An integrated design approach for the design of segmental tunnel lining in an EPB-Shield driven tunnel-A case study in Iran: Ahwaz Metro Project

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ABSTRACT: This paper is intended to present an integrated design approach to provide a flexible design of pre-cast segmental lining in terms of the quantity of steel reinforcements for an EPB (Earth Pressure Balanced Shield) driven tunnel in urban area. The integrated design approach is based on Analytical Solution, Bedded-Spring Method, and Numerical Analysis. In this way, the solicitations obtained by different methods have been compared in such a way as to consider the variability in membrane forces, for which the structural verifications should be satisfied. In addition, the reliability of the integrated method is rationally high due to structural examination by different methods. The case to be analyzed by proposed integrated design approach was metro project of Ahwaz, a big city in the Southwest of Iran. The membrane forces in the segmental lining have been analyzed by proposed method and a quite good tendency in results was observed.

1 PROJECT DEFINITION

Currently Iran is planning and executing large infrastructural projects to meet the country development program, above all the increase in accessibility of big cities have recently been emphasized. The population of Ahwaz city, the capital of Khuzistan Province, which according to a census conducted in 2006 stood at 1.38 million, is forecasted to reach 1.6 million in 2021. The city will, therefore, urgently need modern transportation systems, particularly urban railway networks, in order to retain its position as a leading agricultural, industrial and educational centre. Fulfilment of this project reduces urban traffic congestion and air pollution thereby provides comfort and welfare for all citizens. Saving in expenses of journeys within the city and offering the most effective and safest transportation system are the other advantages of this project. The metro alignment stretches from North-East to South-West through the city centre, crossing the Karun River in the zone of Naderi, which is in vicinity of the Ahwaz fault. The total length of line is about 23km and it has 24 stations as represented in Fig.1. The double tunnels, 6.5m in diameter, of Ahwaz Metro are excavated using shielded TBM with the control of the pressure at the face (Earth Pressure Balance Shield, EPB type).

The application of an integrated design method for the design of the segmental lining, particularly the estimation of the reinforcement quantity to meet with both structural verifications and economical considerations is focused in this paper.

Generally speaking, the design and dimension of segmental lining must satisfy the structural verifications for three phases, starting from prefabrications phase (demoulding, storing, transporting, assembling), following advancement phase (TBM thrusting force), ending in service stage (imposing ground and water loads, taking into account the seismic effects and probably future constructive loads).

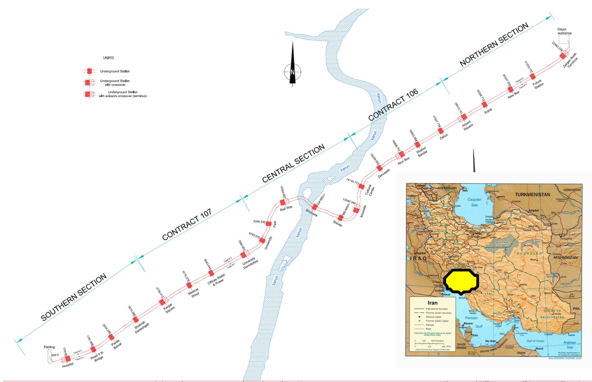


Figure 1. The layout of Ahwaz metro alignment crossing the Karun River.

2 GEOLOGICAL-GEOTECHNICAL SETTING

Based on the results attained from site investigations, four typical geological and geotechnical settings have been identified along the tunnel alignment as listed below:

- domain of sandstone, mudstone and claystone (the Aghajari Formation);
- domain of sandstone, mudstone and claystone tectonically disturbed (the Aghajari Formation in faulted zone in the vicinity of the Karun River);
- a mainly clayey sedimentary sequence related to recent sediments;
- a mainly sandy and silty sedimentary sequence corresponding to recent sediments.

Figure 2 presents the failure envelopes of different geotechnical units. According to a comprehensive geological surveys and investigation along the tunnel alignment, whole tunnel routes will be excavated under water table, which is about 5 meter below surface level, apart from the under-passing of the Karun River, where it is coincided with the level of the river and thus upon the ground level. Due to absence of investigation related to in-situ stress field, the prediction of its value was an assumption of the total overburden load (lithostatic load condition). The value of stress ratio (k) was determined based on the value of earth coefficient at rest, as proposed by Jaky (1944).

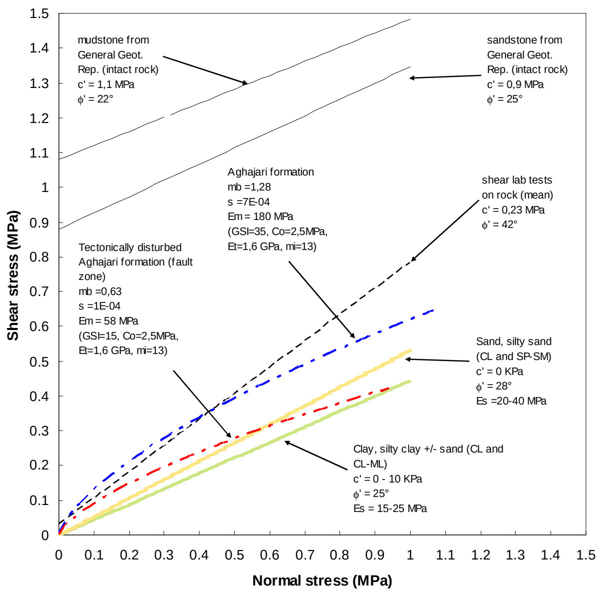


Figure 2. Geotechnical characteristic and Mohr-Coulomb failure envelopes for different geological units.

3 SEGMENTAL LINING AND TBM CHARACTERISTICS

Two EPB Shields (cutting diameters of 6.78m) were chosen to excavate the running tunnels. A nominal thrusting force of around 37800kN are distributed on 22 jacking cylinders and transmitted on 11 shoes. The length of the shield is around 9.5m.

Figure 3. EPB shields foreseen for the excavation of the running twin tunnels

A universal types of ring (5+1) including three base rectangular segments, two trapezoidal counter segments, and one key was proposed (see Fig.4). The thickness of the segment is 30cm and the class of concrete was chosen as C45/55 (E=40300MPa, R_{ck}≥55MPa). The internal diameter is 5.9m and the ring length is 1.4m. The taper of ring allows a theoretical minimum bend radius of 250 m. The connection system between segments and ring are of bolt types. A total of 12 bolts connect the segments in a ring and each ring links to adjacent ring by means of 11 longitudinal bolts.

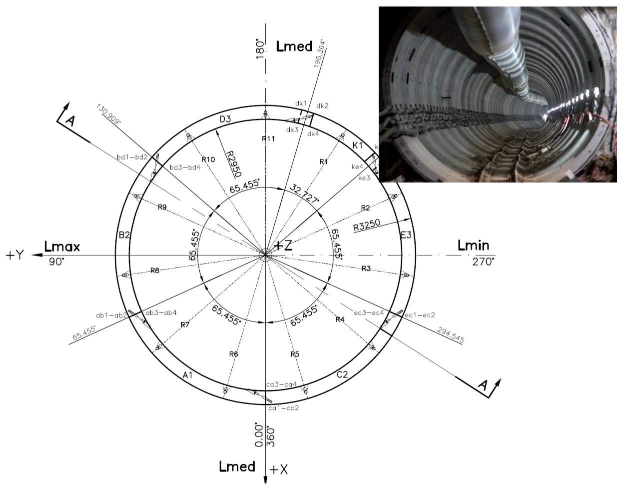
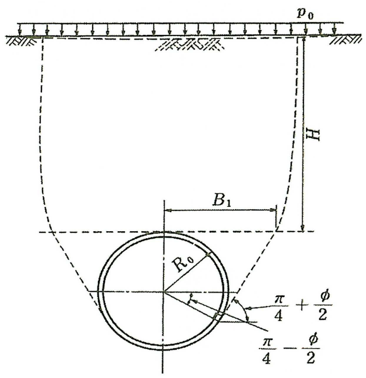


Figure 4. Geometry of the segmental ring 5+1.

4 ESTIMATES OF EARTH AND WATER PRESSURE

The most important potential loads acting on underground structures are ground load (earth pressure) and pore water pressure. One of the important steps in dimensioning the permanent lining for a tunnel is that of determining the ground load for the long term condition since any misjudgements in the design of lining can lead to either under-design and costly failures or over-design and high tunnelling costs. In order to estimate the ground pressure, the concept of Terzaghi's solid (1946), formulations of JSCE (2006), and Unal (1983) have been used respectively for the soil and rock conditions. It is worth pointing out that due to circular shape of the tunnel and continuous longitudinal grouting for filling the gap between the lining and ground during excavation, the loading (ground and water) was considered to act radially in Bedded-Spring model as shown in Fig. 6.



$$B_1 = R_0 \cdot \cot\left(\frac{\frac{\pi}{4}+\frac{\phi}{2}}{2}\right) \quad (1)$$

$$\sigma_v = \frac{B_1(\gamma - c/B_1)}{K_0 \tan\phi} \cdot \left(1 - e^{-K_0 \tan\phi \cdot H / B_1}\right) + p_0 \cdot e^{-K_0 \tan\phi \cdot H / B_1} \quad (2)$$

$$h_0 = \frac{B_1\left(1 - (c/B_1)\right)}{K_0 \tan\phi} \cdot \left(1 - e^{-K_0 \tan\phi \cdot H / B_1}\right) + \frac{p_0}{\gamma} \cdot e^{-K_0 \tan\phi \cdot H / B_1} \quad (3)$$

σ_v= Terzaghi's earth pressure, h_0=effective overburden thickness, K_0= the ratio of horizontal earth pressure to vertical earth pressure, Φ=internal friction angle, p_0= surcharge load, γ= unit weight of soil, c=cohesion of soil.

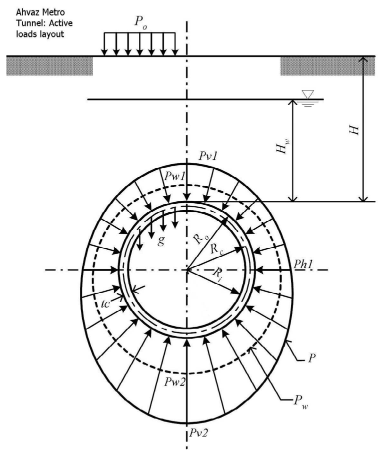
Figure 5. Terzaghi load concept and calculation of ground load for soil ground.

ITA WG (2000) suggested that the value of the coefficient of lateral earth pressure (λ) to be used in the design calculation should be between the value of the coefficient of lateral earth pressure at rest (K_0) and the value of the coefficient of lateral active earth pressure (K_a). It was proposed by JSCE (2006) that: (1) the value of K_0 can be regarded as λ when the horizontal ground reaction is difficult to be obtained, and (2) the value of K_a or a reduction of K_0 can be used as λ when the horizontal ground reaction is available. Following these suggestions, the value of λ is taken as half of the sum of K_0 and K_a:

$$\lambda = \frac{1}{2}(K_0 + K_a) \ , \quad (4)$$

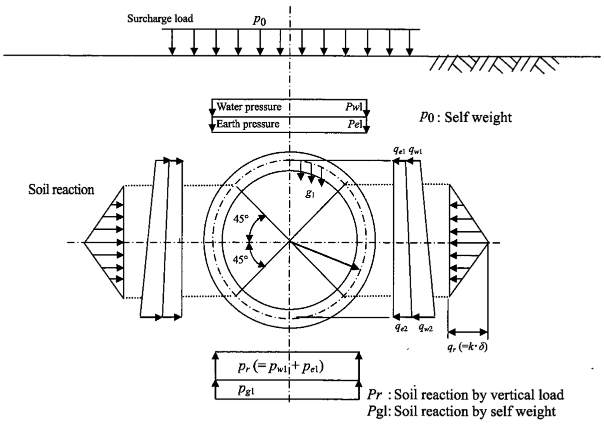
K_a is the coefficient of lateral active earth pressure. K_a can be calculated using equation proposed by Rankine (Aysen, 2005):

$$K_a = \tan^2\left(\frac{\pi}{4} - \frac{\phi}{2}\right) \ , \quad (5)$$



Load	Rock	Soil	Unit
Pw1	130	280	kPa
Pw2	195	345	kPa
Pv1	50.7	112	kPa
Pv2	128.7	173.7	kPa
Phl	51.13	81.5	kPa

Figure 6 Radial active ground and water loads (ground and water pressure) acting on tunnel used in Bedded-Spring Model.



Load estimation (JSCE, 2006)	
p_{e1}	143.9 kPa
p_{w1}	280 kPa
q_{e1}	83.1 kPa
q_{w1}	280 kPa
q_{e2}	118.7 kPa
q_{w2}	345.0 kPa
q_r	10.7 kPa
p_r	423.9kPa
p_{g1}	23.6 kPa

Figure 7. Active loads (ground and water pressure) acting on tunnel based on JSCE (2006) used in Analytical Solution.

On the other hand, in rock ground condition, the rock load (P) based on Unal concept is calculated as (Unal, 1983):

$$P = \frac{100 - RMR}{100}\gamma \cdot D \qquad (6)$$

where RMR is Bieniawski's Rock Mass Raring (Bieniawski, 1989), and D is the diameter of the tunnel.

5 INTEGRATED DESIGN METHOD

The integrated design method for the segmental tunnel lining relies on combination of analytical solution, structural method, and numerical analysis. They are only reliable and practical means to dimension the segmental lining:

1. Analytical solutions: mainly based on the ground-lining interaction concept and they are treated either as the simplified solution methods (ITA, 2000; JSCE, 2006, 2010; Duddeck, H & Erdman, J 1982) in which the segmental lining is considered as a solid ring with equivalent flexural rigidity. In current design, the method of JSCE (2006) was applied.
2. Structural method: based on the hyperstatic reaction method (Bedded-Spring model) in presence and absence of segment joints in model. The primary Bedded-Spring method is able to model a staggered ring arrangement and real positions of the ring joints (two adjacent rings with rotation) with definition of rotational and shear springs and their rigidity for existing joints. Alternative structural method is, on the other hand, that of a solid ring with a reduced equivalent uniform rigidity due to presence of the joints and redistribution of the bending moments by introducing transfer ratio of bending moment "ζ". By means of this simplified calculation method, the bending moment in the main segment section is added and that in the joint reduced.
3. Numerical methods, mainly based on Finite Element Method (FEM), are recent method of evaluating the member forces in segmental lining and they are quite capable of modelling the complex excavation stage in shield-driven tunnel even in complex soil ground condition. The finite element methods are able to model the segmental lining ring either as the uniform rigidity ring or a ring with presence of the joint

5.1 *Analytical solution*

The available analytical methods are based on the uniform rigidity ring method, which was first put forward in 1960 in Japan and it is considered as the widely adopted design methods of shield tunnel lining. In this method, the flexural rigidity (EI) of the circular ring is assumed to be uniform throughout the lining ring. The modified version of that method takes into account the reduction of rigidity due to the presence of joints and the increment of bending moment in the joint area by presenting an effective ratio for the bending rigidity (η). Thus, segmental ring is treated as uniform, but less rigid (solid ring with equivalent rigidity). The computational formulas proposed by JSCE (JSCE, 2006), as listed in Table 1, are adopted for computation of member forces of the tunnel lining in this study. This method is based on the assumption that the flexural rigidity of the circular ring is uniform throughout the lining. In other words, the tunnel lining is simplified as a continuous ring.

Table 1.The formulations of the analytical solutions used in this study (JSCE,2006).

Loads	Bending moment	Axial force	Shear force
$(p_{e1}+p_{w1})$	$M=\frac{1}{4}(1-2\sin^2\theta)(p_{e1}+p_{w1})R_c^2$	$N=(p_{e1}+p_{w1})R_c\sin^2\theta$	$Q=(p_{e1}+p_{w1})R_c\sin\theta\cos\theta$
$(q_{e1}+q_{w1})$	$M=\frac{1}{4}(1-2\cos^2\theta)(q_{e1}+q_{w1})R_c^2$	$N=(q_{e1}+q_{w1})R_c\cos^2\theta$	$Q=-(q_{e1}+q_{w1})R_c\sin\theta\cos\theta$
$(q_{e2}+q_{w2}-q_{e1}-q_{w1})$	$M=\frac{1}{48}(6-3\cos\theta-12\cos^2\theta+4\cos^3\theta)$ $(q_{e2}+q_{w2}-q_{e1}-q_{w1})R_c^2$	$N=\frac{1}{16}(\cos\theta+8\cos^2\theta-4\cos^3\theta)$ $(q_{e2}+q_{w2}-q_{e1}-q_{w1})R_c$	$Q=-\frac{1}{16}(\sin\theta+8\sin\theta\cos\theta-4\sin\theta\cos^2\theta)$ $(q_{e2}+q_{w2}-q_{e1}-q_{w1})R_c$
$q_r=k\delta$	When $0\le\theta\le\frac{\pi}{4}$, $M=(0.2346-0.3536\cos\theta)k\delta R_c^2$ When $\frac{\pi}{4}\le\theta\le\frac{\pi}{2}$, $M=(-0.3487+0.5\sin^2\theta+0.2357\cos^3\theta)k\delta R_c^2$ When $\frac{\pi}{2}\le\theta\le\frac{3\pi}{4}$, $M=(0.1513-0.5\cos^2\theta-0.2357\cos^3\theta)k\delta R_c^2$ When $\frac{3\pi}{4}\le\theta\le\pi$, $M=(0.2346+0.3535\cos\theta)k\delta R_c^2$	When $0\le\theta\le\frac{\pi}{4}$, $N=0.3536\cos\theta k\delta R_c$ When $\frac{\pi}{4}\le\theta\le\frac{\pi}{2}$, $N=(-0.7071\cos\theta+\cos^2\theta+0.7071\sin^2\theta\cos\theta)k\delta R_c$ When $\frac{\pi}{2}\le\theta\le\frac{3\pi}{4}$, $N=(\cos^2\theta+0.7071\cos^3\theta)k\delta R_c$ When $\frac{3\pi}{4}\le\theta\le\pi$, $N=-0.3535\cos\theta k\delta R_c$	When $0\le\theta\le\frac{\pi}{4}$, $Q=-0.3536\sin\theta k\delta R_c$ When $\frac{\pi}{4}\le\theta\le\frac{\pi}{2}$, $Q=(-\sin\theta\cos\theta+0.7071\cos^2\theta\sin\theta)k\delta R_c$ When $\frac{\pi}{2}\le\theta\le\frac{3\pi}{4}$, $Q=(-\sin\theta\cos\theta-0.7071\cos^2\theta\sin\theta)k\delta R_c$ When $\frac{3\pi}{4}\le\theta\le\pi$, $Q=0.3535\sin\theta k\delta R_c$
g_1	When $0\le\theta\le\frac{\pi}{2}$, $M=(\frac{3}{8}\pi-\theta\sin\theta-\frac{5}{6}\cos\theta)g_1R_c^2$ When $\frac{\pi}{2}\le\theta\le\pi$, $M=\left[-\frac{1}{8}\pi+(\pi-\theta)\sin\theta-\frac{5}{6}\cos\theta-\frac{1}{2}\pi\sin^2\theta\right]g_1$	When $0\le\theta\le\frac{\pi}{2}$, $N=(\theta\sin\theta-\frac{1}{6}\cos\theta)g_1R_c$ When $\frac{\pi}{2}\le\theta\le\pi$, $N=(-\pi\sin\theta+\theta\sin\theta+\pi\sin^2\theta-\frac{1}{6}\cos\theta)g_1R_c$	When $0\le\theta\le\frac{\pi}{2}$, $Q=(\theta\cos\theta+\frac{1}{6}\sin\theta)g_1R_c$ When $\frac{\pi}{2}\le\theta\le\pi$, $N=\left[(\theta-\pi)\cos\theta+\pi\sin\theta\cos\theta+\theta\sin\theta+\frac{1}{6}\sin\theta\right]g_1R_c$
δ	$\delta=\frac{[2(p_{e1}+p_{w1})-(q_{e1}+q_{w1})-(q_{e2}+q_{w2})+\pi g_1]R_c^4}{24(\eta EI+0.0454kR_c^4)}$		

5.2 *Bedded-Spring model*

The structural method used is based on Bedded-Spring model which is able to model a single solid ring with uniform equivalent flexural rigidity. Apart from taking into account the reduction of ring rigidity by presenting an effective ratio for the bending rigidity (η) due to the presence of joints, the increment of bending moment in the joint area by means of the redistribution of the bending moments is obtained by introducing a transfer ratio of bending moment "ζ" due to staggered arrangement.

The development of the bending moment in a jointed tunnel lining is significantly affected by the joint stiffness and the number of segments in each ring (Lee & Ge 2001; Lee et al. 2002; Teachavorasinskun & Chub-uppakarn, 2010). The jointed ring carries smaller value of the bending moment (at joints) as compared with a continuous ring. Lee at al considered such a reduction of the bending moment generated in a jointed lining due to the existence of segment joints by introducing a coefficient called bending moment ratio "R_m" (Lee et al. 2002). On the other hand, the bending moment (at segments) in a jointed tunnel lining will be larger than its actual value.

The important parameters to be included into the structural analysis are the sub-grade reaction modulus, which are defined as (Galerkin Method). The normal and tangential stiffness are obtained:

$$K_n=\frac{E}{R_t(1+\nu)} \quad (7)$$

$$K_t\approx\frac{1}{3}K_n \quad (8)$$

where E is rock mass deformation modulus, ν is the ground Poisson coefficient, and R_t is the radius of tunnel.

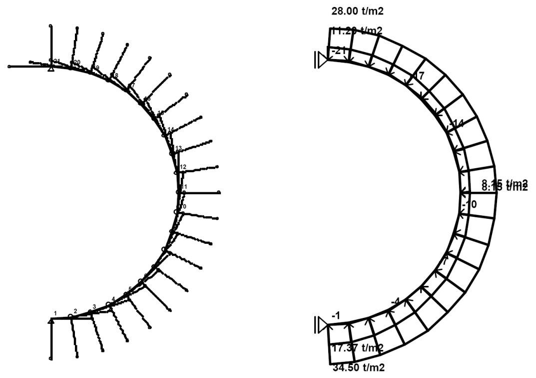


Figure 8. Bedded-Spring Model for soil condition.

5.2.1 *Design criteria*

Figure 9 illustrates the concept used in simplified calculation method, obeying:

- The ring considered singularly is characterised by zones with both high and low flexural inertia, that is, the joints and the segments, respectively;
- A sequence of rings is such that a joint in one ring corresponds to a segment in the previous and subsequent rings (staggered arrangement);
- Such a configuration allows the excess moment that cannot be sustained by the joints in the adjacent segments to be transferred to the previous and subsequent rings.

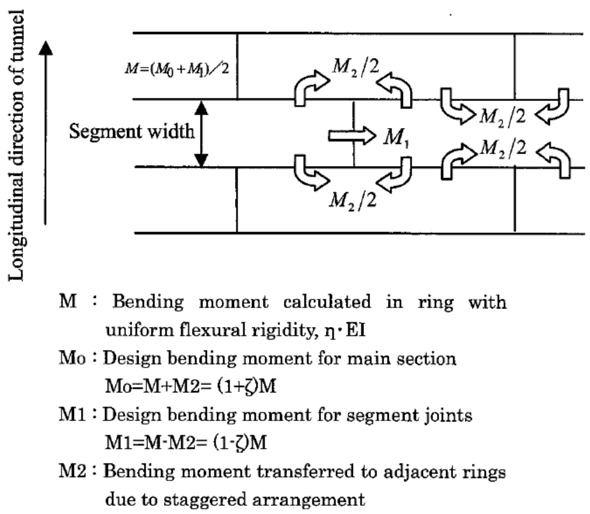


Figure 9. Flexural moments for a segmental lining and transfer of bending moment by joints (JSCE,2006, Article 48).

The transfer ratio of bending moment (ζ) is a ratio of M_2/M, the transfer ratio of bending moment is determined by:

$$\xi = \frac{M_2}{M} \tag{9}$$

M_2 is the bending moment that is transferred to adjacent rings, M is the bending moment calculated in the ring with uniform flexural rigidity (i.e: $\eta.EI$).

The effective ratio of bending rigidity "η" is obtained by:

$$\eta = \frac{EI_e}{EI_n} \tag{10}$$

where E is the elasticity modulus of the segmental lining, In is area-wise moment of complete section without joint, I_e is the equivalent area-wise moment of the section defined by Muir Wood method (1975) as:

$$I_e = I_s + I_n \cdot (\frac{4}{n})^2 \tag{11}$$

where I_e is the equivalent area-wise moment of the section and n is the number of segment and n>4 (small key-segment counted not counted).

However, it should be noted that the coefficient "η" depends not only on types of segment / joint and staggered arrangement, but also on ground condition. Consequently, a careful consideration should be given to determine the value of η.

Of course, Koyama (2003) and Teachavorasinskun & Chub-uppakarn (2010) have indicated that the rigidity of the continuous lining should be reduced by 20-40%. i.e. the effective bending rigidity ratio "η" varies between 0.6 and 0.8. It is evident that the η=1.0 stands for the continuous ring case without any joint. Koyama (2003) and Teachavorasinskun & Chub-uppakarn (2010) have alleged that, based on the results obtained from simplified design method of JSCE (2006), the segmental joint should be designed to carry only 60-80% of the maximum bending moment carrying by the main segment and the rest amount of the bending moment are to be transferred into adjacent segment. So the transfer ratio of the bending moment "ζ" varies, in most cases, between ranges of 0.2 and 0.4. However, in some cases the value of 0.5 was back-calculated.

In this point, it could be concluded a correlation between the effective ratio of bending rigidity (η) and the transfer ratio of bending moment (ζ):

$$\eta = f(\xi) \tag{12}$$

Thus, in simplified calculation method it can be assumed:

$$\eta \approx (1-\xi) \tag{13}$$

Considering the geometry of the segments and joints, the effective ratio of bending rigidity "η" is obtained as 0.76 which means that rigidity of a continuous ring should be reduced by 24% to simulate the jointed ring. Consequently, an additional bending moment of 24% is assumed to be transferred to the adjacent segment while the joints could carry only 76% of the deduced bending moment.

5.3 *Numerical method by means of FEM*

The numerical models were developed to simulate the tunnel construction process in soil and rock domains. Here only the worst ground condition, i.e. for clay, silty clay + sandy soil domain is interested. The main advantages of numerical analysis is that of taking into consideration the history of field stresses, tunnelling excavation steps and timing of lining installation. None of which can be, in contrast, modelled in the structural model. Furthermore, the volume loss control method was integrated in numerical analysis to correlate the relaxation with the ground volume loss obtained from ground surface subsidence profile.

In this method akin to convergence-confinement method, volume loss is prescribed rather than proportion of unloading prior to lining construction (Potts & Zdravkovic´, 2001). The prescribed volume loss (as a fixed-parameter) corresponds to a relaxation factor "λ" (as a variable parameter). Such a relaxation factor is related to the equivalent nodal force acting on the boundary of the excavation (see Fig.10).

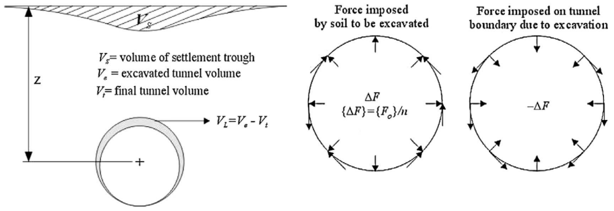


Figure 10. (a) Volume loss method; (b) modelling excavation of solid elements (after Potts & Zdravkovic´, 2001).

Even though the stress redistribution and the deformations occurring during tunnel face advance can be more properly simulated only if 3D numerical models are applied, for the sake of the simplicity and time saving in many cases, 2D plane strain assumption suffices to calculate simple tunnel geometry using certain approximations that can account for the 3D face effect. These approximations in 2D FE analysis would reflect the deformations, which occur between the removal of certain parts of the ground in the tunnel area, and the application of the lining.

The Finite Element analysis by means of PLAXIS 2D was used to analyze the segmental lining. Fig. 11 illustrates the finite element model while the geotechnical properties of the soil (the worst ground condition) are given in Table 2. In order to accurately model the excavation of twin tunnels by EPBS and to investigate the effect of previously excavated tunnel on newly foreseen tunnel, a staged model was considered. In each tunnel driving steps, after nullifying the excavation area, the nodal forces (fictitious inner pressure) which are equal to in-situ stress are applied. The nodal forces are decrease gradually until the segmental lining is installed (the nodal pressure is 85 % of in-situ stress at this stage. i.e. a relaxation of λ=15%). The staged model is capable of simulating the convergence of tunnel which occurs before installation of the lining. The reduced equivalent uniform rigidity approach was used in PLAXIS analysis to comply with the Muir Wood concept.

Table 2 Geomechanical parameters of soil material used in finite element analysis

Analysis type	Finite Element Method (FEM), plane strain 2-D
Stress-strain regime	Elastic-Perfectly plastic
Material type	Isotropic
Failure criterion	Mohr-Coulomb Hardening Soil model
Soil unit weight [kN/m^3]	19.5
Tunnel depth from top of tunnel [m]	26.5
Stress ratio [k]	0.57= (1-sin Φ)
Friction angle Φ	25˚
Young's modulus E [MPa]	20
Poisson's ratio ν	0.3
Cohesion [kPa]	5.0

The time interval between excavations of two tunnels should be chosen in such a way that until the first tunnel runs far enough (~ 2 TBM length), the excavation of second tunnel does not start. It is due to the fact that in such a poor soil material of Ahwaz, any disturbance in ground stress and strain must be avoided. A uniform loading of 0.01 MPa at the top of boundary was applied to simulate the existence of The Karun River.

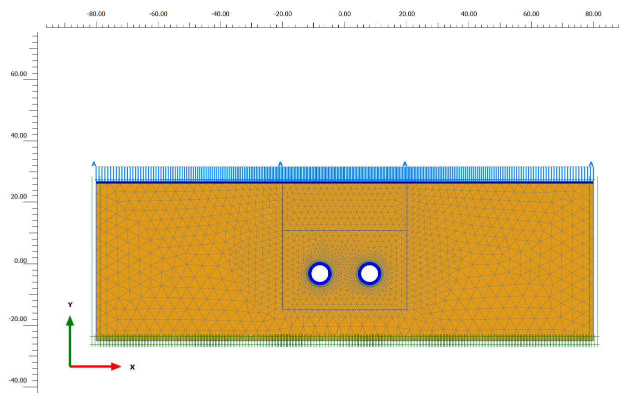


Figure 11. 2D-Finite Element Model by PLAXIS

5.4 *The comparison of the results*

The integrated design approach takes into account the solicitations obtained by different methods. In this way a comparison between the results is made in such a way as to consider the variability in amount of induced axial force and bending moment lining, for which the structural verifications should be satisfied. As far as the soil domain is concerned, Figures 12 and 13 put forward the points that the induced bending moment obtained by Bedded-Spring method is higher than that of analytical and numerical methods. However, the range of induced axial force by such a method lies between the upper bound of FEM and lower bound of analytical solution. The induced axial force obtained by FEM is rather higher that that obtained by Bedded-Spring and analytical methods. Further, the bending moment obtained by FEM is between the ranges of analytical and Bedded-Spring methods.

5.5 *Alternatives for reinforcement quantity*

The intensity of the reinforcements foreseen for the lining should satisfy all load combinations obtained by different methods .i.e. analytical, Bedded-Spring, and numerical methods for both rock and soil condition. Accordingly, a flexible lining design in terms of reinforcement intensity is the matter of interest.

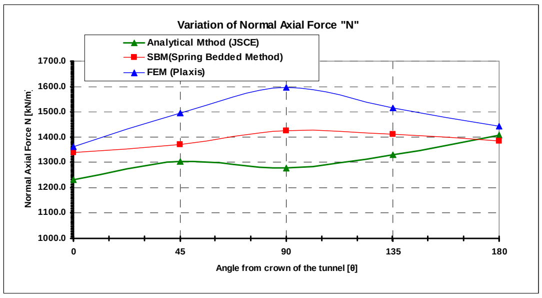


Figure 12. Distribution of the induced axial force in lining, soil condition.

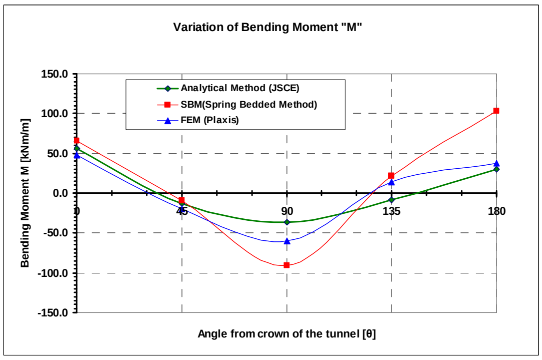


Figure 13. Distribution of the induced bending moment in lining, soil condition.

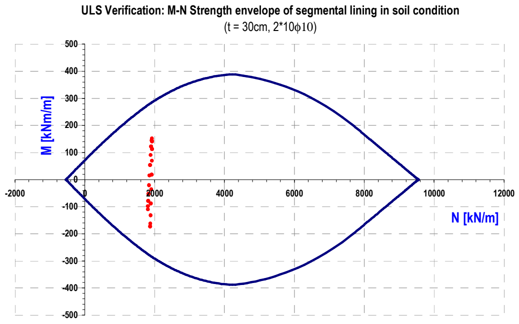
Based on the results obtained for both soil and rock domains, the criteria for lining design are differed as:

- Case A : Lighter reinforcement design for N-E section (rock)
- Case B : Heavier reinforcement design for S-W and The Karun River sections (soil or very weak rock mass of faulty zone).

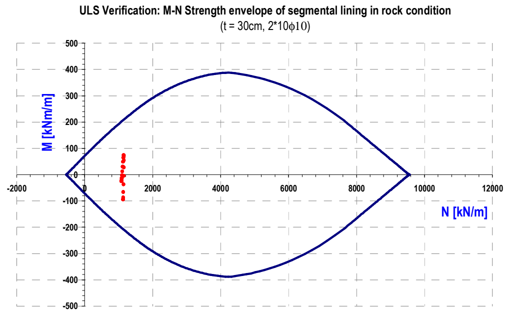
The TBM nominal thrusting force of 37800kN and resulting bursting compressive and tensile splitting stresses inside the segment dominated an equal application of principal reinforcement quantity for both light and heavy segments. A reinforcement quantity of 785 mm^2/m (*10Φ10*) has satisfied the TBM thrusting force. However, the reinforcement quantity ratio has been differently chosen for longitudinal direction of segment depending on ground load, joint action, and different values of axial force and bending moment that act in segment and joints.

The structural verifications were successfully carried out based on EUROCODE (ENV 1992-1-1). The structural verifications in terms of Ultimate Limit State (ULS) are presented in

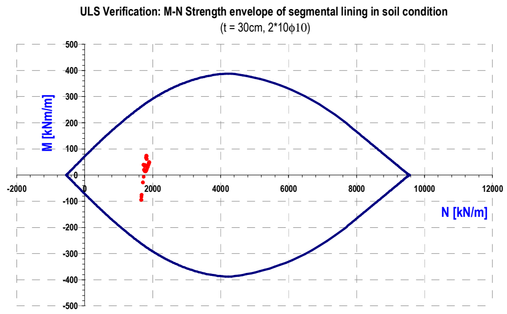
Figure 14 while Table 3 summarizes the quantity of the reinforcements given for the alternative of both light and heavy segments



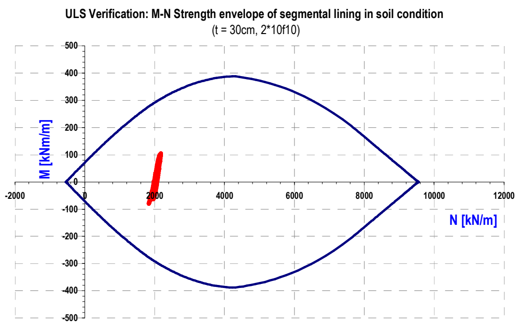
(a)



(b)



(c)



(d)

Figure 14. ULS verification of the segmental tunnel lining: (a) ULS verifications for the combination resulted from Bedded-Spring model in soil condition (b) ULS verifications for the combination resulted from Bedded-Spring model in rock condition (c) ULS verifications for the combination resulted from analytical solution in soil condition (d) ULS verifications for the combination resulted from numerical analysis in soil condition.

Table 3. Quantity of reinforcements for light and heavy segments.

Parameters	Type A: rock part (Aghajari rock formation)	Type B: soil (Quaternary and recent sediment)
Avg. weight of segment [kg]	4015	4015
Avg. volume of segment [m^3]	1.606	1.606
Nominal yield strength of steel f_y [MPa]	420	420
Steel elastic modulus [GPa]	210	210
Weight of reinforcement [kg]	124	154
Quantity of reinforcement [kg/m^3]	77.7	96.0

6 CONCLUSIONS

An integrated design method has been proposed in this paper for the design of segmental lining under difficult geological conditions where it varies from soil to rock even mixed-condition and includes significant variability and uncertainties in ground geotechnical characteristics.

In view of the fact that the integrated design method combines analytical solution, structural model, FEM and takes into account the variability in resulting membrane forces in lining, the safety degree of this method is to a large extent.

Such an approach has successfully been applied in designing the segmental lining of Metro Ahwaz in Iran.

Applying integrated design method made it possible to optimize the design of segmental lining to meet with both technical and economical requirements.

For this purpose, the integrated design method aimed at providing two sets of segments in terms of reinforcement quantity (lighter vs heavier) satisfying structural verifications.

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