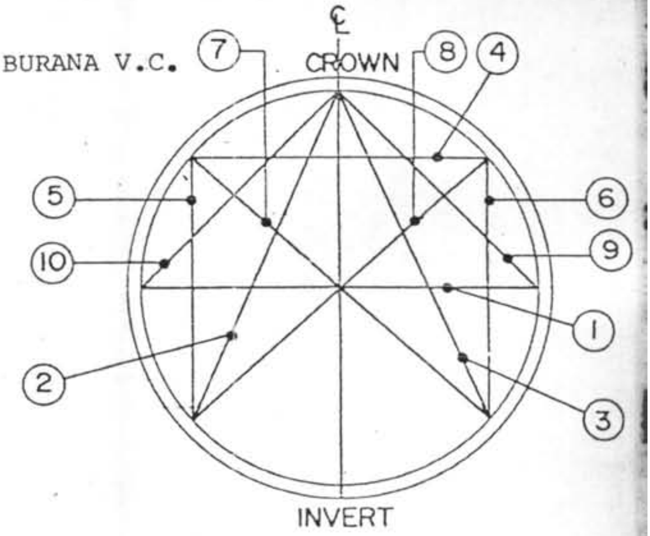


Table 5

LOOKING TOWARDS MACHINE

DATE	TEMP	TAPE REF.	EXTENSO-METER REF.	READING (MM.)	AV	DISTORTION VALUES	EXTENSO-METER POSITION	COMMENTS
31-1-79	36°C	1		1.81,1.82,1.83	1.82			
		2		8.01,8.05,8.06	8.04			
		3		17.72,17.73,17.70	17.72			
19-2-79	36°C	1		2.49,2.51,2.52	2.51	0.69		
		2		7.44,7.44,7.45	7.43	-0.61		
		3		17.16,17.12,17.14	17.14	-0.58		
19-3-79	37°C	1		3.30,3.26,3.26	3.27	1.45		
		2		7.38,7.37,7.38	7.38	-0.66		
		3		17.03,17.03,17.03	17.03	-0.69		
5-5-79	36°C	1		4.18,17.13,17.14	4.15	2.33		
		2		7.06,7.08,7.09	7.08	-0.96		
		3		16.74,16.73,16.74	16.74	-0.98		
18-5-79	36°C	1		4.31,4.27,4.28	4.29	2.47		
		2		7.04,7.00,6.96	7.00	-1.04		
		3		16.60,16.59,16.59	16.59	-1.13		

NOTATIONS : R = RIGHT CR = CROWN 1 = INVERT

BANGKOK WATER TRANSMISSION TUNNEL

DISTORTION VALUES IN TUNNEL LINING

SITE REFERENCE lat ya riser to tha phra rat burana V.C.

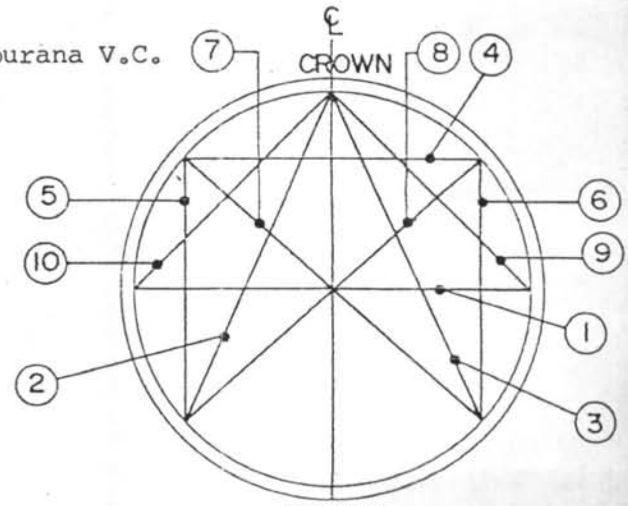
RING NO. 64 SEGMENT TYPE NORMAL

LOCATION 44.5 m. FROM ℄ OF LAT YA R.S.

SOIL CHARACTER STIFF CLAY

DATE OF ERECTION 26-12-78

DATE OF GROUTING 26-12-78



LOOKING TOWARDS MACHINE

DATE	TEMP	TAPE REF.	EXTENSOMETER REF.	READING (M M.)	AV	DISTORTION VALUES	EXTENSOMETER POSITION	COMMENTS
2-6-79	37°C	1		4.48,4.52,4.50	4.50	2.68		
		2		6.51,6.52,6.52	6.52	-1.52		
		3		16.15,16.15,16.16	16.15	-1.57		
6-7-79	38°C	1		4.71,4.68,6.70	4.69	2.87		
		2		6.47,6.47,6.48	6.47	-1.57		
		3		16.14,16.13,16.17	16.15	-1.57		
1-8-79	38°C	1		5.03,5.07,5.06	5.05	3.23		
		2		6.25,6.25,6.25	6.25	-1.79		
		3		15.88,15.86,16.89	15.88	-1.84		
21-9-79	37°C	1		5.07,5.13,5.13	5.11	3.29		
		2		6.02,6.01,5.98	6.00	-2.04		
		3		15.77,15.71,15.74	15.74	-1.98		

BANGKOK WATER TRANSMISSION TUNNEL

DISTORTION VALUES IN TUNNEL LINING

SITE REFERENCE LAT YA R.S. TO THA PHRA RAT BURANA V.C.

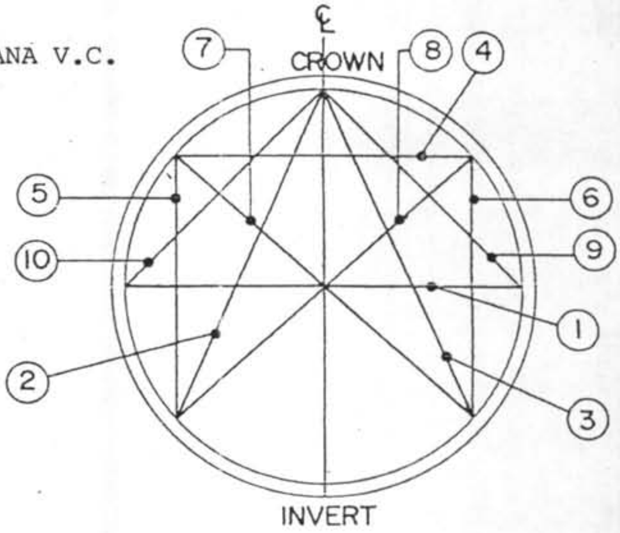
RING NO. 65 SEGMENT TYPE NORMAL

LOCATION 45.5 m. FROM ℄ OF LAT YA R.S.

SOIL CHARACTER STIFF CLAY

DATE OF ERECTION 26-12-78

DATE OF GROUTING 26-12-78



LOOKING TOWARDS MACHINE

DATE	TEMP	TAPE REF.	EXTENSOMETER REF.	READING (M.M.)	AV	DISTORTION VALUES	EXTENSOMETER POSITION	COMMENTS
31-1-79	36°C	1	142	8.62,8.61,8.62	8.62			
		2		7.62,7.65,7.67	8.65			
		3		7.90,7.91,7.92	8.91			
19-2-79	36°C	1		9.42,9.40,9.39	9.40	0.78		
		2		7.13,7.13,7.13	7.13	-0.52		
		3		7.36,7.37,7.41	7.38	-0.53		
19-3-79	37°C	1		10.08,10.03,10.04	10.05	1.43		
		2		6.99,6.94,6.96	6.97	-0.68		
		3		7.14,7.17,7.17	7.16	-0.75		
5-5-79	37°C	1		10.88,10.90,10.90	10.89	2.27		
		2		6.56,6.52,6.53	6.54	-1.11		
		3		6.93,6.94,6.90	6.92	-0.99		
18-5-79	36°C	1		11.10,11.11,11.15	11.12	2.50		
		2		6.46,6.46,6.49	6.47	-1.18		
		3		6.85,6.85,6.85	6.85	-1.06		

BANGKOK WATER TRANSMISSION TUNNEL

DISTORTION VALUES IN TUNNEL LINING

SITE REFERENCE LAT YA RISER TO THA PHRA RAT BURANA V.C.

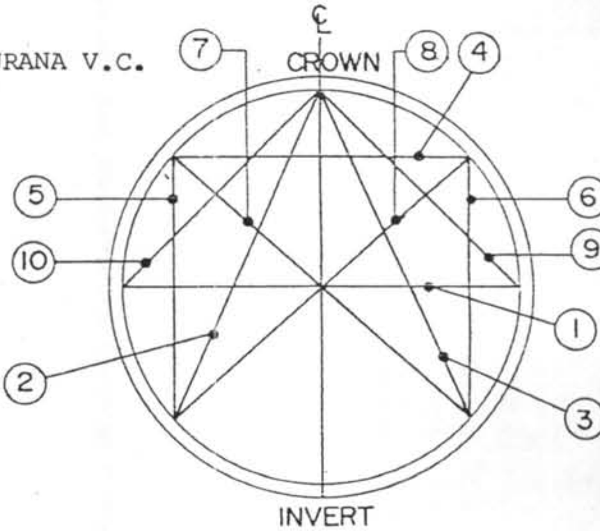
RING NO. 65 SEGMENT TYPE NORMAL

LOCATION 45.5 m. FROM ℄ OF LAT YA R.S.

SOIL CHARACTER STIFF CLAY

DATE OF ERECTION 26-12-78

DATE OF GROUTING 26-12-78



LOOKING TOWARDS MACHINE

DATE	TEMP	TAPE REF.	EXTENSO-METER REF.	READING (M M.)	AV	DISTORTION VALUES	EXTENSO-METER POSITION	COMMENTS
2-6-79	37°C	1		11.33,11.30,11.35	11.33	2.71		
		2		6.24,6.23,6.24	6.24	-1.41		
		3		6.63,6.65,6.61	6.63	-1.28		
6-7-79	38°C	1		11.53,11.55,11.54	11.54	2.92		
		2		6.13,6.10,6.16	6.13	-1.52		
		3		6.56,6.57,6.57	6.57	-1.34		
1-8-79	38°C	1		11.72,11.73,11.73	11.73	3.11		
		2		5.82,5.85,5.87	5.85	-1.80		
		3		6.36,6.30,6.31	6.32	-1.59		
21-9-79	37°C	1		11.83,11.81,11.81	11.82	3.20		
		2		5.49,5.47,5.49	5.48	-2.17		
		3		6.11,6.10,6.10	6.10	-1.81		

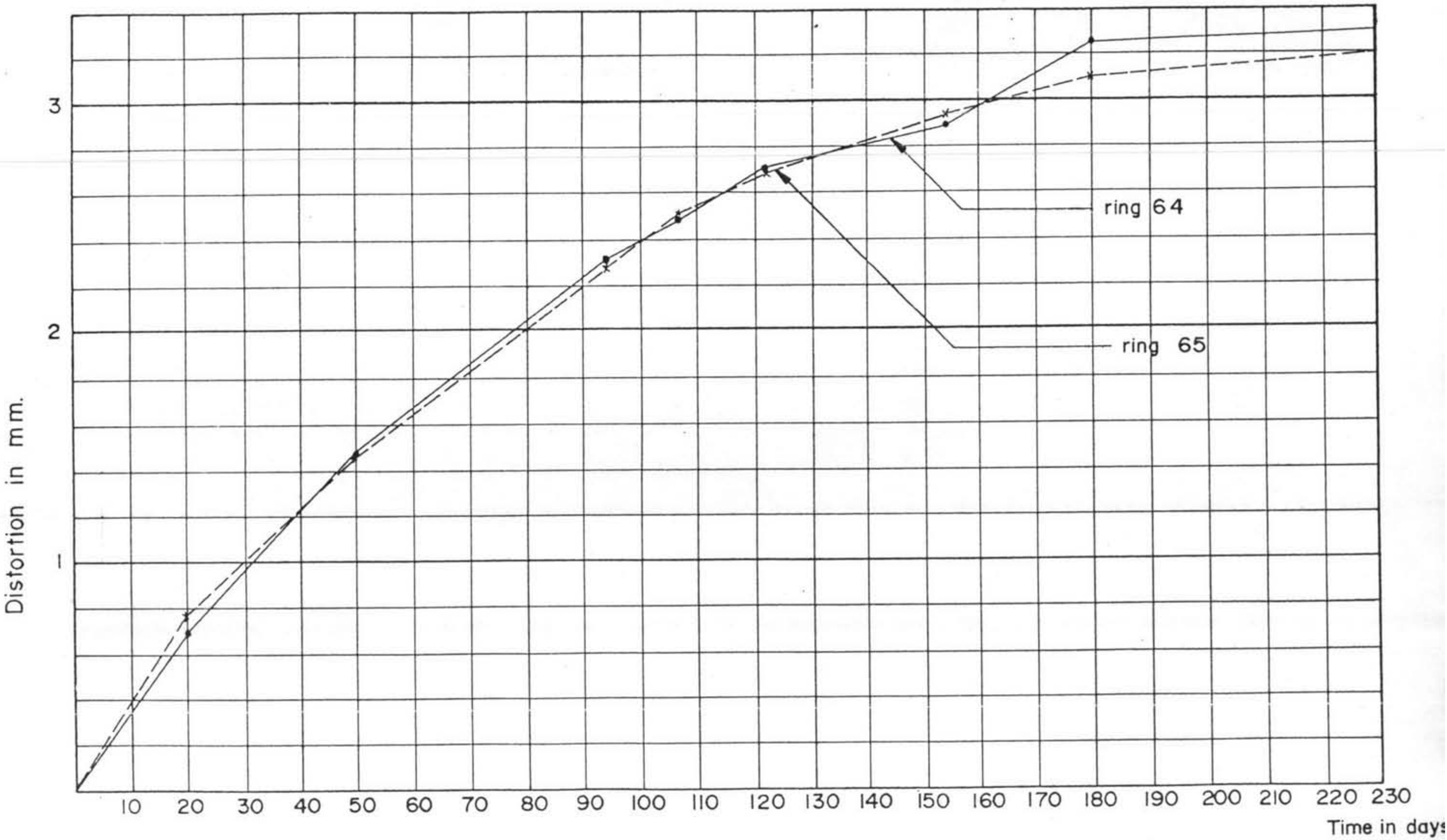


Fig. 54 Relationship between distortion of primary lining and elapsed time of line I.

of grouting during construction, inflow of material with ground water entering the tunnel at improtected place

a) Settlement Calculation

The sources contributing to ground settlement and displacement in the unproperly filled voids tunnel are

1. The settlement resulting from the longitudinal tensile stress developed in clay, that is the effect of the squeezing of ground into the annular void around the lining.

2. Delayed settlement resulting from the consolidation of remolded clay and may increase its compressibility.

From Szechy[15]

In Fig. 55

$$u = u_1 + \frac{u_o}{2} \qquad \text{(Protodyakonov's theory)}$$

$$u_o = 2\delta + \Delta$$

where

u_o = the maximum annular space.

δ = thickness of tail skin.

Δ = clearance between lining and inner surface of tail skin.

u_1 = settlement due to the loosening zone above the crown of tunnel.

u = total space thickness above the tunnel lining.

$$u_1 = \frac{h^2 \gamma}{2M}$$

$$h = \frac{B}{2 \tan \phi}$$

if

h = height of loosening.

γ = unit weight of loosening.

M = modulus of soil reaction.

B = the maximum of loosening at the top of tunnel.

$$B = 2r\,(\tan \beta + \sec \beta)$$

$$\beta = 45^{\circ} + \frac{\phi}{2}$$

$$t = H - \frac{h}{2}$$

$$u = \frac{B^2}{8M \tan^2 \phi} + \frac{2\delta + \Delta}{2}$$

$$S_{max} = \frac{2\pi B\, u \left(\frac{h + t}{t}\right)}{8\,(B + 2h \times \tan \beta)(1 + \frac{h}{2t}) + t. \cot \phi} \qquad \ldots\ldots\ 2.95$$

where

S_{max} = the maximum settlement depth.

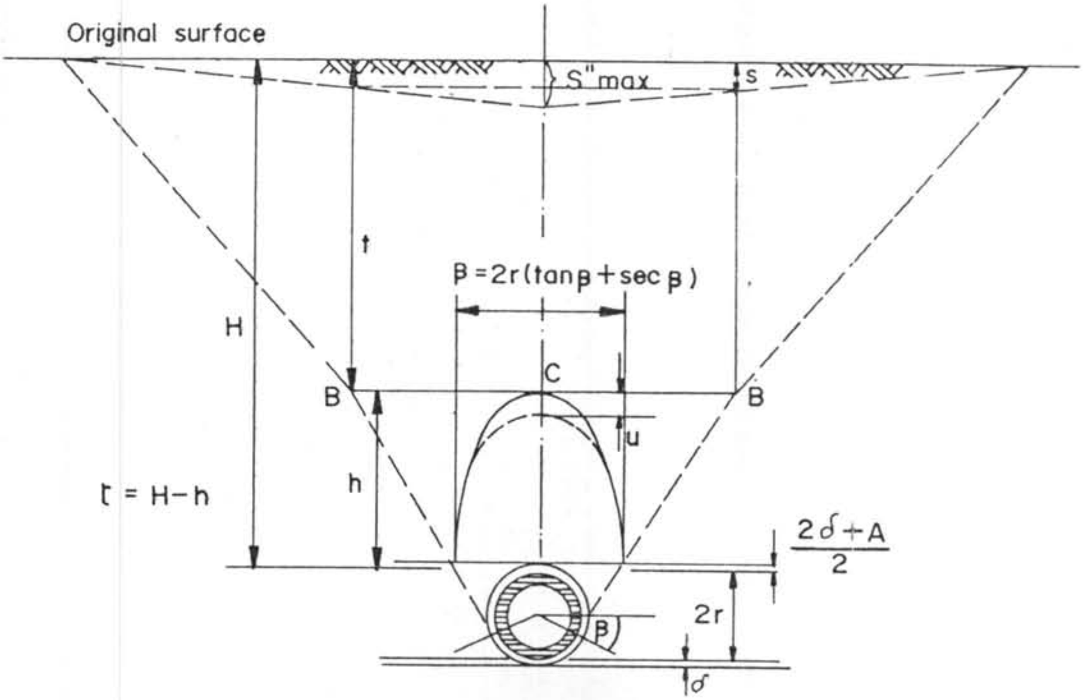


Fig. 55 Settlement trough and unfilled void around tunnel

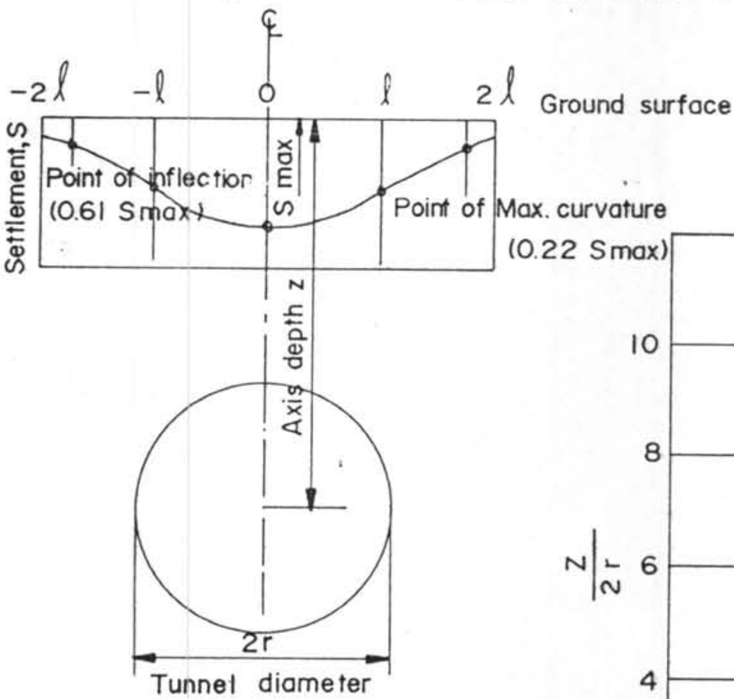


Fig. 56 Error function representing cross section of settlement trough above circular tunnel

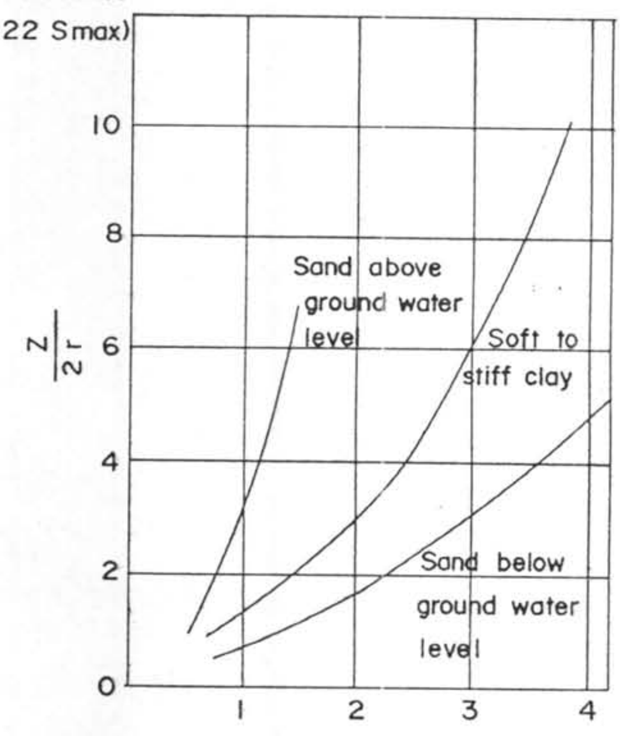


Fig. 57 Relation between related width of settlement trough and related depth

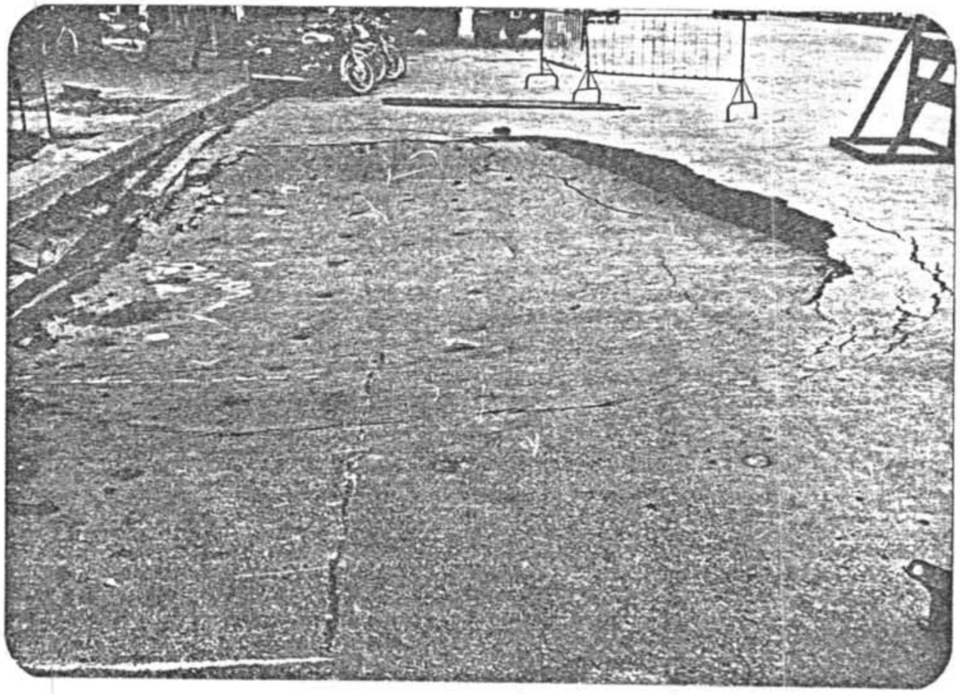


Fig. 58a

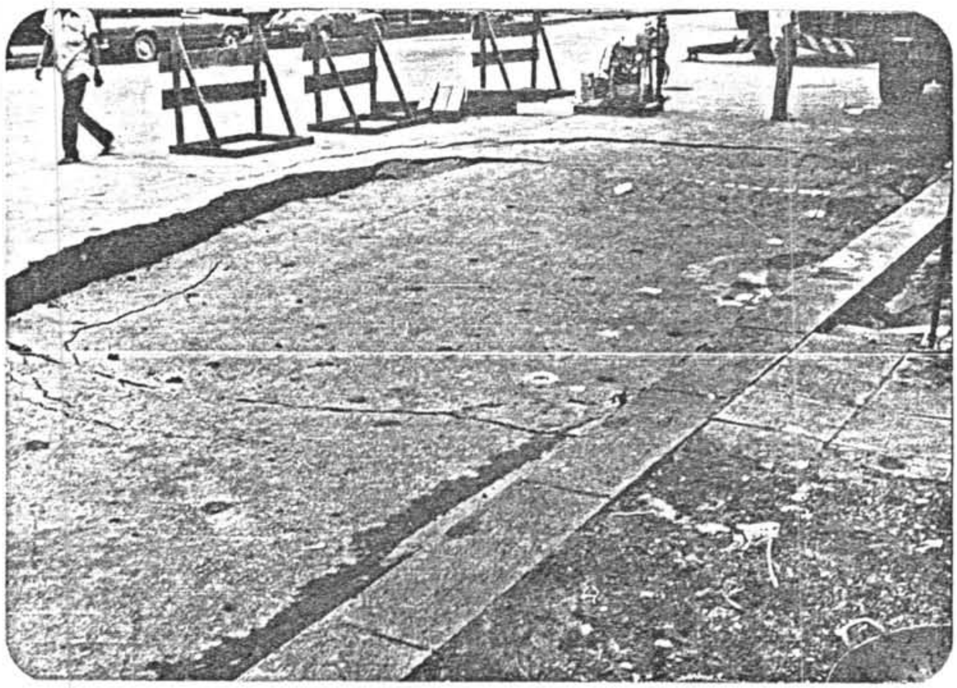


Fig. 58b

Fig. 58 Surface settlement because of inproperly filled voids between lining and surrounding soil.

b) Volume of Settlement

By Peck's observation, he pointed out in the "Deep excavation and tunnelling in soft ground" (Szechy[15], the volume of settlement is

$$V = 2.5.\ell.S_{max} \qquad \ldots\ldots\ldots\ldots\ldots\ldots \; 2.96$$

where

V = volume of settlement.

ℓ = distance from the center line of the tunnel to the point of maximum curvature.

S_{max} = the maximum settlement depth.

By the error function and its relationship of the dimensions and relative depth of tunnel, the settlement curve can be plotted as shown in Fig. 56 and 57.

8. Secondary Lining

Secondary lining or cast in place lining is the internal lining subject to inside pressures while primary lining was designed for external pressure withstanding.

Secondary lining construction is deferred until tunnel distortion is in the least rate. It is necessary to find the time interval between the construction of primary lining and secondary lining in each surrounding soil.

Two generally types of secondary lining are

1. reinforced concrete secondary lining.
2. steel pipe secondary lining.

8.1 Secondary Lining Design

The design is based on Timoshenko and Goodier formulae[21]. The stress of the hollow cylinder that is submitted to internal pressure, p_i in Fig. 59 are

$$\sigma_r = \frac{a^2 p_i}{b^2-a^2}\left(1 - \frac{b^2}{r^2}\right) \qquad \text{.................. 2.97}$$

$$\sigma_\theta = \frac{a^2 p_i}{b^2-a^2}\left(1 + \frac{b^2}{r^2}\right) \qquad \text{.................. 2.98}$$

where

σ_r = radial stress of the cylinder.

σ_θ = circumferential stress of the cylinder.

p_i = internal pressure of the cylinder.

a = inner radius of the cylinder

b = outer radius of the cylinder.

r = radius at any point of the cylinder ($a \leqslant r \leqslant b$).

If the internal pressure, p_i is more than the external pressure, σ_r is always a compressive stress and σ_θ a tensile stress.

At inner surface of the cylinder

$$\sigma_r = -p_i \qquad \text{.................} \quad 2.99$$

$$\sigma_\theta = p_i\left(\frac{a^2+b^2}{b^2-a^2}\right) \qquad \text{.................} \quad 2.100$$

At outer surface

$$\sigma_r = 0 \qquad \text{.................} \quad 2.101$$

$$\sigma_\theta = \frac{2a^2 p_i}{b^2-a^2} \qquad \text{.................} \quad 2.102$$

a) Reinforced Concrete Secondary Lining (Fig. 60)

As the secondary lining is in high water pressure condition, tension cracks on concrete, that water can permeate into reinforcing steels and destroy them are not permitted (from Ekkasit[1]).

By controlling the tension crack on inner surface of concrete, the strength and thickness of concrete are more effective in desining than reinforcing steel. For the longitudinal steel, it is used as temperature steel.

According to ACI-318-77[2], the modulus of rupture of concrete is $2.0\sqrt{f'_c}$

If

E_c = modulus of elasticity of concrete.

E_s = modulus of elasticity of reinforcing steel.

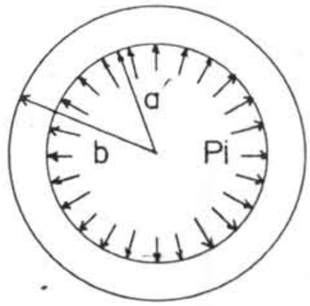
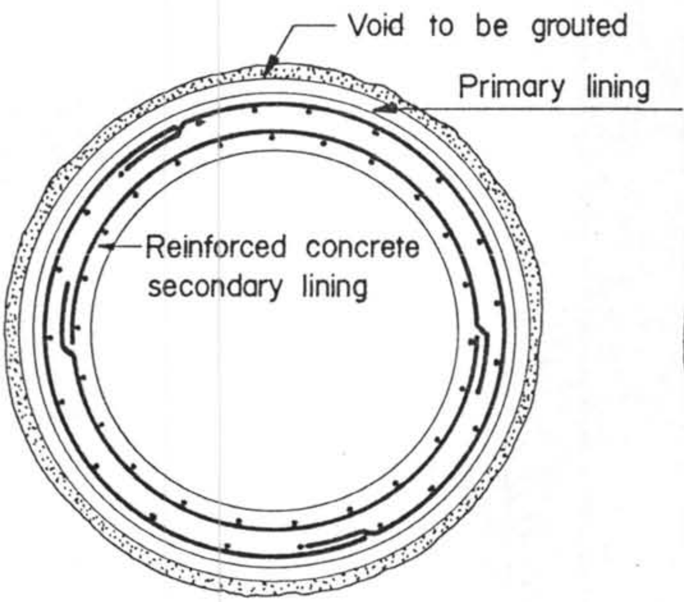


Fig. 59 The hollow cylinder submitted to uniform inner pressure.



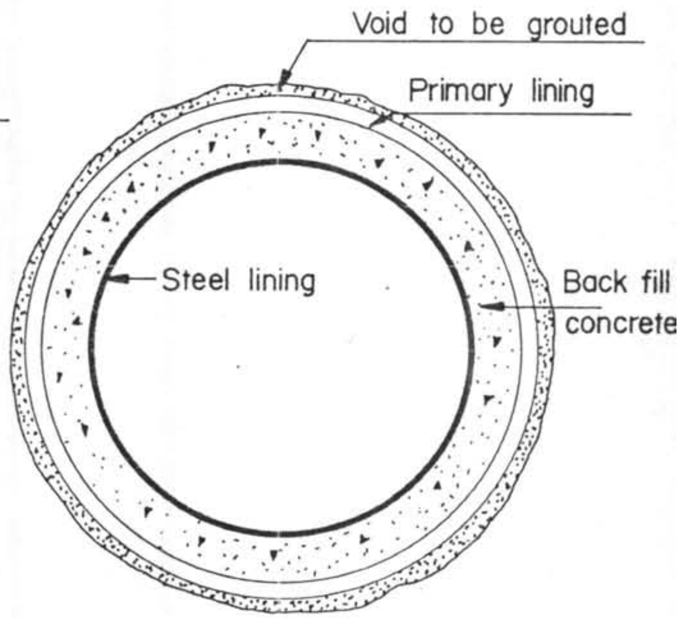


Fig. 60 Reinforced concrete secondary lining. Fig. 61 Steel pipe secondary lining.

f_t = tensile stress of concrete.

f_s = tensile stress of reinforcing steel.

ε_s = tensile strain of reinforcing steel.

ε_t = tensile strain of concrete.

$$\varepsilon_t = \frac{f_t}{E_c}$$

allowable $f_t = 2.0\sqrt{f'_c}$ 2.103

$$\varepsilon_t = \frac{2.0\sqrt{f'_c}}{E_c}$$

$$\varepsilon_s = \frac{f_s}{E_s}$$

but

$$\varepsilon_s = \varepsilon_t$$

$$f_s = E_s \varepsilon_t$$

allowable $f_s = \dfrac{2.0E_s\sqrt{f'_c}}{E_c}$ 2.104

b) <u>Steel Pipe Secondary Lining (Fig. 61)</u>

In the design of steel pipe secondary lining, all stresses from the internal pressure are only on the steel pipe. The backfill concrete is to fill the void between the primary lining and the steel pipe.

If f_s all is the allowable tensile stress of steel plate.

$$\sigma_\theta \leq f_s \text{ allow}$$

c) Example of Secondary Lining Calculation

If

The maximum water pressure in the tunnel; p_i = 6 ksc.

Inner radius of primary lining; a = 4.30 m.

1. Reinforced Concrete Secondary Lining

Assume

The thickness of secondary lining = 0.30 m.

Inner radius of secondary lining; a = 1.00 m.

Outer radius of secondary lining; b = 1.30 m.

Circumferential steel # 9 @ 270 mm.

Longitudinal steel # 4 @ 455 mm.

Cylindrical stress of concrete = 300 ksc.

Yield stress of reinforced steel = 1400 ksc.

From Eq. 2.103

$$\text{allowable } f_t = 2.0\sqrt{f'_c}$$

$$= 2.0\sqrt{300} = 34.6 \quad \text{ksc.}$$

For concrete

From Eq. 2.99 - 2.102

Radial stress; σ_r

At inner surface of the lining $= -6$ ksc.

At outer surface $= 0$ ksc.

Circumferential stress; σ_θ

At inner fiber of the lining $= \dfrac{6(1^2 + 1.30^2)}{1.30^2 - 1^2}$

$= 23.4$ ksc. $<$ allowable f_t

At outer fiber $= \dfrac{2 \times 1^2 \times 6}{1.30^2 - 1^2}$

$= 17.4$ ksc. $<$ allowable f_t

For reinforcing steel

Covering of steel $= 5$ cm.

Distance from inner circum-

-ferential steel to inner

fiber of concrete $= 5 + 4.55 + \dfrac{1.35}{2} = 10.9$ cm.

$\therefore \; r = 1.00 + 0.109 = 1.11$ m.

From Eq. 2.98

Tensile stress at inner

reinforcing steel, σ_θ $= \dfrac{1^2 \times 6}{1.30^2 - 1^2}\left(1 + \dfrac{1.30^2}{1.11^2}\right)$

$= 20.64$ ksc. $<$ allowable f_s

2. Steel Pipe Secondary Lining

Assume

Inner radius of secondary lining $= 1.00$ m.

Steel pipe thickness $= 0.05$ m.

Backfill concrete thickness $= 1.30 - 1.00 - 0.05$

$= 0.25$ m.

Allowable tensile stress of steel plate $= 2,500$ ksc.

From Eq. 2.100 and 2.102

Circumferential stress; σ_θ

At inner fiber σ_θ $= 6\left(\frac{1^2 + 1.05^2}{1.05^2 - 1^2}\right)$

$= 123$ ksc. $< f_s$ all.

At outer fiber σ_θ $= \frac{2 \times 1^2 \times 6}{1.05^2 - 1^2}$

$= 117$ ksc. $< f_s$ all.

8.3 Secondary Lining construction

Concrete for the secondary lining is mixed at the nearest concrete plant (Fig. 62) on the ground surface and transporting to the working area. It is necessary to control the concrete properties, method of concrete mixing, transporting and placing.

Secondary lining concrete will be deposited in place within 45 minutes after the addition of water. Concrete is shot down from the surface to the tunnel through 20 cm. diameter pipes with careful operations without high velocity discharge and resultant separation.

a) Reinforced Concrete Secondary Lining

The forms of secondary lining (Fig. 63) are true to the required shape and sizes, strong and rigid so as there

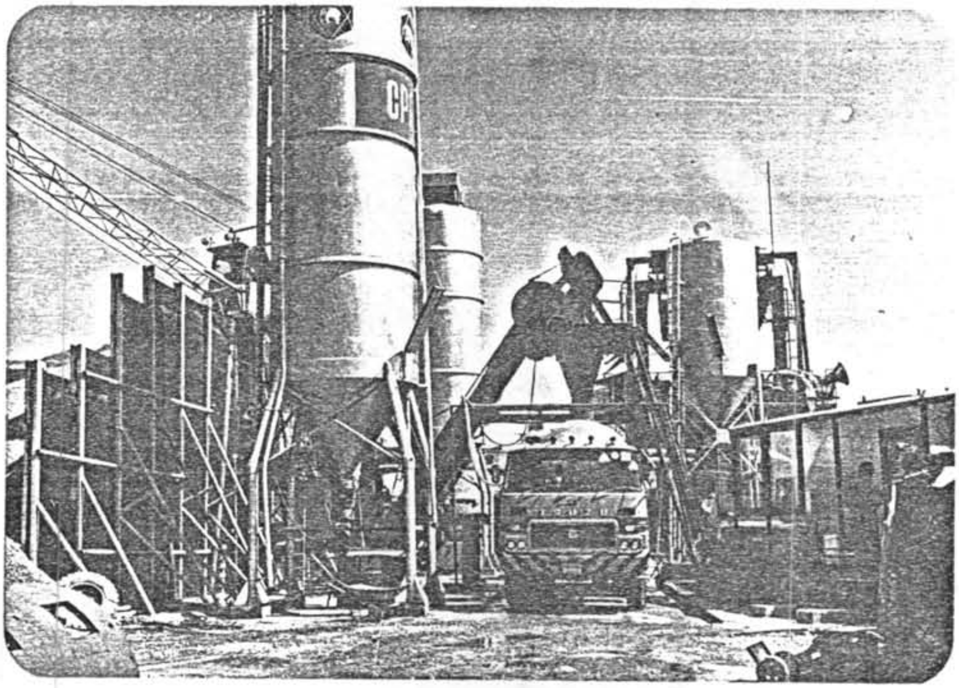


Fig. 62 Concrete mixing plant in working area

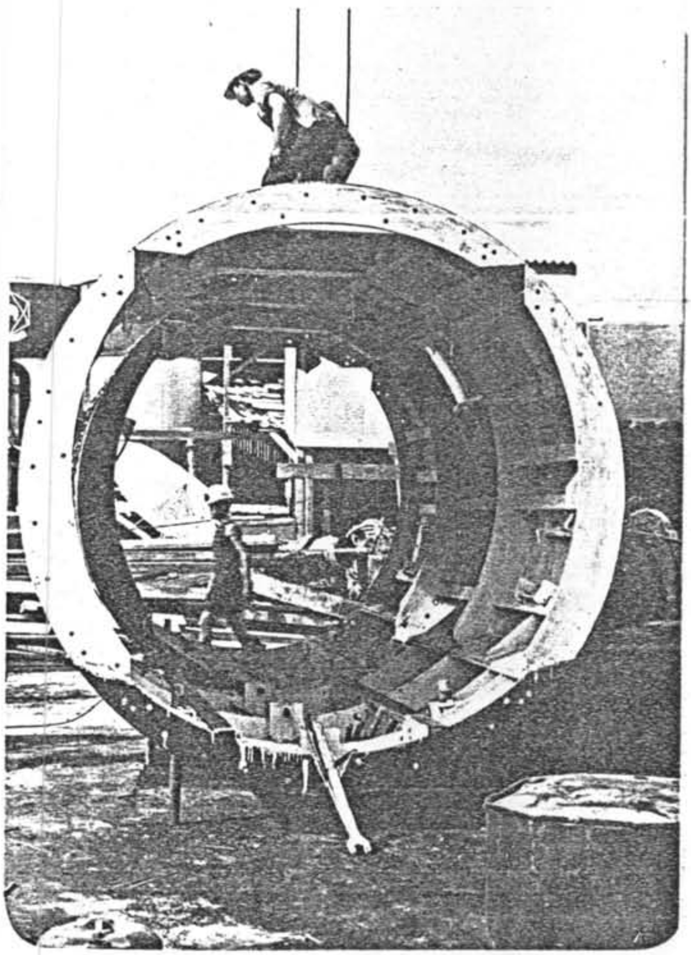


Fig. 63 Secondary lining form

will be no deformation while concrete placing. Full round forms have a tendency to float, they must be blocked to the primary lining to prevent their movement. Concrete is shot through the pipe and discharged on the top of the form running on both sides until it is into all panels of secondary lining and the fill the tunnel arch completely.

Consolidation of concrete are electrically or pneumatically driven with immersion type vibrators and form vibrators. Immersion type vibrators must be handled through a window in the form with vibrating speeds of 6,000 to 7,000 rpm.

Fig. 64 and 65 show the reinforcing steel and the finished reinforced concrete secondary lining.

b) Steel Pipe Secondary Lining

After steel pipes are placed in the proposed position and fixed rigidly to the primary lining. Lean concrete is used for backfill the void between the steel pipe and the primary lining.

The approximate cost relationship for reinforced concrete and steel pipe secondary lining are in the rate of 1.0 to 1.5.

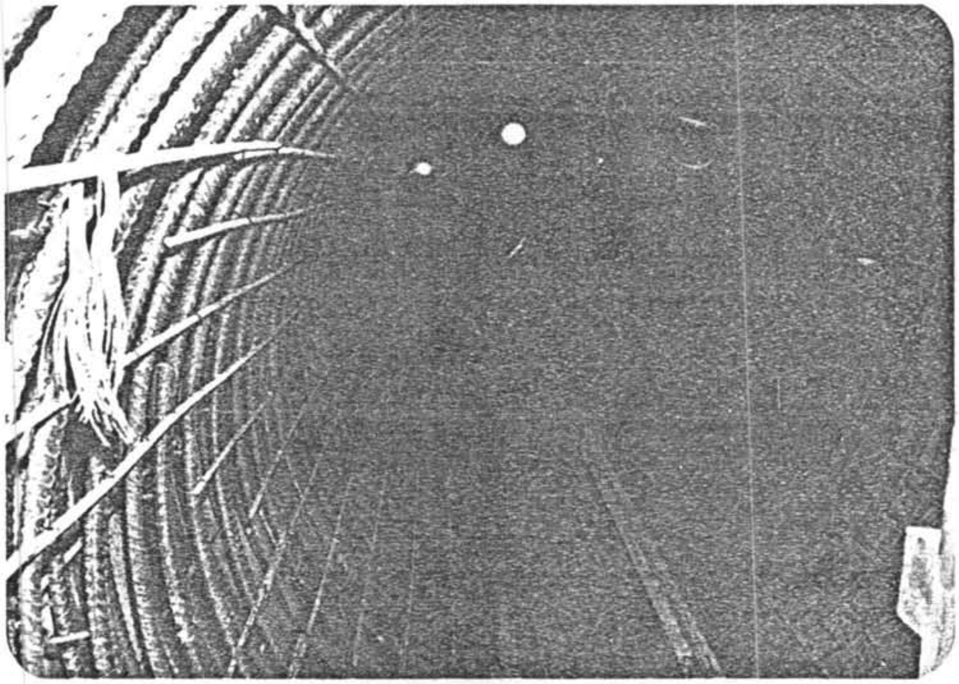


Fig. 64 Reinforcing steel for secondary lining

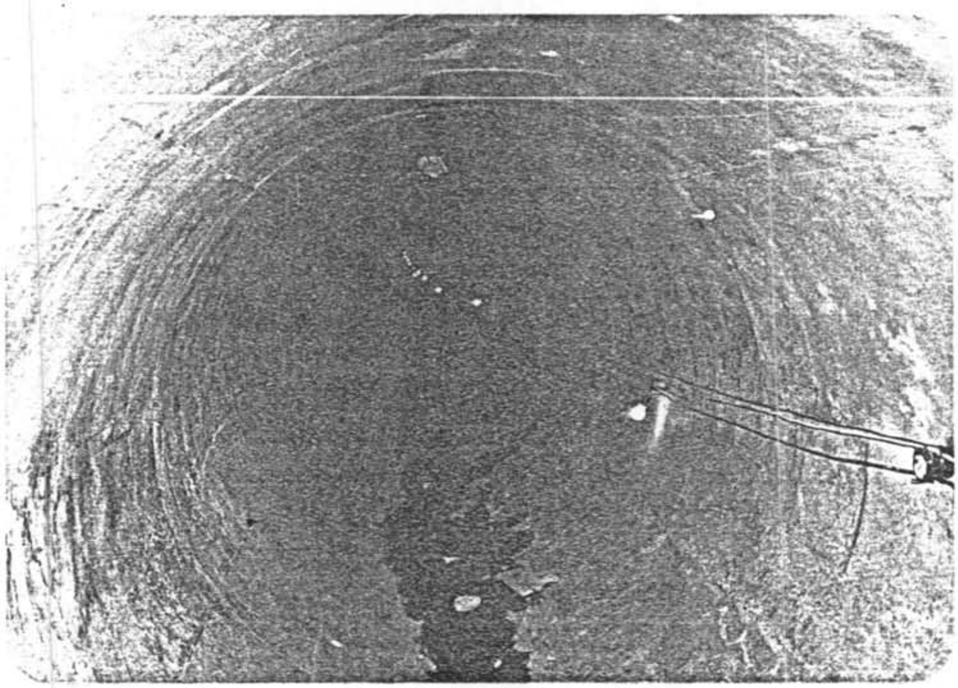


Fig. 65 Reinforced concrete secondary lining