POLITECNICI DI MILANO

School of Civil, Environment and Management Engineering

Master of Science in Civil Engineering for Risk Mitigation

Modern approach for safety assessment of underground structures

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Contents

Abstract

Nation-wide subway construction supports the rapid development of society in China. Despite the occurrence of several accidents, shield tunnelling is the main construction method of these infrastructures. This document illustrates the main criteria and standards for the safety assessment of shield tunnel lining. Finite element analysis is combined with Chinese national code for the verification of a case study.

Key words:

Shield tunnel method, structure design, stress analysis, finite element method.

Sommario

Il rapido sviluppo della società cinese è supportato anche dalle molteplici linee di metropolitana in costruzione a livello nazionale. Nonostante i frequenti incidenti, il principale metodo di costruzione adottato in questo ambito è basato sulla tecnica che prevede perforazioni schermate. Questa tesina illustra i metodi di valutazione della sicurezza di queste strutture secondo i criteri e le normative correnti. Inoltre, si presentano i risultati di uno studio condotto utilizzando il metodo degli elementi finiti combinato con le normative nazionali cinesi.

Parole chiavi:

perforazioni schermate, progettazione strutturale, analisi delle sollecitazioni, metodo ad elementi finiti

Chapter 1. Introduction

With the rapid development of society and economy, the scale of cities continues to expand, resulting in the reduction of space for human ground activities. Especially in economically developed cities, the rapid expansion of cities cannot be limited to the development of above-ground space. The construction of urban infrastructure such as cross-river tunnels and large underground parking lots can better share the pressure of road traffic. With the effective development and utilization of underground space, complex structural systems such as "underground comprehensive commercial streets" have recently emerged. "Underground comprehensive commercial street" is an underground activity center integrating commercial, catering, transportation, office, and other functions. It is foreseeable that the development and utilization of underground space will receive unprecedented attention in the 21st century [1]. For the development of underground space, China has used the shield method since the 1990s.

1.1 Underground construction in China

As of June 2016, 43 cities in China have owned or have been approved to build subways. By the end of 2016, the total mileage of urban rail transit approved by the National Development and Reform Commission was about 10207.81km, and the total mileage of 3649.88km has been completed, accounting for about 35.76%, of which the total mileage of subway operations reached 3168.7km. This means that in the next few years, China's subway construction will continue. In the process of subway construction, the characteristics of geotechnical complexity and unpredictability of unidentified objects, the complexity of construction procedures and construction equipment, concealment, and long construction period all increase the risk of the construction process. And from the beginning of the subway construction, various risk factors have always existed. In addition, the amount of subway construction works is large, and a large number of personnel are required during the construction process. Once an accident occurs, it is likely to cause serious personal and property losses.

Based on statistics of subway construction accidents in the past 15 years, three indicators were selected: annual number of accidents, number of deaths, and death rate (number of deaths / number of accidents in a year). Statistics are carried out according to the year, and the statistical results are shown in Figure 1. Since 2011, the number of subway construction accidents has decreased compared with previous years, but it is showing a growing trend. The death rate is generally proportional to the total number of years and increases as the number of accidents increases. As the main structure of subway construction, the safety assessment of the shield tunnel becomes very important.

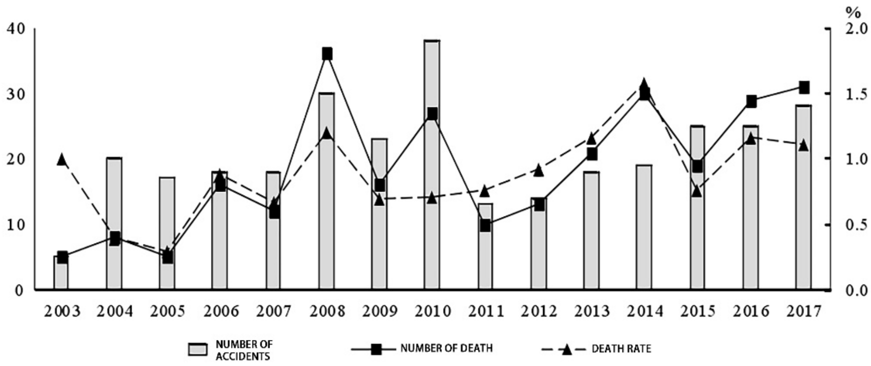

Fig. 1. Statistics of subway construction accidents in China from 2003 to 2017 [2]

1.2 Research status for safety assessment of shield tunnel

Many researchers have done safety assessment of shield tunnel from different aspects. In order to better describe the situation of the shield tunnel under complex geological environment, to provide a reference for the design, Koyama [3] has presented a structure model: beam-spring model and interaction model considering actions around the model, improved requirement for design.

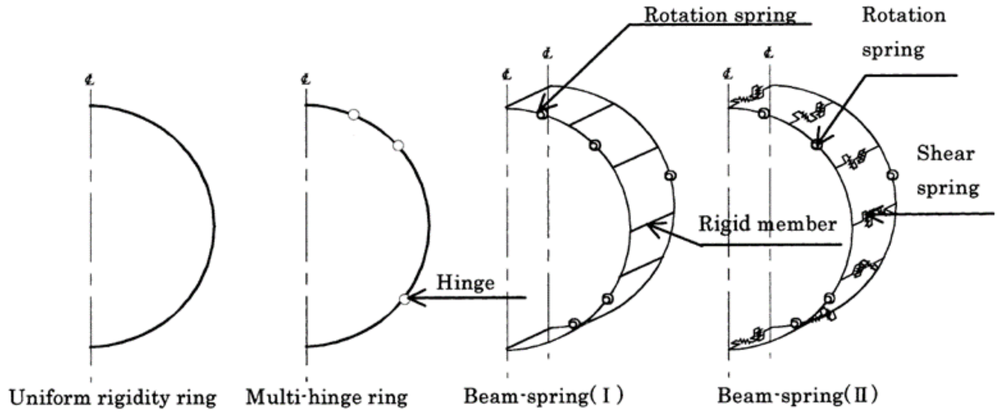

Fig. 2. Structural models of the segment ring [3]

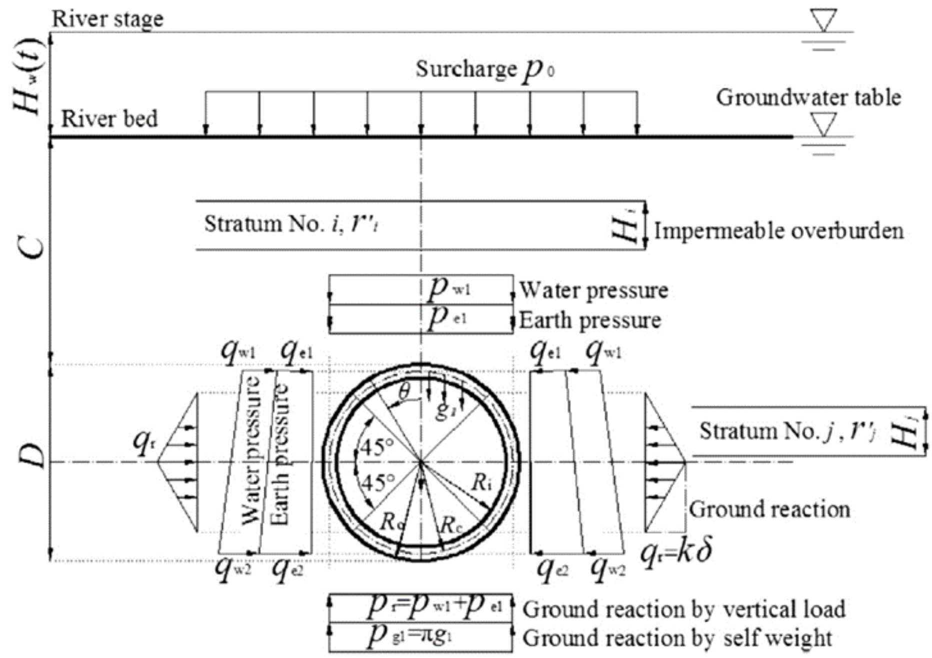

Fig. 3. Conventional model [3]

On his basis, Zhu [4] prove that the consistency of two different beam-spring models: straight beam-spring model and curved beam-spring model. These models are the basis simulation and provide reference for analysis safety assessment of shield tunnel.

As for the safety assessment of shield tunnel lining, stress analysis is a common way, the distribution of stress of precast concrete lining segments under different situation is studied. Chih-Chieh and Jin-Hung [5] did safety assessment for a shield tunnel in a liquefiable deposit and assess the possible damage to shield tunnels by soil liquefaction, they found during the dynamic effective stress analysis, the safety assessment of the tunnel was performed by checking whether the internal forces histories at critical points on the lining (e.g., the moment, shear, and axial forces) exceeded the axial force-moment capacity curve (Pu-Mu) and axial force-shear capacity curve (Pu-Vu). They also did safety assessment of a shield tunnel damaged in 1995 Kobe earthquake [6] to analyze the collapse mechanism. Dynamic time-history analysis and a modified cross-section deformation method are used, seismic behavior of shield tunnel is studied along the time.

About safety assessment of shield tunnel during construction, ground settlement always leads to damage to surrounding structures, which is unavoidable and unignorable. Shu [7] uses three-dimensional FEM analysis, builds a numerical model to simulate shield tunneling process and monitor the ground settlement through whole processing and get the displacement diagram. In order to provide adequate mitigation measures, it is important to assess possible damage not only to shield tunnel but also above structures. Massimino [8] builds a tunnel-soil-aboveground building system and uses FEM to model the seismic behavior of this system. For the characteristics of metro earth pressure balance shield tunnel construction, YOU and MOU [9] use the idea of system engineering identifying the risk factors of shield tunnel construction then identify the risk levels by fuzzy comprehensive assessment theory, finally they form a safety assessment model， which can provide reference for similar projects construction.

Chapter 2. Shield tunnel methodology

The shield method [10] is to use a steel component with a specific shape to dig forward through the excavated soil according to the designed tunnel axis. During the excavation phase, the steel component is used to maintain the stability of the soil and protect the safety of the operator. When using a shield to build a tunnel underground, ensure that the palm face is stable and prevent the soil on the front of the palm face from appearing landslide, using the sturdiness of the shield machine's shell to safely carry out tunnel excavation and segment assembly in the underground.

2.1 Basic principle of shield construction

The basic principle of shield construction is to try to minimize the disturbance to the rock mass during the excavation of the soil body and to minimize the impact on the ground buildings and underground facilities. The shield tunnel is mainly excavated on the premise of maintaining the stability of the surrounding rock mass through its rigid shield shell and segment stiffness. The closed chest shields are mainly earth pressure balance and mud water pressurized shields, which use the pressure of mud compartment or mud water to balance the water and soil pressure on the palm surface, so as to ensure the stability of the palm surface.

The form of shield machine is different, and the construction process and principle are not the same. Take the construction of earth pressure balance shield machine as an example:

1) Build a shaft or foundation pit at one end of the tunnel to be excavated to install the shield machine in place.
2) The shield machine starts from the reserved hole in the wall of the shaft or foundation pit, and advances in the ground along the design axis to the reserved

hole in the shaft or foundation pit at the other end.

3) The front end of the shield machine is equipped with a device for excavating earth. During the propulsion process, an appropriate amount of earth is continuously discharged from the excavation surface, and it is transported to the bottom of the shaft by the vehicle.
4) The shell of the shield machine is composed of a steel cylinder, and several jacks required for jacking are installed along the periphery in the middle section of the steel cylinder. The resistance of the formation during the advancement of the shield machine is transmitted through the jack to the segment ring assembled at the rear of the shield machine.
5) The tail of the steel cylinder is a shell with a certain space. Each time a ring distance is pushed, a ring segment is assembled under the support of the shield tail. A tunnel lining ring composed of several segments can be placed in the shield tail.
6) Since the outer diameter of the steel cylinder at the tail of the shield is larger than the outer diameter of the lining, as the shield machine advances, a certain annular space will be left between the lining and the surrounding rock. To prevent the tunnel and ground from sinking, the grouting body should be pressed into the gap behind the tail of the shield in time.

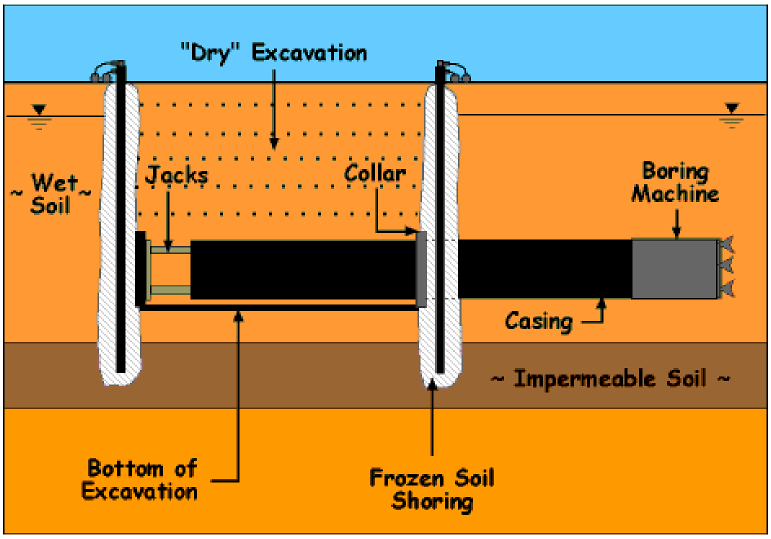

Fig. 4. Shield method schematic [10]

2.2 The structure of shield machine

The basic structure of shield machine is mainly composed of shell, cutting system, propulsion system, earthmoving system, assembly system, etc.

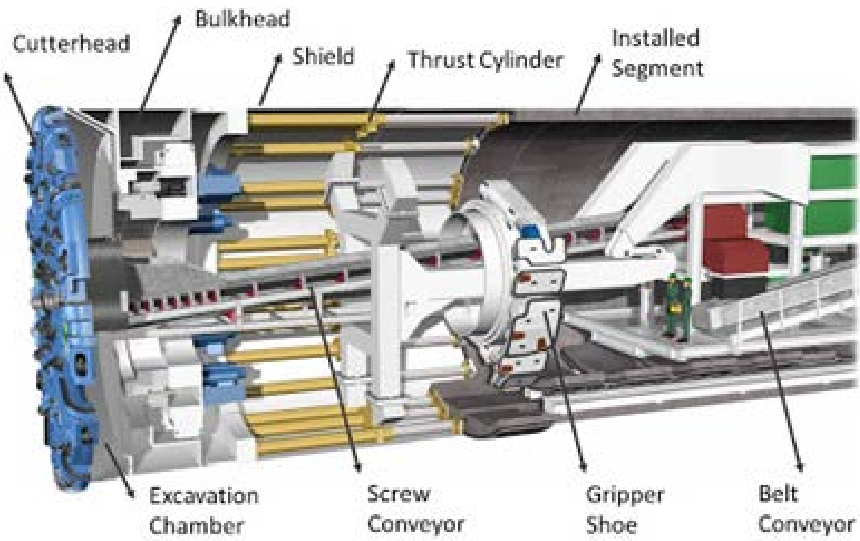

Fig. 5. Shield method schematic [10]

2.2.1 Shield machine shell

The purpose of installing the shield machine shell is to protect the safety of all working equipment such as excavation, earthmoving, propulsion, construction lining, etc., so the whole shell is made of steel plate and reinforced and supported by ring beam.

2.2.2 Cutting system

For the mechanical shield machine, the cutting system is cutterhead, the cutterhead is a disk-shaped excavator that rotates or shakes. It is composed of cutting tools for cutting strata, a face plate that stabilizes the excavation face, an excavation notch, a driving mechanism for turning or shaking, and a bearing mechanism. The cutter head is set at the front of the shield machine, which can not only excavate the soil of the stratum, but also play a certain supporting role on the excavation surface, thereby ensuring the stability of the excavation surface.

2.2.3 Propulsion system

The propulsion system includes a group of propulsion jacks and control equipment arranged on the annular center beam inside the shield machine shell, where the jack is a key component that advances the shield machine in the soil layer.

2.2.4 Earthmoving system

Earthmoving system consists of bucket, sliding guide, funnel, belt conveyor or screw conveyor, and sludge discharge pipe.

2.2.5 Assembly system

The segment assembling system is set at the rear of the shield machine and is composed of a weight-lifting arm and a true circle retainer. The weight-lifting arm is a device for quickly assembling the lining segment in the tail of the shield according to the position required by the design and quickly assembling the lining ring. The true circle retainer can keep the lining ring circular under the self-weight and soil pressure.

2.3 shield tunnel lining

The tunnel lining is the wall of the tunnel. It usually consists of precast concrete segments which form rings. The waterproof of the segment itself is mainly achieved by improving the impermeability of the concrete and the precision of the segment production. Concrete strength plays an important role in structural safety.

2.4 Influence of shield tunneling on stratum

Shield construction will inevitably cause disturbance of the stratum and deform the stratum, especially the weak stratum. When the burial depth is shallow, the deformation will spread to the surface and cause settlement of the surface. Deformation of the ground layer beyond a certain range will seriously affect the safety of nearby buildings and underground pipeline networks, causing a series of environmental geotechnical problems.

The main cause of ground settlement is caused by the loss of stratum during construction. The factors that cause formation loss are:

1) Excavation movement.
2) When dewatering measures are adopted, the effective stress of the soil increases, which again causes consolidation and deformation of the soil.
3) Formation loss due to friction and shear between shell movement and formation.
4) Under the action of water and soil, the deformation of the tunnel lining causes the formation loss

2.5 Technical measures for controlling ground settlement and formation loss

The key to controlling the surface deformation during shield construction lies in two aspects: one is to maintain the stability of the shield face during excavation; the other is to reduce the formation loss. The stability of the shield face during shield excavation is mainly controlled by adjusting and optimizing the shield tunneling parameters; the reduction of formation loss is mainly achieved by timely filling grouting in the tail of the shield and timely grouting in the cave. The specific technical measures are as follows:

1) Adjust and optimize shield tunneling parameters

   The stability of the shield excavation surface can be controlled by adjusting and optimizing the tunneling parameters. The driving parameters include: cutter head and earth cabin pressure, dumping volume and propulsion speed, screw speed, total jack thrust, grouting pressure and time, grouting volume method, slurry performance, shield slope, shield posture and pipe Piece assembly deviation, etc. In order to optimize the construction parameters, the operation of the shield machine must be skillfully mastered, and the actual measurement feedback should be carried out according to the ground deformation curve to verify the rationality of the selected construction or adjust and optimize the construction parameters accordingly. By setting the advancing speed, adjusting the amount of dumping or setting the amount of dumping to adjust the advancing speed, the balance between the pressure in the soil cabin and the formation pressure can be achieved to maintain the stability of the shield face during the excavation of the shield.

2) Simultaneous filling grouting and secondary grouting in the cave

   When the lining ring comes off the shield tail, synchronous grouting in time, and appropriately increase the amount of grouting to fully fill the construction gap between the tunnel and the ground (grouting uses hardening slurry); at the same time, the shield tail should also be strengthened seal. Secondary grouting is an effective auxiliary means to reduce ground subsidence. At the completion of the assembly of the shield about 10 rings (the specific grouting timing is adjusted according to the digging situation), the second (or multiple) grouting is made to the back of the lining to compensate for the synchronization The lack of grouting. According to the construction (structure) and underground pipeline conditions and the nearby geological conditions, determine a reasonable slurry ratio, grouting pressure, grouting volume and grouting start time. The double-liquid slurry is used for the simultaneous construction and secondary grouting of the tunnel under construction at close range.

3) Tunneling in curve section

In the curve section, in order to reduce the excessive deflection angle between the shield axis and the tunnel axis, the ground deformation caused by over-excavation and excessive formation loss is caused. Need to slow down the digging speed, timely correction, increase grouting volume and other work.

4) Construction monitoring

In the process of shield construction, systematic and comprehensive tracking and measurement are carried out, and information construction is carried out. According to the monitoring results, timely adjust the tunneling construction parameters of the shield, or verify the rationality of the selection of the construction parameters, so as to maintain the stability of the shield face of the excavation of the shield and reduce the formation loss.

Chapter 3. Safety assessment of shield tunnel

Safety assessment is an ongoing process that begins with the start of the investigation and continues as the worker gathers more extensive information. It is based on safety criteria and standards from national code.

Safety assessment of shield tunnel can be divided into three aspects:

1) Safety assessment of design of shield tunnel lining

2) Safety assessment of shield tunnel during construction

3) Safety assessment of existing shield tunnel

For each aspect, safety criteria and standards are introduced.

3.1 Safety criteria and standards for design of shield tunnel lining

As the main structure of shield tunnel, the safety criteria [11] of design of shield tunnel lining are:

1) the maximum crack width of shield tunnel lining is 0.2mm
2) the section tunnel lining structure is deformed, the diameter deformation is ≤0.4% ~ 0.6% D (D is the outer diameter of the tunnel), and the opening of the ring longitudinal joint is within 2mm.
3) Durability design according to the designed service life of 100 years.
4) The waterproof level of the interval structure is Grade 2.
5) The structural design should be checked against floating stability according to the most unfavorable conditions. Anti-floating safety factor should not be less than 1.05 when excluding formation side friction resistance.
6) The tunnel lining should be subjected to structural strength calculations during its construction and normal use, and if necessary, stiffness and stability calculations should also be carried out. For concrete and reinforced concrete structures, crack resistance verification or crack width verification should also be performed.

To fulfill these criteria, standards from Chinese national code are introduced.

1) The maximum crack width [12]

The load standard combination or quasi-permanent combination and the maximum crack width considering the long-term effect can be calculated according to the following formula:

$$w_{max} = \alpha_{cr}\Psi\frac{\sigma_s}{E_s}(1.9c_s + 0.08\frac{d_{eq}}{\rho_{te}})$$

$$\Psi = 1.1 - 0.65\frac{f_{tk}}{\rho_{te}\sigma_s}$$

$$d_{eq} = \frac{\sum n_i d_i^2}{\sum n_i v_i d_i}$$

$$\rho_{te} = \frac{A_s + A_p}{A_{te}}$$

$$\sigma_s = \frac{N_q(e - z)}{A_s z}$$

$$e = \eta_s + y_s$$

$$\eta_s = 1 + \frac{1}{4000e_0/h_0}$$

Where, α_{cr} is the characteristic coefficient of component stress, using according to Table 1.

Ψ is the coefficient of strain non-uniformity of longitudinal tensile reinforcement between cracks, when $\Psi < 0.2, \Psi = 0.2; \Psi > 1.0, \Psi = 1.0$

σ_s is longitudinal tensile stress of reinforced concrete members calculated according to quasi-permanent combination of loads or equivalent stress of longitudinal tensile reinforcement of prestressed concrete members calculated according to standard combinations. For Eccentric compression member:

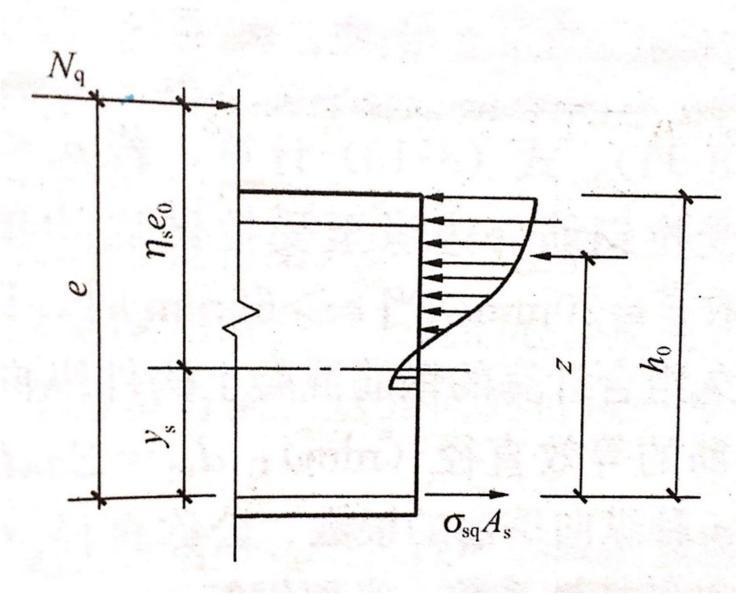

Fig. 6. Calculation diagram for eccentric compression member

E_s is reinforcement modulus of elasticity

c_s is the distance from the outer edge of the outermost longitudinal tensile steel bar to the bottom edge of the tensile zone(mm): when $c_s < 20, c_s = 20; c_s > 65, c_s = 65$

ρ_{te} is reinforcement ratio of longitudinal tensile reinforcement calculated according to effective tensile concrete cross-sectional area, when $\rho_{te} < 0.01, \rho_{te} = 0.01$;

A_{te} effective tensile concrete cross-sectional area, for axial tension members, take the cross-sectional area of the member; for bending, eccentric compression and eccentric tension members, $A_{te} = 0.5bh + (b_f - b)h_f$, where b_f and h_f are width and height of tension flange, respectively.

A_s is cross-sectional area of longitudinal reinforcement in tension zone

A_p is cross-sectional area of longitudinal prestressed tendons in tension zone

d_{eq} is equivalent diameter of longitudinal reinforcement in tension zone

d_i is nominal diameter of the i-th longitudinal reinforcement in the tension zone

n_i is number of the i-th longitudinal reinforcement in the tension zone

v_i is relative bonding characteristic coefficient of the i-th longitudinal reinforcement in tension zone, using according to Table 2.

N_q is design axial force calculation by quasi-permeant load combination.

e is the distance between N_q and the resultant point of tensile rib A_s

z is the distance from the longitudinal tension bar resultant point to the compression zone resultant point

type	α_{cr}	
	Reinforce concrete	Precast concrete
Bending, eccentric compression	1.9	1.5
eccentric tension	2.4	-
Axial tension	2.7	2.2

Table. 1.characteristic coefficient of component stress [12]

Type of steel	steel		Pre-tensioned prestressing tendon			Post-tensioned prestressing tendons		
	Smooth rebar	Ribbed rebar	Ribbed rebar	Spiral rib steel wire	Steel stranded wire	Ribbed rebar	Steel stranded wire	Smooth steel wire
v_i	0.7	1.0	1.0	0.8	0.6	0.8	0.5	0.4

Table. 2. relative bonding characteristic coefficient of steel bar [12]

2) Durability design according to the designed service life of 100 years

    1. The minimum strength grade of reinforced concrete structure is C30; the minimum strength grade of prestressed concrete structure is C40
    2. The maximum chloride ion content in concrete is 0.05%
    3. It is advisable to use non-alkali active aggregates. When using alkali active aggregates, the maximum alkali content in concrete is 3.0kg / m^3

3) Waterproof grade 2

No water leakage is allowed, there may be a small amount of wet stains on the surface of the structure, the total wet stain area should not be greater than 2/1000 of the total waterproof area. The wet area on any 100m^2 area should not exceed 3, and the maximum area of a single wet stain should not be greater than 0.2m. Different impermeability grades according to different engineering burial depths shown in table 3.

burial depths H (m)	Design impermeability grades
$H < 10$	$P6$
$10 \leq H < 20$	$P8$
$20 \leq H < 30$	$P10$
$H \geq 30$	$P12$

Table. 3. Waterproof concrete design impermeability grade [12]

Impermeability grade: according to the maximum water pressure that the concrete specimen can withstand during the impermeability test, the impermeability grade of the concrete is divided into six grades such as P4, P6, P8, P10, P12, and greater than P12, corresponding to the resistance of 0.4, 0.6, 0.8 , 1.0 and 1.2MPa hydrostatic pressure without water seepage.

4) Anti-floating stability check

$$W/F \geq 1.05$$

Where W is the sum of the structure's self-weight and the standard value of the permanent load acting on it.

F is the floating force

5) Material

The engineering materials of underground structures should be selected according to the structure type, stress conditions, use requirements and environment, as well as their reliability, durability and economy. For shield tunnel method the design strength grade of concrete under general environmental conditions is not less than that specified in Table 4.

Assembled reinforced concrete segment	C50
Monolithic reinforced concrete lining	C35

Table. 4. The lowest design strength grade of concrete under general environmental conditions [11]

Reinforcement in ordinary reinforced concrete structure should be selected according to the following regulations:

1. HRB400, HRB500, HRBF400, HRBF500 steel bars should be used for longitudinally stressed beams of beams and columns. HPB300, RRB400 steel bars can also be used for other longitudinally stressed steel bars.
2. Stirrups should adopt HRB400, HRBF400, HPB300, HRB500, HRBF500 steel bars

6) Type of loads

Type of loads is introduced by Chinese national code (GB5009-2012) and is shown in table 5.

Type of load		Name of load	Value of load
Permanent load		Structure self-weight	By actual weight
		Cover weight	soil bulk density provided by detailed
		Overburden	20kN/m^2 per floor
		Lateral water and soil load	Calculated by positive soil pressure
		Floating	Calculated by water level
		Device weight	
		Foundation sinking	
Variable load	Basic variable load	Ground overload	20kPa uniform live load
		Lateral load because of	
		Crowd load	
		Metro vehicle load and	Train load is determined according to
	Other variable	Influence of	
		Construction load	
Accidental load		Seismic load	The seismic load is considered
		Civil defense load	The air defense is considered according

Table. 5. Type of load [13]

7) Load combination

Load combination is determined by Chinese national code (GB5009-2012) regulations and the most unfavorable situation that may occur. The structural calculation mainly considers the load ultimate limit state (for reinforcement calculation) and service limit state (for crack width verification). The structural importance factor is taken as 1.1, and the civil air defense load and seismic load are calculated. The calculation conditions and combination coefficients are shown in table 6.

Serial number	Load / combination	Permanent load	Variable load	Accidental load	
				Civil defense load	Seismic load
1	Basic combination strength calculation	1.35（1.0）〔control〕1.20（1.0）	1.4	-	-
2	Calculation of cracks and deformations of basic composite components	1.0	0.5	-	-
3	Air defense accidental combination Component strength combination	1.2（1.0）	0	1.0	-
4	Earthquake accidental combination Component strength combination	1.2（1.0）	1.2×0.5（1.0 ×0.5）	-	1.3

Table. 6. Load combination [13]

8) Calculation of shear strength of rectangular section

When $h_0/b \leq 4$

$$V_c \leq 0.25\beta_c f_c b h_0$$

Where V is design value of maximum shearing force on inclined section of member.

β_c is concrete strength influence coefficient: when the concrete strength level does not exceed C50, β = 1.0 is taken; when the concrete strength level is C80, β = 0.8 is taken; the linear interpolation method is used to determine.

b is width of rectangular section

h_0 is effective height of section

For section without web reinforcement, shear strength of rectangular section can be calculated:

For distributed load:

$$V = 0.7 f_t b h_0$$

Where, f_t is tensile strength of concrete

9) The rectangular section eccentric compression member's normal section compressive bearing capacity should meet the following requirements:

$$N_u \leq \alpha_1 f_c b x + f_y' A_s' - f_y A_s$$

$$N_u e \leq \alpha_1 f_c b x \left(h_0 - \frac{x}{2}\right) + f_y' A_s' (h_0 - a_s')$$

$$e = e_i + \frac{h}{2} - a_s$$

$$e_i = e_0 + e_a$$

$$e_0 = M/N$$

Where N_u is design value of compressive bearing capacity

α_1 is coefficient, for C50 is 1.0

e is the distance between the point where the axial force acts and the total force point of the tensile bar A_s

e_i is the initial eccentricity

e_0 is the eccentricity of the axial force to the center of gravity of the section

e_a is additional eccentricity, take the larger of 1/30 and 20mm of the cross-sectional dimension in the eccentric direction.

x is height of concrete compression zone

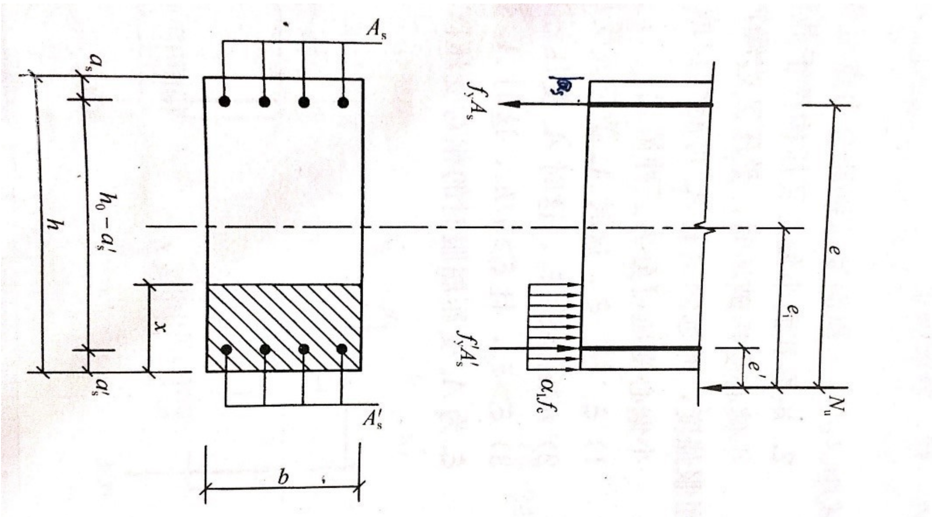

Fig. 7. Diagram of calculation of bearing capacity of large eccentric compression section

3.2 Guidelines for the design of shield tunnel structure

1) Adherence to specification, code or standard

The tunnel to be constructed should be designed according to appropriate normative standards, code, or standards. The normative standards are determined by the project leader or discussed and determined by these people and the designer.

2) Decision on inner dimension of tunnel

The inner diameter of the tunnel to be designed should be determined according to the space required by the tunnel function. This space depends on:

- In railway tunnels, construction gauges and automobile regulations.

- In terms of highway tunnels, traffic volume and number of lanes.
- Discharge in the case of water tunnels and sewer tunnels.
- For public pipelines, the types of facilities and their sizes.

3) Determination of load condition

The load acting on the lining includes earth pressure and water pressure, static load, reaction force, additional force and thrust of the shield jack. Designers should choose cases that are critical to design lining.

4) Determination of lining condition

The designer should determine the lining conditions, such as the size (thickness) of the lining, the strength of the material, and the arrangement of the reinforcement.

5) Computation of member forces

Designers should use appropriate models and design methods to calculate component forces, such as bending moments, lining axial forces and shear forces.

6) Safety check

The designer should check the safety of the lining based on the calculated component force.

7) Review

If the designed lining cannot resist the design load, the designer should change the lining conditions and design the lining. If the designed lining is safe but not economical, the designer should change the lining conditions and redesign the lining.

8) Approval of the design

After the designer determines that the designed lining is safe, economical, and optimal, the project leader should approve the design document.

These steps are shown below:

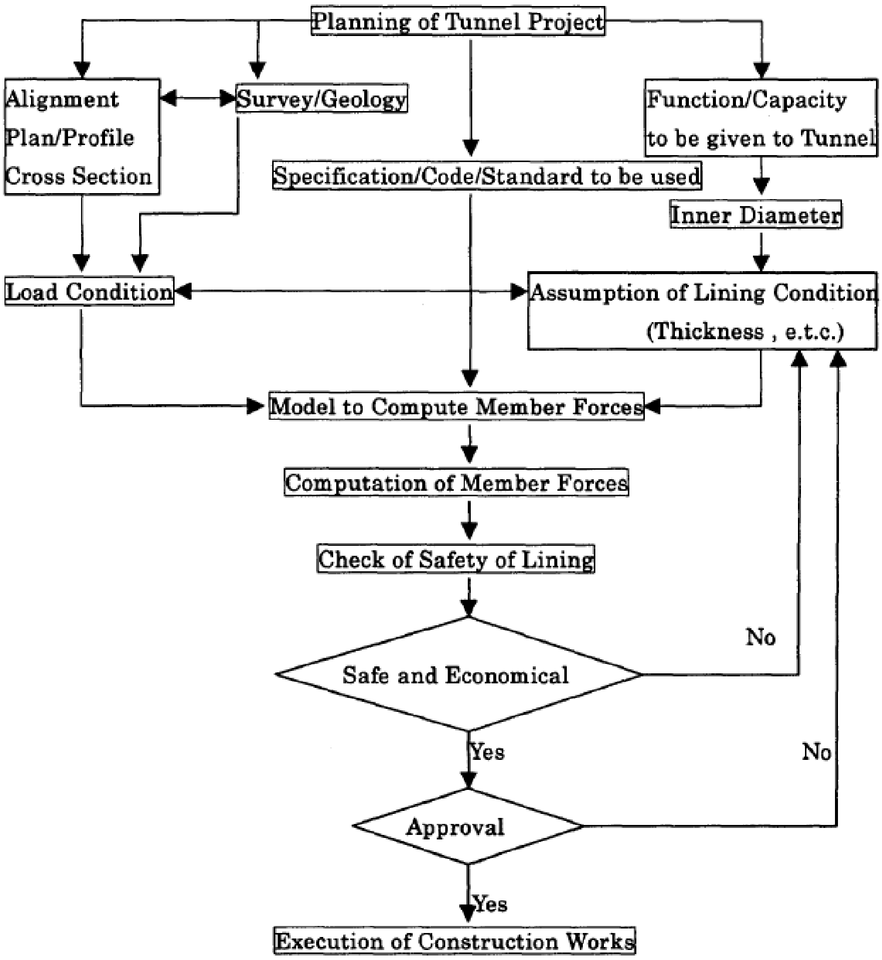

Fig. 8. Flow chart of shield tunnel lining design [14]

3.3 Safety criteria and standards for shield tunnel construction

Shield tunnel construction will produce more or less ground settlement without exception, the safety criteria for shield tunnel construction mainly focus on the protection of affected ground buildings and underground pipelines. Chinese national code organizes the degree of impact in Table 7 and Table 8.

The ground settlement and uplift caused by the tunnel construction should be controlled within the design range; the design must take stable and reliable measures and design according to the surrounding environment, the building foundation condition, and the sensitivity of the underground pipeline to deformation; during the shield construction The amount of ground deformation and settlement should be controlled within 30mm, and the amount of uplift should be controlled within 10mm; when passing through main buildings or underground pipelines, it should be determined according to the actual situation; in open areas, it can be appropriately relaxed.

Maximum surface settlement (mm)	Brittle material	Ductile material
$w_{max} \leq 10$	Pipeline stress increase is not significant	
$w_{max} > 10$	The influence of formation deformation on pipelines should be evaluated in detail	-
$w_{max} > 25$	The pipeline stress will definitely increase significantly, and the small diameter pipeline may be damaged	-
$w_{max} > 50$	Large diameter pipeline may be damaged	The pipeline stress may increase significantly, and the effect of formation deformation on the pipeline should be evaluated in detail

Table. 7. Preliminary estimation of the influence of stratum deformation on underground pipelines [15]

Type of structure	δ / L(L is the length of structure,δ is the differential settlement)	Response of structure
General brick wall load-bearing structure, including the structure with inner frame, the length-to-height ratio of the building is less than 10, and there are ring beams	1/150	The partition wall and the load-bearing brick wall have a considerable number of cracks, which may cause structural damage
General reinforced concrete frame structure	1/150	Severely deformed
	1/500	Cracks started to appear
High-rise rigid building	1/250	Observation of building tilt
Single-storey bent structure factory building with bridge driving	1/300	Difficult operation of bridge driving, difficult to operate without adjusting the level of the rail surface, cracks in the partition wall
Frame structure with diagonal braces	1/600	At a safe limit
Machine foundation generally sensitive to settlement response	1/850	The machine may be difficult to use and is in the limit of operability

Table. 8. Structure response under different settlement differences [15]

3.4 Safety assessment of existing shield tunnel

There are no specific codes or standards presented to illustrate safety criteria for existing shield tunnel, considering that shield tunnel structure is made of concrete so safety criteria applicable to concrete structures can be used for shield structures.

For the conceptual considerations in the assessment of existing concrete structures, general procedure for structural safety assessment showing below:

1. Apply code rules for load and resistance models; assess structural safety.

2. Apply actualized material properties in resistance models from codes; re-assess structural capacities and corresponding structural safety.

3. Apply actualized load models on the action side; re-assess structural safety.

4. Apply limit analysis tools, i.e. failure mechanisms and stress fields; re-assess structural safety within structural safety classification system.

5. Apply probabilistic assessment methods.

Also, a classification system which is largely independent of national codes is introduced. The classification includes "good", "sufficient" and "insufficient" categories, which are represented by green, yellow and red, respectively. The limitation of the "sufficient" category also qualitatively indicates the inherent accuracy of the evaluation criteria considered. If a structure can be completely classified as "good", no further evaluation is required for a long time. For objects classified as "sufficient", monitoring should be strengthened, and repair measures may need to be taken within the current storage period. If a structure is completely classified as "insufficient", you need to take immediate measures, for example: Of course, the conversion between structural safety levels is non-stop, and evaluation engineers and owners (if applicable) should make Security decision authority.

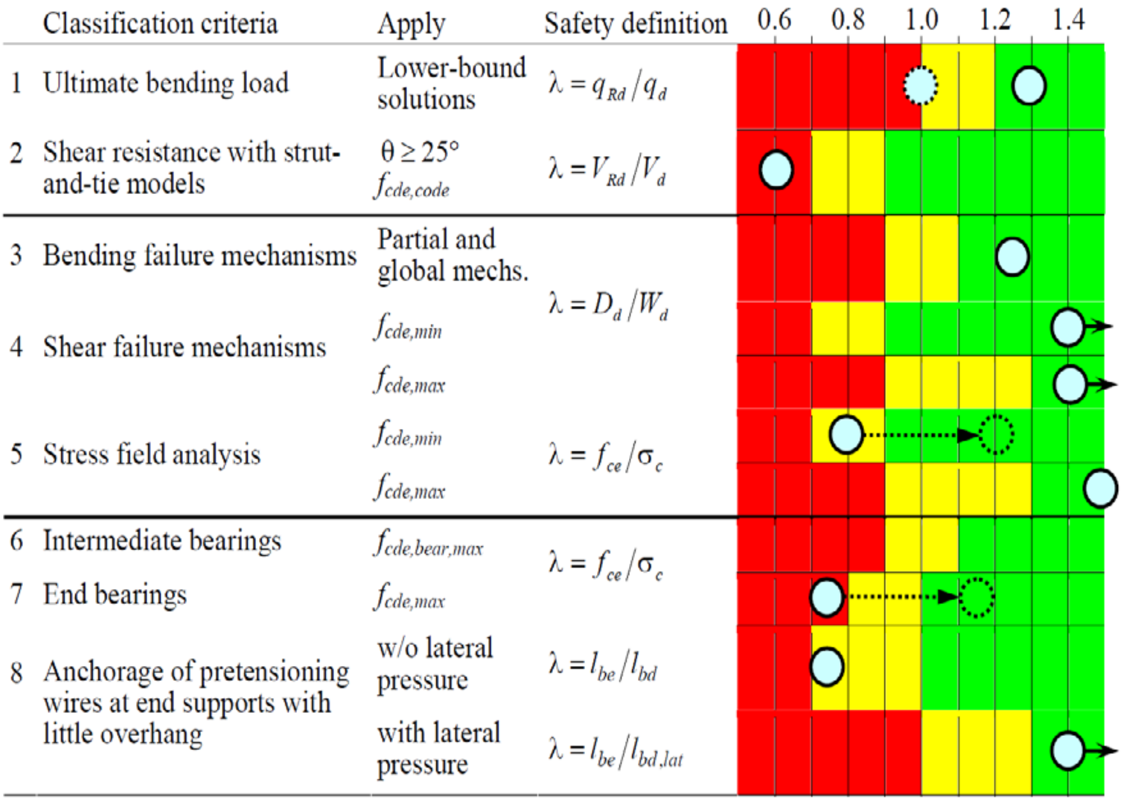

Fig. 9. Structural safety classification system [16]

Application of limit analysis method

The application of the failure mechanism corresponds to the upper limit method of limit analysis, comparing the energy dissipated Dd with the work of the external load Wd.

Ultimate bending load

The limit bending load q_{Rd} of the whole system is calculated according to the lower limit theory of limit analysis. They can be found by assuming the bending resistance M_{Rd} of the entire cross section in the maximum stress section and installing a maximum possible envelope between them so that the flexural strength will not be exceeded anywhere in the structure.

Shear resistance

It is assumed that the tensile strength V_{Rd} is determined by the strut-tie model of the web concrete, and it is assumed that the stirrup of the present invention has been fully utilized and the tilt angle θ of the strut is limited to 25 °. Compare the stress in the web concrete with the limit values of the applied (national) regulations. As a guide value, it can be assumed that $f_{cde,code} = 0.4 * f_{cd}$, where $f_{cd} =$ design value of compressive strength of curved concrete. Then compare the design value of shear strength with the design shear force V_d.

Bending failure mechanisms

For compact cross-sections (e.g. box girder), the so-called inclined collapse crack failure mechanism should be studied, which is carried out without activating energy dissipation in the web concrete, considering the reduction of interlaced stirrups and longitudinal reinforcement. For structures with open cross-sections, such as flat girder bridges, partial failure mechanisms should be studied.

Shear failure mechanisms

These so-called web crushing failure mechanisms are primarily characterized by the activation of dissipation energy in the web concrete. For the effective compressive strength of the web concrete

$f_{cd} = (1.2 - 25\varepsilon_1) * f_{cc}^{2/3}$ and $f_{cc}^{2/3} \leq f_{cd} \leq 1.1 * f_{cc}^{2/3}$

where the major principal strain, $\varepsilon_1 = \varepsilon_x + (\varepsilon_x - \varepsilon_2) * \cot^2 \theta$ can be calculated from the longitudinal strain, , ε_x the minor principal strain, ε_2 – usually taken as 0.002 –, and the inclination of the compression fields, θ, respectively. f_{cc} is the characteristic cylinder concrete compressive strength.

Stress field analysis

The most heavily loaded part of a structure can be easily determined from shear envelopes. The stress fields are developed assuming full exploitation of the present stirrup resistance and considering no restrictions for the compression field inclinations.

Chapter 4. Case study

The project is based on underground metro structural design. Shield Tunneling Method are used. Shield tunneling interval is from Futian station to Yitian parking lot in city Shenzhen, which is located in southeast of China.

4.1. Description of shield tunnel

This interval starts from a downhill slope of 2 ‰ and turns to an uphill slope of 25 ‰ at TCDK0 + 280.000. The buried depth of the tunnel is 9 ~ 18.5m. According to the geological survey report and on-site survey, the area where the interval tunnel is located is the sea alluvial plain landform, and the ground elevation is about 3.1 ~ 5.3m. The top of the tunnel is covered with soil about 5.5 ~ 18.8m. Interval tunnels basically pass through silty cohesive soil, medium sand, coarse sand, gravel, round gravel, pebble soil, gravelly cohesive soil, silty clay, fully-weathered granite. The bottom of the tunnel structure is mostly located in silty cohesive soil, medium sand, coarse sand, gravel, round gravel, pebble soil, full and strong weathered granite.

Fig 10 is the longitudinal section of shield tunnel interval. According to the unfavorable principle, the design should be carried out in the largest buried depth (18.5m). Fig 11 shows the soil stratification of this section. Table 9 is from the Geotechnical Investigation Report, shows basic properties of each layer.

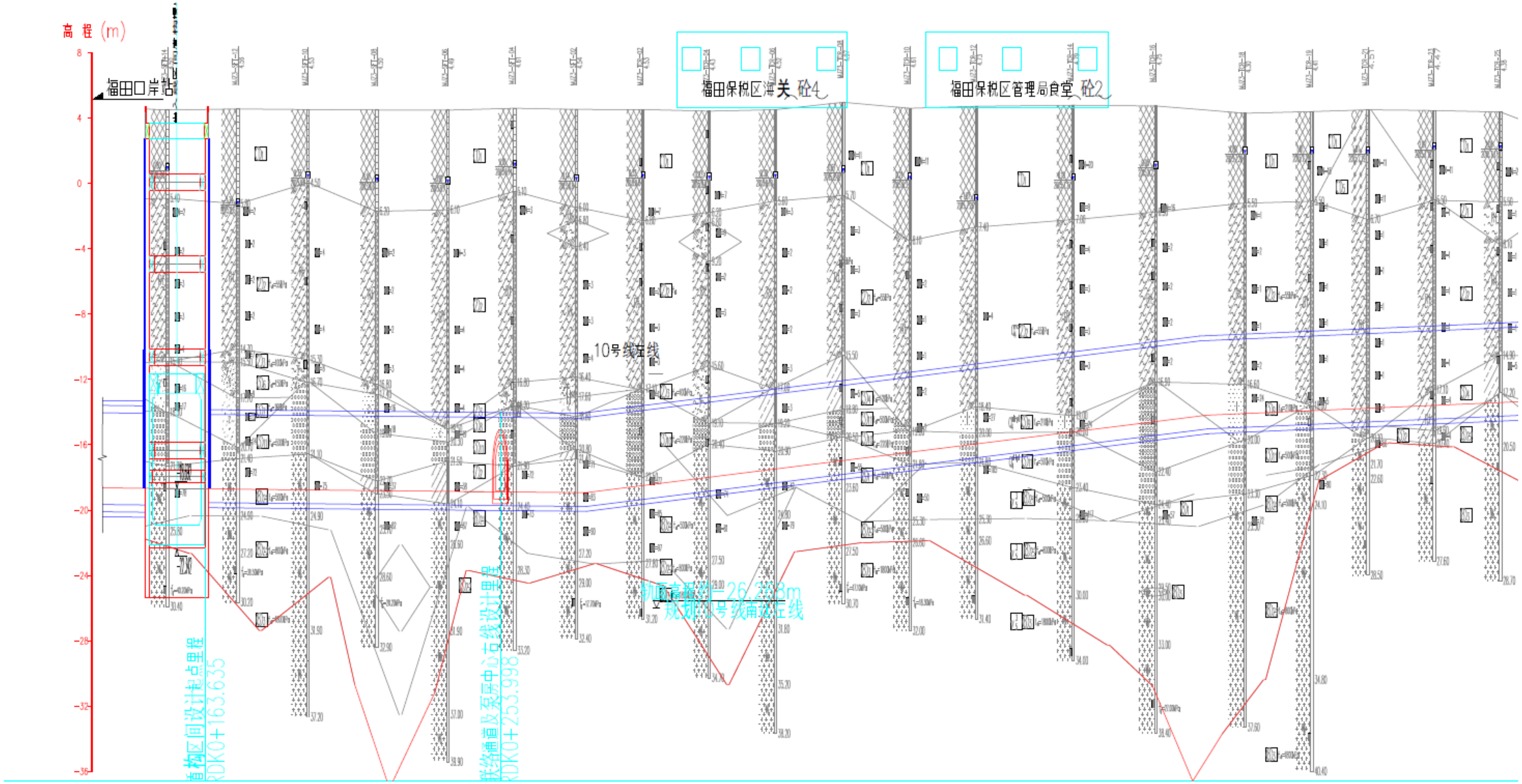

Fig 10. longitudinal section of shield tunnel interval

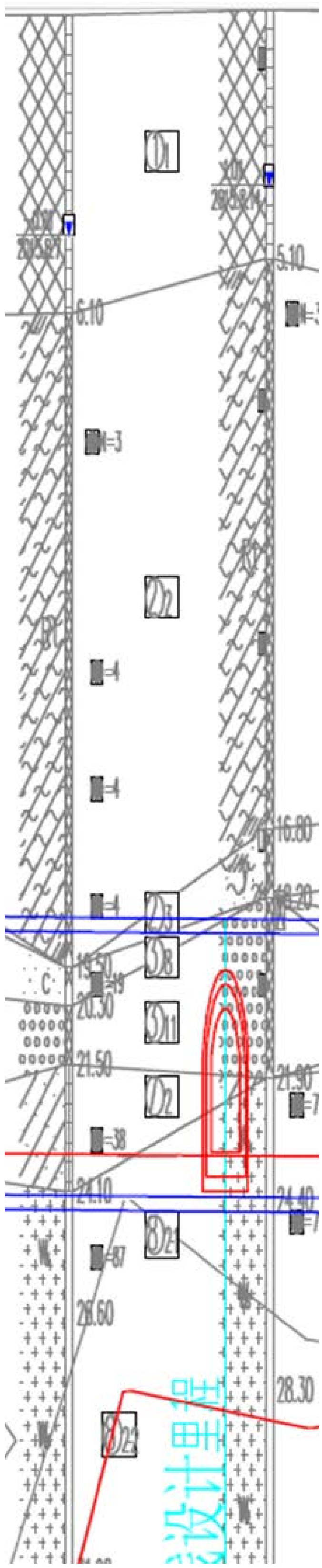

Fig 11. soil stratification

Stratum number	Geotechnical name	Natural water content ω (%)	Natural density per unit volume γ (kN/m^3)	Natural porosity ratio e_0	saturation Sr (%)	Liquid Limit $ω_L$ (%)	Plastic limit ωp (%)	Plasticity index I_P	Liquidity Index I_L
①$_1$	Plain fill		18.6						
①$_5$	Miscellaneous soil		20.8						
②$_1$	silt	60.8	16.1	1.71	96.6	51.9	28.3	23.64	1.37
②$_2$	Muddy clay	47.8	17.0	1.34	95.8	43.2	24.2	18.93	1.23
②$_3$	Muddy sand		16.0						
③$_2$	Muddy clay	52.7	16.9	1.46	98.3	51.1	30.5	20.60	1.08
③$_4$	Silty clay	27.9	19.5	0.79	95.5	31.6	18.9	12.74	0.67
③$_5$	Silt		19.0						
③$_6$	Fine sand		19.5						
③$_7$	Medium sand		19.6						
③$_8$	Coarse sand		19.8						
③$_9$	Gravel		20.0						
③$_{10}$	Round gravel		20.0						
③$_{11}$	Pebble		22.5						
⑦$_2$	Sandy clay	22.4	19.1	0.72	83.1	27.9	17.2	10.74	0.46
⑧$_1$	Fully weathered granite	16.2	19.4	0.61	72.3	23.8	15.2		
⑧$_{2-1}$	Sandy strongly weathered granite	20.0	18.9	0.69	76.4	24.5	15.6		
⑧$_{2-2}$	Massive strongly weathered granite		22.0						
⑧$_3$	Medium weathered granite		26.0						

Table 9. Basic properties of soil stratification

Principle and standard of calculation

The safety level of the interval structure is Grade 1, and the durability is designed according to the design service life of 100 years.

The waterproof level of the interval structure is Grade 2.

The maximum crack width of the reinforced concrete segment is: 0.2mm

The section tunnel lining structure is deformed, the diameter deformation is ≤0.4% ~

0.6% D (D is the outer diameter of the tunnel), and the opening of the ring longitudinal joint is within 2mm.

Fig 11 is cross-section with:

Shield tunnel inner diameter: 5.4m

Tunnel outer diameter: 6m,

Segment thickness: 0.3m,

Segment width: 1.5m.

According to Chinese national code, materials of shield tunnel are listed below:

Reinforced concrete segment

Concrete class: C50, P8

Steel bar: HRB400, HPB300

Steel: Q235B

Cover of segment reinforcement is 40mm outside and 25mm inside

Length of each segment: 5m

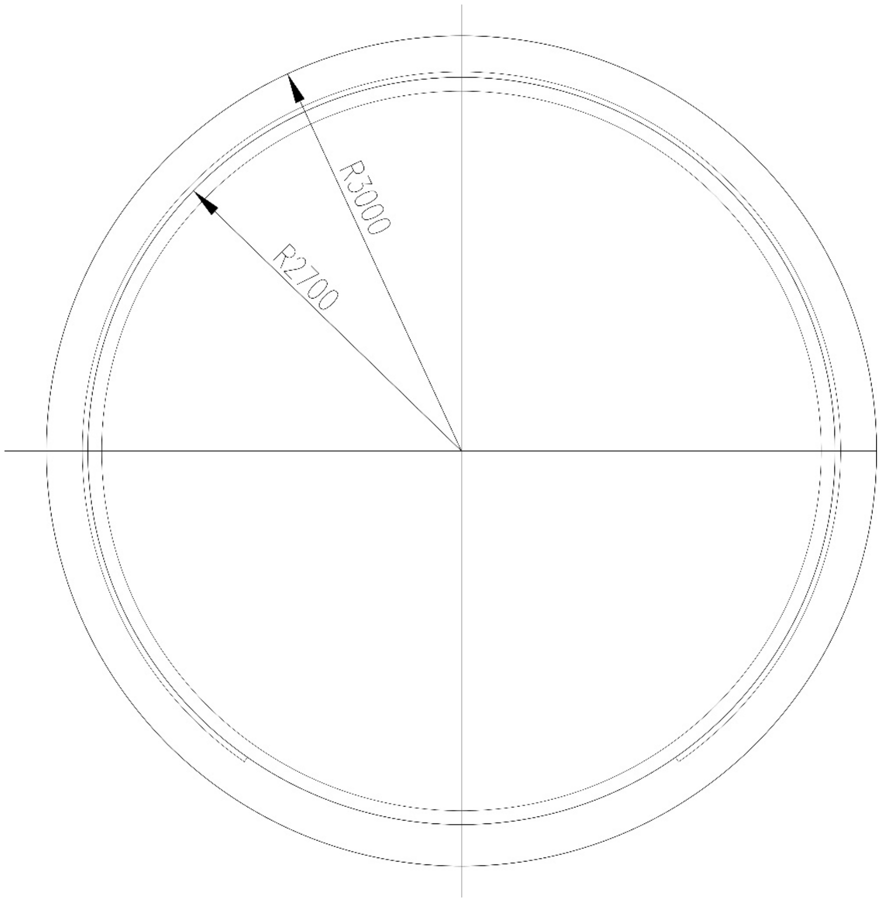

Fig. 12. Shield tunnel cross-section

4.2 Loads and loads combination

4.2.1 Loads

According to the former research, loads acting on the shield wall can be simulated by the interaction model which considers earth pressure, soil reaction, water pressure and self-weight.

Vertical earth pressure

Assuming that vertical earth pressure acts on the upper part of the segmented ring, there are two cases, one is to consider all overlying layers, and the other is to reduce the layer considering the shear strength of the soil. For both pressures, the load is assumed to be uniform. The latter is calculated according to Terzaghi's formula. The upward earth pressure at the bottom of the tunnel is the same in size and distribution as the downward earth pressure at the top.

Horizontal earth pressure

It is assumed that the horizontal earth pressure is a load that changes uniformly with increasing depth. Therefore, the horizontal earth pressure at the top of the ring is obtained by multiplying the vertical earth pressure by the horizontal earth pressure coefficient.

Water pressure

It is assumed that the water pressure on the tunnel acts in the direction of the center of the ring, and this pressure increases in the depth direction from the groundwater level.

Self-weight

So far, it is believed [17] that during the assembly of the tunnel ring in the shield machine, there is no support around the shield tunnel, and the residual stress in each segment during ring assembly is retained. Therefore, in the conventional shielding section design method, the bending moment due to its Self-weight has been calculated without considering the ground reaction force.

However, the bending moments due to the earth and hydrostatic pressure have been calculated taking account of the ground reaction. The sum of these calculated bending moments has been used to determine the material and the cross-sectional dimensions of the segments. As the research of Massimo and Ishimura, at the stage of assembling shield segments in the shield machine, small bending moments in the shield segments occurred due to their self-weight. The ground reaction to the self-weight of the segments could be considered by considering the reaction of a spring support.

4.2.2 Calculation model

The tunnel has a buried depth of 9 ~ 18.5m. The tunnel basically passes through silty clay, medium sand, coarse sand, gravel, round gravel, pebble soil, gravelly clay, silty clay, full and strong weathered granite. Design based on comprehensive consideration of various factors such as the buried depth of the tunnel, the stratum below it, and the surrounding structures ,the location of the poor stratum (coarse sand) and the largest buried depth (18.5m) is selected for reinforcement calculation.

The water pressure is calculated using full hydraulic pressure. The design uses the actual overlying soil height of 18.5m as the calculated soil column height. The lateral pressure coefficient is 0.33. The calculation model is built and shown in Fig 13.

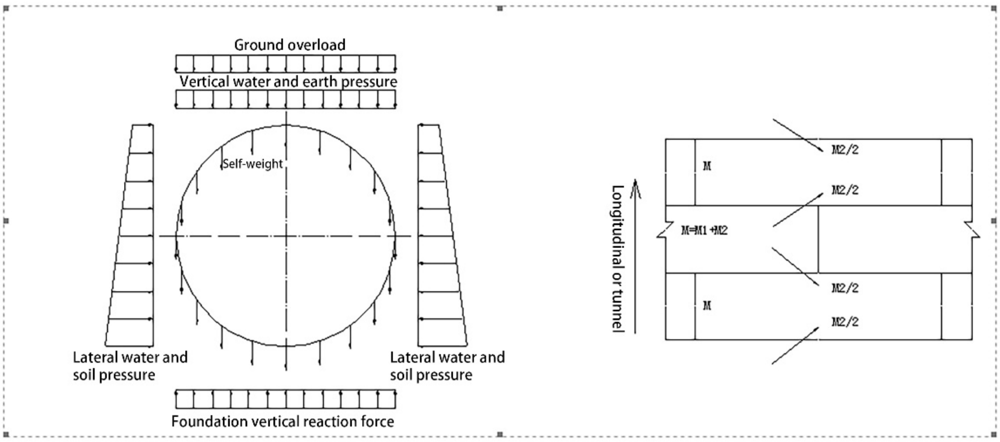

Fig. 13. Calculation model

4.2.3 Load combination

According to Chinese national code (GB5009-2012), basic combination is used to strength check and quasi-permanent combination is used to crack width check.

4.2.4 Anti-floating check

Take 1.5m in the longitudinal direction for calculation and take the cross section at 9m in the shallowest overlying soil for calculation.

Self-weight $= 8.055 \times 25 = 201kN$

Overburn $= 9 \times 10 \times 1.5 \times 6 = 810kN$

Floating force $= 3.14 \times 3^2 \times 10 \times 1.5 = 424kN$

Anti-floating coefficient (excluding side friction resistance) $= \frac{(201+810)kN}{424kN} > 1.05$

Meet anti-floating requirements.

4.4 Finite element analysis

To order to check verify the structure design of shield tunnel structure, finite element analysis of MATLAB coding is introduced. The shield tunnel structure is analyzed as plane section. Because the longitudinal dimension of the shield is much larger than the cross-sectional dimension, plain strain hypothesis is taken. The plane is meshed rough and fine uniformly into 320 nodes, 160 elements and 640 nodes, 320 elements, respectively, with a unit width B=1m. Considering the safety check of cross-section bearing capacity and crack width, basic load combination (1.35 to permanent load, 1.4 to variable load) and quasi-permanent load combination (1 to permanent load, 0.5 to variable load) are used.

4.4.1 Material definition

According the reference of physical properties of concrete, the elastic modulus and Poisson's ratio of concrete C50 are:

$$E = 3.45 * 10^4 MPa$$

$$\upsilon = 0.2$$

Where E is the elastic modulus, υ is Poisson's ratio of concrete C50.

4.4.2 Loads applied and constrains set

4.4.2.1 self-weight

According the reference of physical properties of concrete, the density of concrete C50 is $2500kg/m^3$, the unit weight $25kN/m^3$ is applied to every element as volume load.

4.4.2.2 Earth pressure and water pressure

Earth pressure and water pressure can be calculated as:

$$p_i = \gamma_i * h_i$$

Where i is i-th stratum and h_i is the height of i-th stratum, γ_i is the density per unit volume of i-th stratum. Data can be accessed according to Geotechnical Investigation Report.

4.4.2.3 Constrains

Three constrains are set to get a static determinate model in order to get symmetrical compressed and expanded deformations, constrains are set on the in both horizontal and vertical direction on left side of the tunnel and in the vertical direction on the right side of the tunnel.

4.4.3 Numerical results

The deformed configuration and stress distributions are shown below:

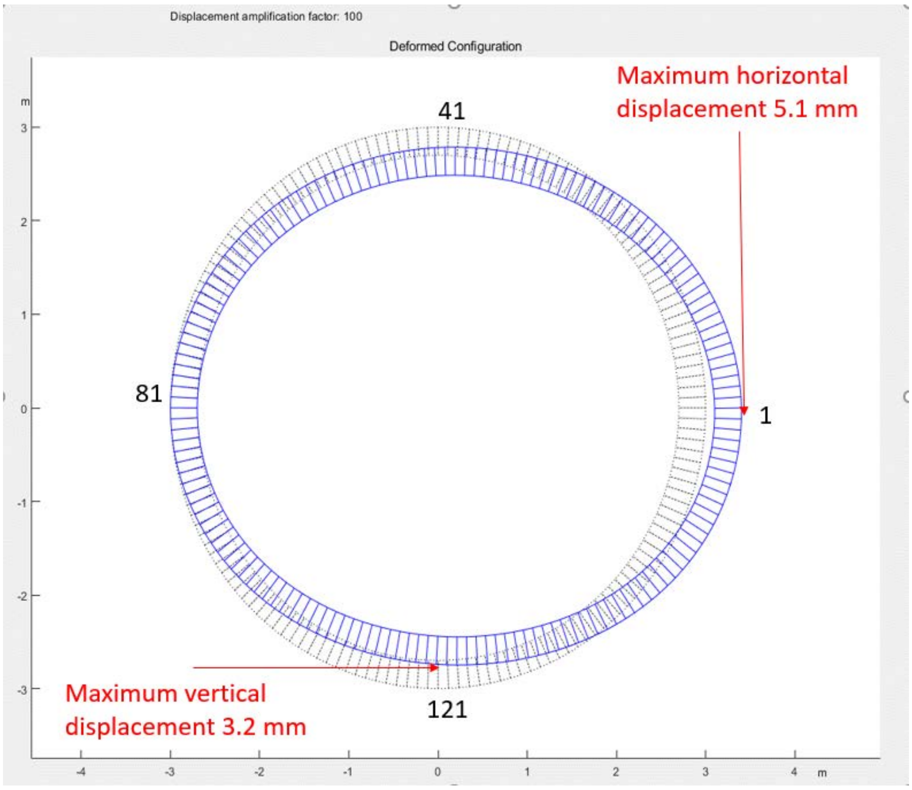

Fig. 14. Deformed Configuration (Basic Combination, Rough)

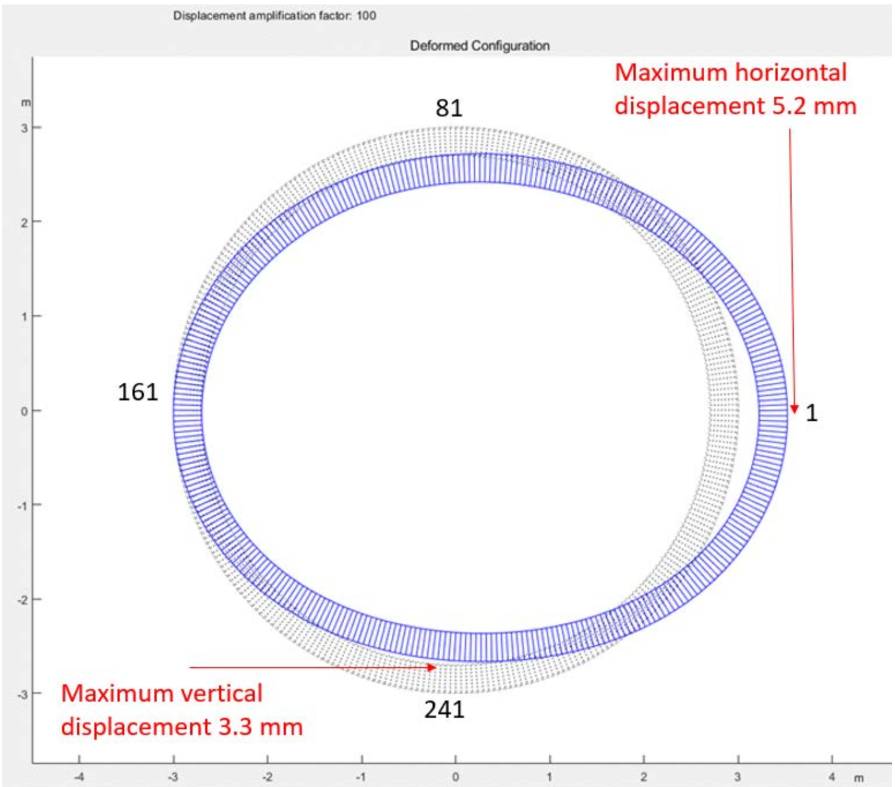

Fig. 15. Deformed Configuration (Basic Combination, Fine)

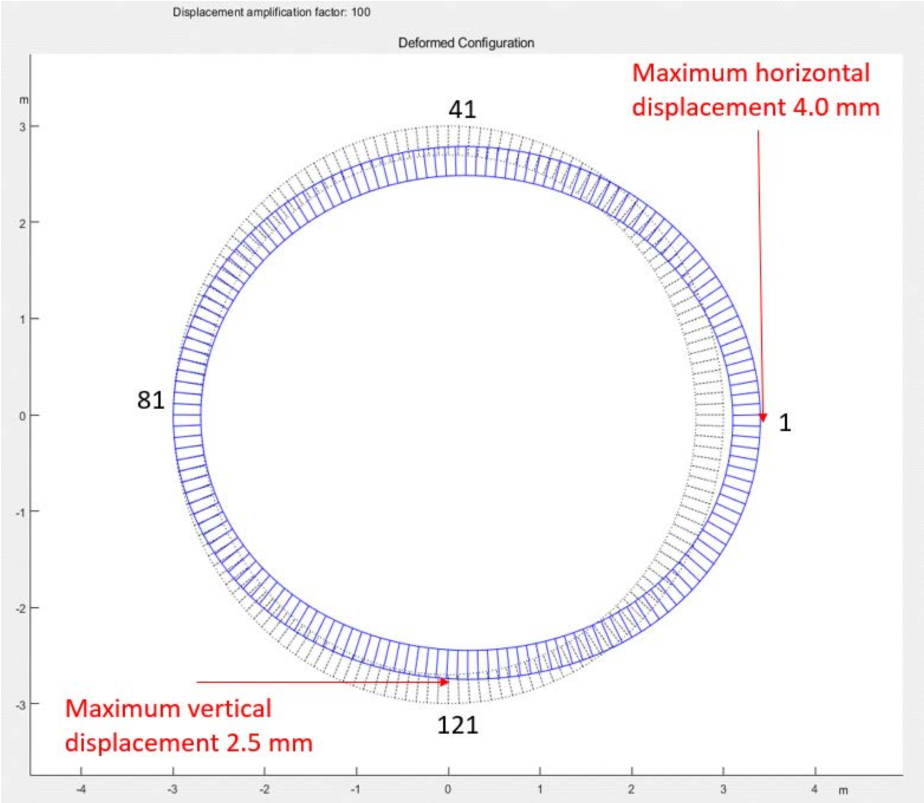

Fig. 16. Deformed Configuration (Quasi-permanent Combination, Rough)

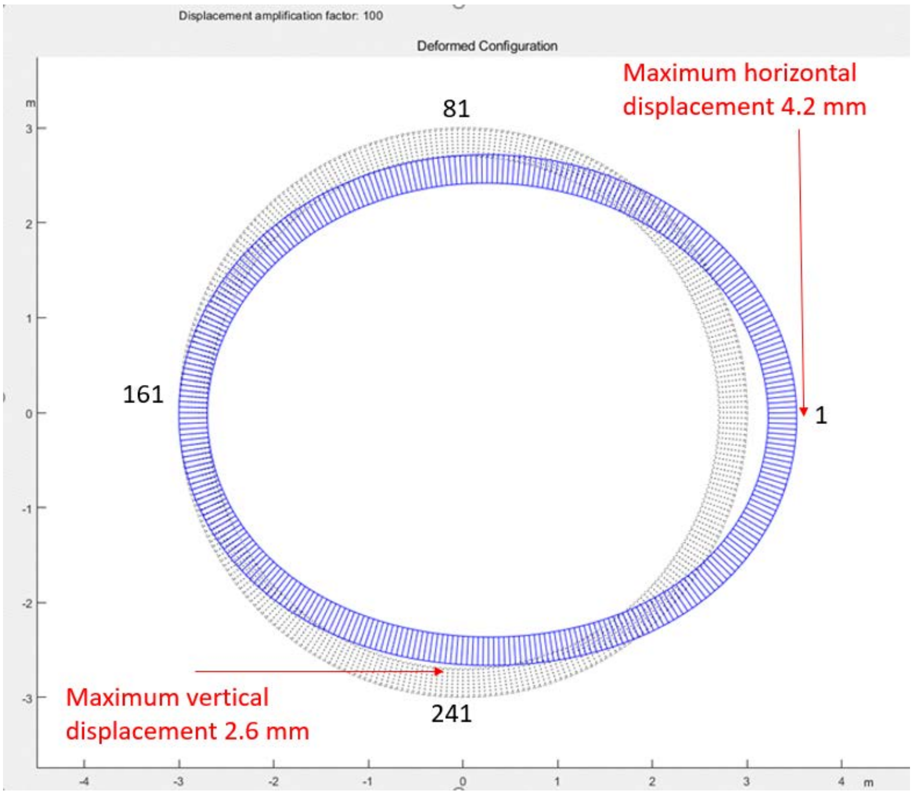

Fig. 17. Deformed Configuration (Quasi-permanent Combination, Fine)

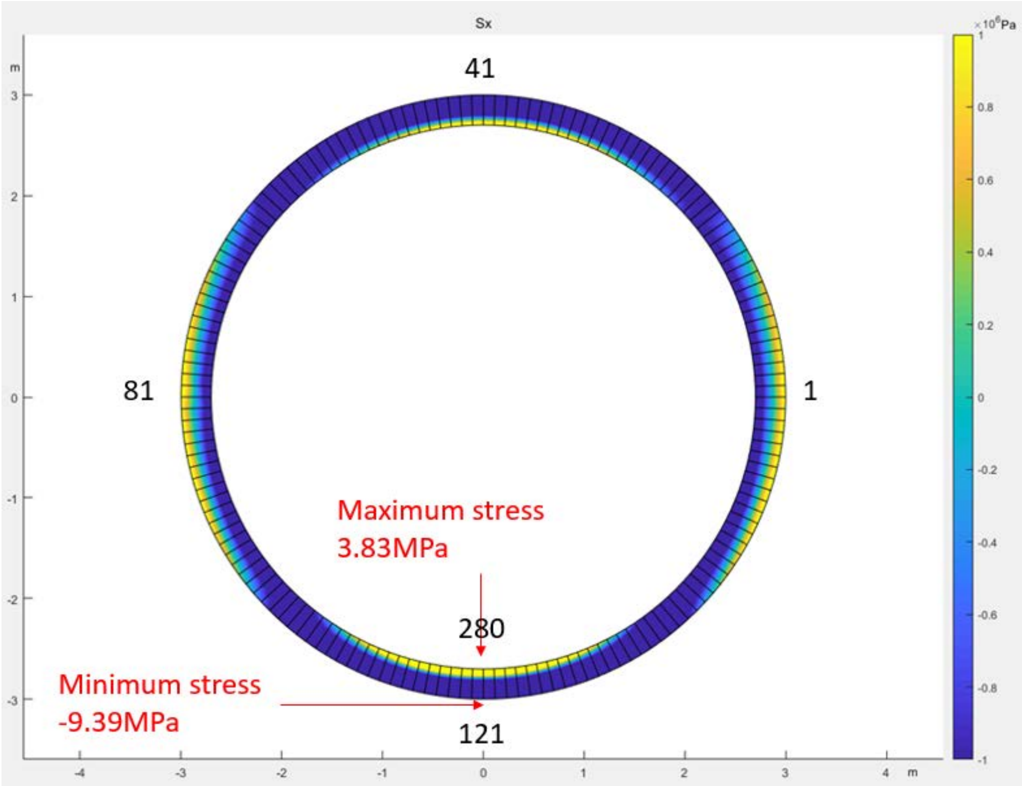

Fig. 18. σ_x (Basic Combination, Rough)

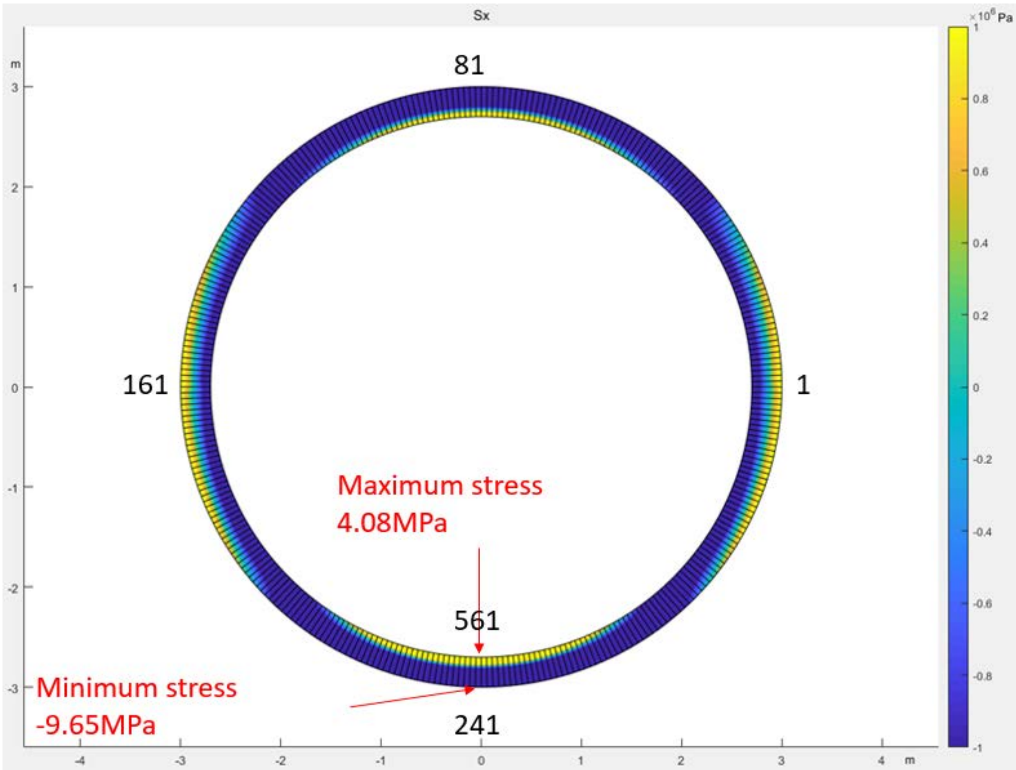

Fig. 19. σ_x (Basic Combination, Fine)

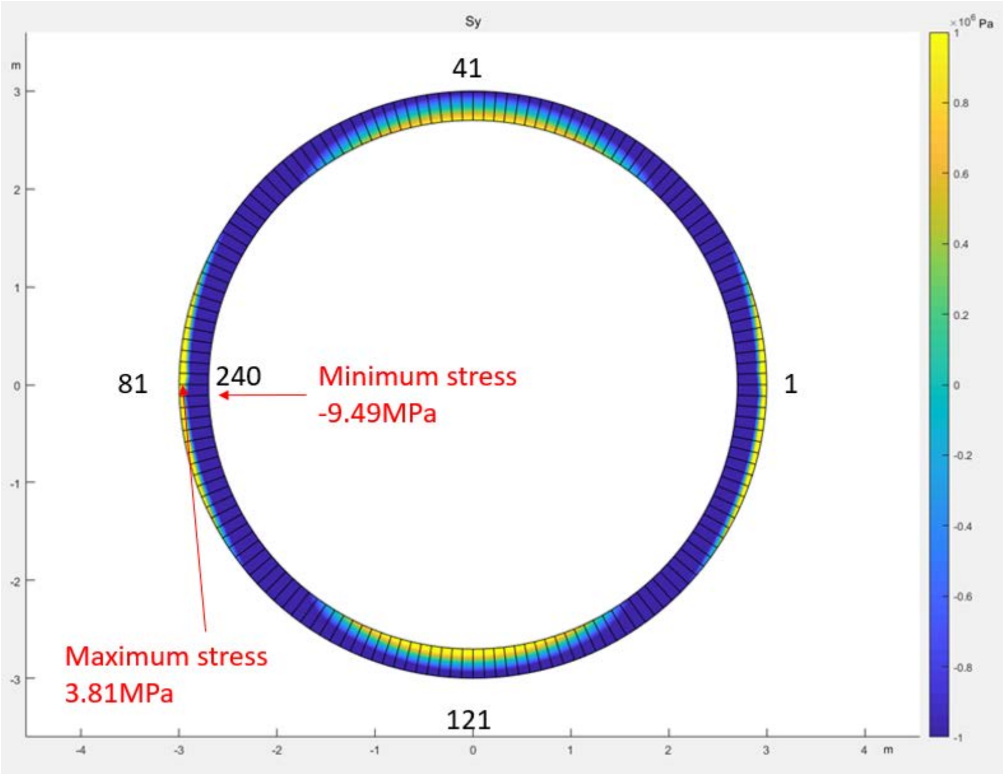

Fig. 20. σ_y (Basic Combination, Rough)

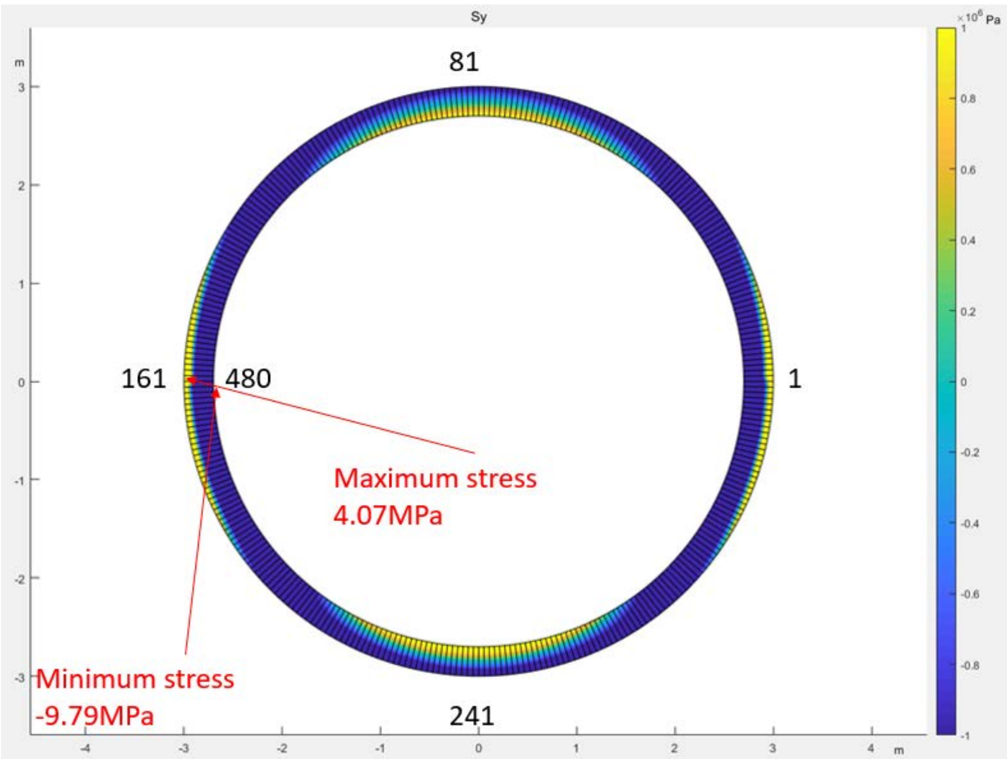

Fig. 21. σ_y (Basic Combination, Fine)

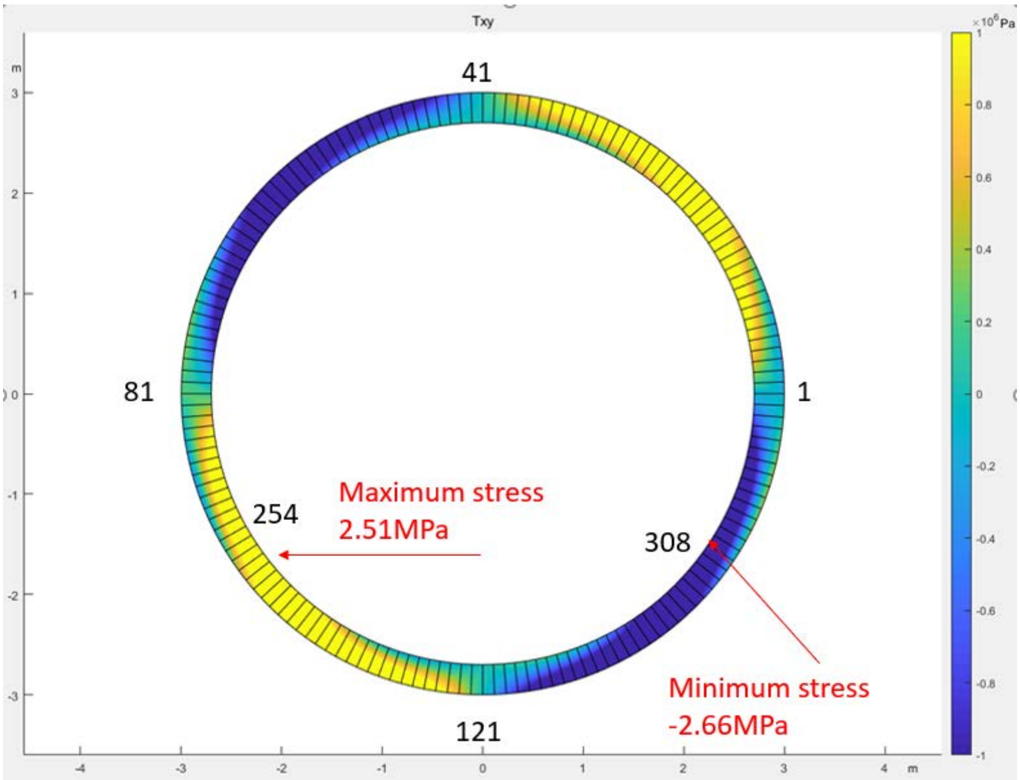

Fig. 22. τ_{xy} (Basic Combination, Rough)

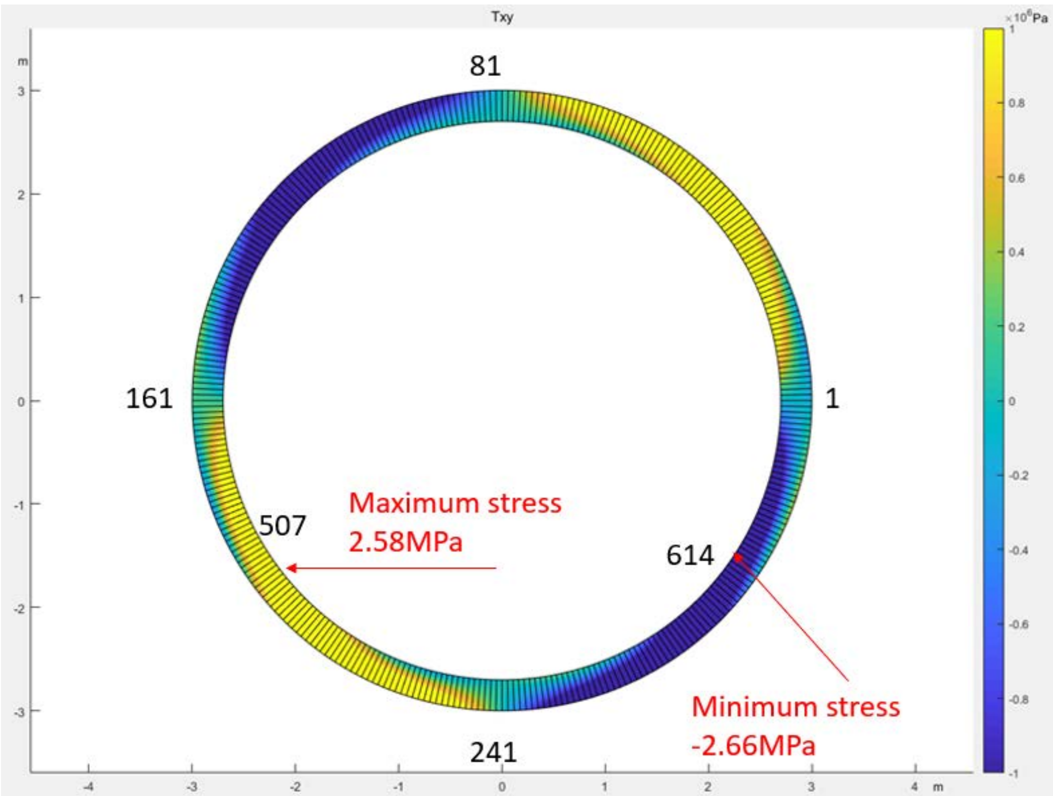

Fig. 23. τ_{xy} (Basic Combination, Fine)

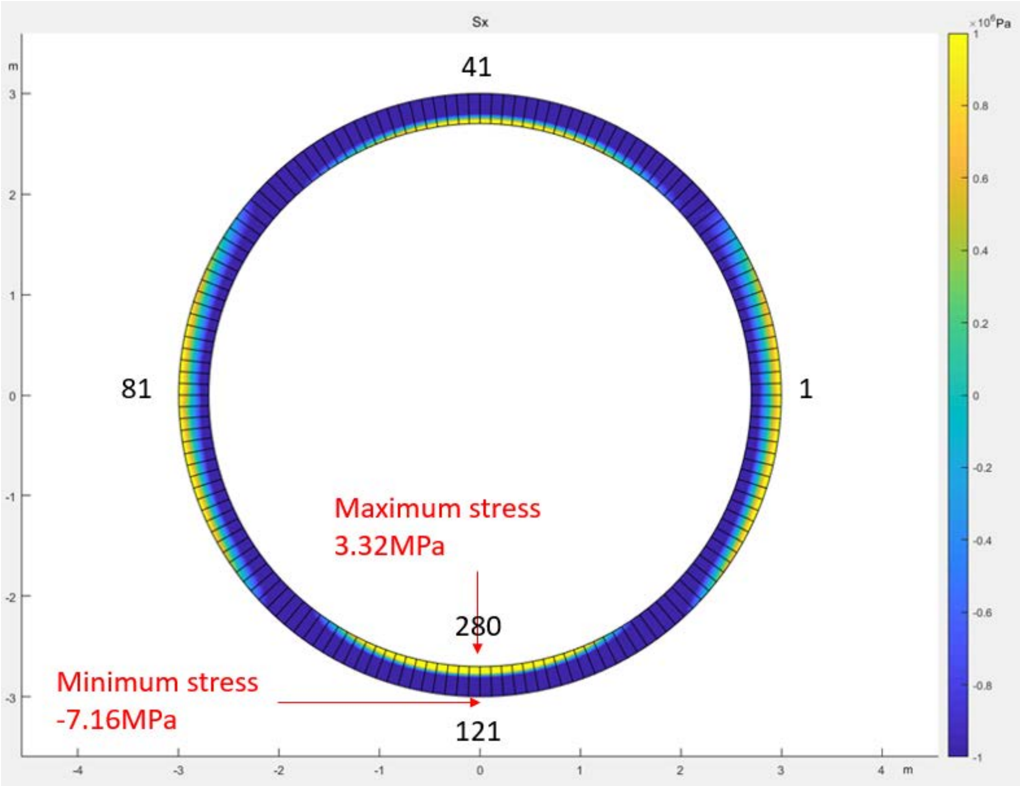

Fig. 24. σ_x (Quasi-permanent Combination, Rough)

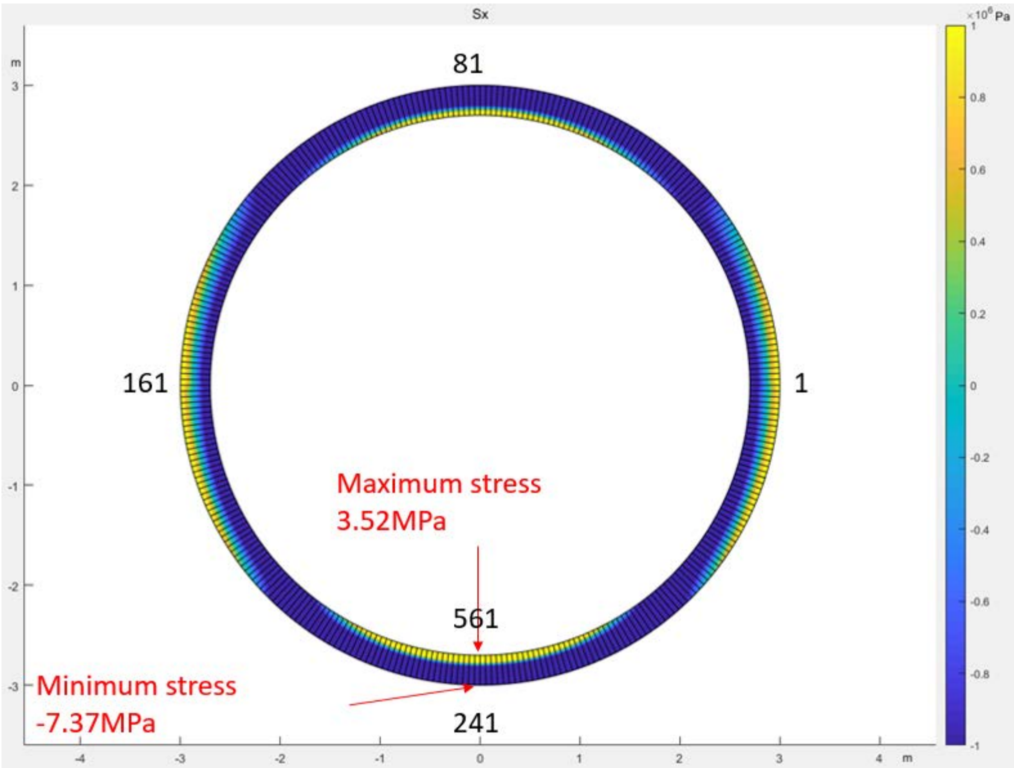

Fig. 25. σ_x (Quasi-permanent Combination, Fine)

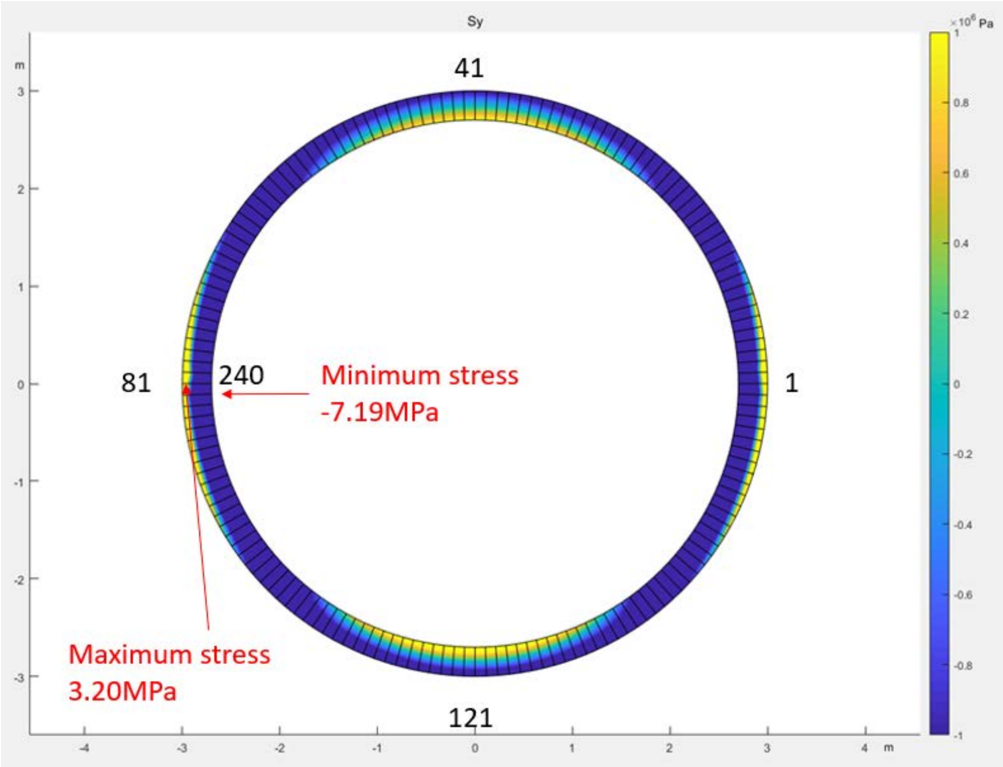

Fig. 26. σ_y (Quasi-permanent Combination, Rough)

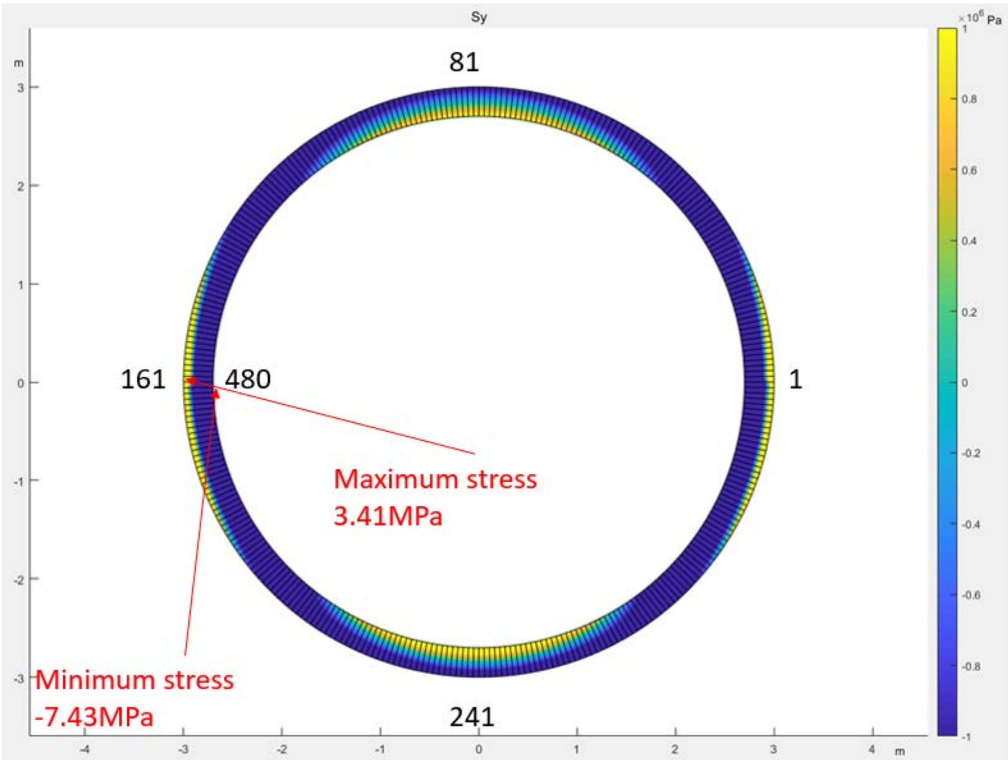

Fig. 27. σ_y (Quasi-permanent Combination, Fine)

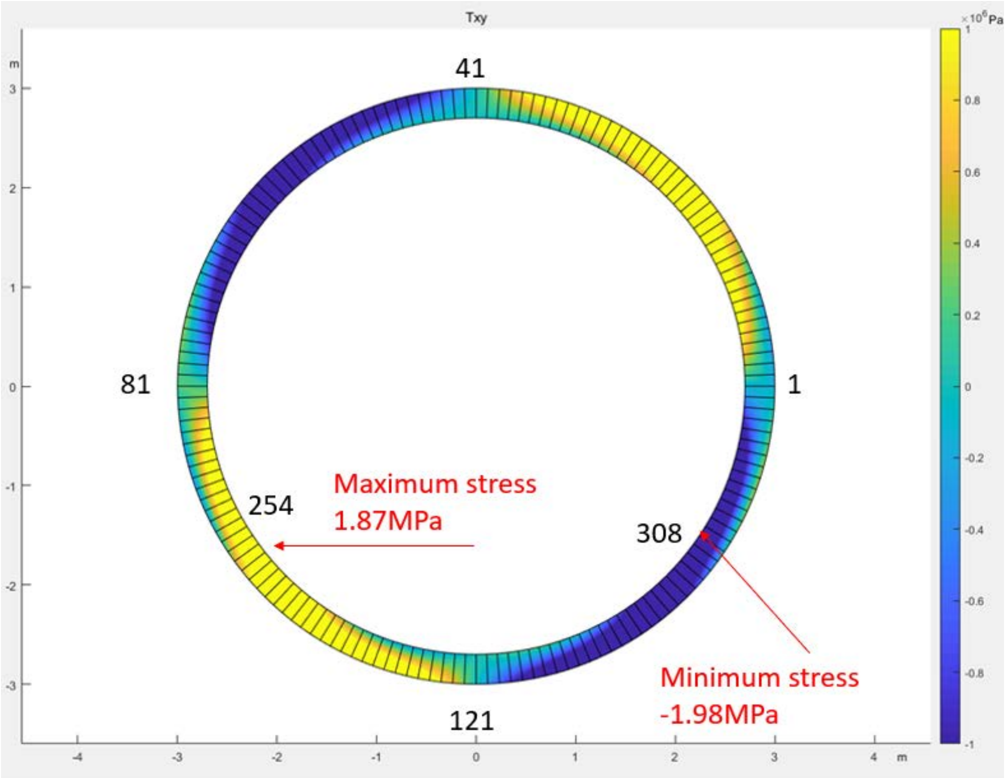

Fig. 28. τ_{xy} (Quasi-permanent Combination, Rough)

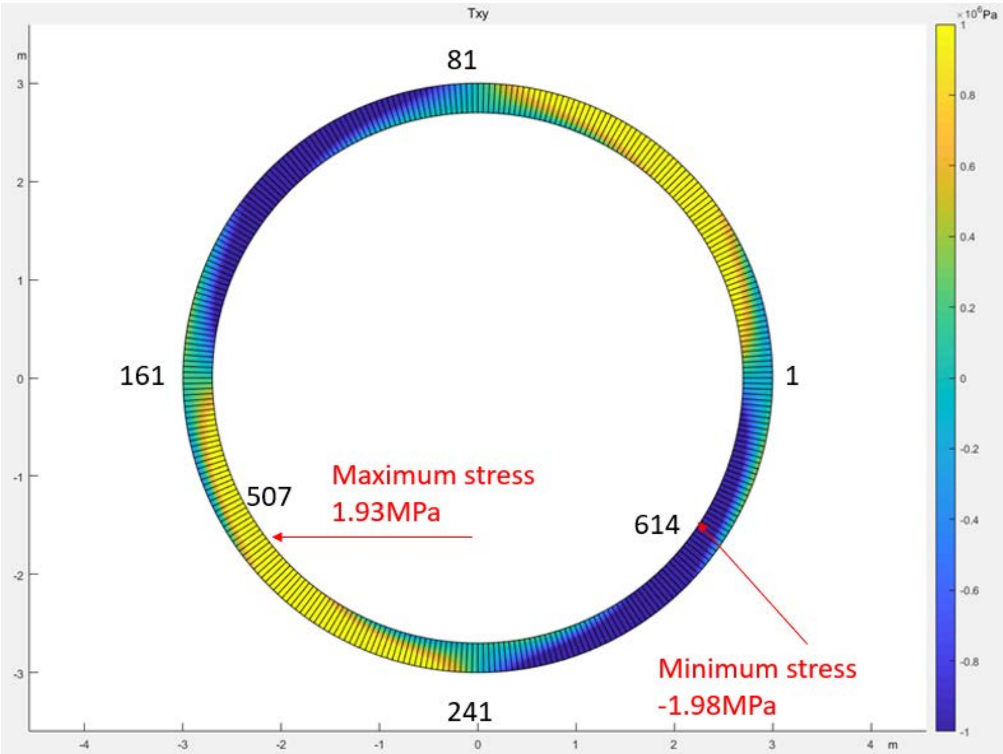

Fig. 29. τ_{xy} (Quasi-permanent Combination, Fine)

According to nodal stresses shown above, assuming that the internal force is linearly distributed, internal force can be introduced by equations:

$$N = \int \sigma_a dA$$

$$T = \int \tau_a dA$$

$$M = -\int \sigma_a y dA$$

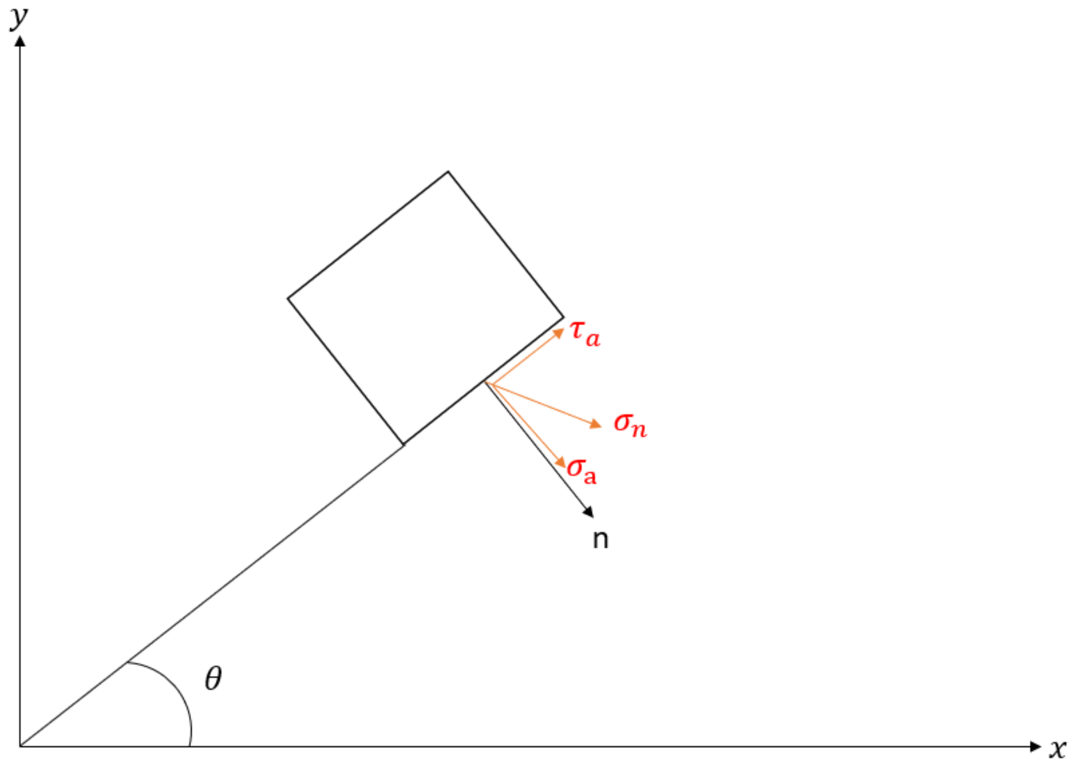

Fig. 30. Normal stress calculation model

$$\sigma_a = \frac{\sigma_x + \sigma_y}{2} - \frac{\sigma_x - \sigma_y}{2} cos2\alpha - \tau_{xy} sin2\alpha$$

$$\tau_a = \frac{\sigma_x - \sigma_y}{2} sin2\alpha - \tau_{xy} cos2\alpha$$

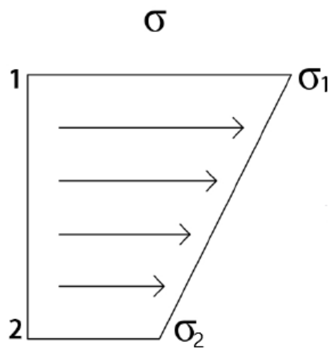

Fig. 31. Normal Stress distribution

Maximum and minimum bending moment and corresponding axial force, maximum shear stress and maximum displacements are shown in Table 10.

Critical	Basic Combination, Rough	Basic Combination, Fine	Quasi-permanent Combination, Rough	Quasi-permanent Combination, Fine
Maximum positive bending moment (KN*m)	200.5	218.0	141.3	152.5
Corresponding axial force (KN)	1250.3	1252.4	863.4	865.1
Maximum negative bending moment (KN*m)	159.3	177.9	117.1	122.0
Corresponding axial force (KN)	1475.3	1595.9	1107.3	1210.3
Maximum shear stress	247.5	255.1	195.2	201.3
Maximum displacement (mm)	5.1	5.2	4.0	4.2

Table. 10. Critical internal force and maximum displacement

4.5 Verification

4.5.1 Check of diameter deformation

The maximum displacement is 5.2 mm

Allowable deformation range is $0.4\% \times 6000mm \sim 0.6\% \times 6000 = 24mm \sim 36mm$

$$5.2mm < 24mm$$

Verified

4.5.2 Structural reinforcement calculation and verification

Compare the results of fine meshes and rough meshes, similar results presented, fine meshes output are used for reinforcement calculation. Convert the calculation result into a 1.5m wide segment structure for reinforcement calculation.

4.5.2.1 Checking calculation of cross-section bias strength

Reinforcement calculation on the inside of segment

For the inside of the segment, maximum bending moment under basic load combination M_u is $327(kN * m)$, and corresponding axial force N_u is 1879 kN.

The cross-section with $b = 1500mm, h = 300mm$. The concrete cover of inside of segment a_s is 40mm, outside of segment a_s' is 25mm. Calculation length $l_c = 5m$

The effective height of cross-section $h_0 = h - a_s = 260mm$.

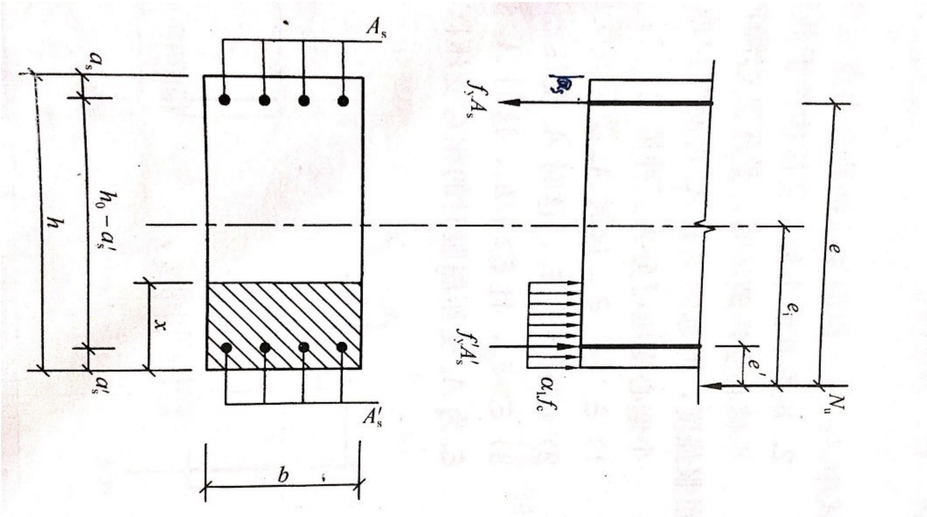

Fig. 32. Diagram of calculation of bearing capacity of large eccentric compression section

For concrete C50:

Coefficient $\alpha_1 = 1.0$;

Uniaxial compressive strength of concrete $f_c = 23.1\ N/mm^2$;

Limit of relative height of the compression zone $\xi_b = 0.518$;

Limit of height of the compression zone $x_b = \xi_b * h_0 = 134.7mm$.

For steel HRB400:

Strength of steel $f_y = 360\ N/mm^2$.

The eccentricity of the axial force to the center of gravity of the section:

$$e_0 = \frac{M_u}{N_u} = 174mm$$

The additional eccentricity

$$e_a = \max\left(\frac{h}{30}, 20\ mm\right) = \max(10{,}20\ mm) = 20mm$$

The initial eccentricity:

$$e_i = e_0 + e_a = 194mm$$

The distance between the point where the axial force acts and the total force point of the tensile bar A_s:

$$e = e_i + \frac{h}{2} - a_s = 304mm$$

Assuming that cross-sectional area of longitudinal reinforcement in compression zone is zero, $A_s' = 0$.

The height of concrete compression zone can be calculated by formula:

$$N_u e = \alpha_1 f_c b x (h_0 - x/2)$$

$$x = 74mm < x_b = 134.7mm$$

Calculated by large eccentric.

The cross-sectional area of longitudinal reinforcement in tension zone:

$$A_s = (\alpha_1 f_c b x - N_u)/f_y = 2304mm^2$$

Reinforcements on the inside of segment are 4 with diameter 22mm+ 8 with diameter 20 mm, the actual area of reinforcement is $4034mm^2$.

The actual height of concrete compression zone:

$$x = 96mm < x_b = 134.7mm$$

Large eccentric verified.

Reinforcement calculation on the inside of segment

For the outside of the segment, maximum bending moment under basic load combination M_u is $267(kN*m)$, and corresponding axial force N_u is 2394 kN.

The eccentricity of the axial force to the center of gravity of the section:

$$e_0 = \frac{M_u}{N_u} = 112mm$$

The initial eccentricity:

$$e_i = e_0 + e_a = 132mm$$

The distance between the point where the axial force acts and the total force point of the tensile bar A_s:

$$e = e_i + \frac{h}{2} - a_s = 242mm$$

Assuming that cross-sectional area of longitudinal reinforcement in compression zone is zero, $A_s' = 0$.

The height of concrete compression zone can be calculated by formula:

$$N_u e = \alpha_1 f_c bx(h_0 - x/2)$$

$$x = 75mm < x_b = 134.7mm$$

Calculated by large eccentric.

The cross-sectional area of longitudinal reinforcement in tension zone:

$$A_s = (\alpha_1 f_c bx - N_u)/f_y = 569mm^2$$

Reinforcements on the outside of segment are 12 with diameter 18 mm, the actual area of reinforcement is $3054\ mm^2$.

The actual height of concrete compression zone:

$$x = 100.8mm < x_b = 134.7mm$$

Large eccentric verified.

4.5.2.2 Check of crack width

Check of crack width on the inside of segment

For the inside of the segment, maximum bending moment under quasi-permanent load combination M_q is 228.8 $(kN * m)$, and corresponding axial force N_q is $1297\ kN$.

The actual area of reinforcement is $4034\ mm^2$

reinforcement modulus of elasticity $E_S = 200000N/mm^2$

Standard value of uniaxial tensile strength of concrete $f_{tk} = 2.64\ N/mm^2$

The distance from the outer edge of the outermost longitudinal tensile steel bar to the bottom edge of the tensile zone

$$c_s = 30mm$$

The reinforcement ratio of longitudinal tensile reinforcement calculated according to effective tensile concrete cross-sectional area

$$\rho_{te} = \frac{A_s + A_p}{A_{te}} = \frac{4034 + 4034}{1500 * 300} = 0.017929$$

The distance from the longitudinal tension bar resultant point to the compression zone resultant point

$$Z = 206mm$$

The distance between N_q and the resultant point of tensile rib A_s

$$e = 319mm$$

Equivalent stress of longitudinal tensile reinforcement of prestressed concrete members

$$\sigma_s = \frac{N_q(e-z)}{A_s z} = 169N/mm^2$$

The coefficient of strain non-uniformity of longitudinal tensile reinforcement between cracks

$$\Psi = 1.1 - 0.65\frac{f_{tk}}{\rho_{te}\sigma_s} = 0.5337$$

relative bonding characteristic coefficient $v = 1.0$

The equivalent diameter of longitudinal reinforcement in tension zone

$$d_{eq} = \frac{\sum n_i d_i^2}{\sum n_i v_i d_i} = (4*22^2 + 8*20^2)/(4*22 + 8*20) = 20.7\text{mm}$$

For rectangular eccentric compression member, characteristic coefficient of component stress $\alpha_{cr} = 1.9$

$$w_{max} = \alpha_{cr}\Psi\frac{\sigma_s}{E_s}\left(1.9c_s + 0.08\frac{d_{eq}}{\rho_{te}}\right) = 0.19mm \leq 0.20mm$$

Verified

Check of crack width on the outside of segment

For the inside of the segment, maximum bending moment under quasi-permanent load combination M_q is 183 $(kN*m)$, and corresponding axial force N_q is $1815\ kN.$

The actual area of reinforcement is $3054\ mm^2$

reinforcement modulus of elasticity $E_S = 200000N/mm^2$

Standard value of uniaxial tensile strength of concrete $f_{tk} = 2.64\ N/mm^2$

The distance from the outer edge of the outermost longitudinal tensile steel bar to the bottom edge of the tensile zone

$$c_s = 30mm$$

The reinforcement ratio of longitudinal tensile reinforcement calculated according to effective tensile concrete cross-sectional area

$$\rho_{te} = \frac{A_s + A_p}{A_{te}} = \frac{3054 + 3054}{1500 * 300} = 0.013573$$

The distance from the longitudinal tension bar resultant point to the compression zone resultant point

$$Z = 192mm$$

The distance between N_q and the resultant point of tensile rib A_s

$$e = 247mm$$

Equivalent stress of longitudinal tensile reinforcement of prestressed concrete members

$$\sigma_s = \frac{N_q(e - z)}{A_s z} = 159N/mm^2$$

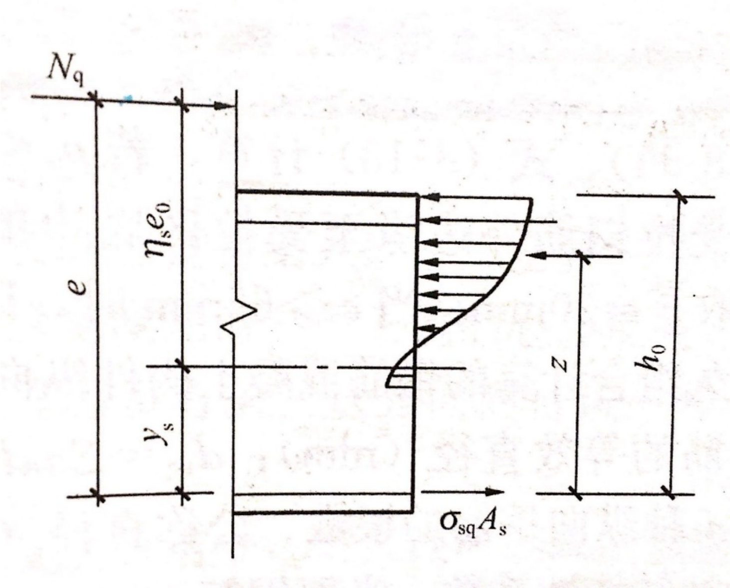

Fig. 33. Calculation diagram for eccentric compression member

The coefficient of strain non-uniformity of longitudinal tensile reinforcement between cracks

$$\Psi = 1.1 - 0.65\frac{f_{tk}}{\rho_{te}\sigma_s} = 0.3028$$

relative bonding characteristic coefficient $v = 1.0$

The equivalent diameter of longitudinal reinforcement in tension zone

$$d_{eq} = \frac{\sum n_i d_i^2}{\sum n_i v_i d_i} = (12 * 18^2)/(12 * 18) = 18\text{mm}$$

For rectangular eccentric compression member, characteristic coefficient of component stress $\alpha_{cr} = 1.9$

$$w_{max} = \alpha_{cr}\Psi\frac{\sigma_s}{E_s}\left(1.9c_s + 0.08\frac{d_{eq}}{\rho_{te}}\right) = 0.13mm \leq 0.20mm$$

Verified

4.5.2.3 Rectangular section shear strength check

The maximum shear force is $382.7\ kN$

Tensile strength of transverse reinforcement $f_{yv} = 360\ N/mm^2$

For C50 concrete

The design value of uniaxial tensile strength of concrete $f_t = 1.89\ N/mm^2$

The concrete strength influence coefficient $\beta_c = \ 1.0$

Check the dimension of rectangular

$$0.25\beta_c f_c b h_0 = 2252kN > 382.7kN$$

Verified.

The shear strength of cross-section without web reinforcement

$$V_c = 0.7 * f_t b h_0 = 516kN > 382.7kN$$

Reinforcement arranged according to minimum ratio of stirrups

$$\rho_{sv,min} = 0.24 * \frac{f_t}{f_{yv}} = 1.26 * 10^{-3}$$

The arrangement of stirrups should be

$$\frac{A_{sv}}{s} = \rho_{sv,min} * b = 1.89mm$$

10@200 (5 ribs per meter in the longitudinal direction of the segment)

$$\frac{A_{sv}}{s} = 1.96mm > 1.89mm$$

Verified

The calculation results are listed in following tables.

Checking calculation of cross-section bias strength											Take $A_s' = 0$ distinguish Large / small eccentricity			Single-sided reinforcement	
Position	$M(kN*m)$	$N\ (kN)$	$B\ (mm)$	$H\ (mm)$	$h_0(mm)$	$l_0(mm)$	$f_y(N/mm^2)$	Concrete class	$\alpha_1 f_c$	$e\ (mm)$	$x\ (mm)$	ξ_b	Large / small eccentricity	$A_s\ (mm^2)$	$\rho(\%)$
Maximum positive bending moment (bottom of tunnel)	327.0	1879	1500	300	260	5000	360	C50	23.1	304	74	0.518	Large eccentricity	2304	0.42
Maximum negative bending moment (slides of the tunnel)	266.9	2394	1500	300	260	5000	360	C50	23.1	241	75	0.518	Large eccentricity	569	0.13

Table. 11.Calculation table of section bearing capacity (basic combination)

Checking Computation of Crack Width of Bias Pressure																		
Position	M $(kN*m)$	$N\ (kN)$	B (mm)	H (mm)	$h_0(mm$	$l_0(mm)$	$20<c<65$	d_{eq} (mm)	A_s (mm^2)	Class	f_{tk} $(N/mm2)$	e (mm)	Z (mm)	σ_{sk} $(N/mm2)$	ρ_{te}	ψ	ω_{max} mm	ρ (%)
Maximum positive bending moment (bottom of tunnel)	228.8	1297	1500	300	260	5000	30	20.7	4034	50	2.64	319.2858	206	169	0.017929	0.5337	0.19	0.90
Maximum negative bending moment (slides of the tunnel)	183.0	1815	1500	300	260	5000	30	18	3054	50	2.64	246.8056	192	159	0.013573	0.3028	0.13	0.68

Table. 12.Cross-sectional crack width check table (quasi-permanent combination)

Rectangular section shear strength check calculation (non-concentrated load action; f_{yv} is f_y, $f_{yv} \leq 360$)											
V(KN)	B(mm)	H(mm)	h_0(mm)	f_{yv}(N/mm^2)	Concrete class	$\beta_c f_c$	f_t	V_c (KN)	Check cross-section	$\frac{A_{sv}}{s}(mm)$	Shear resistance without reinforcement (KN)
382.7	1500	300	260	300	50	23.1	1.89	2252	Satisfied	1.96	516

Table. 13. Rectangular section shear strength check

According to calculation considering the geological conditions, burial depth and location of the structure, the tunnel is in a weak stratum with a maximum burial depth of 18.5m. The reinforcements on the inside are 4 with diameter 22 mm + 8 with diameter 20 mm, on the outside 12 with diameter 18 mm.

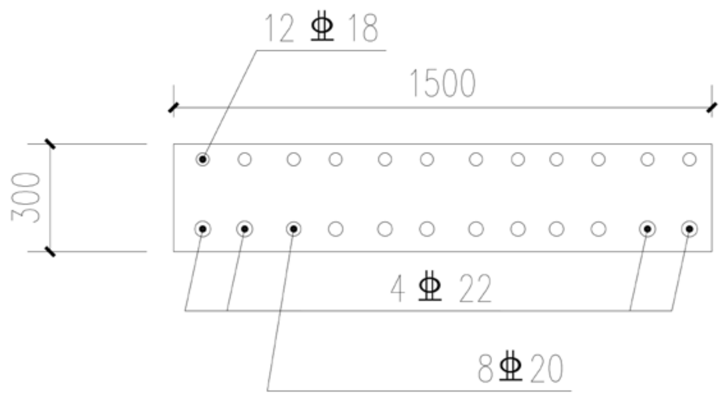

Fig. 32. Arrangement of reinforcement

Chapter 5. Conclusion

In the era of rapid development of China's subway, the shield tunnel method is widely used as the main construction method. The complex environment of geotechnical engineering and the particularity of shield construction methods make accidents happen from time to time. Safety assessment of shield tunnel becomes particularly important.

Safety assessment of shield tunnel can refer to:

1) Design of shield tunnel lining

2) Construction phase

3) The conditions of existing shield tunnel

For each aspect, safety criteria are introduced.

In addition to the requirement of codes and standards, the stress distribution and deformation of the structure are the basis of the design. Calculations by the finite element method return reasonable results in terms of stress distribution and deformation.

In the presented case study, two different meshes are used to check the accuracy of the numerical simulations. Small differences in different meshes lead to high accuracy.

From results, excessive designs referring to the concrete class (C50), dimension of cross-section (1.5m width and 0.3 height) and area of reinforcement show in the aspects of displacement of structure and cross-section bearing capacity.

The design with concrete class C50 leads to a small displacement (maximum 5.2mm) compared to allowable displacement (range from 24mm to 36mm).

The actual design area of reinforcement in inside (outside) of segment is $4034\ mm^2$ ($3054\ mm^2$)which is higher than what is calculated in cross-section bearing capacity

$2304\ mm^2$ ($569\ mm^2$). And in the shear resistance check, the cross-section shear bearing capacity (2250kN) is much higher than the maximum shear stress(382.7kN).

As the demand of crack width check (margin is 0.01mm) as well as minimum single-side reinforcement ratio (0.2%) and minimum total reinforcement ratio (0.55%), excessive reinforcement designs and dimension designs in displacement of structure and cross-section bearing capacity are necessary.

In design stage, approach of safety assessment of shield tunnel lies on how to convert a real problem into checking whether the stress distributed on shield lining have exceeded the limit.

In construction stage, engineer focus more on the ground settlement and stratum deformation, approach of safety assessment of shield tunnel lies on how to monitor and measure the deformation of formation.

Reference

[1] Mengshu Wang. Discussion on the Development of Tunnels and Underground Space in China in the 21st Century, Journal of Railway Science and Engineering, Vol. 1, 2004, Page 7-8.

[2] Yong Wang. A Statistical Analysis of Metro Construction Accidents in China, Journal of Engineering Management, Vol. 32, 2018, Pages 2-4.

[3] Yukinori Koyama, Present status and technology of shield tunneling method in Japan, Tunnelling and Underground Space Technology, Vol. 18, 2003, Pages 145-159.

[4] Hehua Zhu, Beam-spring system model for analysis of lining force of shield tunnel, Rock and Soil Mechanics, Vol. 19, 1998, Pages 27-31.

[5] Chih-Chieh Lu, Jin-Hung Hwang, Safety assessment for a shield tunnel in a liquefiable deposit using a practical dynamic effective stress analysis, Engineering Failure Analysis, Vol. 102, 2019, Pages 369-383.

[6] Chih-Chieh Lu, Jin-Hung Hwang, Nonlinear collapse simulation of Daikai Subway in the 1995 Kobe earthquake: Necessity of dynamic analysis for a shallow tunnel, Tunneling and Underground Space Technology, Vol. 87,2019, Pages 78-90.

[7] Yong Shu, Research on Calculation Method and Controlling Measures of Ground Settlement Induced by Subway Shield Tunneling Construction (in Chinese). Nanchang Hang kong University, 2014, Pages 12-30.

[8] Glenda Abate, Sebastiano Corsico, Maria Rossella Massimino, FEM Modelling of the Seismic Behavior of a Tunnel-soil-Aboveground Building System: A Case History in Catania (Italy), Procedia Engineering, Vol. 158, 2016, Pages 380-385.

[9] YOU Pengfei, Mou Ruifang, Safety assessment model of metro EPB shield tunnel construction, Chinese Journal of Rock Mechanics and Engineering, Vol. 29,2010, Pages 2663-2668.

[10] CHEN Kui, HONG Kairong, Shield Construction Technology, China communication press,2009, Pages 164-172.

[11] Beijing Planning Commission, Code for design of metro (GB 50157-2013), China Construction Industry Press, 2013.

[12] Ministry of Housing and Urban-Rural Development of the People's Republic of China, Code for design of concrete structures (GB50010-2010), China Construction Industry Press, 2010.

[13] Ministry of Housing and Urban-Rural Development of the People's Republic of China, Load code for the design of building structures (GB5009-2012), China Construction Industry Press, 2012.

[14] Guidelines for the design of shield tunnel lining, Tunneling and Underground Space Technology, Vol. 15, 2000, Pages 303-331.

[15] Ministry of Housing and Urban-Rural Development of the People's Republic of China, Code for construction and acceptance of shield tunnelling method (GB50446-2017), China Construction Industry Press, 2017.

[16] Zwicky, Daia, Structural Safety Assessment and Classification of Concrete Structures. IABSE Symposium Report, 2005. Pages 40-47.

[17] H. Mashimo, T. Ishimura, Evaluation of the load on shield tunnel lining in gravel, Tunneling and Underground Space Technology, Volume 18, 2003, Pages 233-241.