OFFICIAL REPORT OF THE
INTERNATIONAL TUNNELLING ASSOCIATION

Guidelines for the Design of Shield Tunnel Lining

Working Group No. 2, International Tunnelling Association

Abstract—*These guidelines, prepared by Working Group 2 (Research) of the International Tunnelling Association, are presented in three parts: Part I describes the outline of the procedure of design. Part II presents the detailed design methods. Part III provides references, including examples of design. Because the methods of designing shield tunnel linings vary, the guidelines do not recommend that priority be given to any one method. Rather, this report presents the basic concepts of shield tunnel linings, in order to provide reference and guidance in designing tunnel linings.*

Preface

Although these guidelines present the basic concepts of shield tunnel lining, they do not supersede relevant specifications relevant to each country and each project. The aim of the guidelines is to promote advances in the design of shield tunnels in accordance with the objectives of the International Tunnelling Association prescribed in Section II of the Statutes of the ITA (ITA 1976).

Work on the guidelines began at the meeting of ITA Working Group 2 – Research in Amsterdam in 1993. After much study, discussion and investigation, the guidelines were completed in December 1999.

The guidelines consist of three parts:

Part I outlines the shield tunnel design procedure.

Part II presents the detailed design methods.

Part III provide references, including examples of shield tunnel design.

Because there are various competent methods of designing shield tunnel linings, these guidelines give no priority to any specific method. Rather, they introduce design methods generally and widely used throughout the world.

Shield tunnels are usually excavated in soft ground rather than in rock. The parameters of linings, such as dimension and strength of material, are subject not only to ground conditions but also to construction conditions.

The practice of designing tunnel linings requires much experience, practical and theoretical knowledge. It is therefore not expected that these guidelines will cover every point of tunnel lining design, but instead will provide basic knowledge useful to design practitioners. It is hoped that this knowledge will be continuously improved through progress in tunnelling technology.

As Vice-Animateur of ITA Working Group 2, I wish to acknowledge the important contributions of the following persons: Mssr. Yann Leblais, who led the study as Animateur; Professor Andre Assis and Professor Z. Eisenstein, who guided our study as Tutors; working group members Herr Dr. Harald Wagner, Professor Teodor Iftimie, Dr. Birger Schmidt, Signor Piergiorgio Grasso, all of whom contributed greatly to the study; and members of Research and Development Committees of the Japan Tunnelling Association, chaired by Professor Toru Konda, who prepared the draft of the guidelines.

For the International Tunnelling Association,

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Part I: Outline of Shield Tunnel Lining Design Procedure

Following the planning works for the tunnel, the lining of a shield tunnel is designed according to the following sequence, as a rule:

1. Adherence to specification, code or standard. The tunnel to be constructed should be designed according to the appropriate specification standard, code or standards, which are determined by the persons in charge of the project or decided by discussion between these persons and the designers.

2. Decision on inner dimension of tunnel. The inner diameter of the tunnel to be designed should be decided in consideration of the space that is demanded by the functions of the tunnel. This space is determined by:

- The construction gauge and car gauge, in the case of railway tunnels;

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Pergamon

- The traffic volume and number of lanes, in the case of road tunnels;
- The discharge, in the case of water tunnels and sewer tunnels;
- The kind of facilities and their dimensions, in the case of common ducts.

3. Determination of load condition. The loads acting on the lining include earth pressure and water pressure, dead load, reaction, surcharge and thrust force of shield jacks, etc. The designer should select the cases critical to the design lining.

4. Determination of lining conditions. The designer should decide on the lining conditions, such as dimension of the lining (thickness), strength of material, arrangement of reinforcement, etc.

5. Computation of member forces. The designer should compute member forces such as bending moment, axial force and shear force of the lining, by using appropriate models and design methods.

6. Safety check. The designer should check the safety of the lining against the computed member forces.

7. Review. If the designed lining is not safe against design loads, the designer should change the lining conditions and design lining. If the designed lining safe but not economical, the designer should change the lining conditions and redesign the lining.

8. Approval of the design. After the designer judges that the designed lining is safe, economical and optimally designed, a document of design should be approved by the persons in charge of the project.

In Figure I-1, these steps are shown on a flow chart for designing tunnel linings. A schematic example of step-by-step design procedures is summarized in Part I-A, which immediately follows this section.

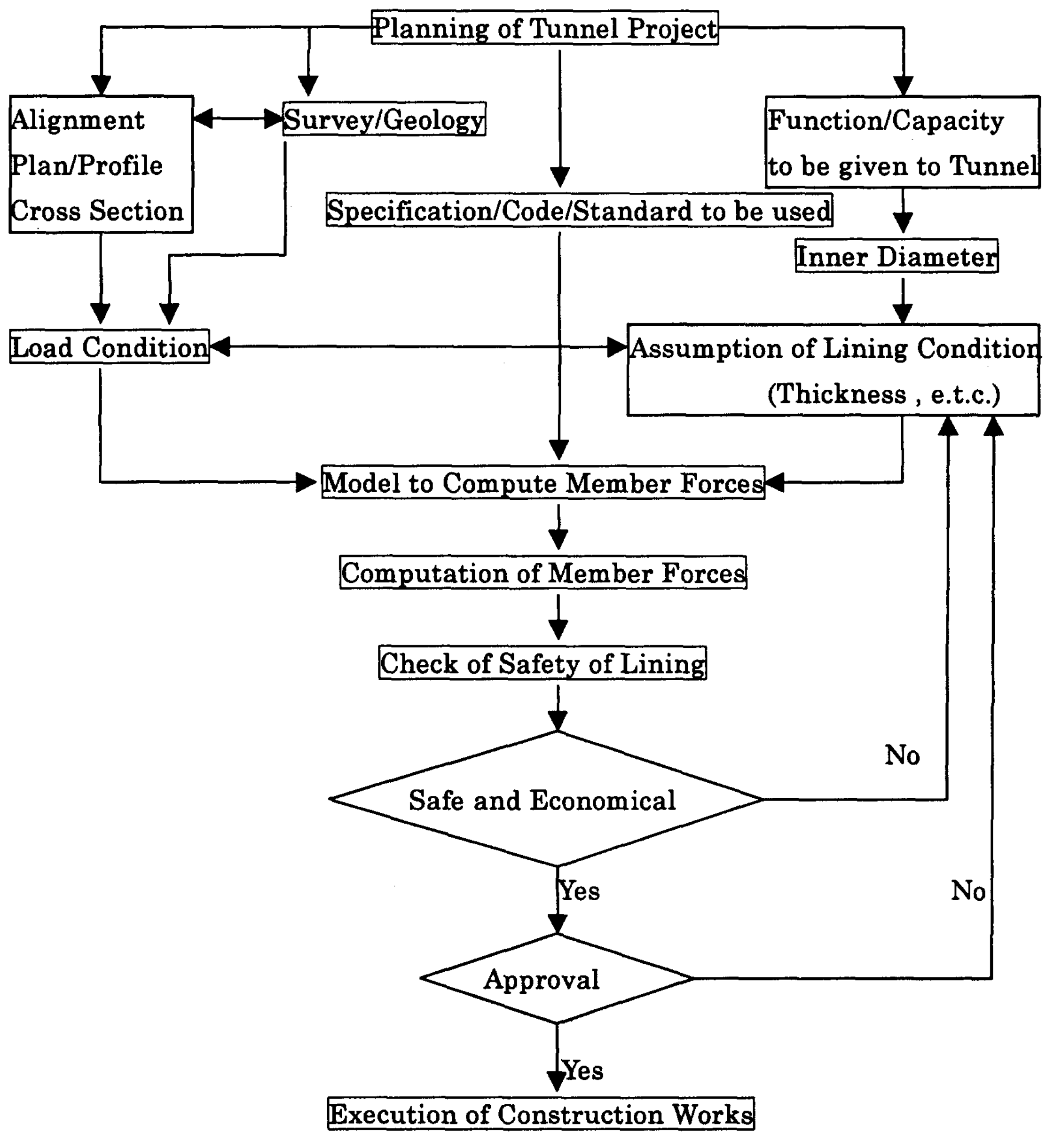


Figure I-1. Flow chart of shield tunnel lining design.

Part I-A. Schematic Example of Step-by-Step Design Procedure

This appendix provides a summarized schematic example of step-by-step design procedure.

Step 1: Define Geometric Parameters

Alignment, excavation diameter, lining diameter, lining thickness, average width of ring, segment system, joint connections.

Step 2: Determine Geotechnical Data

Specific gravity, cohesion (unconfined and effective), friction angle (unconfined and effective), modulus of elasticity, modulus of deformation, K_0-value.

Step 3: Select Critical Sections

Influence of overburden, surface loads, water, adjacent structures.

Step 4: Determine Mechanical Data of TBM

Total thrust pressure, number of thrusts, number of pads, pad geometry, grouting pressure, space for installation.

Step 5: Define Material Properties

Concrete class, compressive strength, modulus of elasticity, steel type, tensile strength, gasket type, gasket width, elastic capacity, allowable gap.

Step 6: Design Loads

6.1: Geostatical loads

Analyse load effects on lining segments and ground (Figs. A-1 through A-5).

6.2: Thrust jacking loads

Analyse load effects distributed on segment types by thruster pads (Fig. A-6).

6.3: Trailer and other service loads

Including main bearing loads, divided by number of wheels (Fig. A-7)

6.4: Secondary grouting loads

Extending regular grout pressure (Fig. A-8).

6.5: Dead load, storage and erection loads

Bending moment influence (Fig. A-9).

Step 7: Design Model

The three- dimensional condition has to be simulated by symbolic computation as two-dimensional conditions.

7.1: Analytical model

Using formulas in accordance with national stan dards and with superposition of selected design loads (Fig. A-10).

7.2: Numerical model

Using Finite-Element programs with constitutive laws in accordance with national standards to

LOADING 1: Initial state of stress

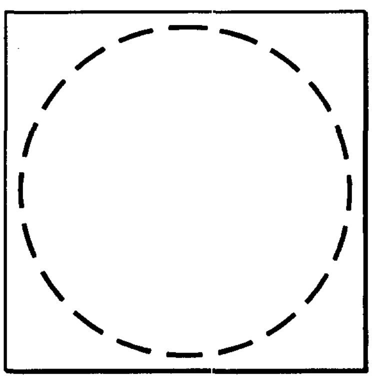


Figure A-1. Loading Case 1.

LOADING 2: Initial stress relief

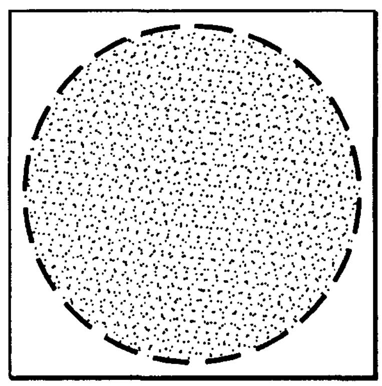


Figure A-2. Loading Case 2.

LOADING 3: Excavation supported by shield

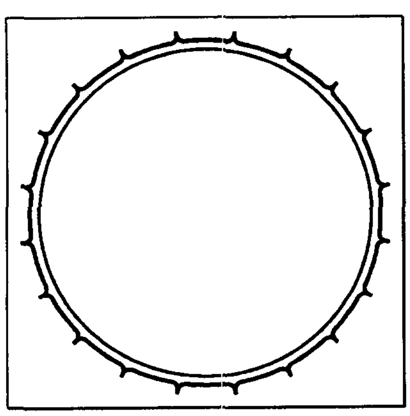


Figure A-3. Loading Case 3.

LOADING 4: Excavation supported by grouted segment

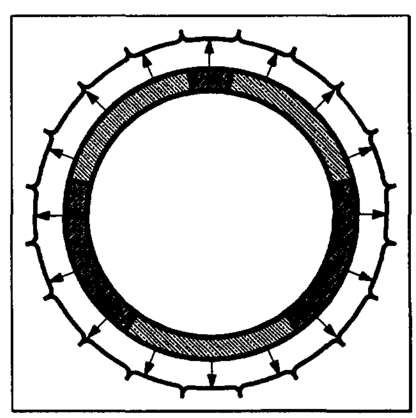


Figure A-4. Loading Case 4.

achieve stresses and strains under elasto-plastic conditions, allowing simulation of detailed construction stages (Fig. A-11).

Step 8: Computational Results

Are represented in table format as normal and shear forces, bending moments and deflections, defining the design loads and subsequently reinforcement of the segments.

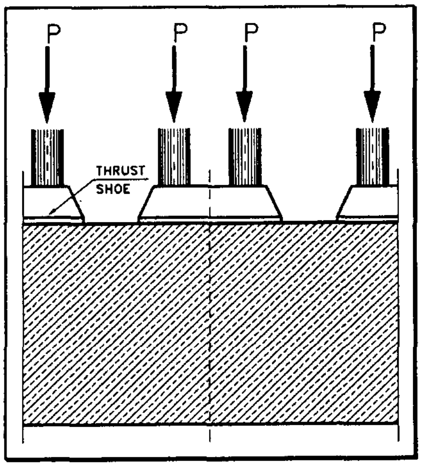


Figure A-6. Thruster pads distribution.

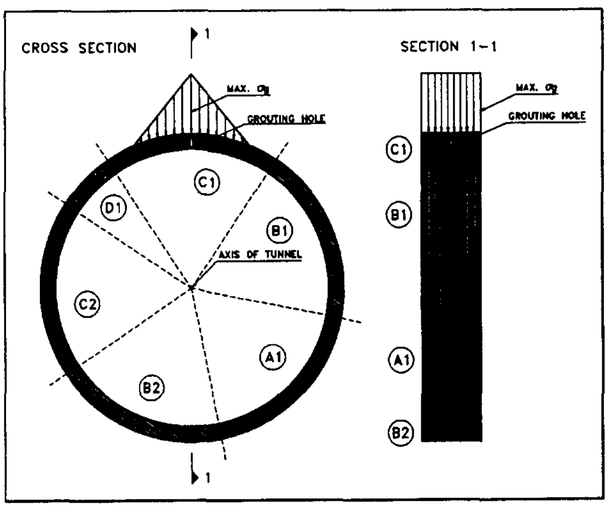


Figure A-8. regular grout pressure.

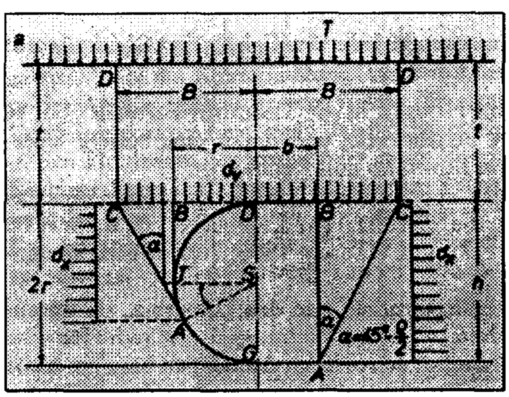


Figure A-10. Design load – Assumption of Terzaghi.

LOADING 5: Long term deformation

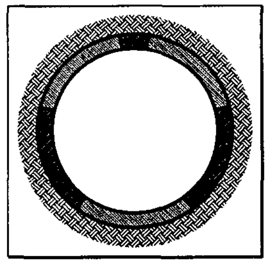


Figure A-5. Loading Case 5.

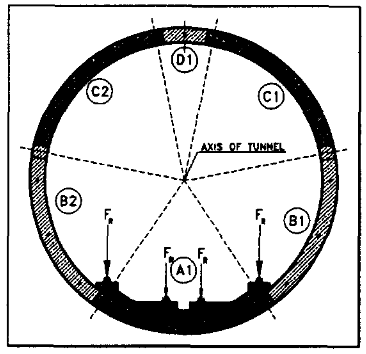


Figure A-7. Trailer load distribution.

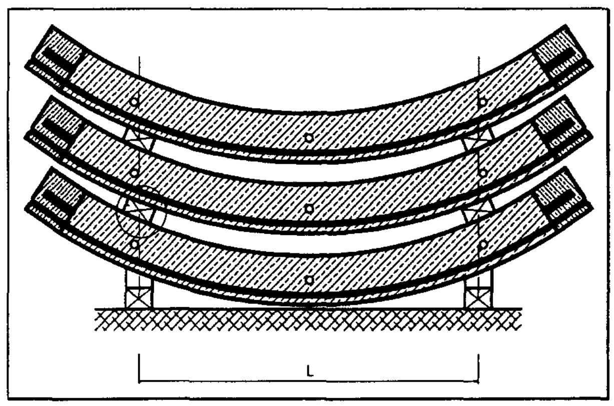


Figure A-9. Self weight of segments on stock.

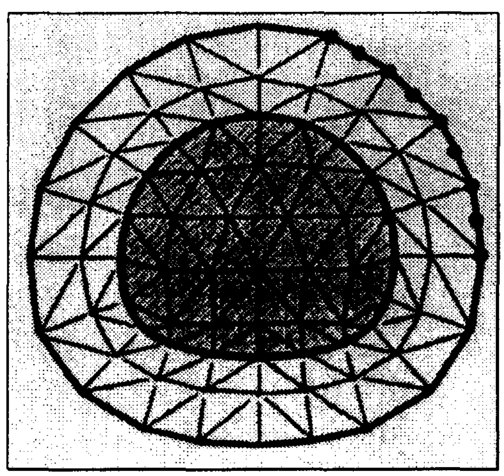


Figure A-11. FEM network configuration.

Part II: Design Method of Shield Tunnel Lining

1. General

1.1 Scope of Application

These guidelines provide general requirements for the design of *i)* segmental linings made of reinforced concrete, and *ii)* the secondary lining of shield tunnel constructed in very soft ground such as alluvial or diluvial layers. They can be applied to segmental linings of rock tunnels which are excavated in earth or soft rock by Tunnel Boring Machine (TBM). The physical characteristics of soft ground are as follows:

$N \leq 50$

$E = 2.5 \times N \leq 125$ MN/m^2

$q_u = N/80 \leq 0.6$ MN/m^2 Eq.1.1.1

where N: N value given by standard penetration test,
E: E elastic modulus of soil, and
q_u: Unconfined compressive strength of soil.

1.2 Design Principle

It is a design principle to examine the safety of lining for a shield tunnel for its purpose of usage. The calculation processes—including the prerequisite of design, the assumption and the conception of design, and the design lifetime—should be expressed in the report in which the tunnel lining is examined in terms of its safety.

1.3 Definition of Terms

The following terms are defined for general use in this recommendation.

Segment: Arc-shaped structural member for initial lining of shield tunnel; these guidelines are intended for precast concrete segment (see Fig. II-1).

Segmental lining: Tunnel lining constructed with segments; 1 ring of the lining comprises some pieces of segments (see Fig. II-2).

Segmental lining completed in shield: The segmental lining system in which all of the segments are assembled inside the shield and the lining is completed inside the shield.

Enlarged segmental lining: The segmental lining system in which all segments except the key segment are assembled inside shield and, right after shield, the key segment is inserted and the lining is completed.

Thickness: Thickness of the lining of the cross-section of the tunnel.

Width: Length of segment in longitudinal direction.

Joint: Discontinuity in the lining and contact surface between segments.

Types of joints:

- Plain joints:
  - with connecting elements
    - straight steel bolts
    - curved steel bolts
    - reusable inclined steel bolts
    - plastic or steel connector
  - without connecting elements
  - with guiding bars
- Tongue and groove joints
- Hinge joints:
  - with convex-concave faces
  - with convex-convex faces
  - with centering elements – steel rod link
  - without centering elements
- Pin joints

Circumferential joint: joint between rings.

Radial joint: joint between segments in longitudinal direction.

Bolt for joints: Steel bolt to joint segments.

In actual design and construction, lining makeup, segment shapes, joint and waterproofing details, and tolerances should be selected for effective, reliable and rapid erection, considering the following.

- Method and details of erection and erection equipment.
- Functional requirements of the tunnel, including lifetime and watertightness requirements.
- Ground and groundwater conditions, including seismic conditions.
- Usual construction practice in the location of the tunnel.

1.4 Notation

The following notations are used in these guidelines (see Fig. II-3):

t: Thickness
A: Area
E: Modulus of elasticity
I: Moment of inertia of area
EI: Flexural rigidity
M: Moment
N: Axial force
S: Shearing force
ζ: Ratio between the increment of moment transmitted through the adjacent segment at the joint and M $(1+\zeta)$M is transmitted through the segment. $(1-\zeta)$M is transmitted through the joint.
D: Diameter of lining
Dc: Diameter of centroid
Ro,Rc,Ri: Outer radius, radius of controid and inner radius of the lining, respectively.
$\gamma, \gamma', \gamma_w$, γc: weight of soil, submerged unit weight of soil, unit weight of water and unit weight of concrete, respectively.
H: Overburden
$\gamma w \times Hw$: Groundwater pressure at crown of lining
Po: Surcharge
W: Weight of lining per meter in longitudinal direction
p_g: Dead Load
p_{e1}: Vertical earth pressure at crown of lining
p_{w1}: Vertical water pressure at crown of lining applied to the elastic equation method (See 5.2.)
q_{e1}: Horizontal earth pressure at crown of lining
q_{w1}: Horizontal water pressure at crown of lining applied to the elastic equation method (See 5.2.)
p_{e2}: Vertical earth pressure at bottom of lining
p_{w2}: Vertical water pressure at bottom of lining applied to the elastic equation method (See 5.2.)
q_{e2}: Horizontal earth pressure at bottom of lining.
q_{w2}: Horizontal water pressure at bottom of lining applied to the elastic equation method (see 5.2).
pw: Water pressure.
Fw: Buoyancy.
λ: Coefficient of lateral earth pressure.
k: Coefficient of subgrade reaction.
δ: Displacement of lining.
p_k: Subgrade reaction/ la reaction/ Bettung.
C: Cohesion of soil / La cohesion du sol / Kohaesion vom Boden.
ϕ: Angle of internal friction of soil.
f'_{ck}: Nominal strength of Concrete (Characteristic Compressive Strength of Concrete)
f_y: Yield strength of steel
E_s: Modulus of elasticity of steel

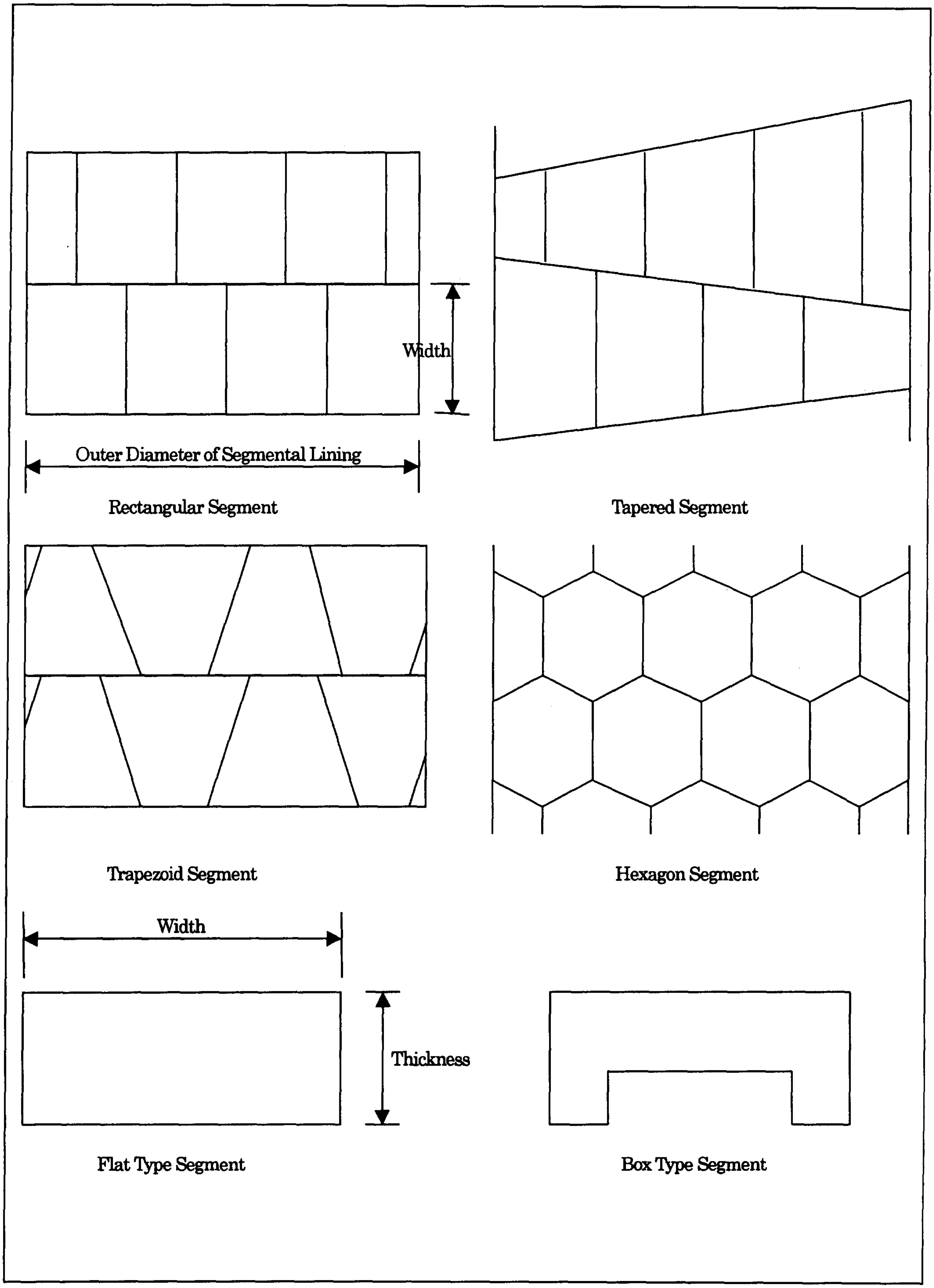


Figure II-1. Types of segments.

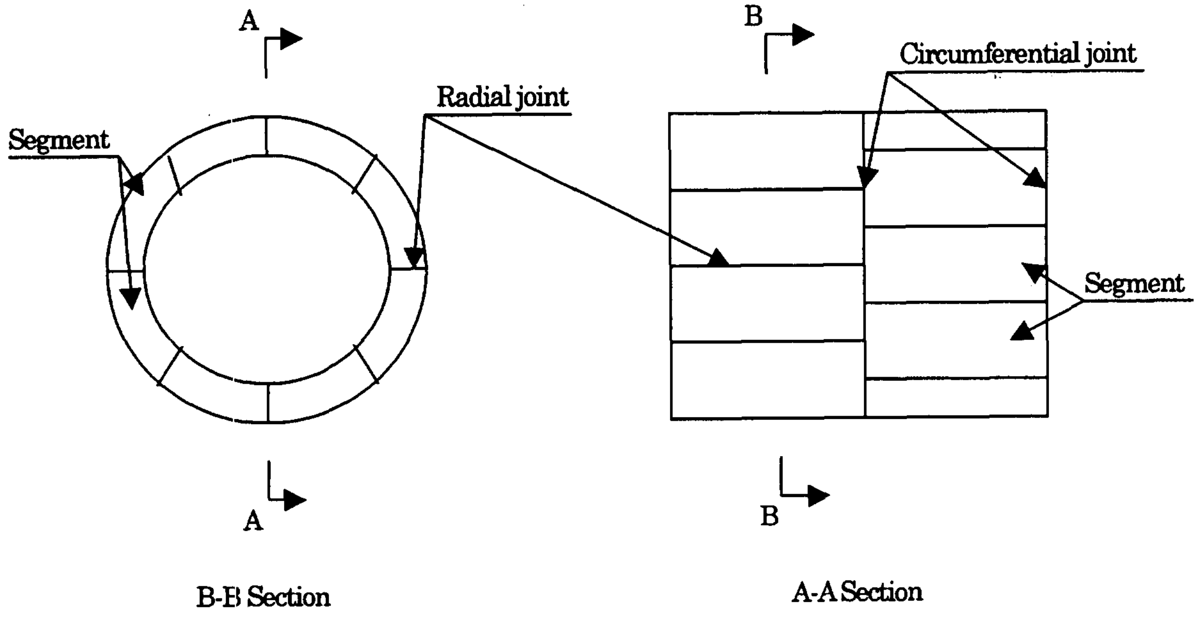


Figure II-2. Segmental lining.

2. Loads

2.1 Kinds of Loads

The following loads should be considered in the design of the lining.

These loads must always be considered:

(1) Ground pressure
(2) Water pressure
(3) Dead load
(4) Surcharge
(5) Subgrade reaction

If necessary, the following loads should be considered:

(6) Loads from inside
(7) Loads during construction stage
(8) Effects of earthquake

Special loads:

(9) Effects of adjacent tunnels
(10) Effects of settlement
(11) Other loads

2.2 Ground Pressure

Figure II-4 shows a section of tunnel and surrounding ground. The ground pressure should be determined in accordance with appropriate analysis. For example, the ground pressure should act radially on the lining or be divided into the vertical ground pressure and the horizontal ground pressure. In the latter case, the vertical ground pressure at the tunnel crown should be a uniform load and, as a rule, should be equal to the overburden pressure, if the designed tunnel is a shallow tunnel. If it is a deep tunnel, the reduced earth pressure can be adopted in accordance with Terzaghi's formula (see Formula 2.2.1), Protodiaconov's formula or other formulae.

The horizontal ground pressure should be the uniformly varying load acting on the centroid of lining from the crown to the bottom. Its magnitude is defined as the vertical earth pressure multiplied by the coefficient of lateral earth pressure (see Fig. II-6(1)). It can be evaluated as the uniform load or the uniformly varying load with the pentagon model (see Fig. II-6(2)). The value of coefficient of lateral earth pressure to be used in the design calculation should be between the value of coefficient of the lateral earth pressure at rest and the value of coefficient of the active lateral earth pressure. The designer should decide this value considering relaxation and construction conditions.

Concerning the unit weight of soil for the calculation of earth pressure, the wet unit weight should be used for soil above the groundwater table and the submerged unit weight should be used for soil below the groundwater table.

$$P_{e1} = P_0 + \Sigma\, \gamma iHi + \Sigma\, \gamma jHj \qquad \text{Eq. 2.2.1}$$

where,

P_0 = Surcharge.
γi = Unit weight of soil of stratum No. i, which is located above the groundwater table.
Hi = Thickness of stratum No. i, which is located above the groundwater tunnel.
γj = Unit weight of soil of Stratum No. j, which is located below the groundwater table.
Hj = Thickness of stratum No. j, which is located below the groundwater table.
$H = \Sigma\, Hi + \Sigma\, Hj$
$h_0 = B_1\, [1 - C/B_1\, \gamma) \}\, \{1 - \exp\, (-k_0 \tan\, (\phi)\, H/B_1)]/K_0 \tan\, (\phi) + P_0 \exp\, \{-K_0 \tan\, (\phi)\, H/B_1\}/\gamma$
$B_1 = R_0 \cot\, (\pi/8 + \phi/4)$
$P_{e1} = \gamma\, h_0$ (if the tunnel is located above the groundwater table)

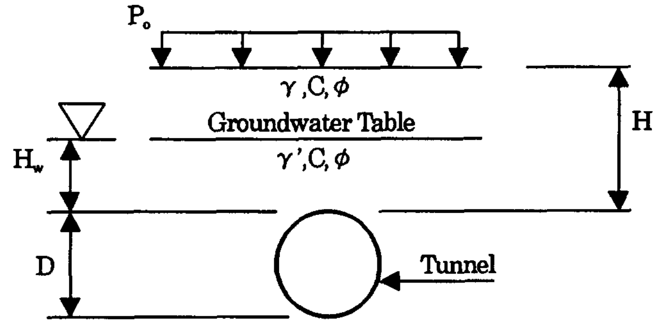


Figure II-3. Examples of notation used in the guidelines.

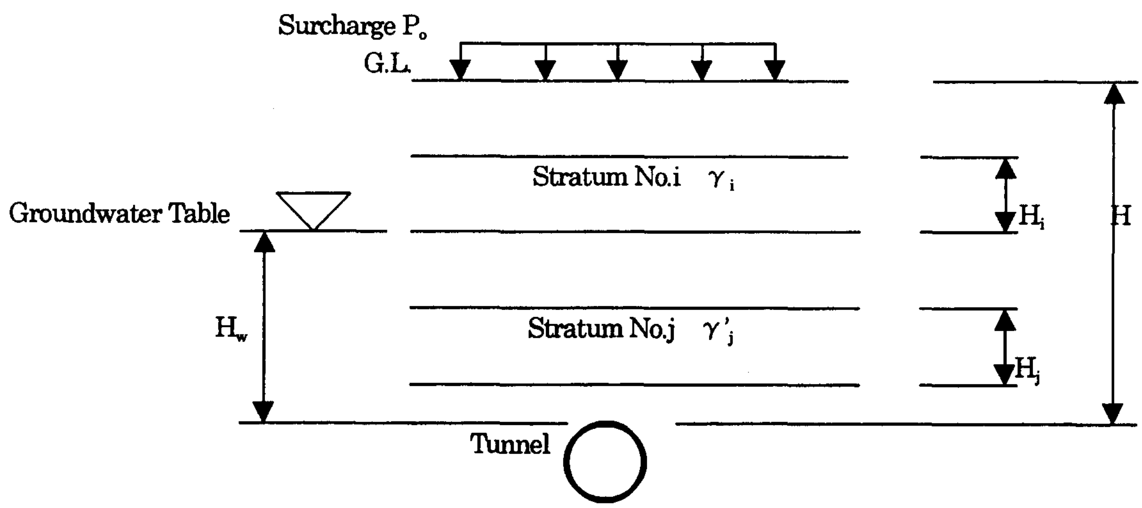


Figure II-4. Section of tunnel and surrounding ground.

$P_{e1} = \gamma' h_0$ (if $h_0 \leq Hw$)
Terzaghi's formula (Formula 2.2.1)

where,

h_0 = Reduced earth pressure divided by unit weight of soil

K_0 = Ratio between lateral earth pressure and vertical earth pressure = 1

2.3 Water Pressure

As a rule, the water pressure acting on the lining should be the hydrostatic pressure (see Fig. II-8). The resultant water pressure acting on the lining is the buoyancy. If the resultant vertical earth pressure at the crown and the dead load is greater than the buoyancy, the difference between them acts as the vertical earth pressure at the bottom (subgrade reaction). If the buoyancy is greater than the resultant vertical earth pressure at the crown and the dead load, the tunnel would float.

2.4 Dead Load

The dead load is the vertical load acting along the centroid of the cross-section of tunnel. It is calculated in accordance with Eq. 2.4.1:

$p_g = W/(2\pi\ Rc)$

$p_g = \gamma_c \times t$ (if the section is rectangular)

Eq. 2.4.1

2.5 Surcharge

The surcharge increases earth pressure acting on the lining. The following act on the lining as the surcharge:

- road traffic load,
- railway trafic load, and
- weight of buildings.

2.6 Subgrade Reaction

When computing the member forces in the lining, we must determine the acting range, the magnitude and the direction of the subgrade reaction. The subgrade reaction is divided into

i) the reaction independent of the displacement of ground, such as p_{e2} (see Fig. II-9), and

ii) the reaction dependent on the displacement of ground.

It is assumed that the latter subgrade reaction is proportional to the displacement of ground, and its factor of proportionality is defined as the coefficient of subgrade reaction. The value of this factor depends on the ground stiffness and the dimension of lining (radius of lining). The subgrade reaction is the product of the coefficient of subgrade reaction and the displacement of the lining which is decided by the ground stiffness and the rigidity of segmental lining. The rigidity of the segmental lining depends on the rigidity of segment, and the number and the type of joint(s).

The bedded rigid frame model can evaluate the subgrade reaction as the spring force (see Fig. II-6, Fig. II-10 and Fig. II-16).

If the member forces are computed using the FEM, plain strain elements simulating ground are evaluated as spring for subgrade reaction.

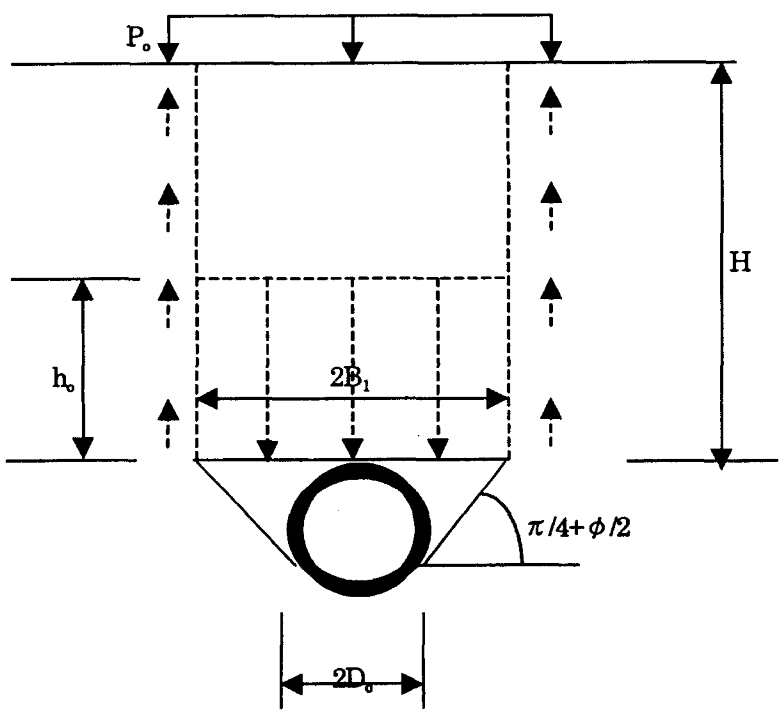
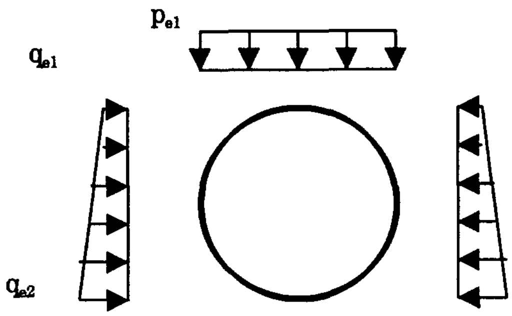


Figure II-5. Reduced earth pressure calculated by Terzaghi's formula.



Where;

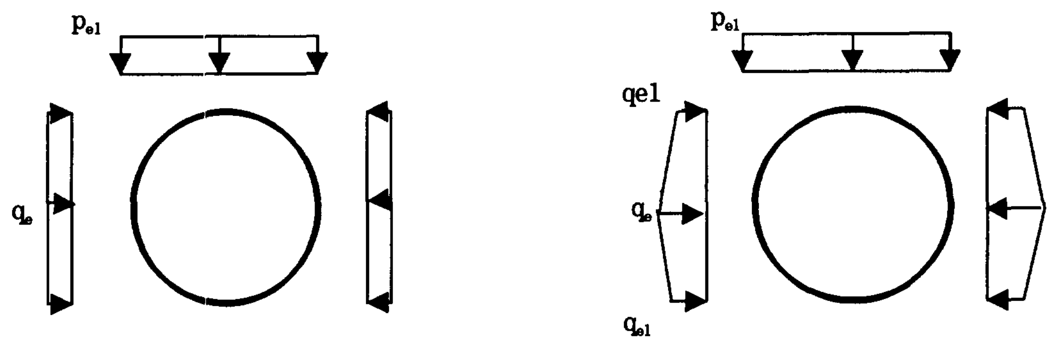
$q_{e1}=\lambda\,(p_{e1}+\gamma\times t/2)$ (if tunnel is located above groundwater table.)

$q_{e1}=\lambda\,(p_{e1}+\gamma'\times t/2)$ (if tunnel is located under groundwater table.)

$q_{e2}=\lambda\,\{p_{e1}+\gamma\times(2R_c-t/2)\}$ (if tunnel is located above groundwater table.)

$q_{e2}=\lambda\,\{p_{e1}+\gamma'\times(2R_c-t/2)\}$ (if tunnel is located under groundwater table.)

Figure II-6 (1). Ground pressure acting on lining (1).



Where,

$qe=(q_{e1}+q_{e2})/2$

Figure II-6 (2). Ground pressure acting on lining (2)

2.7 Loads from Inside

Loading caused by facilities suspended from the ceiling of the tunnel or by inner water pressure should be investigated.

2.8 Loads during Construction Stage

The following loads act on the lining during the construction stage.

1. *Thrust force of shield jacks.* When segments are produced, the strength of the segment against the thrust force of shield jacks should be tested. For the analysis of influence of shield jack forces to segments, the designer should examine shear and bending forces resulting from credible eccentricity, including cases of placement at the limit of tolerance.
2. *Loads during transportation and handling of segments.*
3. *Pressure of backfill grouting.*
4. *Load by operation of erector.*
5. *Others*, e.g. dead load of backup carriages, jack force of segment reformer, torque of cutterhead.

2.9 Effects of Earthquake

The static analysis, such as the seismic deformation method, the seismic coefficient method, or the dynamic analysis, should be used for the seismic design. The seismic deformation method is usually adopted to investigate the effect of earthquake on tunnels. Details should be presented separately from these guidelines.

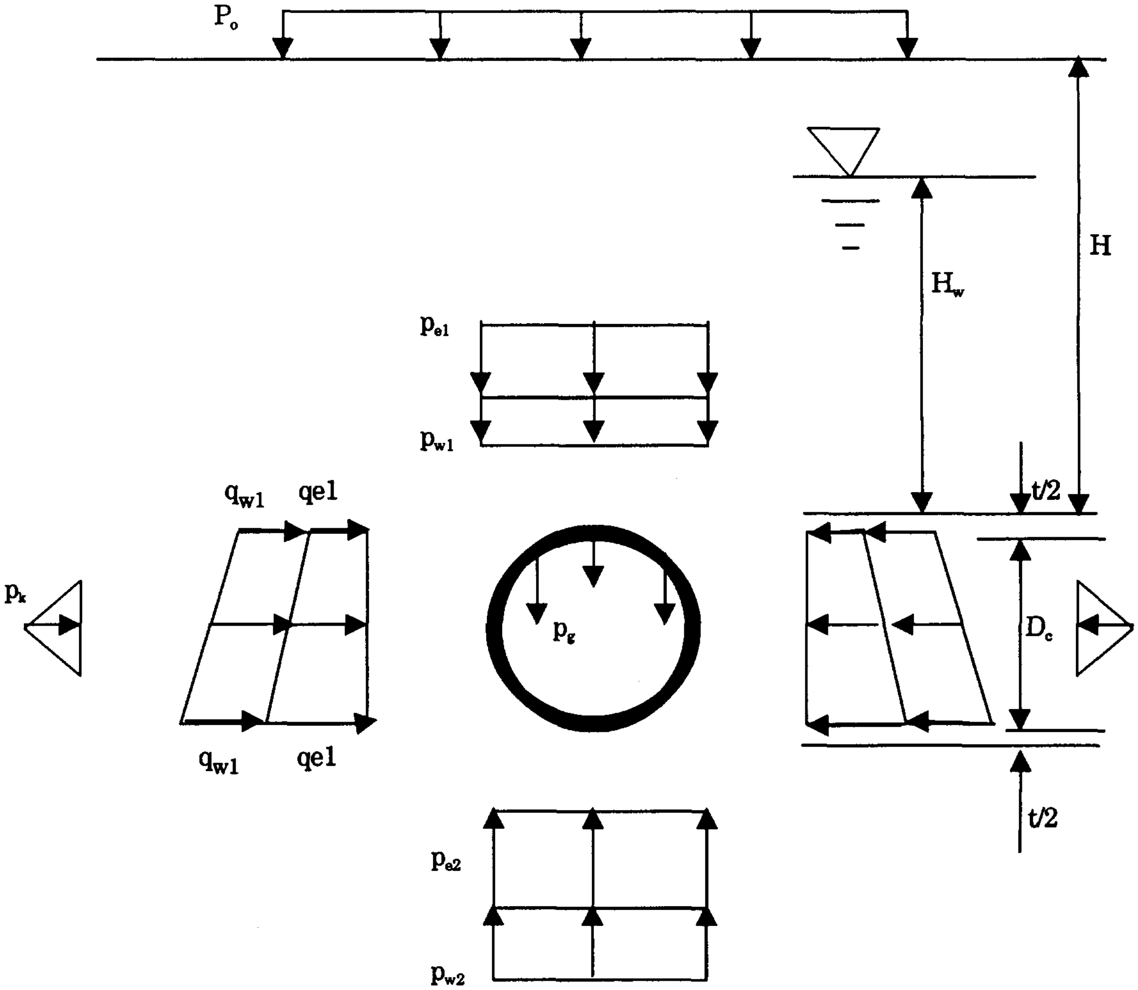
2.10 Other Loads

If necessary, the effect of adjacent tunnels and/or the effect of unequal settlement should be investigated.

3. Materials

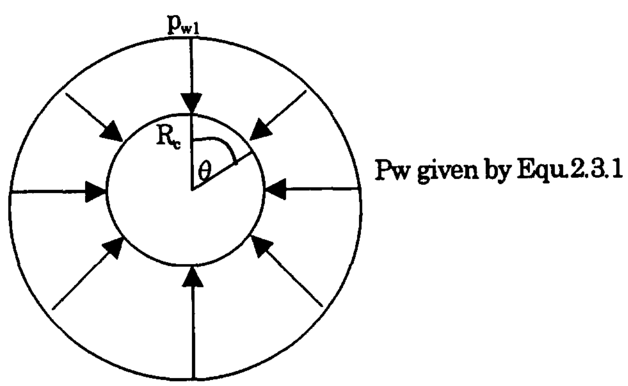
These guidelines are intended for the reinforced concrete segment as the material of the initial lining and for cast-in-place concrete as the material of the secondary lining. The Japan Industrial Standard (JIS), Deutsche Industrie-Norm (DIN) and American Concrete Institute (ACI) Standard specify the test methods of materials.

There may not be a cast-in-place inner lining. If the outer segmental lining is designed and constructed to meet lifetime tunnel lining demands, then a one-pass lining is certainly permitted.



$p_{e2}=p_{e1}+p_{w1}+\pi p_g-p_{w2}$

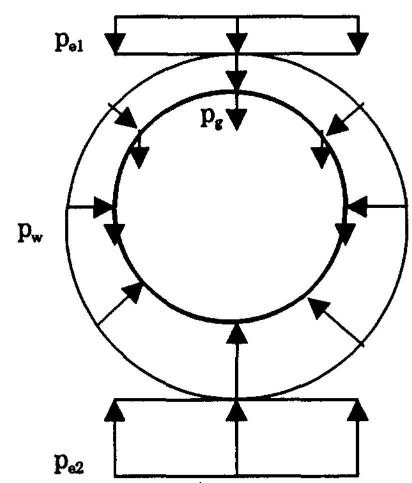
Figure II-7. Load condition of elastic equation method.



$P_{w1} = \gamma_w H_w$ (water pressure at tunnel crown)

$P_w = P_{w1} + \gamma_w Rc (1 - \cos\theta)$ Eq. 2.3.1

Figure II-8. Hydrostatic pressure.



$2rp_{e2} + \pi r^2\gamma_w = 2rp_{e1} + 2\pi rpg$

$p_{e2} = p_{e1} + \pi pg - \pi r \gamma_w/2$

Figure II-9. Subgrade reaction independent of the displacement of ground (p_{e2}).

3.1 Modulus of Elasticity

Table II-1 shows the moduli of elasticity of concrete and steel, as a reference.

3.2 Stress-Strain Curve

Figures II-11 and II-12 show the stress-strain curves of concrete and steel, respectively.

4. Safety Factors

The safety factors should be based on the ground loading and should be defined in accordance with the structural requirements and codes, for example, the national standard specification for design and construction of concrete structures, for each project. Construction procedure and performance should be linked with the safety factors. For their application to the design computation, refer to Section 5.3 below, "How to Check the Safety of the Section". If the tunnel is designed as a temporary structure, the safety factors can be modified.

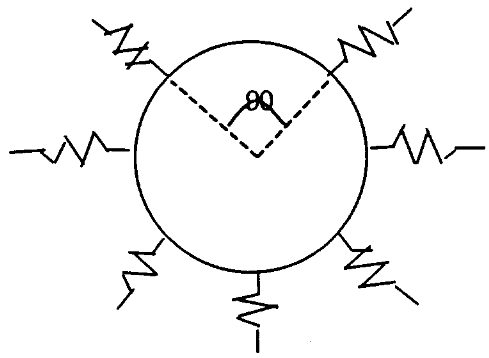


Figure II-10. Model of subgrade reaction.

5. Structural Calculation

SI units should be used in the structural calculation of the lining.

5.1 Design Principles

The design calculation of the cross-section of tunnel should be done for the following critical sections (see Fig. II-13).

1. Section with the deepest overburden.
2. Section with the shallowest overburden.
3. Section with the highest groundwater table.
4. Section with the lowest groundwater table.
5. Section with large surcharge.
6. Section with eccentric loads.
7. Section with unlevel surface.
8. Section with adjacent tunnel at present or planned one in the future.

5.2 Computation of Member Forces

The Member Forces (M,N,S) are calculated using various structural models.

5.2.1 Model for computation

The member forces should be computed by using the following methods (refer to Fig. II-19).

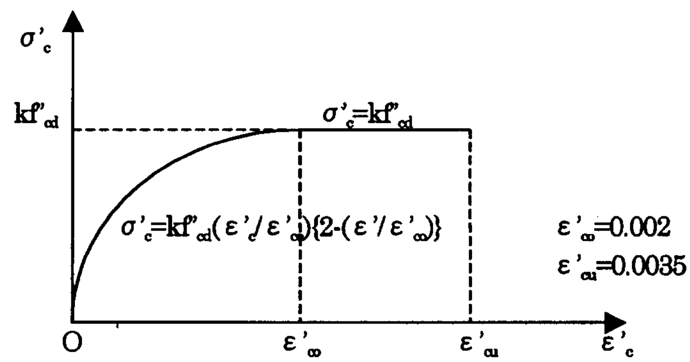


Figure II-11. Stress-strain curve of concrete.

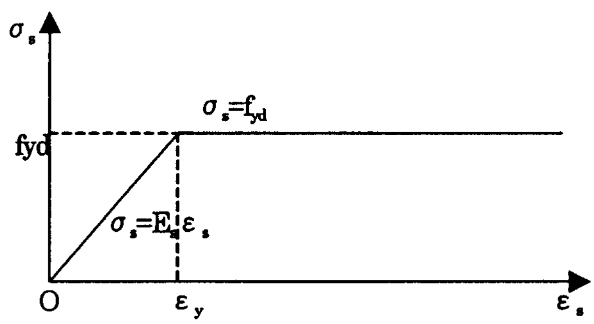


Figure II-12. Stress-strain curve of steel.

Table II-1. Modulus of elasticity of concrete and steel.

Nominal strength f_{ck} (MN/m²)	18	24	30	40	50	60
Modulus of elasticity of concrete E_c (× 10⁴ MN/m²)	2.2	2.5	2.8	3.1	3.1	3.5
Modulus of elasticity of steel	E_s = 210,000 (MN/m²)					

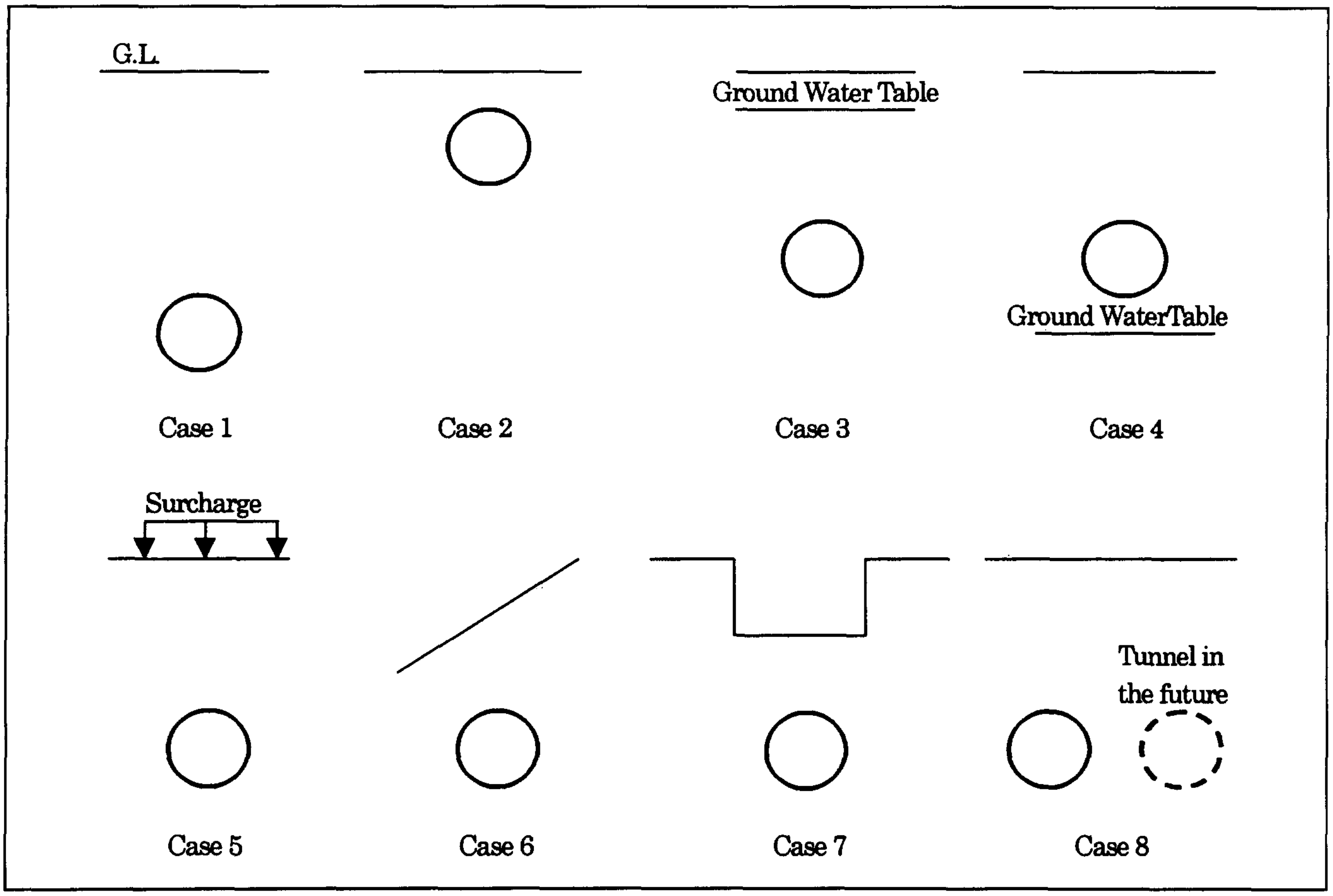
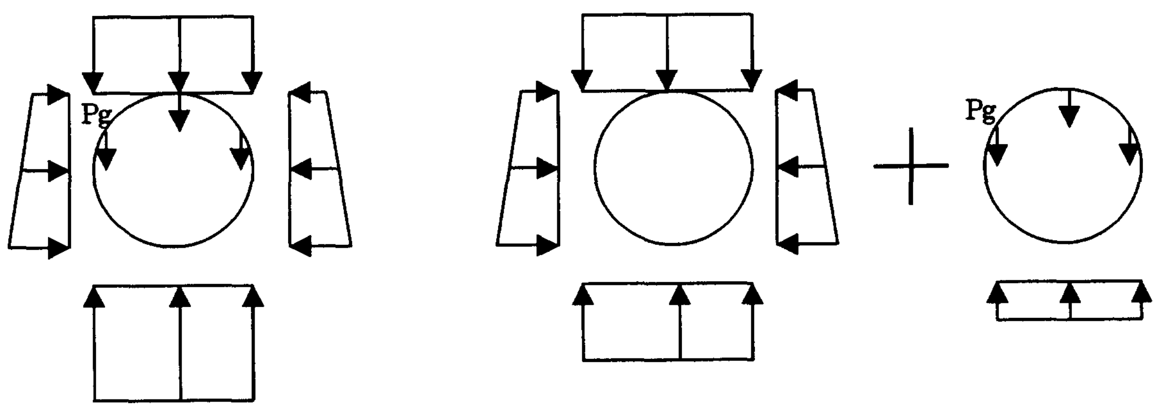


Figure II-13. Critical sections to be checked by the designer of the lining.



Model (a) Model (b)

The subgrade reaction against displacement due to dead load can be evaluated in Model (a) and not evaluated in Model (b).

Figure II-14. Models for bedded frame method to calculate member forces.

1. Bedded frame model method (see Figs. II-14, II-15, and II-16).
2. Finite Element Method (FEM) (Fig. II-17).
3. Elastic equation method (see Fig. II-18 and Table II-2).
4. Schultze and Duddeck Model.
5. Muir Wood Model.

The bedded frame model method is a method to compute member forces with a matrix method using a computer because this model is mutiple statically indeterminate. This method can evaluate the following conditions:

1) nonuniformly varying load due to change of soil condition (see Fig. II-15(b)).
2) eccentric loads (see Fig. II-15(c).).
3) hydrostatic pressure (see Section 2.3, "Water Pressure").
4) spring force to simulate subgrade reaction (see "2.6 Subgrade Reaction").

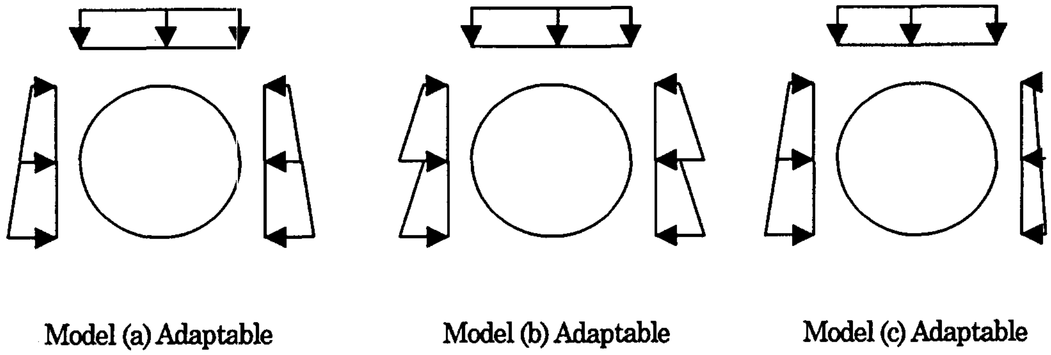
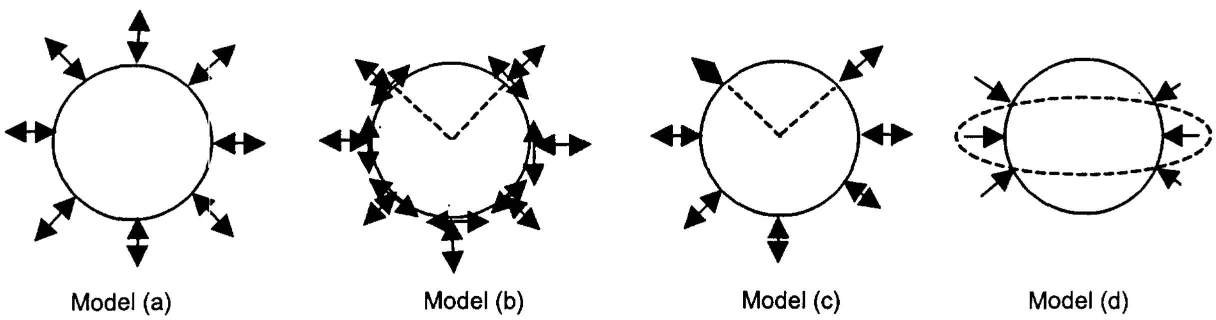


Figure II-15. Adaptable loading models for bedded frame model.



Model	Range of Bedding	Direction of Bedding	Bedding Compression/Tension
a	Full round	Normal	Compression and tension
b	Without crown	Normal and tangential	Compression and tension
c	Without crown	Normal	Compression and tension
d	Dependent on displacement	Normal	Compression only

Figure II-16. Range and direction of subgrade reaction of calculation model of bedded frame method.

5) effect of joint by simulating joints as hinges or rotation springs (semi-hinge) (see Section 5.2.2, "Evaluation of Joints").

If the subgrade reaction against displacement due to dead load cannot be expected, the member forces caused by dead load must be independently calculated and superposed with member forces caused by the other loads. In this case, the member forces caused by dead load can be computed by the elastic equation method.

This method can adopt not only the subgrade reaction in normal direction, but also the subgrade reaction in the tangential direction. The options on the range of the subgrade reaction are as follows (and see Figure II-16):

1. Full round-bedded model.
2. Bedded model without subgrade reaction at crown.
3. Full round-non-tension bedded model.

The FEM is based on the theory of continuous body and has been adopted with the developement of computer. In the FEM, Young's modulus and the Poisson's ratio of soil must be parameters. In design by the FEM, the segmental lining is evaluated as a beam element. The FEM can compute not only the member forces of tunnel lining, but also the displacement and stress-strain state of the surrounding ground and the influence of tunnel construction on overlying or adjacent structures.

The FEM model can reproduce the behavior of interaction of lining and massive ground realistically, with the following merits:

1. The behavior of massive ground can be evaluated in consideration of the initial state of stress of ground, the parameters of ground such as unit weight of soil, Young modulus and Poisson's ratio, the shape and size of tunnel section, and the execution method, including its procedure.

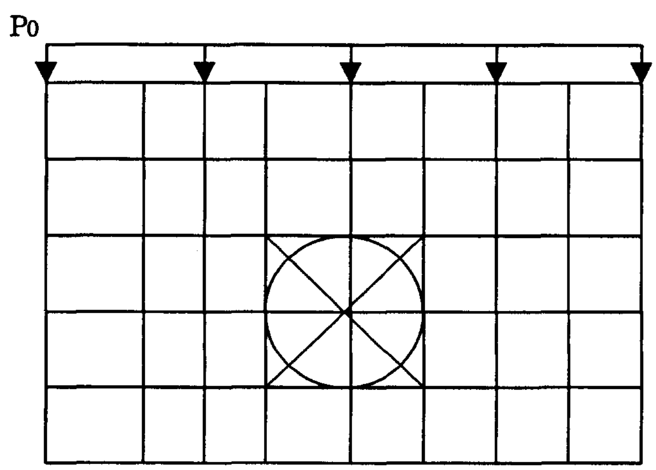
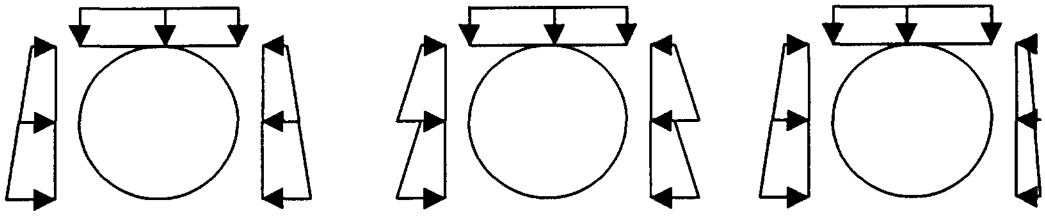


Figure II-17. FEM mesh layout.



Model (a) Adaptable Model (b) Unadaptable Model (c) Unadaptable

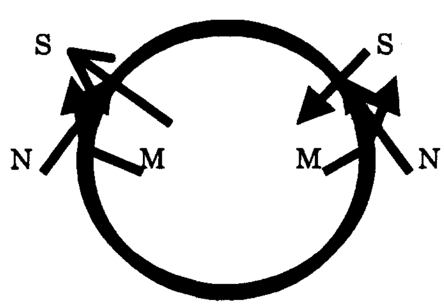
Figure II-18. Loading models for elastic equation method to calculate member forces.

Table II-2. Elastic equations to compute member forces.

Load	Moment (M) (@R_c^2)	Axial Force (N) (@R_c)	Shear Force (S) (R_c)
Uniform load in vertical direction ($P=p_{e1}+p_{w1}$)	(1-2S2)@P/4	S2@P	-SC@P
Uniform load in lateral direction ($Q=q_{e1}+q_{w1}$)	(1-2C2)@Q/4	C2@Q	-SC@Q
Triangular lyvarying load in lateral direction ($Q'=q_{e2}+q_{w2}$) (Q-Q')	(6-3C-12C2+4C3)@(Q-Q')/48	(C+8C2-4C3)@(Q-Q')/16	(S+8SC-4SC2)@(Q-Q')/16
Subgrade reaction in lateral direction (kδ)	$0 \leqq \theta \leqq \pi/4$ (0.2346-0.3536C)@ k δ $\pi/4 \leqq \theta \leqq \pi/2$ (-0.3487+0.5S2+0.2357C3) @ k δ	$0 \leqq \theta \leqq \pi/4$ 0.3536C@ k δ $\pi/4 \leqq \theta \leqq \pi/2$ (-0.7071C+C2+0.7071S2C)@ k δ	$0 \leqq \theta \leqq \pi/4$ 0.3536S@ k δ $\pi/4 \leqq \theta \leqq \pi/2$ (SC-0.7071C2S) @k δ
Dead load (g)	$0 \leqq \theta \leqq \pi/2$ {(3/8) π - θ S-(5/6)C}@g $\pi/2 \leqq \theta \leqq \pi$ {- π /8+(π - θ)S-(5/6)C-(1/2) π S2} @g	$0 \leqq \theta \leqq \pi/2$ {θ S-(1/6)C}@g $\pi/2 \leqq \theta \leqq \pi$ {- π S+ θ S+ π S2-(1/6)C}@g	$0 \leqq \theta \leqq \pi/2$ {θ C-(1/6)S}@g $\pi/2 \leqq \theta \leqq \pi$ {-(π - θ)C+ θ S+ π SC-(1/6)S}@g
Lateral displacement at spring (δ)	δ ={(2P-Q-Q')+ π g}@R_c^4/{24(η EI/h+0.045k R_c^4)		

θ =Angle from crown

S=sin θ S2=sin² θ S3=sin³ θ C=cos θ C2=cos² θ C3=cos³ θ



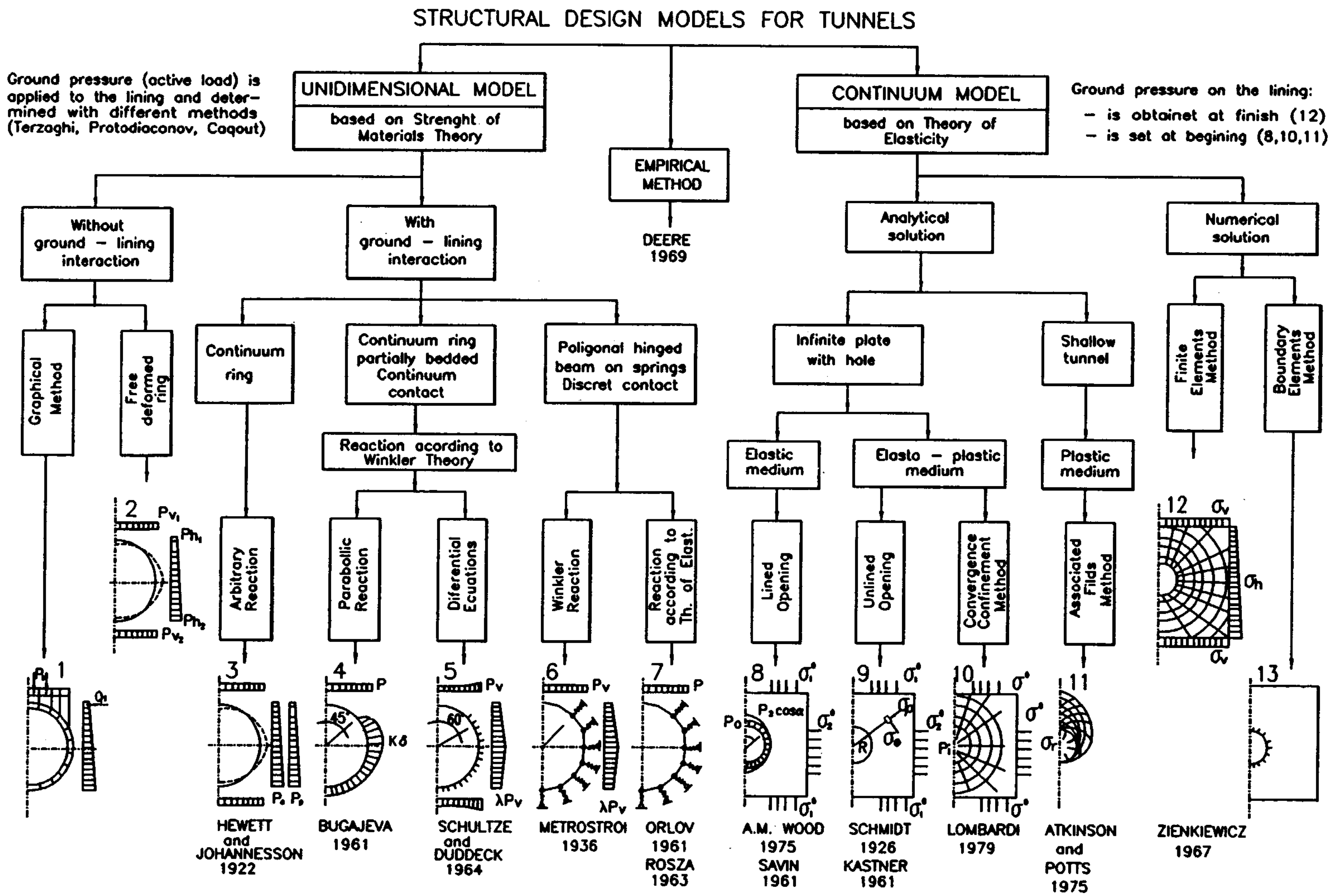


Figure II-19. Structural models to compute member forces (Iftimie 1994).

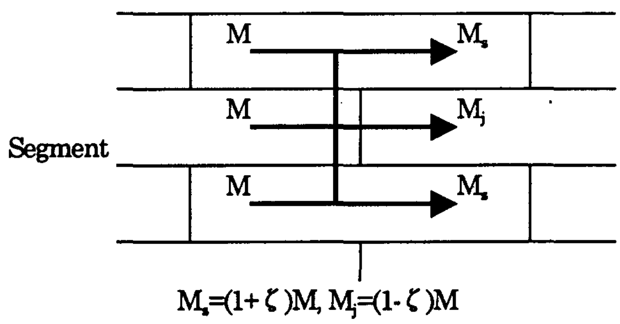


Figure II-20. Distribution of moment at joint.

2. The behavior of the lining which resists the loads depends on the lining structure (number of segments, their configuration and joint type), the characteristics of the backfill grouting and its efficiency, and the loading given by the ground. These factors can be evaluated.
3. The degree of relaxation depends on the ground condition, the construction method (such as the type of shield method), and backfill grouting method, including the size of tail void. These factors can be evaluated.

The elastic equation method is a simple method for calculating member forces without a computer. However, it cannot evaluate the above-mentioned conditions 1) to 5) (see Fig. II-18). In this method, water pressure should be evaluated as the combination of vertical uniform load and horizontally uniformly varying load. The horizontal subgrade reaction should be simplified as a triangularly varying load (see Fig. II-7).

5.2.2 Evaluation of joints

If the segmental lining is jointed with or without bolts, its actual flexural rigidity at the joint is smaller than the flexural rigidity of the segment. (Structurally, the segmental ring can be modeled as a multiple hinged ring or lining having a rigidity between a perfectly uniform rigid ring and a multiple hinged ring.) If the segments are staggered, the moment at the joint is smaller than the moment of the adjacent segment. The actual effect of the joint should be evaluated in the design.

5.3 How to Check the Safety of Section

According to the calculation result of member forces, the safety of the most critical sections must be checked using the limit state design method or the allowable stress design method. These are as follows:

1. Section with the maximum positive moment.
2. Section with the maximum negative moment.
3. Section with the maximum axial force.

The safety of the lining against the thrust force of the shield jacks should be checked.

5.3.1 Limit state design method

The relationship between the design axial capacity and the design flexural capacity of the member cross-sections subjected to axial load and flexural moment is described by a curve, as shown in Figure II-21. Therefore, as a rule, the safety for combined axial load and flexural moment is examined by confirming that the point (M_d, N_d) is located inside of the (M_{ud}, N'_{ud}) curve, that is, at the side of the origin, as shown in Figure II-21. (M_{ud}, N'_{ud}) are calculated by Eq. 5.3.1 and Eq. 5.3.2, respectively. Concerning the stress-strain curve of concrete and steel, refer to Section 3.4, "Stress-Strain Curve". In Figure II-21 and Eqs. 5.3.1 and 5.3.2, γ_b and γ_s are the safety factors of concrete and steel, respectively.

5.3.2 Allowable stress design method

If the extreme fiber stress of concrete and the stress of reinforcement are not more than their allowable stresses, the segmental lining should be safe against the design loads. (See Eq. 5.3.4 and Eq. 5.3.5.)

$$\sigma_c \le \sigma_{ca} = f_{ck}/F_c \qquad \text{Eq. 5.3.3}$$

$$\sigma_s \le \sigma_{sa} = f_{yd}/F_s \qquad \text{Eq. 5.3.4}$$

where,

- σ_c : Extreme fiber stress of concrete
- σ_{ca} : Allowable stress of concrete
- f_{ck} : Characteristic compressive strength (Nominal strength) of concrete
- F_c : Safety factor of concrete
- σ_s : Stress of reinforcement
- σ_{sa} : Allowable stress of reinforcement
- f_{yd} : Yield stress of steel
- F_s : Safety facror of steel

Figure II-22 shows the state of stress and strain distribution.

5.4 Structural Calculation of Joints

At joints, bolts are evaluated as reinforcement. The safety of the joint should be checked by the same method as that used to check the safety of segment, described in Section 5.3, "How to check the safety of section". Because the locations of joints are indefinite before the segments are assembled, the design calculation should be done for the three most critical sections, also as described in Section 5.3.

If bolts are used for erection only and are removed after erection, the joint should transmit a moment limited by the normal force across the joint. Between rings, the force to be transferred from one ring to another is governed by geometric interlock, and microcracks in the segment are propagated by the thrust force of the shield jacks. They influence the longevity of the segmental lining. The quality control for tensile strength of concrete of segment should be considered in order to prevent an increase in microcracks when the segments are produced.

5.5 Check of Safety against Thrust Force of Shield Jacks

The safety of the lining aginst the thrust force of the shield jacks should be checked with the following equation, as a minimum:

$$f_{ck}/F_c \le F_s/A$$

where

- f_{ck}: characteristic compressive strength (nominal strength) of concrete
- F_c: safety factor of concrete
- F_s: total thrust force of shield jacks
- A: area of cross-section area of lining

If more critical conditions are expected by the selection of used jacks, such cases should be checked because the bending moment is caused by it.

Microcracks in the segments are propagated by the thrust force of shield jacks. They influence the longevity of segmental lining. Quality control of the tensile strength of concrete of segments should be considered to prevent an increase in microcracks when segments are produced.

6. Structural Details

6.1 Dimension and Shape of Segment

The fewer the number of pieces that compose one segmental ring, the better the efficiency of manufacturing and

① Ultimate Limit State I

$\varepsilon'_u = \varepsilon'_l = \varepsilon'_{cu}$

$N_{ud} = N_{max}$, $M=0$

② Ultimate Limit State II

$\varepsilon'_u = \varepsilon'_{cu}$, $\varepsilon'_l = 0$, $x = t$

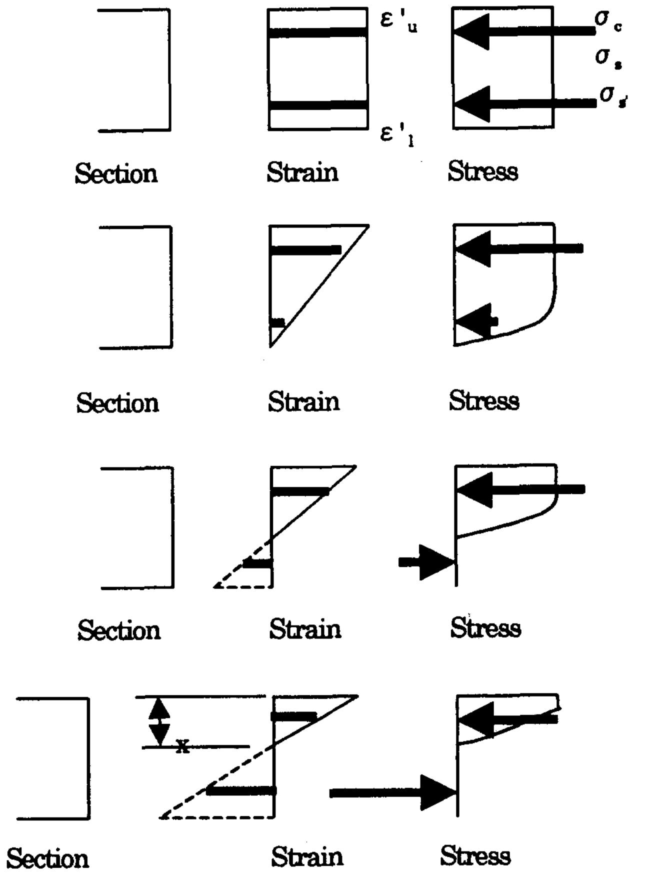
③ Ultimate Limit State III

$\varepsilon'_u = \varepsilon'_{cu}$, $\varepsilon'_l < 0$, $x_o < x < t$

④ Ultimate Limit State IV

$\varepsilon'_u = \varepsilon'_{cu}$, $\varepsilon'_l < 0$, $x = x_o$

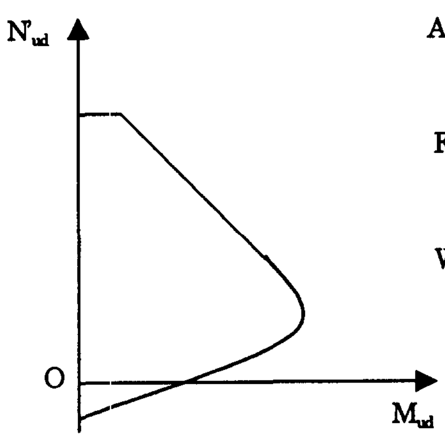
$N_{ud} = 0$



Where, ε'_u=Upper extreme fiber strain

ε'_l=Lower extreme fiber strain

x=Distance between upper extreme fiber and neutral axis



Axial Capacity

$N'_{ud} = \int \sigma(y) b\,dy/\gamma_b + (T_s + T'_s)/\gamma_s$ Equation 5.3.1

Integrate between –h/2 and h/2

Flexural Capacity

$M_{ud} = \int \sigma(y) y b\,dy/\gamma_b + \{T_s(h/2-t) - T'_s(h/2-t')\}/\gamma_s$ Equation 5.3.2

Integrate between –h/2 and h/2

Where, $T_s = A_s \sigma_s$; $T'_s = A'_s \sigma'_s$

Figure II-21. Transition of ultimate limit states and M_{ud}-N'_{ud} diagram.

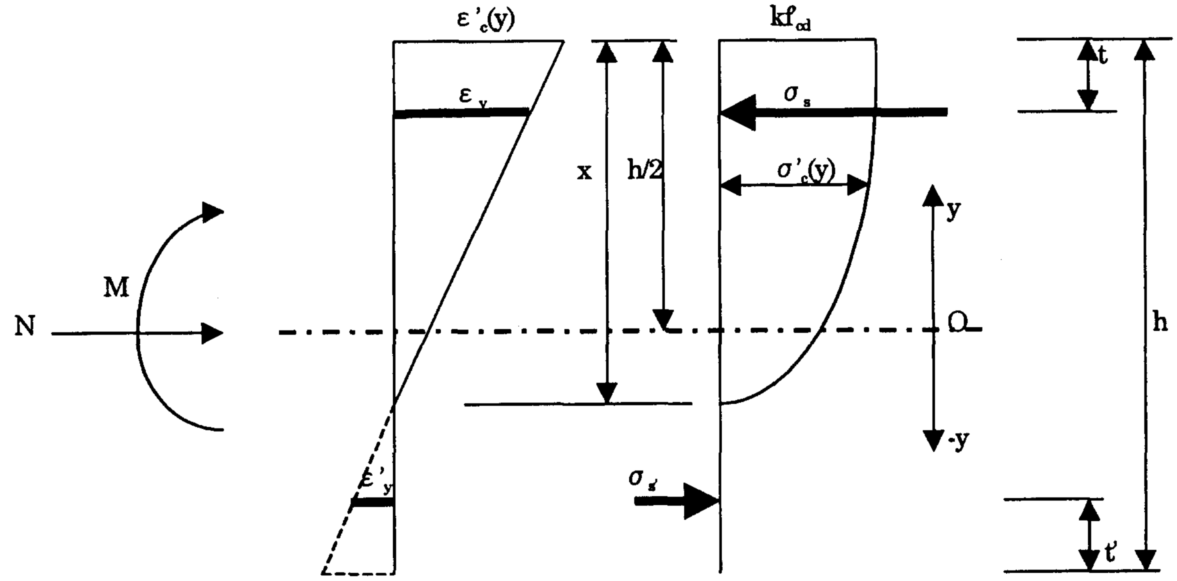


Figure II-22. State of stress and strain distribution.

assembling segments. However, in consideration of the transportation and handling of segments, the length of arc and the weight of one segment should be determined.

6.2 Measures Against Leakage

If allowable leakage discharge is designed, a drainage system can be installed in the tunnel. If not, measures against leakage are necessary. Watertightness requirements should be determined based on the ultimate use and the functional requirements of the finished tunnel. An initial lining that is followed by a cast-in-place inner lining (whether or not a waterproofing membrane is applied) should be sufficiently tight to permit the placement of an inner lining without compromising its quality. Sealing strips should then be applied as necessary. One-pass lining segments below the groundwater table should be furnished with one or two gaskets to seal the tunnel. If only one gasket is used, then provisions should be made to place caulking if excessive leakage should occur (see Fig. II-23).

The sealing method is divided into gasket sealing and paint sealing; the former method is usually adopted. In gasket sealing, the gasket is stuck onto the surface of the joint of the segment. Materials used in the manufacture of gaskets are butyl non-sulfide rubber, deformation butyl rubber, solid rubber, special synthesis rubber, and/or water-expansive material. The water-expansive gasket is a compound of polymer that reacts with water and natural rubber or urethane. If the tunnel is excavated in ground with high ground-water pressure, a two-line gasket should be stuck onto the joints of the segments. In some cases, butyl rubber is not sufficiently resilient to provide an adequate seal under significant external water pressure. In this case, it can be used as sealing strips in an initial segmental lining, which is followed by an inner lining.

In the caulking method, the groove that is made on the inside surface of the segment is filled with caulking materials. The main chemicals used in caulking are epoxy resin, thiokol and urea resin. The caulking should be executed after rebolting of the segment, cleanup of the groove and painting of primer.

If leakage cannot be stopped by gasket sealing and the caulking, urethane injection may be effective. In such a case,urethane is injected through holes made in the segment; the urethane then reacts with groundwater and expands to protect against water invasion.

If the quality of the selected waterproofing system is not proven through previous tests or construction records, the system should be tested in the laboratory under the ex-

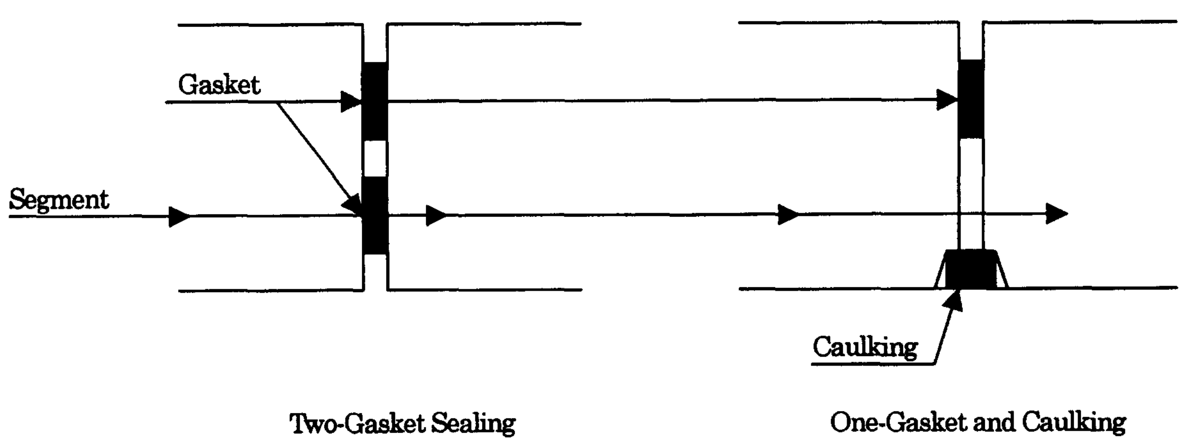
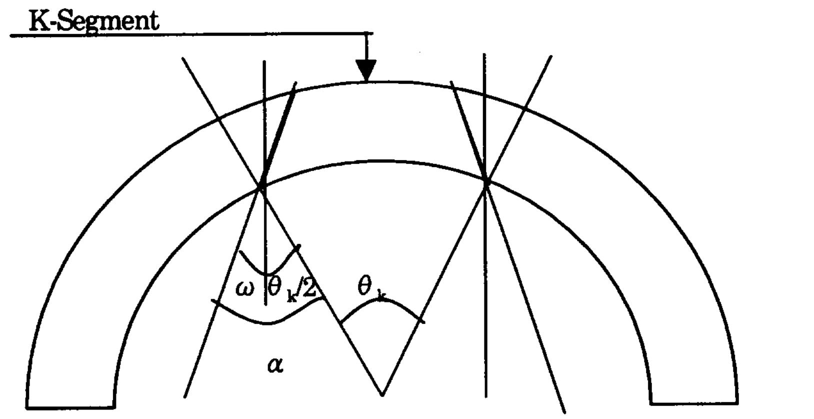


Figure II-23. Gasket sealing and caulking.

pected maximum pressure (with a suitable safety factor) and with joint geometry incorporating the maximum permitted out-of-tolerance placement of the segment at the joint. Where groundwater is aggressive to components of the lining or components installed in the tunnel, full waterproofing should be applied, including the use of waterproof concrete or external waterproofing of segments, or both. (For example, salt groundwater or groundwater high chloride or sulphate content is aggressive to these components.)



$\alpha = \theta_k/2 + \omega$ *(both side-tapered K-segment)* Eq. 6.6.1

$\alpha = \theta_k + \omega$ *(one side-tapered K-segment)*

where α = *angle of joint of K-segment*

θ_k = *central angle of K-segment*

ω = *spare angle to insert K-segment (usually,* $2° \le \omega \le 5°$*)*

Figure II-24. Angle of joint of K-segment.

6.3 Structural Details to Handle Segments and Grout

When the segments are assembled with an erector, equipment should be provided to handle and hang the segment. The recently developed vacuum-type erector can handle segments without the above-mentioned equipment for hanging a segment.

If backfill grouting is performed through segments, each segment should have a grout hole with an inner diameter of about 50 mm in order to inject grout uniformly. A grout hole can be used for the equipment used to hang the segments.

6.4 Angle of Joint of Key-Segment

The type of K-segment is divided into two types: the K-segment inserted in the radial direction (Kr-Segment), and the K-segment inserted in the longitudinal direction (Kl-segment). If this angle is too large, axial force acting (segment works as a force to slide the joint (see Figs. II-2 and II-25).

The Kl-segment can prevent the influence of axial for because the angle of its joint is very small.

The design of the K-segment, if used, should consid the geometry of the erection system in the shield (and vic versa).

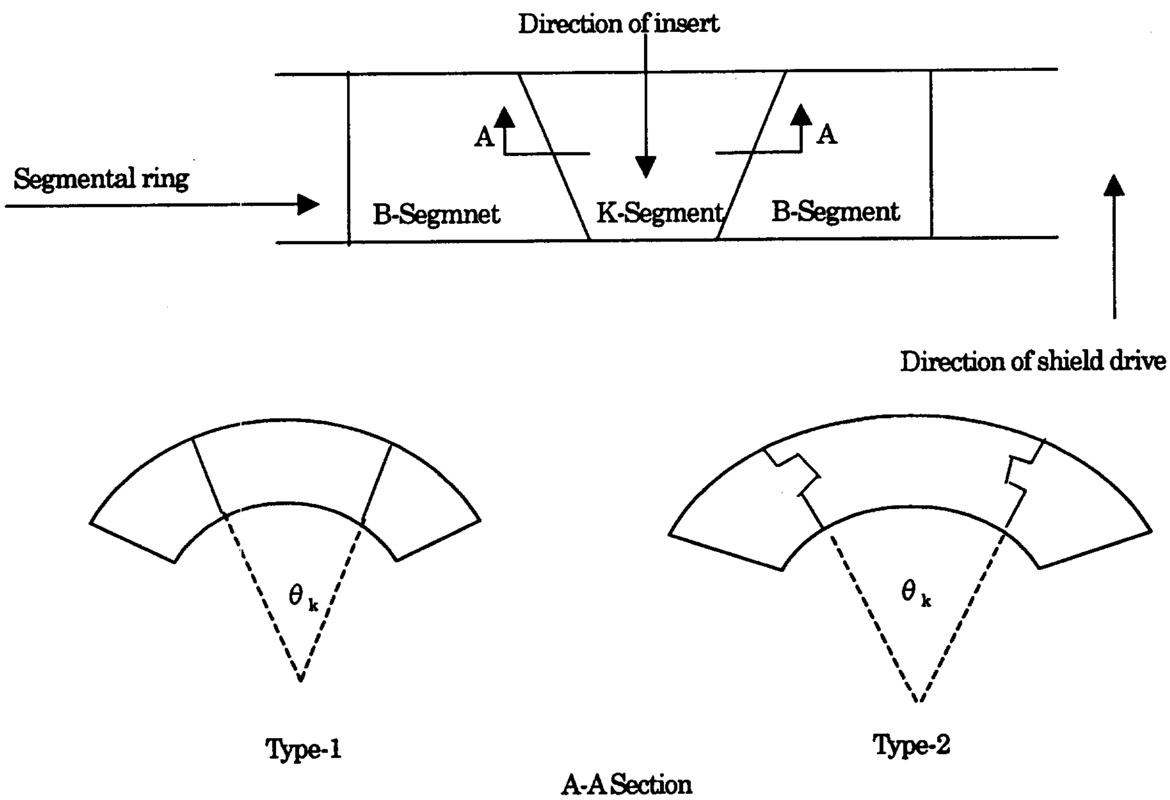


Figure II-25. Joint of K-segment inserted in longitudinal direction.

6.5. Tapered Segments

Tapered segments are used for the construction of curved alignment or the direction control of the shield. The difference between the maximum width and the minimum width can be calculated by using Eq. 6.7.1:

$$\delta = \{(m/n)\,S + S'\}\,D/(R + D/2) \qquad \text{Eq. 6.7.1}$$

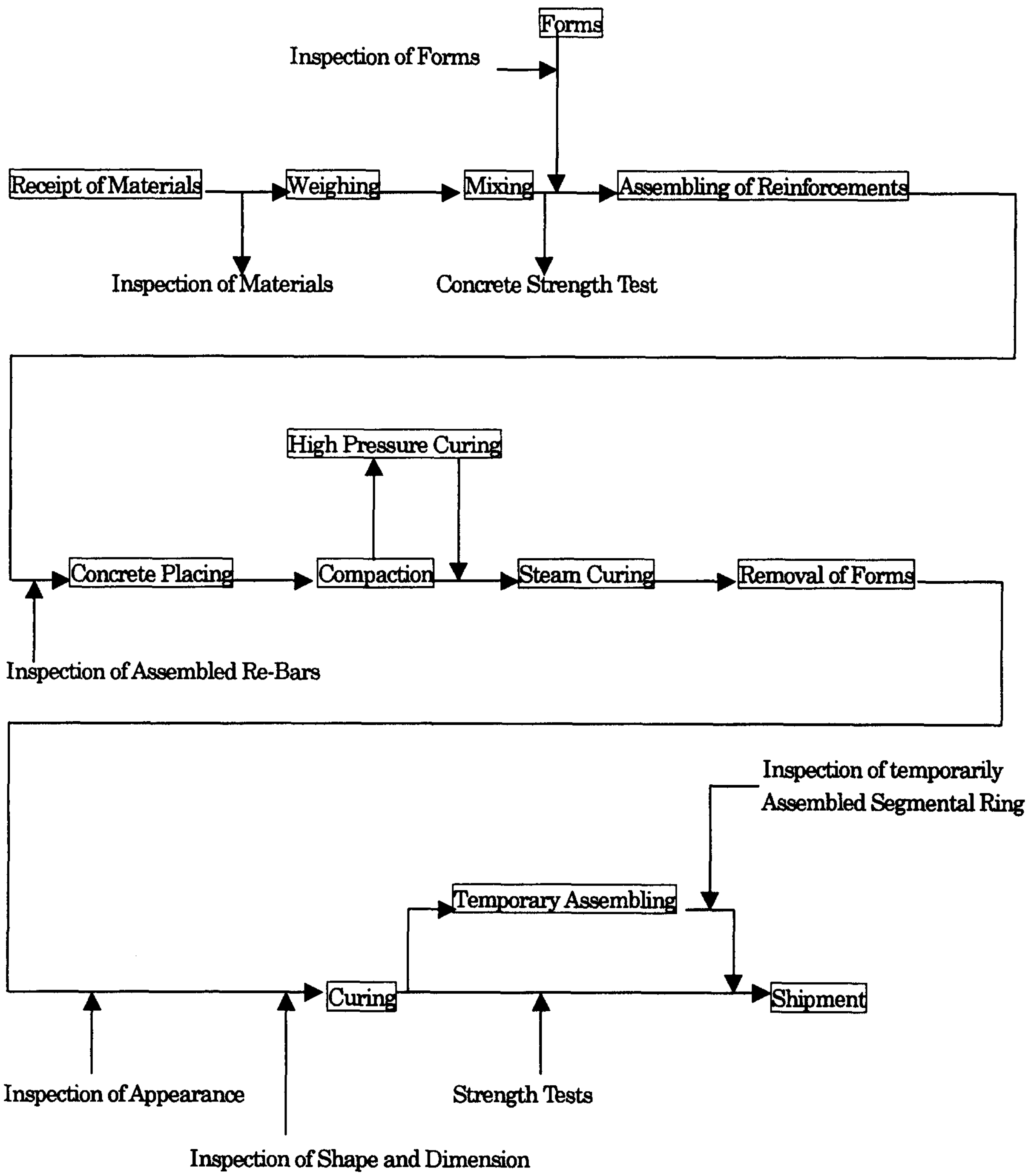
where

- δ = difference between maximum width and minimum width of tapered segmental ring;
- S = width of standard segmental ring;
- S' = maximum width of tapered segmental ring;
- m = number of standard segmental rings in curved section;
- n = number of tapered segmental rings in curved section;
- D = outer diameter of tunnel;
- R = radius of alignment at the center of tunnel.

7. Production of Segments

7.1 Tolerance of Dimension

The errors of dimension of produced segments should not be more than the tolerance. They should be minimize to



Extract from Standard Specification for Design and Construction of Tunnelling, issued by the Japan Society of Civil Engineers

Figure II-26. Sequence of manufacture of segments.

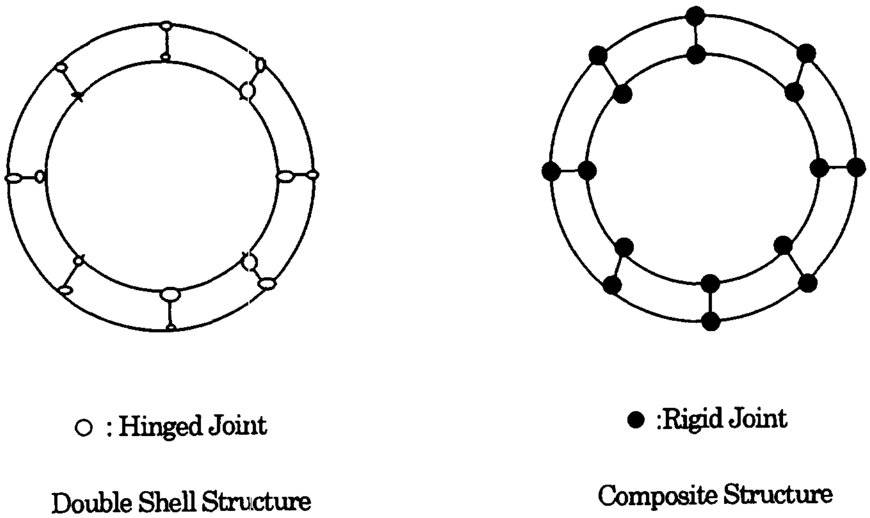


Figure II-27. Bedded frame model.

prevent lealkage and to assemble segments easily and accurately.

7.2 Inspection

The following inspections should be made for for quality control of segments:

1. Inspection of materials.
2. Inspection of appearance.
3. Inspection of shape and dimension.
4. Temporary assembly inspection of temporarily assembled segmental ring.
5. Performance tests (Strength tests).
6. Other tests.

Figure II-26 shows the sequence of segment production.

8. Secondary Lining

8.1 General

The secondary lining is constructed with cast-in-place concrete. It is divided into the non-structural member and the structural member. The former is executed to reinforce segments, to prevent corrosion and vibration, to improve the appearance of the lining, and to correct alignment. In the latter case, the secondary lining is constructed as a structural member combined with the segmental lining.

8.2 Thickness

The thickness of the secondary lining as a non-structural member usually ranges from 15 cm to 30 cm. The thickness of the secondary lining as a structural member is decided in accordance with the result of design calculation.

8.3 Computation of Member Forces

If the secondary lining is constructed as a structural member, member forces of secondary lining should be computed by using loads that act on the lining after the completion of secondary lining. In this case, the tunnel lining combined by the segmental lining and the secondary lining is divided into the double-shell structure and the composite structure in accordance with the smoothness of border between both linings. In the case of the double-shell structure, only axial force must be transmitted through the border of both linings; shear force need not be transmitted through it. In the case of the composite structure, both the axial force and the shear force must be transmitted through the border of both linings by dowelledly jointing both linings or by making the surface of the border uneven. As a rule, a tunnel lining combined by a segmental lining and a secondary lining should be treated as a double-shell structure.

If the secondary lining is a non-structural member, design calculation for it can be omitted; however, for safety, the calculation might be made by using dead load as the load condition. If a waterproofing membrane without a drainage system is placed before casting the secondary lining, the secondary lining should be designed for the full water pressure, as maximum.

Assuming that the tunnel lining combined by segmental lining and secondary lining is a double-shell structure, the member forces of the secondary lining should be computed by any rational method that properly considers the interaction between the initial lining and the secondary lining and that is compatible with the design of the initial lining. Examples of methods for computing member forces are discussed below.

8.3.1 Bedded frame model method

When the member forces of the secondary lining are computed by the bedded frame model, the double-ring frame model should be used. In this model, the outer ring simulates the segmental lining, and the inner ring simulates the secondary lining. Figure II-27 shows how to compute member forces of secondary lining with the bedded frame model.

8.3.2 Elastic equation method

This method assumes that loads acting on lining are sustained by the segmental lining and secondary lining in proportion to the magnitude of the flexural rigidity. Eq. 9.3.1 calculates the ratio between loads sustained by the secondary lining and the total loads. When member forces of the secondary lining are calculated, the loads multiplied by μ replace the corresponding loads, and $EI_1 + EI_2$ replace EI in Table II-2.

$$\mu = (E_2 I_2/Rc_2^4) / (E_1 I_1/Rc_1^4 + E_2 I_2/Rc_2^4)$$

8.4. How to Check the Safety of the Section

The safety of the section should be checked by using the limit state design method or the allowable stress design method, which are the same methods as are used for checking the segmental lining.

Part III: Design Example 1[1]

1. Function of Tunnel

The planned tunnel is to be used as a sewer tunnel.

2. Design Conditions

2.1 Dimensions of segment

Type of segment: RC, Flat type
Diameter of segmental lining: Do = 3350 mm
Radius of centroid of segmental lining: Rc = 1612.5 mm
Width of segment: b = 1000 mm
Thickness of segment: t = 125 mm

2.2 Ground conditions

Overburden: H=15.0 m
Groundwater table: G.L.-2.0 m Hw=15.0-2.0=13.0 m
N Value: N=30
Unit weight of soil: γ=18 kN/m^3
Submerged unit weight of soil: γ'=8 kN/m^3
Angle of internal friction of soil: ϕ=32 Degree
Cohesion of soil: C=0 kN/m^2
Coefficient of reaction: k = 20 MN/m^3
Coefficient of lateral earth pressure: λ = 0.5
Surcharge: p_0=10 kN/m^2
Thrust force of shield jacks: T=1000 kN@10 pieces
Soil condition: Sandy
Allowable stresses of materials:
Concrete: Nominal strength f_{ck}=42 MN/m^2
Allowable compressive strength σ_{ca} = 15 MN/m^2
Reinforcement (SD35):
Allowable strength σ_{sa} = 200 MN/m^2
Bolt (Material 8.8):
Allowable strength: σ_{Ba} = 240 MN/m^2

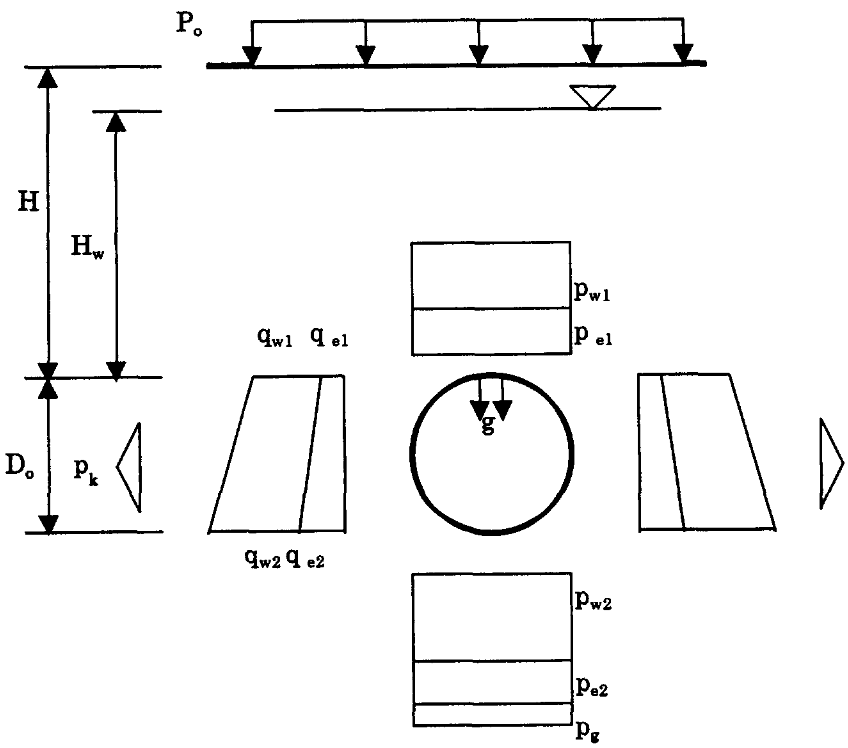


Figure III-1. Load condition for design example 1.

In checking the safety of segmental lining against the thrust force of shield jacks, modified allowable stresses that are 165% of the above-mentioned ones can be adopted because a segmental lining can be evaluated as a temporary structure.

2.3 Design Method

This shield tunnel shall be designed in accordance with the Specification for Design and Construction of Shield Tunnel issued by the Japan Society of Civil Engineers.

- *How to check compute member forces*: Elastic equation method (See Table II-2 in these guidelines).
- *How to check the safety of lining:* Allowable stress design method.

3. Load Conditions

3.1 Computation of Reduced Earth Pressure at Tunnel Crown

The vertical earth pressure at the tunnel crown (p_{e1}) is computed by Terzahgi's Formula.

p_{e1} = MAX ($\gamma' h_0$, 2 γ' Do)
h_0 = 4.581 m (given by Terzahgi's Formula; see formula 2.2.1 in Section 2.2, "Ground pressure of Guidelines") < 2 Do = 6.7 m
p_{e1} = 2 γ' Do = 53.60 kN/m^2

3.2 Computation of Loads

Dead load: g = γc @ t=3.25 kN/m ^{2}
where γc = Unit weight of RC segment = 26 kN/m^3
Reaction of dead load at bottom: $p_g = \pi$ g = 10.21 kN/m^2

Vertical pressure at tunnel crown:
Earth pressure: p_{e1}= 2 γ' Do = 53.60 kN/m^2
Water pressure: $p_{w1} = \gamma_w$Hw = 130.00 kN/m^2
$p_1 = p_{e1} + p_{w1}$=183.60 kN/m^2

Vertical pressure at tunnel bottom:
Water pressure: $p_{w2} = \gamma_w$ (Do + Hw)
= 163.50 kN/m^2
Earth pressure: $p_{e2} = p_{e1} + p_{w1} - p_{w2}$
= 20.10 kN/m^2

Lateral pressure at tunnel crown:
Earth pressure: $q_{e1} = \lambda \gamma'$ (2 Do + t/2)
= 27.05 kN/m^2
Water pressure: $q_{w1} = \gamma_w$ (Hw +t/2)
= 130.63 kN/m^2
$q_1 = q_{e1} + q_{w1}$=157.68 kN/m^2

Lateral pressure at tunnel bottom:
Earth pressure: $q_{e2} = \lambda \gamma'$ (2Do +Do – t/2)
= 39.95 kN/m^2
Water pressure: $q_{w2} = \gamma_w$ (Hw + Do – t/2)
= 162.88 kN/m^2
$q_2 = q_{e2} + q_{w2}$=202.83 kN/m^2

Reaction:
δ = (2p_1 – q_1 – q_2)/{24 (EI+0.0454kRc^4)}
= 0.00016374 m
p_k = kδ =3.27 kN/m^2
where
δ = Displacement of lining at tunnel spring
E = Modulus of elasticity of segment = 33000000 kN/m^2
I = Moment of inertia of area of segment = 0.00016276 m^4/m

Table III-1. Member forces of segmental lining.

θ (deg)	M (KNm/m)	N (kN/m)	Q (kN/m)
0	6.52	278.00	00.00
10	5.96	279.07	-3.93
20	4.39	282.02	-7.05
30	2.12	286.36	-8.77
40	-0.39	291.31	-8.76
50	-2.65	296.05	-7.12
60	-4.29	299.88	-4.36
70	-5.07	302.44	-1.18
80	-4.98	303.78	1.68
90	-4.20	304.29	3.68
100	-3.00	304.25	4.76
110	-1.61	303.88	5.00
120	-0.26	303.59	4.99
130	0.87	303.65	3.46
140	1.68	304.14	2.26
150	2.15	305.01	1.17
160	2.37	305.99	0.43
170	2.43	306.76	0.09
180	2.44	307.05	0.00

k = Coefficient of reaction=20 MN/cm^3

Figure III-1 shows the load condition to compute the member forces by using the elastic equation method.

4. Computation of Member Forces

Table III-1 shows the result of computation of member forces of the segmental lining.

The maximum positive moment occurs at the tunnel crown (Section A) and the maximum negative moment occurs at the spring which is located at 70 degrees from the tunnel crown (Section B). Figure III-2 shows the arrangement of bars in the segment.

5. Check of Safety of Segmental Lining

The safety of the segmental lining should be checked at Section A, Section B and the joint part. Its safety against the thrust forces of shield jacks also should be checked.

5.1 Section A and Section B

Figure III-3 shows the distribution of stress at Section A and Section B. Table 5.1 shows the computation result of the check of the safety of Section A and Section B.

Both Section A and Section B are safe.

5.2 Joint

The resisting moment of the joint shall be not less than 60% of the resisting moment of the segment body.

5.2.1 Resisting moment of segment body (Mr)

x = Depth between compressive extreme fiber and neutral axis when N=0

x = $-n(A_s+A_s')/b+[\{n(A_s+A_s')/b\}2+2b(A_s d+A_s' d')]1/2$

= 3.711 cm (See Figs. III-2 and III-3).

M_{rc}= Resisting moment of segment body when the compressive extreme fiber stress reaches 15 MN/m^2, which is the allowable compressive stress of concrete.

M_{rs}= Resisting moment of segment body when the reinforcement reaches 200 MN/m^2, which is the allowable stress of reinforcement.

M_{rc}= $[bx(d-x/3)/2+nAs'(x-d')(d-d')/x]\ \sigma_{ca}$

= 22.24 kNm/Ring

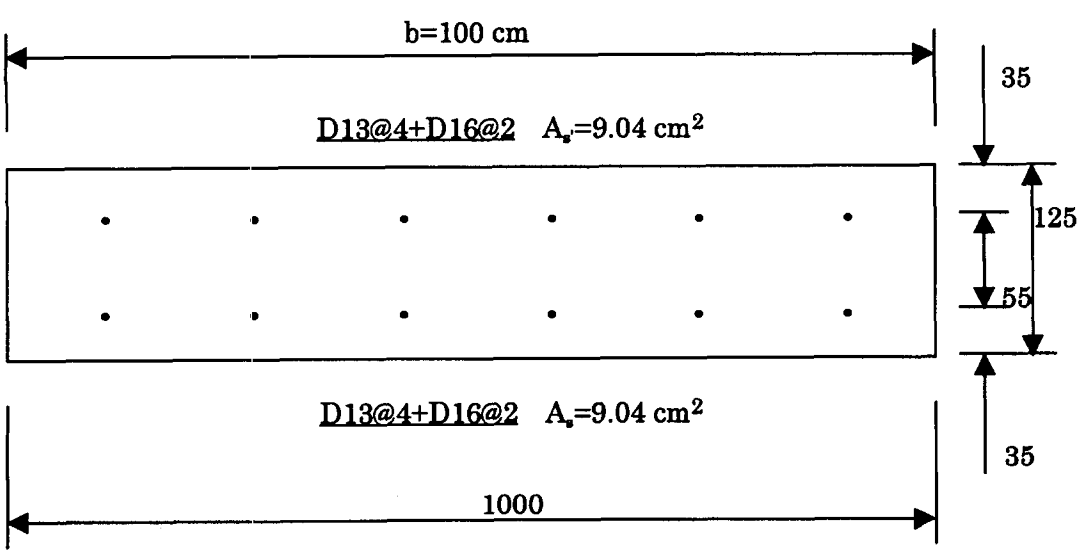


Figure III-2. Section of segment and arrangement of bars.

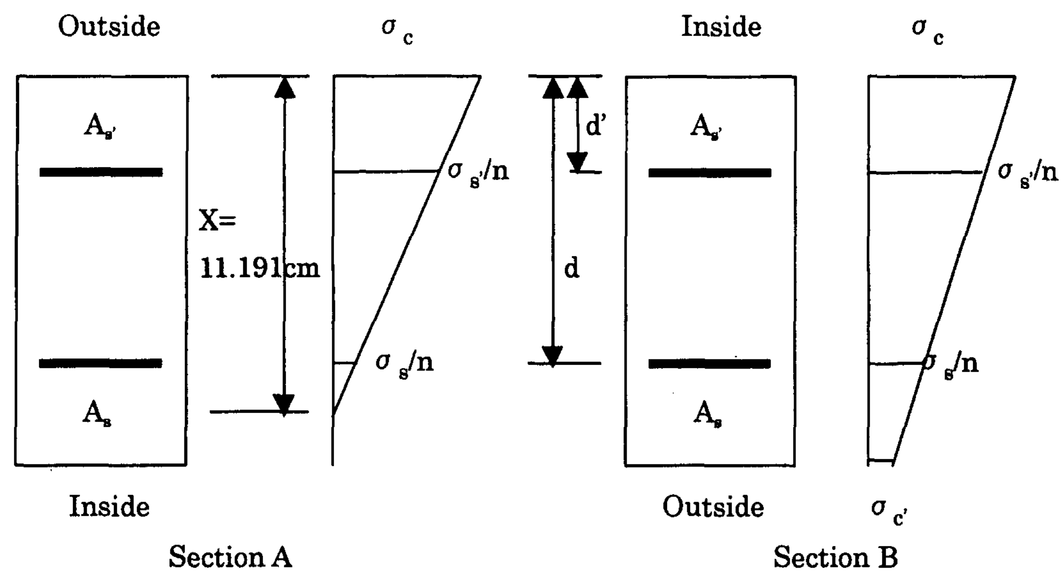


Figure III-3. Distribution of stress of critical sections of segmental lining.

M_{rs} = $[\{bx(d-x/3)/2+nAs'(x-d')(d-d')/x\}]\ x/\{n(d-x)\}\sigma_{ca}$
= 13.87 kNm/Ring
M_r = $\mathrm{Min}(M_{rc}, M_{rs}) = M_{rs}$ =13.87 kNm/Ring

5.2.2 Resisting moment of joint (M_{jr})

x = Depth between compressive extreme fiber and neutral axis when N = 0

x = $nA_B[-1+\{1+2bd/(nA_B)\}^{1/2}]/b$ = 3.011 cm (see Fig. III-4)

M_{jrcv} = Resisting moment of joint when the compressive extreme fiber stress reaches 15 MN/m², which is the allowable compressive stress of concrete.

M_{jrB} = Resisting moment of joint when the reinforcement reaches 240 MN/m², which is the allowable stress of the bolt.

M_{jrc} = $[bx(d-x/3)/2]\ \sigma_{ca}$ =15.80 kNm/Ring

Table III-2. Computation result of the check of the safety of Section A and Section B.

Section	A	B
M (kNm/m)	6.52	-5.07
N (kN/m)	278.000	302.44
σ_c (MN/m²) (Compressive)	4.09	3.72
$\sigma_{c'}$ (MN/m²) (Compressive)	–	0.26
σ_s (MN/m²) Tensile)	12.02	-18.42
$\sigma_{s'}$ (MN/m²) (Compressive)	42.19	41.23

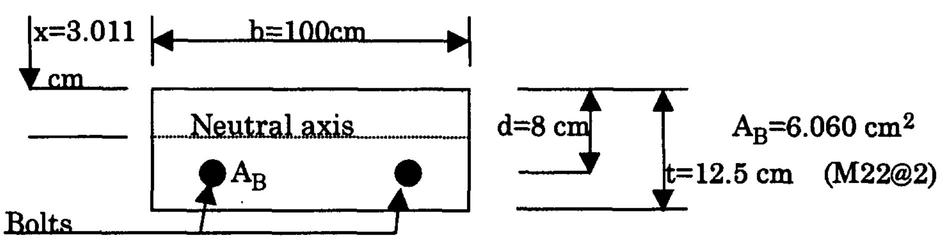


Figure III-4. Section of joint.

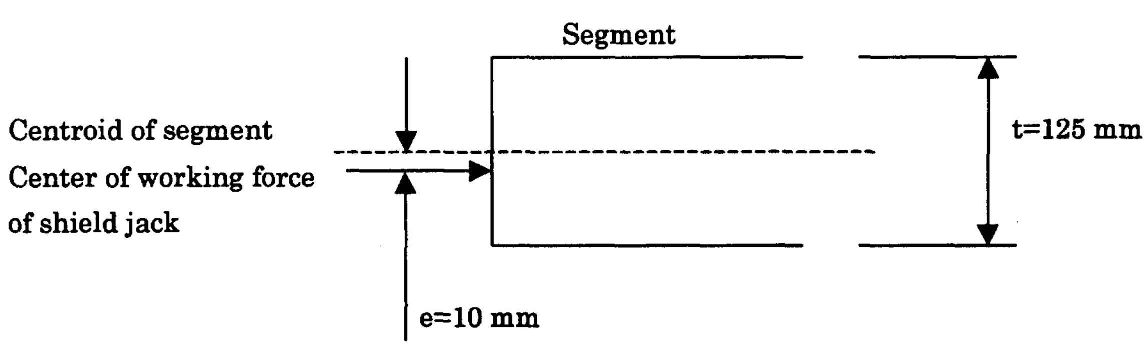


Figure III-5. Segment and thrust force of shield jack.

$M_{rs} = [A_B(d-x/3')\sigma_{Ba} = 10.18$ kNm/Ring
$M_{jr} = \mathrm{Min}(M_{jrc}, M_{jrB}) = M_{rs} = 10.18$ kNm/Ring
$M_{jr}/M_r = 10.18/13.87 = 0.733 > 0.6$
OK

5.3 Check of Safety Against Thrust Forces of Shield Jacks

e = Eccentricity between center of working thrust force by one jack and centroid of segmental lining = 1 cm
l_s = Space between adjacent two jacks = 10 cm
A = Touching area of spreader of one jack on segmental lining = Bt,

where

t = Thickness of segment = 12.5 cm
B = 2π Rc/Nj$-l_s$ = 2π 1.6125/10 - 0.1 = 0.9123 m, where Nj = Number of shield jacks = 10 pieces
A = Bt = 0.1141 m², I = Bt³/12 = 0.00014863 m⁴
σ_c = Maximum compressive stress of concrete of segment
= $P/A + P_e(h/2)/I = 13$ MN/m²
$< \sigma_{ca} = 15 \times 1.65 = 24.75$ MN/m² OK,

where P = Thrust force of one shield jack = 1000 kN (see Fig. III-5)

6. Conclusion

The designed segmental lining is safe against the design loads.

Notes

[1] This design example was prepared by Japan the Tunnelling Association.

Part III: Design Example 2[1]

1. Function of Tunnel

The planned tunnel is to be used as a subway tunnel.

2 Design Condition

2.1 Dimensions of Segment

Type of segment: RC, Flat type
Diameter of segmental lining: Do = 9500 mm
Radius of centroid of segmental lining: Rc = 4550 mm
Width of segment: b = 1200 mm
Thickness of segment: t = 400 mm

2.2 Ground Condition

Overburden: H = 12.3 m
Groundwater table: G.L. + 0.6 m Hw=12.3+0.6=12.9 m
N Value: N = 50
Unit weight of soil: $\gamma = 18$ kN/m³
Submerged unit weight of soil: $\gamma' = 8$ kN/m³
Angle of internal friction of soil: $\phi = 30°$
Cohesion of soil: C = 0 tf/m²
Coefficient of reaction: k = 50 MN/m³
Coefficient of lateral earth pressure: k = 0.4
Surcharge: $p_0 = 39.7$ kN/m²
Soil condition: Sandy
Allowable stresses of materials:
Concrete: Nominal strength $f_{ck}=48$ MN/m²
Allowable compressive strength $\sigma_{ca}=17$ MN/m²
Allowable shear strength $\tau_{ca}=0.55$ MN/m²
Reinforcement (SD35):
Allowable strength: $\sigma_{sa} = 200$ MN/m²
Bolt (Material 8.8):
Allowable tensile strength $\sigma_{Ba} = 240$ MN/m²

2.3 Design Method

This shield tunnel shall be designed in accordance with Specification for Design and Construction of Shield Tunnel

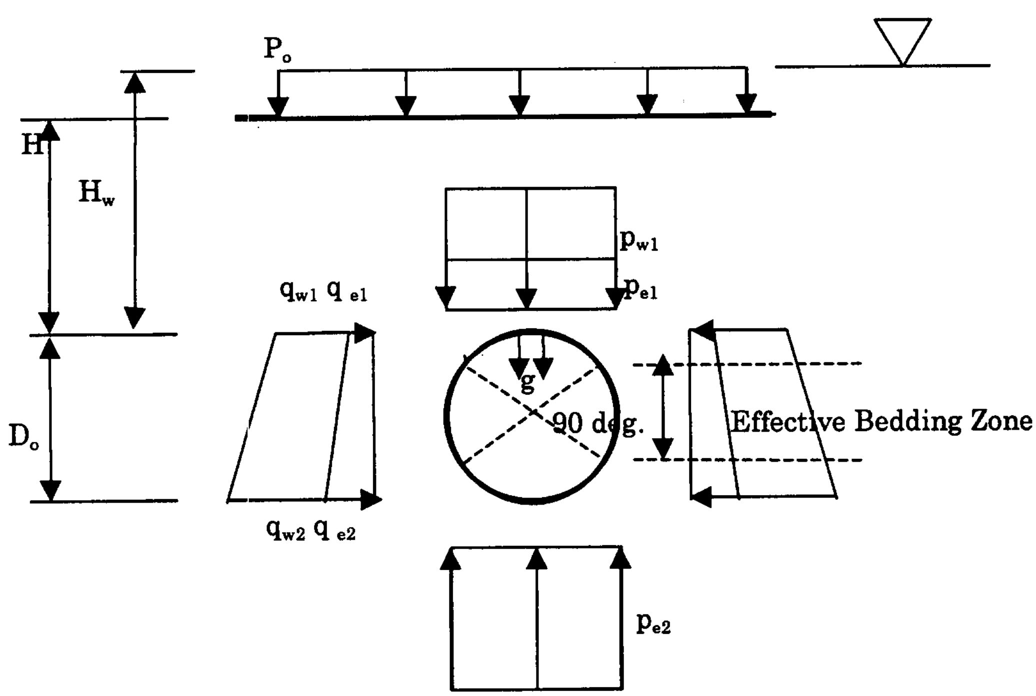
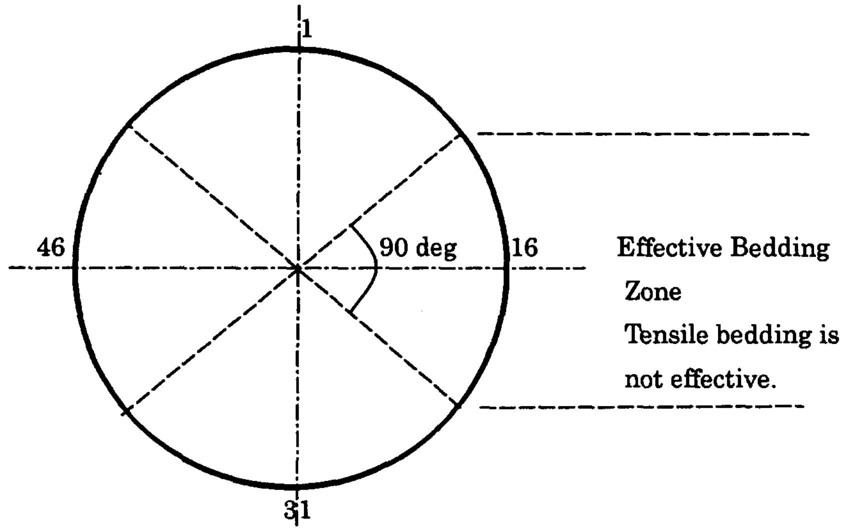
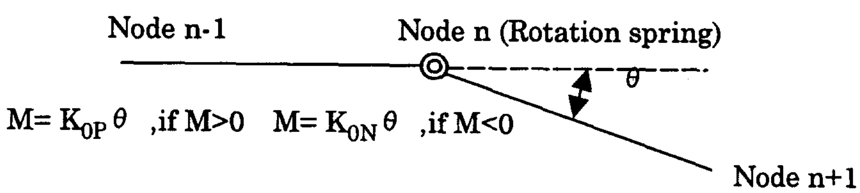


Figure III-6. Load condition for design example 2.



R = Rc = 4550 mm
E = Modulus of elasticity of segment = 39000000 kN/m^2
I = Moment of inertia of area of segment = 0.006400 m^4
A = 0.48 m^2
K_{0P}= Constant of rotation spring for positive moment = 18070 kNm/rad
K_{0N}= Constant of rotation spring for negative moment=32100 kNm/rad

Figure III-7. Bedded frame model to compute member forces.



Hinged joint: K_0 = 0, Rigid joint: K_0 = Infinite

Figure III-8. Model of rotation spring.

Table III-3. Member forces of segmental lining.

Critical Condition		Node	M (kNm)	N (kN)
Segment	+Max	3	+205.83	1178.09
	-Max	11	-169.05	1675.45
Joint	+Max	58	+20.10	1578.24
		3 (@0.6)	+123.50	1178.09
	-Max	50	-22.70	1448.58
		11 (@0.6)	-101.43	1675.45
S_{max}		31	S_{max} = 178.70 kN	

issued by the Japan Society of Civil Engineers.

- *How to compute member forces:* Bedded frame model.
- *How to check the safety of lining:* Allowable stress design method.

3. Load condition

The full overburden earth pressure is adopted as the vertical earth pressure at the tunnel crown (p_{e1}).

Dead load:

$g = B\,\gamma_c t$=1.2 @ 26.5 @ 0.4=12.72 kN/m^2

where,

γ_c = unit weight of RC segment = 26.5 kN/m^3

:Reaction of dead load at bottom:

$p_g = \pi$ g=39.96 kN/m^2

Vertical pressure at tunnel crown:

Earth pressure: $p_{e1} = B(p_0+\gamma' H)$ = 1.2 @ 138.1 =165.7 kN/m^2

Water pressure:

$p_{w1} = B\gamma_w Hw$ = 1.2 @ 129.0 = 154.8 tf/m^2

$p_1 = p_{e1} + p_{w1}$= 320.5 kN/m^2

Vertical pressure at tunnel bottom:

$p_2 = p_1 + p_g$=320.5 + 39.96 =360.46 kN/m^2

Lateral pressure at tunnel crown:

Earth pressure: $q_{e1}=B\lambda\{p_0+\gamma'(H+t/2)\}$ = 1.2 @ 55.88 = 67.1 kN/m^2

Water pressure: $q_{w1}= B\gamma_w(Hw + t/2)$ = 1.2 @ 131.0 = 157.2 kN/m^2

$q_1 = q_{e1} + q_{w1}$ =224.3 kN/m^2

Lateral pressure at tunnel bottom:

Earth pressure:

$q_{e2} = B\lambda\{p_0+\gamma'(H+Do - t/2)\}$ = 1.2 @ 85.00 = 102.0 kN/m^2

Water pressure:

$q_{w2}= B\gamma_w (Hw+Do - t/2)$ = 1.2@ 222.0 = 266.4 kN/m

$q_2 = q_{e2} + q_{w2}$ = 368.4 kN/m^2

Figure III-6 shows the load condition.

4. Computation of Member Forces

The member forces are computed with the bedded frame model (see Fig. III-7).

4.1 Model for Computation of Member Forces

A 58-regular polygon having 60 nodes is used to compute the member forces.

Node 16 is the middle point between Node 15 and 17, and Node 46 is the middle point between Node 45 and 47.

Nodes 6, 8, 17, 25, 33, 41, 50 and 58 are located at the joints of the segmental lining. The joint is simulated as rotation spring, and it is assumed that moment(M) is in proportion to the angle of rotation(θ), as follows (see Fig. III-8).

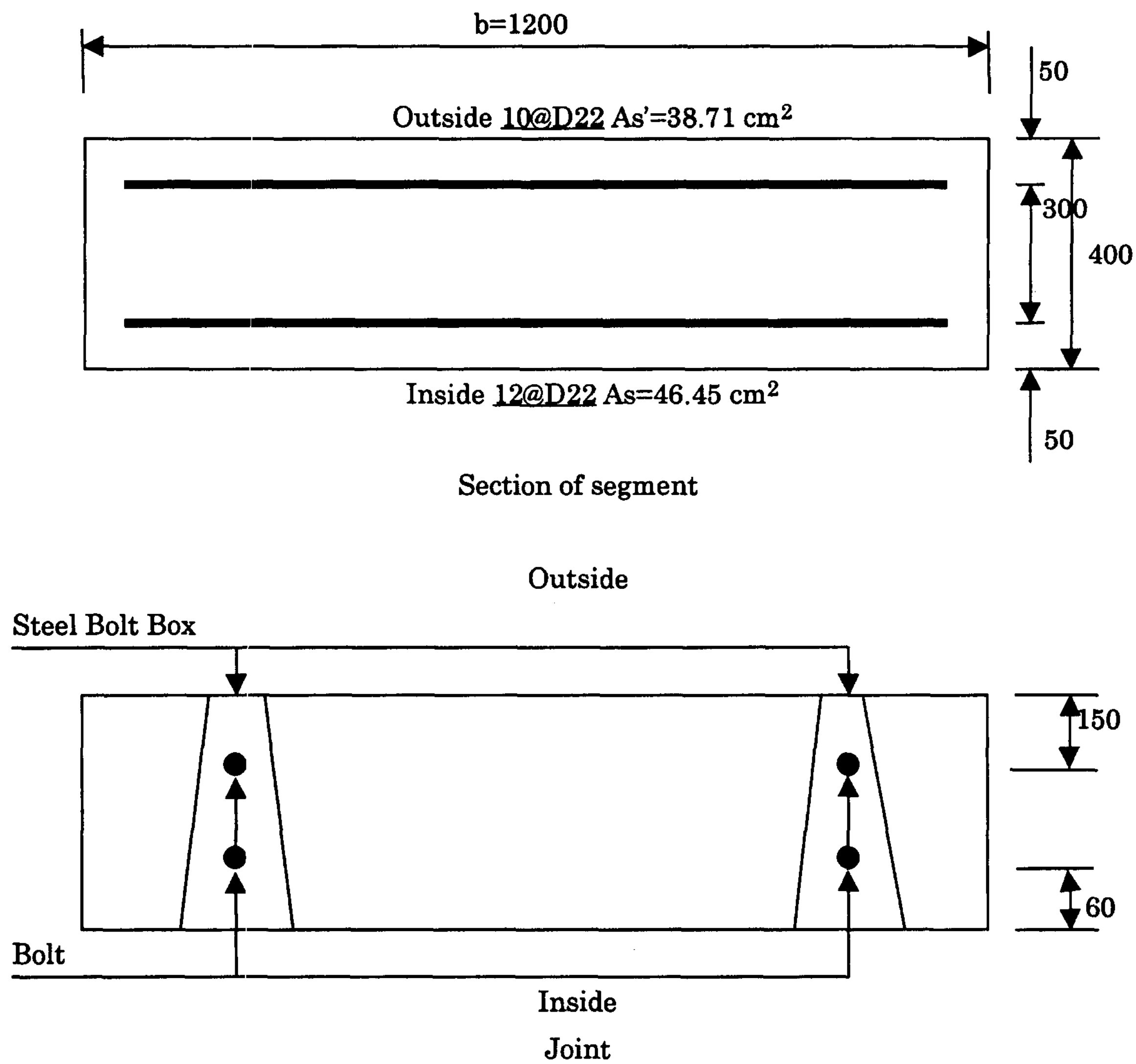
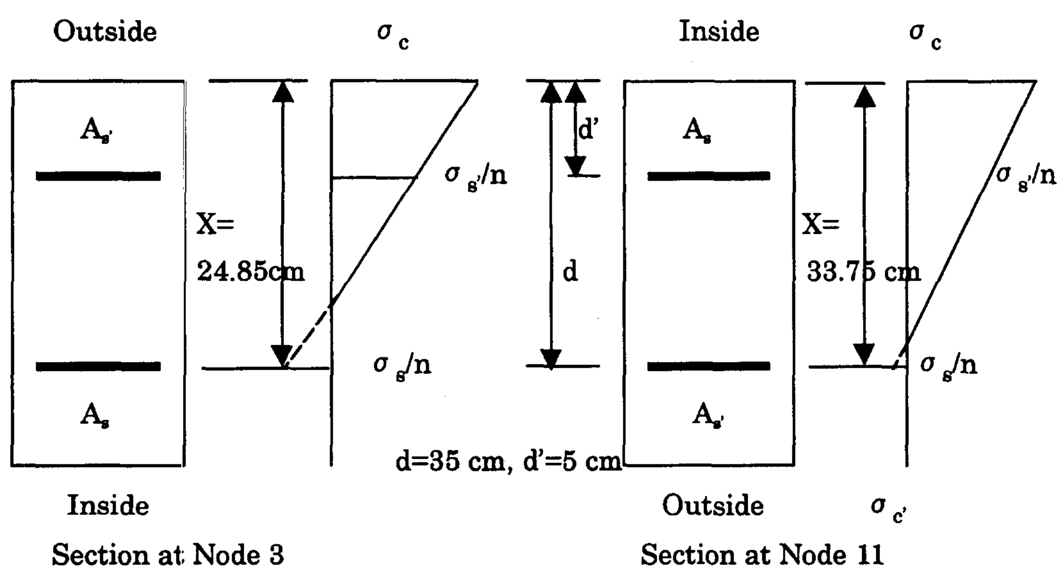


Figure III 9. Section of segment and arrangement of bars and joint.



n=Ratio of moduli of elasticity between reinforcement and concrete=15

Figure III-10. Distribution of stress of critical sections at Node 3 and Node 11.

Table III-4. Computation result of check of safety of segment.

Node	3	11
M (kNm/m)	+205.83	-169.05
N (kN/m)	1178.09	1675.45
σ_c (MN/m^2) (Compressive)	7.1	3.4
σ_s (MN/m^2) Tensile)	43.2	3.6
$\sigma_{s'}$ (MN/m^2) (Compressive)	84.5	82.4

Table III-5. Computation result of check of safety of joint.

Node	58	3	50	11
M (kNm/m)	+20.1	+123.5	-22.7	-101.4
N (kN/m)	1578.2	1178.1	1448.6	1675.5
A_s (cm^2)	11.45	11.45	11.45	11.45
$A_{s'}$ (cm^2)	32.00	32.00	120.00	120.00
d (cm)	34	34	25	25
d (cm)	1	1	7	7
X (cm)	Full section compressive	31.00	31.00	35.10
σ_c (MN/m^2) (Compressive)	3.3	5.1	3.4	5.8
σ_s (MN/m^2) Tensile)	49.8	7.40	46.6	25.0
$\sigma_{s'}$ (MN/m^2) (Compressive)	–	74.1	–	69.5

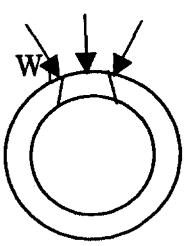


Figure III-11. Check of K-segment.

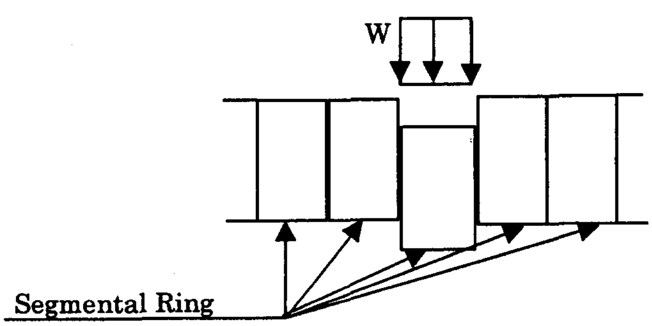


Figure III-12. Check of fall of segmental ring.

4.2 Result of Computation

Table III-3 shows the result of computation of member forces of segmental lining.

In case the safety of the joint is checked, the bigger moment of the maximum moment of the joint and 60% of the maximum moment of the segment is adopted.

Figure III-9 shows the arrangement of bars in the segment and the bolted joint.

5. Check of Safety of segmental Lining

The safety of the segment shall be checked at Node 3 and 11. The safety of the joint shall be checked at Node 58 and 50, and at Node 3 and 11 by using 60% of the moment of each node and the capacity of the joint.

5.1 Check of Segment

5.1.1 Check against moment and axial force

Figure III-10 shows the distribution of stress at Node 3 and Node 11.

Table III-4 shows the computation result of the check of the safety of segment at Node 3 and Node 11. Both of the sections at Node 3 and Node 11 are safe.

5.1.2 Check against shear force

$$\tau = S_{max}/(Bjd)$$
$$= 0.486 \text{ MN/m}^2 < 1.1 \text{ MN/m}^2$$

where

S_{max} = 178.7 kN, B = 120 cm, j = 0.875, d = 35 cm

5.2 Check of Joint

Table III-5 shows the computation result of the safety check of the joint. The steel plate of the bolt box is evaluated as a compressive bar.

5.3 Check of Bolt

Bolt (M27) and bolt (M30) are used between the segment pieces and between the segmental rings, respectively.

5.3.1 Check of bolt between A-type segments and between A-type segment and B-type segment

$\tau = S_{max}/(n_1 A_{BP}) = 54.8\ \text{MN/m}^2 < 150\ \text{MN/m}^2$

where

S_{max} = Maximum shear force among joints = shear force at Node 6 = 125.5 k

n_1 = Number of bolts = 4,

A_{BP} = Area of one bolt (M27) = 5.726 cm^2

5.3.2 Check of bolt between B-type segment and K-type segment

$S\alpha = N\sin\alpha + S\cos\alpha - \mu N = 45.5$ kN

where

$S\alpha$ = Shear force between B-type segment and K-type segment in consideration of angle of joint and friction between both segments

N = Axial force at Node 6 = 1612.7 kN,

S = Shear force at Node 6 = 125.5 kN

α = Angle of joint between B-type segment and K-type segment = 6.7 degree

μ = Coefficient of kinetic energy = 0.2

$\tau = S\alpha/(n_1 A_{BP}) = 19.9\ \text{MN/m}^2 < 150\ \text{MN/m}^2$

5.3.3 Check of fall of K-type segment (Fig. III-11)

$W_1 = \text{Max}(p_b, p_1/B) = p_b$=33 3.3 kN/$m^2$

where

p_b = Pressure of backfill grouting/1.5 = 333.3 kN/m^2

$S_B = 2\pi R_c \times W_1 \times B \times (\theta/360) = 394.92$ kN

$\tau = S_B/(n_2 A_{BP} + n_3 A_{BR}) = 65.8\ \text{MN/m}^2 < 150\ \text{MN/m}^2$

OK

where n_2 = Number of bolts = 8, n_3 = Number of bolts=2,

A_{BR} = Area of one bolt (M30) = 7.069 cm^2

5.3.4 Check of fall of segmental ring (Fig. III-12)

$W = W_1 \times 2 \times Ro \times B + 2 \times \pi \times Rc \times g = 3799.62 + 363.65 = 4163.27$ kN

where

$W_1 \times 2 \times Ro \times B$ = Force acting one segmental ring by pressure of backfill grouting

$2 \times \pi \times Rc \times g$ = Weight of one segmental ring

$\tau = W/(2n_4 A_{BR}) = 101.5\ \text{MN/m}^2 < 150\ \text{MN/m}^2$

OK

References

AFTES (French Tunnelling Association). 19xx. AFTES Recommandations Relatives à la Conception, le Dimmensionnement et l'Execution des Revêtements en Voussoirs Prefabriques en Beton Arme Installés `à l'Arrière d'un Tunnelier. Paris: AFTES.

Ahrens, H.; Lindner, E.; and Lux, K.H. 1982. Zur Dimensionierung von Tunnelausbauten nach den "Empfehlungen zur Berechnung von Tunneln im Lockergestein (1980)". *Die Bautechnik* **8**, 260–311.

Atkinson, J. H. and Potts, D. M. 1978. Calculation of stresses and deformations around shouldow circular tunnels in soft ground by the method of associated fields. Computer Methods in tunnel design, 61–84. London: The Institution of Civil Engineers.

Donovan, H.J. 1974. Expanded tunnel linings. *Tunnels and Tunnelling* (March 1974).

Duddek, H. 1972. Zu den berechnungs methoden und Sicherheit von Tunnelbauten. *Der Bauingenieur* **47**, 44–52.

Duddeck, H. and Erdman, J. 1985. On stuctural design models for tunnels in soft soil. *Underground Space*. 9(5-6), 246–253.

Fujita, K. and Kusakabe, O., eds. 1995. *Underground Construction in soft ground: Proc. of the Int. Symp. on Underground Construction in soft ground, New Delhi, India.* Rotterdam: Balkema.

Iftimie, T. 1998. Overview and a new hypothesis on earth pressure acting on circular tunnel lining. *Proc. of the World Tunnel Congress '98 on Tunnels and Metropolises, São Paulo, Brasil, 25-30 April 1998, Vol.1*, 267–272. Rotterdam: Balkema.

Iftimie, T. and Roatesi S. 1996. The numerical modelling of the shield tunnelling method—Calibration with in situ measurements. *Comptes-rendue des Journées d'études Internationales A.F.T.E.S. (Specifique), Lyon, France, 21–24 Oct. 1996*, 39–47. Paris: AFTES.

Iftimie, T. 1996. A contribution to the concept and structural analysis of precast circular linings for shield driven tunnels. Ph. D. Thesis, Bucharest, Romania.

Iftimie, T. 1994. Prefabricated lining, conceptional analysis and comparative studies for optimal solution. *Proc. of the ITA International Congress "Tunnelling and Ground Conditions", April 1994, Cairo, Egypt,* 339 –346. Rotterdam: Balkema.

Iftimie, T. 1992. Design consideration and testings in shield-driven tunnels. Proc. of the ITA International Congress "Towards New Worlds in Tunnelling", Acapulco, Mexico,16–20 May, 1992, 321 –326. Rotterdam: Balkema.

I.T.A. Working Group on general approaches to the design of tunnels. 1988. Guidelines for the disign of tunnels. *Tunnelling and Underground Space Technology* **3**, 237–249.

Jansen, P. 1984. Tragverhalten von Tunelausbauten mit Gelenk Tubbings. Brauschweig Techn. Univ. Dissertation.

Japan Society of Civil Engineers. 19xx. Specification of Shield Tunnelling for Design and Construction. Tokyo: Japan Society of Civil Engineers.

Kastner, H. 1961. *Statik des Tunnel und Stolenbaues.* Berlin: Springer-Verlag.

Lombardi, G. and Amberg, W. 1979. L'Influence de la mèthode de construction sur l'équilibre final d'un tunnel. *Congres International de Mecanique des Roches. Montreaux , Suisse.*

Muir-Wood, A. M. 1975. The circular tunnel in elastic ground. *Geotechnique* **25**(1), 115–127.

Orlov, S. A. 1961. Metodi staticeskogo rasciota tonnelei. Moscow: Gostroiisdat.

Pantet, A. 1991. Creusement de Galeries à faible profoundeur à l'aide d'un tunnelier à pression de boue. Thèse, Institut National des Sciences Appliquées de Lyon.

Rozsa, L. 1963. Die Bemessung Kreisformiger Tunnelwandungen aus prefabrizierten Stahlbetonelementen nach dem Verfahren der Grenzbelastungen. *Der Bauingenieur* **36**(11), 434– 444.

Schulze, H. and Duddeck, H. 1964. Spannungen in schild-vorgetrieben tunneln. *Beton and Stahlbetonbau* 8, 169– 175.

Sechy, K. 1970. *The Art of Tunneling.* Budapest: Akademiai Kiado.