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Design of linings for shield driven tunnels – A survey on Japanese shield tunneling

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1. INTRODUCTION

This paper is prepared as a part of the study related to a survey on Japanese Shield Tunneling, which was carried out by the Japanese Society of Soil Mechanics and Foundation Engineering — Committee on "Underground Construction in Soft Ground" (JSSMFE TC-28).

Shield tunnel lining in Japan is commonly designed as a ring structure to resist earth and water pressures. The safety of the lining materials vis-a-vis the stress generated in the lining to resist external pressure is confirmed by simulating this external pressure against the ring. As shield tunnel lining is generally composed of segments, attention should be paid to the existence of joints. In addition, the mechanism of the earth and water pressures from the ground acting on a tunnel must be clearly understood. Among the many design elements relating to lining, the earth pressure acting on a tunnel is mainly examined in this paper.

2. CHARACTERISTICS OF EARTH PRESSURE ACTING ON SHIELD TUNNEL LINING

The design process for shield tunnel lining generally starts with computation of the subgrade reaction (resistance earth pressure) which is generated as a result of tunnel deformation or displacement with the earth and water pressures acting as the load. The cross-section of the lining members to resist this load is then decided. According to this design process, the earth pressure acting on a tunnel is determined not only by the tunnel shape, measurements and characteristics of the ground where the tunnel in question is located but also by many other factors, including the digging method, order of digging, rigidity and erection timing of the lining and groundwater conditions.

Imagine the process where a tunnel is excavated by a shield machine, followed by segment lining as illustrated in Fig. 1, the earth pressure (including the subgrade reaction) acting on the lining changes in accordance with the different stages of ground displacement along the tunnel wall as shown in Table 1.

The first step is where the face of the shield machine has not reached a certain place in. Ground displacement starts even at this stage. In the case of open-type shield tunneling work, the ground displaces at the face towards the inside of the tunnel. In contrast, with closed-type shield tunneling work, the ground displaces outwardly away from the tunnel provided that the fluid pressure (slurry pressure or mud pressure) at the face is larger than the earth pressure at the face.

The second step is where the actual digging of the ground by a cutter bit of the shield or by hand occurs. If the face and the ground around the face can be adequately supported by the fluid pressure, there will be little ground displacement. In the case of open-type shield tunneling, even if the compressed air is simultaneously used, displacement large enough to change the earth pressure occurs towards the inside of the tunnel.

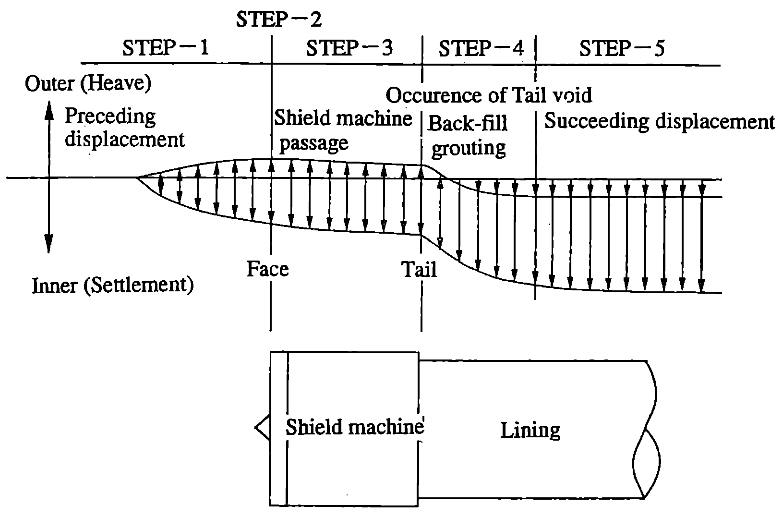


Fig. 1 Advancement of shield machine and ground displacement

Table 1 Summary of changes of earth pressure during shield tunneling operation

Step		Ground Movement	Earth Pressure
1	[Prior to arrival of shield at the face] Balance between the earth pressure at the face during tunneling work and face supporting pressure (slurry pressure, mud pressure and/or air pressure, etc.)	① Ground displacement towards the shield face	Earth pressure becomes active state
		② No ground displacement	Earth pressure remains at rest
		③ Outward ground displacement from the shield face	Earth pressure becomes passive state
2	[Arrival of shield at the face] Balance between the earth pressure of the digging face of the tunnel and the digging face supporting pressure (slurry pressure, mud pressure and/or air-pressure, etc.)	① Ground displacement towards the tunnel cavity	Earth pressure shifts towards active side
		② No ground displacement	No change in earth pressure
		③ Ground displacement to enlarge the tunnel cavity	Earth pressure shifts towards passive side
3	[Passing of shield] Part of the face supporting pressure is conveyed to the digging face of the tunnel.	In general, the digging face moves towards the tunnel cavity to be supported by the shield; shearing of the ground also occurs	Earth pressure shifts towards active side; in some cases, the ground strength is reduced by displacement
4	[Passing of shield tail and completion of back-filling] Immediately after the passing of the shield tail, a void occurs between the back of the segment and the digging face, causing a ground displacement tendency towards the tunnel cavity. This displacement is contained, however, by the back-filling pressure. Deformation of the ring and corresponding subgrade reaction occurs.	① Ground displacement towards the tunnel cavity	Earth pressure shifts towards active side
		② No ground displacement	No change in earth pressure
		③ Ground displacement to enlarge the tunnel cavity	Earth pressure shifts towards passive side
5	[After back-filling] Before the hardening of the back-filling materials, deformation due to release of the grouting pressure and ground rigidity tends to cause displacement of the digging face towards the tunnel cavity which is further encouraged by the long-term consolidation and creep deformation of the back-filling materials.	Ground displacement towards the tunnel cavity	Earth pressure shifts towards active side

The third step is where the shield actually passes the ground in question. At this step, the ground displaces inwardly to the extent of over-excavation of the tunnel and is supported by the outer body of the shield. Even in the case of closed-type shield tunneling work, it is believed impossible to create fluid pressure which is strong enough to prevent ground displacement.

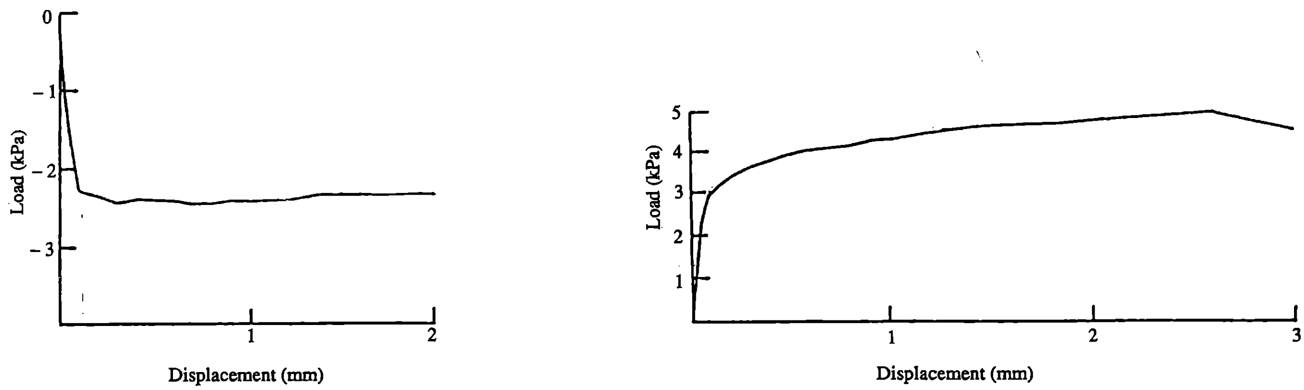
The fourth step is where the shield has completely passed the ground in question and the segment lining assembled inside the tail is detached from the shield tail. At this time, a tail void is created between the lining and the wall of tunnel opening. Any delay in back-filling or delay in the hardening of the back-filling materials leads to ground displacement towards this tail void. If back-filling materials with sufficient strength can be simultaneously grouted to the tail void with the occurrence of the tail void, no ground displacement will occur. There have been some cases in which the back-filling pressure is increased to prevent ground settlement and to push up the ground by pressing hard against the digging face.

The fifth step is where creep deformation of the ground and the back-filling materials gradually occurs with the diminishment of the three-dimensional supporting effect at the face following the further advancement of the shield upon completion of back-filling. At this stage, soft cohesive soil begins to consolidate as its soil structure collapses. The back-filled materials similarly begin to consolidate with the drainage of excess pore water. This consolidation is particularly noticeable when an excessively large grouting pressure is employed.

While ground displacement finally comes to an end after the above process, the earth pressure acting on the tunnel largely depends on the degree of displacement. Fig. 2 (a) shows the correlation between the displacement value of a falling trap-door and the load operating on the trap-door in the trap-door test. Similarly, Fig. 2 (b) shows the correlation between the displacement value of an ascending trap-door and the load operating on the trap-door. These figures indicate that the earth pressure acting on a tunnel from above can easily become loosening earth pressure with minute displacement of the ground above. Conversely, minute ascension generates earth pressure which is larger than the original earth cover load. Fig. 3 shows the correlation between the scale of distortion caused by horizontal displacement of the soil and the earth pressure coefficient. At the sides of a tunnel, displacement at the tunneling wall generates horizontal strain of the soil and Fig. 3 indicates the significant impact of a minor displacement on the earth pressure. In short, the magnitude of earth pressure acting on a tunnel is largely dependent on not only the tunnel depth and soil properties but also on displacement at the tunneling wall in both the vertical and horizontal directions. Moreover, strain in the ground due to displacement may create a plastic zone. With soft cohesive soil, the ground disturbance caused by advancement of the shield reduces the soil strength. This increases the earth pressure acting on the tunnel.

The back-filling to be conducted in the fourth step is an influential factor on substantially changing the earth pressure acting on the tunnel. The larger the residual pressure of the back-filling pressure is, the larger the earth pressure acting on the tunnel is because of the stress generated by the back-filled materials on both the ground and tunnel lining. Unconsolidated back-filled materials have the effect of levelling local fluctuations of the earth pressure. The impacts of the existence of materials which differ from the ground and lining in terms of the characteristics of the two cannot be ignored.

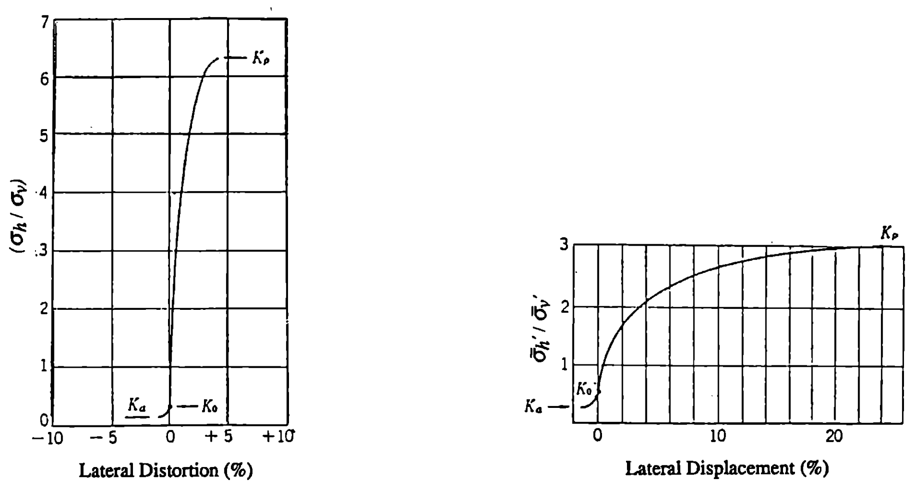
The mechanism of generating the total earth pressure acting on the tunnel is quite complex as seen so far. The distribution and magnitude of the earth pressure substantially change depending on the state of construction even though the geological conditions and tunnel specifications are the same. In practice, the digging method and digging control are believed to be the same throughout a tunnel unless the ground conditions drastically change. As a result, the combined displacement of all the steps does not necessarily represent the actual displacement and should be within the range shown in Fig. 1. Since any tendency of surface settlement is believed to be correlated to the ground displacement near the tunnel, actual observation data on surface settlement can be used to infer such a tendency. Moreover, the movement of groundwater must be taken into consideration at each step, making the phenomenon of earth pressure even more complicated. In short, it is extremely difficult to uniformly determine the earth pressure and subgrade reaction acting on the tunnel. For practical purposes, however, many shield tunnels are uniformly designed by revising the theoretical earth pressure using test results and actual monitoring results. This is one way of determining the earth pressure acting on a tunnel to ensure the safety of all tunnels to be constructed by ordinary methods, allowing for discrepancies between the design earth pressure and actual earth pressure acting on a tunnel.



(a) Descending (Active)

(b) Ascending (Passive)

Fig. 2 Load-displacement curve based on the trap door test (Tanaka, 1991)



(a) Dense Sand

(b) K_0 Consolidated Clay

Fig. 3 Relation between soil's lateral displacement and earth pressure coefficient (Lambe, 1969)

3. LOOSENING EARTH PRESSURE AND LATERAL EARTH PRESSURE

Traditionally, the earth pressure used for tunnel design is considered in terms of the vertical direction and lateral direction. The vertical earth pressure is determined by either of the following 2 methods.

① The weight of the soil of an area loosened by tunneling work above the lining acts as the vertical earth pressure.
② The whole overburden pressure acts as the vertical earth pressure.

One shortcoming here is the lack of a sufficiently proven relationship between the loosening ground area and tunnel deformation. This is caused by the fact that the rigidity of the lining or the ground is not taken into consideration when determining the earth pressure distribution. Neither is the interaction between the tunnel and the ground considered.

There have been many studies on the development of the loosening ground area and its final shape and size. In the case of ground dominated by sandy soil, many studies use the vertical earth pressure from the ground above the tunnel as the earth pressure acting on the sinking floor.

Using the test results on the vertical earth pressure acting on a trap door in the case of sandy soil, Murayama, (1970) showed that 3 distinctive areas will be created in the sand layer following the subsidence of the trap door (Fig. 4). Area I is a directly related area which sinks in accordance with the descent of the trap door. Area II is an indirectly related area which moves towards Area I while loosening static sand grains. Area III is a stational area which disregards the movement of the trap door. The indirect area expands upwardly in accordance with the increased subsidence of the trap door, eventually reaching the ground surface. Assuming the appearance of these 3 areas, Murayama et al. calculated the vertical earth pressure acting on a trap door.

Ono et al. (1983) argued that the stability of the arch in the loosening area requires the working of specific pressure, calculated by multiplying the pressure in the direction of the arch's axis with the coefficient of active earth pressure, on the tunnel lining and, therefore, treated this pressure as the minimum earth pressure acting on the lining. With regard to the earth pressure acting from the side, the earth pressure in question is usually given by multiplying the vertical earth pressure by the coefficient of lateral earth pressure. In general, this earth pressure is believed to be almost equivalent to the earth pressure at rest and the coefficient of pressure at rest relevant to the specific type of soil is used.

All the above-mentioned earth pressures are active earth pressure and tunnel stability is established by the occurrence of earth pressure (subgrade reaction) to resist such active earth pressure. The subgrade reaction is often interpreted in the form of linear spring. As described later, however, there are many theories regarding the distribution of subgrade reaction. Moreover, many views exist on how to decide the coefficient of ground reaction (Fig. 5 and Fig. 6).

In any case, while it is possible to theoretically separate the active earth pressure and subgrade reaction, they cannot be separated in actual measurement because of their simultaneous action nature. In some cases of actual design, these two are consequently not treated separately but are combined to ensure safety.

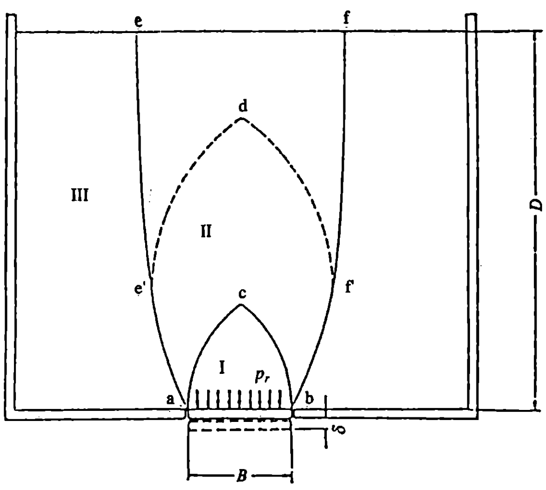
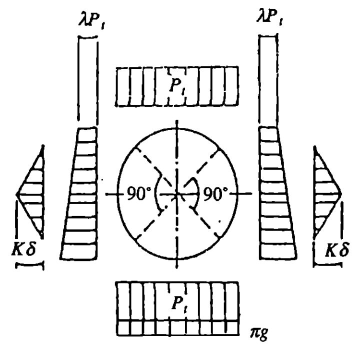
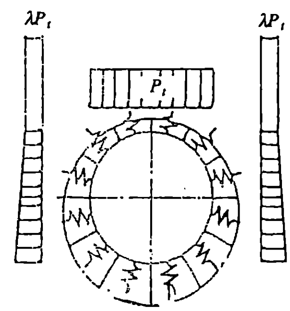


Fig. 4 Different movement areas of sand layer caused by trap door (Murayama, S., 1970)



P_t : vertical earth pressure
λ : lateral earth pressure
K : coefficient of lateral subgrade reaction
πg : reaction to dead weight

Fig. 5 Assumption of subgrade reaction based on common model



P_t : vertical earth pressure
λ : lateral earth pressure

Fig. 6 Assumption of subgrade reaction based on spring bedding model

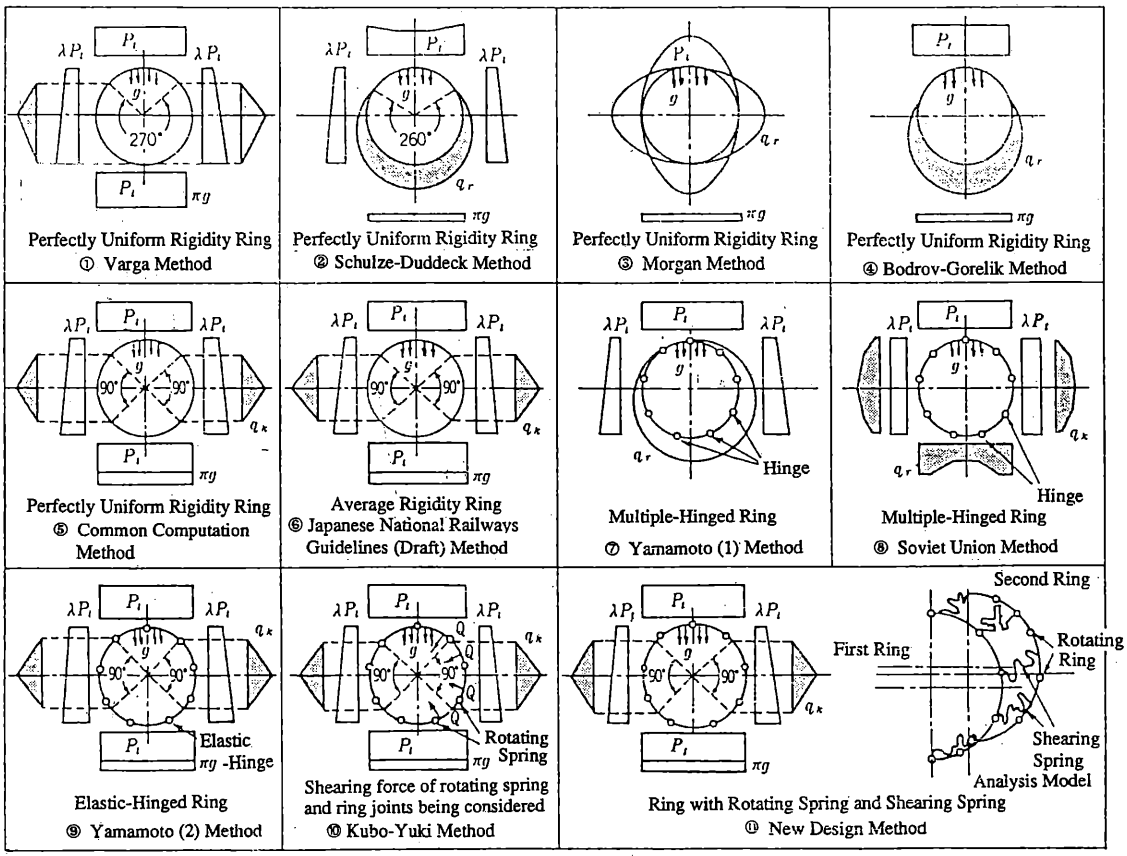
4. COMPUTATION OF EARTH PRESSURE BY CONTINUUM ANALYSIS

With the recent advancements of computer technologies, an analytical method has been developed to treat the ground as a continuum body and this method can be applied to complex issues in terms of the geometry or materials involved. The application of this method to tunneling has also been made easier and, with the inclusion of the time factor, the analysis now fairly accurately represents the actual conditions of the ground. According to the continuum analysis based on FEM, etc., the ground consists of materials showing elasticity, plasticity and viscosity. Any deformation caused by tunneling completes instantaneously in the elastic area. Lining work in the midst of creep deformation causes earth pressure generated by creep or stress relaxation against the lining, the strength of which increases with the passing of time.

In the plastic area, the shear strength declines in accordance with the progress of destruction and expansion of the plastic area due to creep failure increases the tunnel earth pressure with the passing of time. This expansion of the plastic area depends not only on the strength of the ground materials but also on the rigidity of the lining. With the occurrence of a plastic area, the actual earth pressure differs from the theoretical earth pressure based on the FEM analysis method. The portion of the weight in the plastic area which cannot be supported by the shear strength becomes active earth pressure corresponding to the loosening earth pressure described earlier.

In general, the FEM tunnel analysis which treats the ground as an elastic or elasto-plastic body is conducted in the following manner.

① Assuming an unsupported tunnel, the state of the ground stress, especially the plastic area and loosening area, around the tunnel is identified.
② Assuming the placing of the lining as soon as the tunnel has been excavated, the entire load of the initial earth pressure is assumed to be supported by the lining from the elastic or elasto-plastic perspective.



P_t : vertical earth pressure
λ : lateral earth pressure coefficient
g : dead weight
πg : reaction to dead weight (subgrade reaction)
q_k : lateral subgrade reaction (related to displacement)
q_r : radial subgrade reaction
q_u : vertical subgrade reaction

Fig. 7 Segment design model (earth pressure, subgrade reaction and lining structure) (Hanya et al. 1987)

When discussing a visco-elastic body or visco-elasto-plastic body, computation taking time dependence into consideration is possible. Accordingly, the analysis can take the construction processes into consideration, making realistic computation of the earth pressure on the lining, the state of stress and deformation possible.

Although the FEM is frequently used for tunnel analysis, it is hardly ever used for the design computation of the lining for a shield tunnel. Instead, it is often used to evaluate the consequences of neighbouring work or to estimate ground displacement. One reason for this is that the phenomenon of earth pressure in the case of shield tunneling is so complex that it is still difficult to conduct a quantitative evaluation. In order for the FEM analysis method or similar analytical methods to be established as an effective method for the design of shield tunnel lining, it is necessary to accumulate more data through theoretical analyses on the relationship between the earth pressure phenomenon and geological conditions/construction methods and also through actual monitoring at construction sites.

5. DESIGN STATUS OF EARTH PRESSURE AND WATER PRESSURE

Shield tunnel lining design is generally conducted by determining the section specifications vis-a-vis the bending moment, axial force and shearing force generated by such main loads as the active earth pressure and water pressure and the subgrade reaction caused by deformation or displacement of the ring. While the design methods adopt similar earth pressure and water pressure for this purpose, there are many models to determine the subgrade reaction. Fig. 7 shows the main design models and most countries, including Japan, use one of these models. Fig. 8 shows the maximum bending moment and axial tension computed under uniform conditions (Table 2) to compare the different design methods. There is a conspicuous difference in the sectional force depending on the computation method used, indicating the complexity of designing shield tunnel lining.

Table 3 gives the results of the questionnaire survey conducted by the International Tunnel Association circa 1980 on the design of shield tunnel lining. The design methods used are not so diverse, apart from the use of the FEM in addition to those listed in Fig. 7.

Table 2 Ground conditions and structural conditions of segment for computation to compare different design methods (Hanya et al. 1987)

Ground Conditions

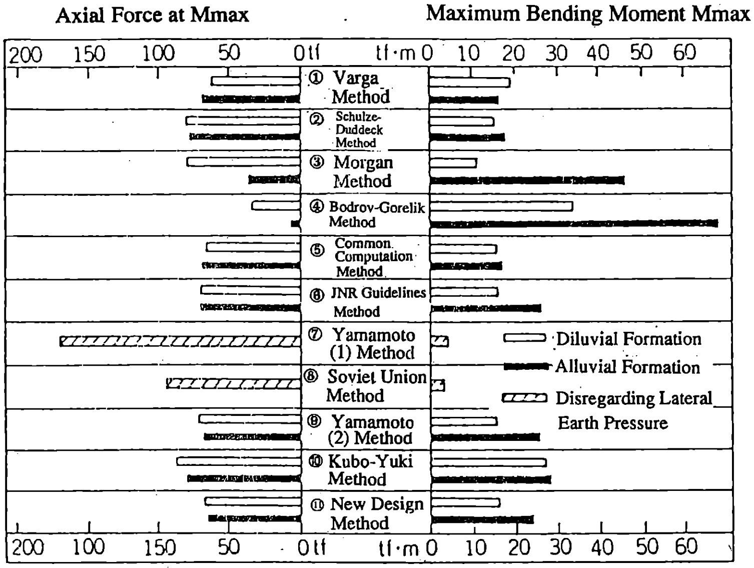
Load Conditions	Diluvial Formation	Alluvial Formation	Applicability
Vertical Earth Pressure (tf/m^2)	30	30	for all methods (including earth and water pressure)
Lateral Earth Pressure Coefficient	0.5	0.7	for ①, ② and ⑤ through ⑩ (single track in ⑦ and ⑧ not considered)
Subgrade Reaction Modulus (tf/m^3)	3,000	0*	for ⑤, ⑥, ⑨, ⑩ and ⑪
		100*	for ②, ③, ④, ⑦ and ⑧
Deformation Modulus of Soil	8,000	300	for ①
Unit Weight of Soil (tf/m^3)	2.0	1.6	for all methods (including earth and water pressure)

* For those cases where design is not feasible without assuming a subgrade reaction modulus, K = 100 tf/m^3 was assumed.

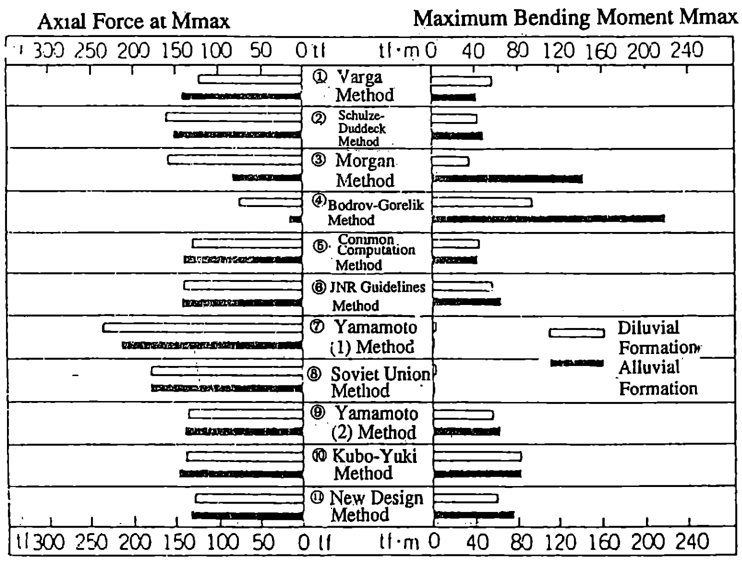
Segment Structure Conditions

Segment / Ring Structure	Double Track		Single Track		Remarks (Application to Different Design Methods)
	Diluvial Formation	Alluvial Formation	Diluvial Formation	Alluvial Formation	
Effective Ratio of Flexural Ridigity (η)	1.0 0.2	1.0 0.5	1.0 0.2	1.5 0.5	for ① through ⑤ for ⑥
Incremental Ratio of Bending Moment (ζ)	0 0.6	0 0.5	0 0.6	0 0.5	for ① through ⑤ for ⑥ and ⑨ (computated based on $\zeta = 1 - k^*/k$)
Rotating Spring Constant k (tf·m/rad)	4,000	10,000	1,000	5,000	for ⑨ (female hinge constant)
Rotating Spring Constant k* (tf·m/rad)	1,600	5,000	400	2,500	for ⑩ and ⑪ (computated based on $k^* = k(1-\zeta)$ (value of ζ from ⑥)
Shearing Spring Constant k_o, k_r (tf/m)	104	105	103	104	for ⑪ (ko: tangential direction; k_r: radial direction)

Note: Different types of joints are used for segments for alluvial formations and segments for diluvial formations.



Segment Ring for Single Track



Segment Ring for Double Track

Fig. 8 Comparison of computed sectional force (Hanya et al. 1987)

6. CONCLUDING REMARKS

As explained earlier, the earth pressure-related phenomena at a tunnel are so complicated that theoretical analysis alone cannot elucidate their mechanisms. Given the fact that the earth pressure is an important factor in the design of tunnel lining, however, there is every reason to promote research on earth pressure phenomena with a view to creating appropriate models. A preliminary step to elucidate these phenomena through theoretical analysis is to conduct detailed measurement of actual phenomena occurring at a tunnel to accumulate vital data. Unfortunately, existing data are so limited that it is difficult to determine whether or not these data reflect the general earth pressure phenomena at a tunnel. Therefore, detailed measurement at many tunneling work sites is highly desirable to elucidate the nature of earth pressure phenomena.

Table 3 International comparison of shield tunnel design methods

Country	Design Model	Design Earth Pressure/ Design Water Pressure	Coefficient of Ground Reaction
Australia	All-around spring model (Muir Wood Curtis model)	Whole overburden pressure Hydrostatic pressure Poisson's ratio	Plate bearing test or reverse analysis of measurement results; perfect connection with ground or connection of which upper limit is frictional force for tangential direction
Austria	All-around spring model	Shallow tunnel: whole overburden pressure (groundwater pressure considered) Deep tunnel: loosening earth pressure formula of Terzaghi	K = Es/r Reaction in the radial direction only considered
Belgium	Schulze-Duddeck method, checked by FEM	Schulze-Duddeck method	No way to accurately determine the coefficient
W. Germany	Earth cover ≤ 2D: partial spring model (excepting crown section) Earth cover ≥ 2D: all-around spring model	Whole overburden pressure	K = Es/r
W. Germany	Partial spring model Schulze-Duddeck method (load along tangential direction not considered)	Whole overburden pressure	No reply
France	All-around spring model or FEM	Whole overburden pressure or Terzaghi's equation λ: empirical value	
China	Empirical method	Whole overburden pressure empirical value	Vertical or horizontal plate bearing test
Spain	Bugera's method taking interaction between ground and lining into consideration	Terzaghi's formula, ignoring cohesion	Only radial direction is considered
UK	Muir Wood method	Whole overburden pressure (+ water pressure) (+ water pressure)	Based on measurement results in similar conditions
UK	Muir Wood method	Initial vertical load and initial lateral load (initial load is whole overburden pressure)	Triaxial compression test or stress-distortion relationship given by stress gauge; friction force is not considered
USA	Elastic support ring	Whole overburden pressure and water pressure	No reply
	(Detailed design of tunnel lining determined without mathematical consideration)		
USA	Elastic support ring	No details given	No details given
USA	Elastic support ring	Whole overburden pressure and water pressure	Based on laboratory test results; friction force is not considered

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