Guidelines for Tunnel Lining Design





Foreword

This guideline consists of 2 Parts.

Part 1	Design Guidelines For Precast Segmental Lining. (Contributed by John Poh)
Part 2	Design Of Sprayed Concrete Lining In Soft Ground. (Contributed by Goh Kok Hun)

Acknowledgements

The production of this *Guidelines For Tunnel Lining Design* was made possible not without much help. The authors are grateful to all the reviewers who have given their personal time freely and often with much great pressures on their time from their own personal work.

PART 1 – DESIGN GUIDELINES FOR PRECAST SEGMENTAL LINING

1.0 INTRODUCTION
1.1 Scope
1.2 Background
1.3 Design Principles
1.4 Definition of Terms
1.5 Notation

2.0 LOADS
2.1 Different kinds of loads
2.2 Ground Loading
2.3 Water Pressure
2.4 Dead Load
2.5 Surcharge

3.0 STRUCTURAL CALCULATIONS
3.1 Design Sections
3.2 Computation of Member Forces
3.2.1 Continuum Analytical Models
3.2.2 Bedded Beam Spring Mdel
3.2.3 Numerical Analysis Models
3.3 Evaluation of joints

4.0 DURABILITY CONSIDERATIONS
4.1 Fire Resistance
4.2 Waterproofing Systems

5.0 TUNNELLING IN CLOSE PROXIMITY

6.0 CONCLUSION

Figure 1 – Flow Chart Of Tunnel Lining Design
Checklist – Step by Step Design Procedure
Example 1

1.0 INTRODUCTION

1.1 Scope

These guidelines provide general requirements for the design of segmental linings made of reinforced concrete in soft ground. They can also be applied to segmental linings of rock tunnels which are excavated in earth or soft rock by Tunnel Boring Machine (TBM).

It will attempt to cover the design of structural linings for driven tunnels to be constructed in most types of ground conditions encountered in Singapore.

1.2 Background

A permanent tunnel lining is the final product of a process that involves planning and evaluation of user needs, geotechnical investigations, analysis of ground lining interaction, construction, and observations and modifications during construction. The designer has to consider the lining context of the many functional, construction, geotechnical requirements that dictate hot the lining is selected and built under practical circumstances. Only by understand how service criteria, construction methods, and geotechnical conditions interrelate within the prevailing system of engineering and contract practice can an effective philosophy of design be established. The handbook will attempt to cover the areas associated with tunnel linings to provide an appropriate background and practical orientation of the subject.

Tunnels provide transportation routes for mass rapid transit, railroads, vehicular traffic, convey both fresh and waste water, etc. They serve as passageways for pedestrians as well as conduits for utilities. Tunnels are built in many underground environments, including soil, mixed soil and rock, and rock, with variations in the ground water conditions, in-situ states of stress, geologic structures. Tunnels may be built using different construction methods including hand excavation, drill and blast method, and the use of a mechanised tunnel boring machine.

Given the wide variety of factors that influence tunnelling, it is difficult to specify any rules of thumb or give prescriptive performance indicators unless many site specific characteristics have been clarified concerning function, ground conditions and tunnelling methods. Experience is essential in this. During the concept or preliminary stages of design, input from experienced site engineers or contractor will enhance the conditions in which a constructable and cost effective lining can be built.

One major concern to a designer is to be able to define operational criteria for the tunnel. Setting up criteria requires review by upper management and senior technical staff. The designer should recognise that operational standards or requirements often will control the characteristics of the final product, including the type and dimension of the lining.

A tunnel lining is often selected based on operational criteria, reviewed according to construction methods, and finally checked according to predicted ground loads. The design may not be governed by the ground loads. As ground and lining are able to share loads when in firm and continuous contact, typically the structural requirements for carrying ground loads can be satisfied easily by many linings.

The use of analytical methods for designing linings should be based on the understanding that analytical precision may greatly exceed the precision with which the principal parameters of the ground can be known. Generally there is great variation in ground conditions along the tunnel route. The main virtue of the analytical studies is their ability to test the lining response to the range of anticipated conditions and to estimate the performance under upper and lower bound conditions. The designer should not use computational elegance as a substitute for judgement and experience.

The expense of a lining can vary substantially as a function of contract practices and specifications even though the lining type and dimensions remain fixed. Constructability is a feature of design that emphasises the practical and economic considerations in construction, It is one of the most important factors affecting cost, and should be a hallmark of the designer's approach to tunnel linings.

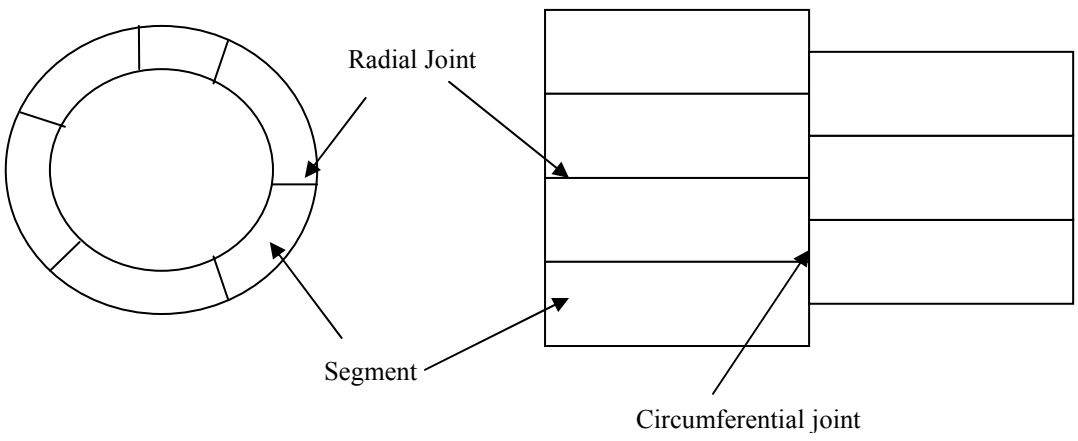
1.3 Design Principles

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

1.4 Definition of Terms

The following terms are defined for general use in this handbook

a) Segment : Arc shaped structural member for initial lining of shield tunnel.
b) Segmental lining : Tunnel lining constructed with segments; One ring of the lining comprises of a number of segments
c) Thickness : Thickness of the lining of the cross section of tunnel
d) Width : Length of segment in longitudinal direction
e) Joint : Discontinuity in the lining and contact surface between segments
f) Types of joints :
    - Plain joint
    - Hinge joint
g) Circumferential joint : Joint between rings
h) Radial joint : Joint between segments in longitudinal direction
i) Bolts for joints : Steel bolts to joint segments



1.5 Notation

The following notations may be used in the guidelines

t	Thickness
A	Area
E	Modulus of Elasticity
I	Moment of inertia of area
EI	Flexural rigidity
M	Moment
N	Axial force
S	Shearing force
D	Diameter
D_c	Diameter of centroid
R_o	Outer radius
R_c	Radius of centroid
R_i	Inner radius
γ	Weight of soil
γ'	Submerged unit weight of soil
γ_w	Unit weight of water
γ_c	Unit weight of concrete
H	Overburden
P_o	Surcharge
W	Weight of lining per metre in longitudinal direction
P_g	Dead load
P_{e1}	Vertical earth pressure at crown of lining
P_{w1}	Vertical water pressure at crown of lining
q_{e1}	Horizontal earth pressure at crown of lining
q_{w1}	Horizontal water pressure at crown of lining
P_{e2}	Vertical earth pressure at invert of lining
P_{w2}	Vertical water pressure at invert of lining
q_{e2}	Horizontal earth pressure at invert of lining
q_{w2}	Horizontal water pressure at invert of lining
δ	Displacement of lining
f_y	Yield strength of steel
E_s	Modulus of elasticity of steel

2.0 LOADS

2.1 Different kinds of load

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

These loads must always be considered

a) Ground pressure
b) Water pressure
c) Dead load
d) Surcharge

The following loads may or may not be considered depending on situation

a) Loads from inside
b) Loads during construction stage
c) Effects of earthquake
d) Effects from adjacent tunnels
e) Effects of settlement
f) Other loads

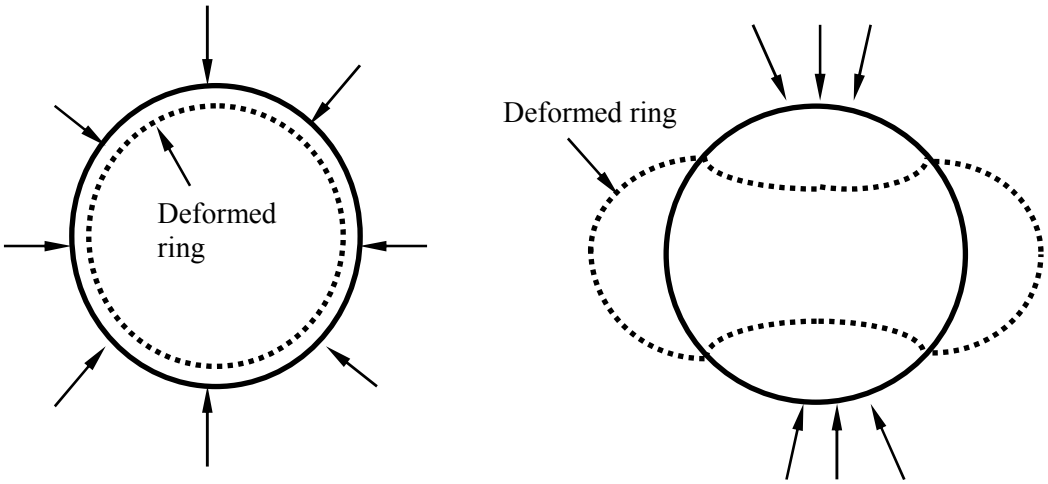
2.2 Ground Loading

Soft ground requires immediate supports as, for example, in driving a shield excavated tunnel or by applying shotcrete with the short time closure of the full ring. Therefore, the general agreement exists on the following assumptions

a) For design model of the linings, it may be sufficient to consider a cross section on the assumption of plane strain conditions for the lining and the ground
b) The active soil pressure on the lining is taken as equal to the primary stresses in the undisturbed ground because the ground is soft. It is thus assumed that for the final stage (years after construction) the ground will eventually return to the same condition as before the tunnelling, except for the passive stresses due to the deflection of the lining. Changing ground water levels, traffic vibration, etc may be the cause of this.
c) Between the lining and the ground there exists a bond either for radial and tangential deformation or for radial deformations only.
d) Because of the lining-ground relationship deformation of the lining results in reaction stresses in the ground. A continuum model includes this effect automatically. For a beam model bedding springs with appropriate bedding moduli have to be applied. The bond at every place around the lining gives rise to a reduction in the loading ground pressure where the lining deflects inwards.
e) The material behaviour of ground and lining is assumed as being elastic

It has been well established that tunnel lining in soft ground will redistribute the ground loading. The ground loading acting on a circular tunnel lining can be divided into two components: the uniform distributed radial component and the distortional component. The uniform distributed radial component will only produce hoop thrust and the lining

will deform in the radial direction with the shape of the ring remaining circular. The distortional component will produce bending moments in the lining, and the crown and invert will be squatted (move inwards) and at the axial level the lining will move outwards, Figure 3. The soil pressure at the crown and invert will be reduced as a result of the inward movement and the soil pressure at the axial level will be increased due to the outward movement of the lining. The redistribution of ground pressure around the ring and the lining deformation will continue until a balance is achieved. The stability of the tunnel lined by concrete segments thus depends on a continuous support / pressure around ring. Any cavity in the annulus of the tunnel lining and the ground will result in excessive distortional loading on the lining and may subject the ring to undergo excessive distortion, causing unacceptable cracking of the segments.



Tunnel lining subjected to uniform distributed loading and distortional loading

2.3 Water Pressure

As a guide and upper limit, the water pressure acting on the lining should be the hydrostatic pressure. The resultant water pressure acting on the lining is the buoyancy. If the resultant vertical earth pressure at the crown and the dead load is greater than the buoyancy, the difference between them acts as the vertical earth pressure at the bottom. If the buoyancy is greater than the resultant vertical earth pressure at the crown and the dead load, the tunnel would float.

The design ground water table is taken at both the ground surface (upper limit) and 3m (lower limit) below the surface for LTA tunnels.

2.4 Dead Load

The dead load is the vertical load acting along the centroid of the cross section of tunnel.

2.5 Surcharge

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

a) Road traffic load

b) Railway traffic load
c) Weight of building

A uniform surcharge of 75 kN/m^2 is considered in the design for LTA tunnels. Typically, a 75 kN/m^2 would have catered for a development load equivalent to a 5 storey building.

3.0 STRUCTURAL CALCULATIONS

The design assumes that the segments in the permanent condition are short columns subject to combined hoop thrust and bending moment. Both ultimate limit state (ULS) and serviceability limit state (SLS) are checked. Ultimate limit state design ensures that the load bearing capacity of the lining is not exceeded while serviceability limit state design checks both the crack-width and deformation of the lining. The following factors are used in the limit state design:

Ultimate limit state:
- Load factor for overburden and water pressure = 1.4
- Load factor for surcharge = 1.6

Serviceability limit state:
- Load factor for overburden, surcharge and water pressure = 1.0

3.1 Design Sections

The design calculations of the cross section of tunnel should be done for the following critical sections

a) Section with the deepest overburden
b) Section with the shallowest overburden
c) Section with the highest ground water table
d) Section with the lowest ground water table
e) Section with the large surcharge
f) Section with eccentric loads
g) Section with uneven surface
h) Section with adjacent tunnel at present or planned one in the future.

Typically, Table 2 shows the load combination consider in the design of LTA tunnels.

Table 2. Load combinations

LOAD COMBINATIONS	ULS					SLS (crack width)					SLS (deflection)	
	1	2	3	4	5	6	7	8	9	10	11	12
Load Factor = 1.4 and 1.6	√	√	√	√	√							
Load Factor = 1.0						√	√	√	√	√	√	√
75kN/m^2 Uniform Surcharge		√		√	√		√		√	√	√	√
Water Table at Ground Surface	√	√				√	√				√	

Water Table 3m Below Ground Surface			√	√	√			√	√	√		√
Full Section Moment of Inertia	√	√	√	√		√	√	√	√			
Reduced Section Moment of Inertia					√					√	√	√
Short Term Concrete Young's Modulus	√	√	√	√		√	√	√	√			
Long Term Concrete Young's Modulus					√					√	√	√
Additional Distortion of 15mm on Diameter					√					√		

The tunnels are to be constructed through soft ground with a tunnel boring machine (TBM). The vertical pressure applied to the lining is thus the full overburden pressure. Distortional loading is derived by using the appropriate K-factor in Curtis formulae according to the soil condition at the tunnel location. The following K-factors are used in accordance with the LTA Design Criteria:

K-factor

Soil Type	**K**
Estuarine, Marine and Fluvial Clays	0.75
Beach Sands, Old Alluvium, Completely Weathered Granite, Fluvial Sands	0.5
Completely Weathered Sedimentary Rocks	0.4
Moderately to Highly Weathered Sedimentary or Granite Rocks	0.3

3.2 Computation of Member Forces

The member forces (M, N, S) are calculated using various structural models, namely

a) Continuum Analytical Models
b) Bedded Beam Spring Model
c) Numerical Models

3.2.1 Continuum Analytical Models

Commonly used continuum analytical models also referred to as “closed form” solutions include those proposed by Muir Wood (1975), Einstein and Schwartz (1979) and Duddeck and Erdmann (1985). All these models are based on excavation and lining of a hole in a stressed continuum. In general, these models yield similar results for normal forces for the same input parameters but the predicted bending moments may differ significantly.

The analytical solutions assume plane stress, an isotropic, homogeneous elastic medium and an elastic lining for circular tunnel, although the Muir Wood-Curtis solutions has been extended by Curtis to viscoelastic ground in 1976. The assumption that the lining is installed immediately after the tunnel is excavated tends to overestimate the loads and

hence judgement is required in deciding the proportion of the original in-situ stresses to apply to the linings.

Some options include applying a reduction factor to the full applied ground stress; any stress relief depends on the ground conditions and the method of construction. This reduced stress can be assumed at 50-70% if the depth to tunnel axis is greater than three diameters (Duddeck and Erdmann, 1985). Alternatively, the Ko value can be set at less than 1.0 to simulate actual behaviour, that is the tunnel squat to match the observed behaviour of segmental tunnels in soft ground.

These models also assumed that the ground is a semi-infinite medium and therefore they should only be used for tunnels where the axis is greater than two tunnel diameters below the surface. Duddeck and Erdmann recommended that full bonding at the ground lining interface be assumed for the continuum models listed above. Most analytical solutions are formulated in total stresses.

The benefit to the designer is that the models are simple quick to use. Information provided on the normal forces, bending moments and deformation and several methods should be applied with a range of input parameters to determine the sensitivity of the lining designs to variations in ground conditions.

3.2.2 Bedded Beam Spring Model

These simulate a tunnel lining as a beam attached to the ground, which is represented by radial and tangential springs, or linear elastic interaction factors, to allow for ground support interaction. The stiffness of the springs can be varied to model conditions at the tunnel extrados from “no slip” to “full slip”, and different combinations can be modelled. Relationships exist for determining the spring stiffness from standard ground investigations tests.

Despite the fact that these models tend to underestimate the beneficial effects of soil-structure interaction, and cannot consider shear stresses in the ground itself, the results can sometimes agree well with those from continuum analytical models.

One of the drawbacks with this method of analysis is the lack of information on movement in the ground and therefore two-dimensional numerical models have tended to replace bedded beam models. It is also difficult to determine the spring stiffnesses.

3.2.3 Numerical Analysis Models

There are two and three dimensional modelling programmes available in the commercial market. The choice of programme depends on whether the ground can be modelled as a continuum or whether the influence of discontinuities, for example faults, bedding surfaces, joints, shear joints, etc requires an assessment of independent block movement.

Soft Ground – This is normally considered as a continuum and hence finite element (FE) or finite difference (FD) methods can be easily applied.

Rock – Jointed rock masses are discontinua and often can be modelled realistically using discrete elements (DE) and boundary element (BE) methods. Discrete element methods include distinct element programmes in which the contacts between elements may deform and discontinuous deformation analysis programmes in which the contacts are rigid. In addition, by means of interface elements, a small number of discontinuities can

be modelled in finite element and finite difference models, but discrete element is required when modelling intersection joints and larger numbers of discontinuities.

The process of building a model with FE and FD is essentially the same and the end products are often very similar. The object to be analysed is represented by a mesh of many elements or zones, in a process of discretisation. The material properties, material behaviour, boundary conditions and loads are assigned to the model and the problem solved.

In FE a stiffness matrix is assembled for the whole mesh in order to relate the displacements to the stresses. These vary in a prescribed manner within each element. The matrix is then solved using standard matrix reduction techniques, in a so-called "implicit" solution technique.

In the FD method, the "dynamic relaxation" solution technique is used. Newton's Law of Motion is expressed as a difference equation and us used to relate explicitly the unbalanced forces at each integration point in a mesh to the acceleration of the mass associated with that point. For a very small time-step the incremental displacements can be calculated. In static mechanical problems this time step is fictitious, i.e. it is not related to real time. The incremental displacements are used to calculate a new set of unbalanced forces (from the constitutive relationships). This calculation step is repeated many times for each integration point in the mesh, in a "time marching" method, until the out-of-balance force has reduced to a negligible value, i.e. equilibrium has been reached for a statical problem. More integration points are required n a FD rather than a FE model because FD used constant strain zones.

In DE method, the individual blocks in a rock mass are modelled and the elements may move and rotate, depending on the movement of adjacent elements. Either FE or FD is used to model the constitutive behaviour within the elements.

In the BE method, the surface of an object is divided into elements, which are modelled mathematically as infinite continua.

A more detailed description of all these numerical methods can be found in Hoek *et al*., 1995.

3.3 Evaluation of joints

If the segmental lining is jointed with or without bolts, it actual flexural rigidity at the joint is smaller than the flexural rigidity of the segment. If the segments are staggered, the moment at the joint is smaller than the moment of the adjacent segment. The actual effect of the joint should be evaluated in the design.

The joints must be detailed to achieve the required watertightness giving consideration to the type of waterproofing material used. Joints must be detailed to achieve adequate bearing area but with reliefs or chamfers to minimise spalling and stripping damage.

Design of the joints should provide for fast and durable connections with sufficient strength to meet the erection sequence support requirements and to maintain compression of the sealing gaskets. Particular attention must be paid to the design of longitudinal joints. High level contact stresses due to joint geometry and ring build may cause

circumferential cracking due to high tensile stresses. Pads can be used to reduce these stresses.

Gasket compression has an important influence on the joint design, as it requires large forces to close the joints and then hold them together. Positioning and size of gaskets for sealing can significantly reduce the cross-sectional areas of joints available for the transfer of compression loads. Relief of loading of the area at the extrados of the segment behind the gaskets can help reduce damage caused by gasket compression. Hence the joint connection, strength, number and position must be designed to ensure and maintain adequate gasket compression.

Consideration should also be given to the relief of the loading at the edges of segment to minimise spalling when ram loads are applied. When completing the ring erection, key sizes and angles must be compatible with the available tail-skin space and shield ram-travel when a ram is used to place the final unit.

Provision of bursting steel may be necessary for large ram loads and loading pads can be helpful in reducing segment damage.

4.0 DURABILITY CONSIDERATIONS

4.1 Fire Resistance

The Singapore Standards SS CP65 Part 2 sets out 3 ways to determine the fire resistance of reinforced concrete members :

a) Tabulated Data
b) Fire Test
c) Fire Engineering Calculations

In all the cases, the size and shape of the element together wil the minimum thickness and cover to reinforcement influence the fire resistance. Allowance is also made for the moisture content of the concrete, the type of concrete, aggregate used and whether any protection is needed.

Two basic options are available for fire protection are available.

a) Protect externally – Protect the concrete against a fast rise in temperature by means of a fire resistant isolation. A degree of protection can be given against relatively low temperature fires by the applications of external systems in form of boarding or spray-applied coatings. Detailed performance criteria and advice should be obtained from specialist suppliers.

b) Protect internally – Protect the concrete against the formation of high vapour stresses. Polypropylene fibres can be added to the concrete mix. These fibres melt at approximately 160^{o}C and form micro-channels, which can prevent or diminish the occurrence of high vapour pressures and hence reduce a tendency of spalling.

4.2 Wateproofing Systems

The strategy put in place for achieving the functional and operational requirements for a project will depend on the design requirements. Guideline relating to watertightness and permissible levels of leakage into sub-surface facilities has been presented by the International Tunnelling Association (ITA). In the absence of any other criteria this provides a reasonable basis for an initial evaluation of design requirements, a useful summary of the effects of water ingress on different types of lining, and the most appropriate repair methods. It also serves as a reminder of the benefits of waterproofing systems. To achieve control over water inflows and seepage into a tunnel there are a number of products available including membranes, gaskets, injected water stops and annular and ground grouting.

4.2.1 Membranes

There are 2 membranes available in the market.

a) Sheet membrane – Sheet membrane that include materials such as PVC (Polyvinylchloride), HDPE (High Density Polyethylene) , and PO (Polyolefin).
b) Spray on membrane – Spray on membrane are a recent innovation and essentially consists of either cement or rubber based compounds.

4.2.2 Gaskets

Gaskets area available in 2 main types

a) EPDM – EPDM or neoprene compression gaskets fitted around individual precast segmental lining
b) Hydrophilic – Hydrophilic seals are made from specially impregnated rubbers or specially formulated bentonite-based compounds that swell when in contact with water.

Bothe EPDM (Ethylene Polythene Diene Monomer) compression gaskets and hydrophilic seals are commonly specified to provide waterproof joints between adjacent segments in a precast segmental lining. These are not for waterproofing the concrete itself, but to prevent water flow through potential apertures. The usual practice is to employ a single EPDM gasket or single trip of hydrophilic seal. A double seal arrangement has been used or gaskets incorporating through thickness barriers. Alternatively a second performed sealing groove with injection points has been provided as a means of remedial sealing.

The long term durability and deterioration of the performance of the seal due to creep and stress relief should also be take into account. The likely fluctuation in water level will also dictate the type of gasket to be employed. Hydrophilic seals may deteriorate if repeatedly wetted and dried. Performance can also be affected by the salinity or chemical content of the groundwater. Different hydrophilic seals are required for saline and fresh water.

The performance of these seals with respect to water pressure, gasket compression characteristics and joint gap tolerance is an important part of the lining design. The specification of the type and performance of the sealing system to be used must be carried out in conjunction with expert suppliers. The exact system should be determined with the contractor as it depends on the type of TBM to be used and the detailed design of the erection equipment.

Gasket compression forces have an important influence on the joint design as they require large forces to close the joints and then hold the joint together while erection continues. The design of the fixings between segments and their performance under load is an integral part of the gaskets' performance. All stages of the erection process must be considered.

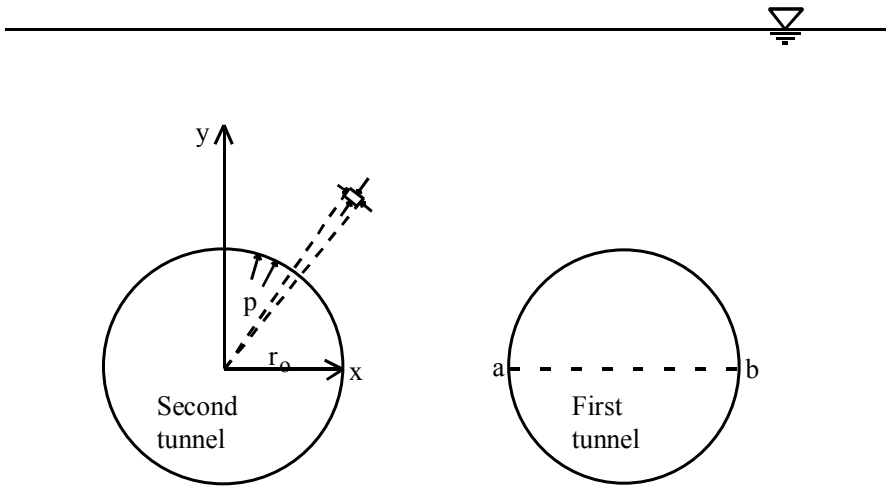
Positioning and size of compression gaskets or hydrophilic sealing systems can significantly reduce the cross sectional areas of joints available for the transfer of compression loads and must be taken into account. Relief behind the gasket can help reduce the damage caused by gasket compression by providing a void for the gasket to flow into thereby preventing the gasket from becoming over compressed and behaving in a hydraulic manner. The joint connection, strength, number and position must be designed to ensure and maintain adequate gasket performance.

5.0 TUNNELLING IN CLOSE PROXIMITY

Additional bending moment in the first tunnel should be considered if the centre to centre distance of the second tunnel to the first is less than 2 times the diameter. The additional bending moment in the first tunnel lining due to the construction of the second tunnel is derived based on the theory of elasticity.

Typically for twin bored tunnels, the second tunnel drive will be some distance behind the first tunnel drive. If there is adequate clearance between the two tunnels, the effect of the second tunnel construction on the erected segmental lining of the first tunnel is negligible. The rule of thumb is that the clearance between the two tunnels should not be less than one tunnel diameter. If the clearance between the tunnels is less than one tunnel diameter, the design should make allowance in the lining of the first tunnel for the effect of the second tunnel construction.

Ground movement due to the second tunnel construction will cause additional distortion to the first tunnel besides that due to the ground loading. This additional distortion is the difference of the movement of the first tunnel at two opposite points a and b, where point a is the closest point to the second tunnel and point b is the furthest point from the second tunnel, see Figure 4. This difference in movement can be calculated based on the theory of elasticity by using the volume loss due to the construction of the second tunnel.



Two tunnels at close proximity

Assuming that the ground is a homogeneous, isotropic, linearly elastic mass, the principal stress σ_r, σ_θ and σ_z and the principal strains ε_r, ε_θ and ε_z can be expressed as follows in terms of the Young's modulus, E and Poisson's ratio, ν:

$-E\varepsilon_r = \sigma_r - \nu (\sigma_\theta + \sigma_z)$
$-E\varepsilon_\theta = \sigma_\theta - \nu (\sigma_z + \sigma_r)$
$-E\varepsilon_z = \sigma_z - \nu (\sigma_\theta + \sigma_r)$

Under the plane strain condition, $\varepsilon_z = 0$, therefore:

$\sigma_z = \nu (\sigma_\theta + \sigma_r)$
$-E_2\varepsilon_r = \sigma_r - \nu_2 \sigma_\theta$
$-E_2\varepsilon_\theta = \sigma_\theta - \nu_2 \sigma_r$

where $E_2 = E/(1- \nu^2)$ & $\nu_2 = \nu/(1- \nu)$, which are elastic parameters for plane strain conditions.

Substituting the radial strain, $\varepsilon_r = du/dr$ and the circumferential strain, $\varepsilon_\theta = u/r$ into the above equations, where u is the radial deformation of the ground at a radial distance r from the centre of the tunnel:

$$\mathbf{-E_2 (d_u/d_r) = \sigma_r - \nu_2 \sigma_\theta} \quad \mathbf{(1)}$$
$$\mathbf{-E_2 (u/r) = \sigma_\theta - \nu_2 \sigma_r} \quad \mathbf{(2)}$$

(2) x ν_2 gives $-\nu_2 E_2 (u/r) = - \nu_2^2 \sigma_r + \nu_2 \sigma_\theta$
(1) + (2) x ν_2 gives $(1-\nu_2^2) \sigma_r = -E_2 (d_u/d_r + \nu_2 u/r)$, thus:

$$\mathbf{\sigma_r = \{-E_2 / (1-\nu_2^2)\}(d_u/d_r + \nu_2 u/r)} \quad \mathbf{(3)}$$

Similarly, (1) x ν_2 gives $-\nu_2 E_2 (d_u/d_r) = - \nu_2^2 \sigma_\theta + \nu_2 \sigma_r$
(2) + (1) x ν_2 gives $(1-\nu_2^2) \sigma_\theta = -E_2 (u/r + \nu_2 d_u/d_r)$, thus:

$$\mathbf{\sigma_\theta = \{-E_2 / (1-\nu_2^2)\}(u/r + \nu_2 d_u/d_r)} \quad \mathbf{(4)}$$

The equilibrium equation in the radial direction can be written as:

$$\mathbf{\frac{d\sigma_r}{dr} + \frac{(\sigma_r - \sigma_\theta)}{r} = 0} \quad \mathbf{(5)}$$

Substitute Equations (3) and (4) into Equation (5) gives:

$$\mathbf{\frac{r^2 d^2 u}{dr^2} + \frac{r du}{dr} - u = 0} \quad \mathbf{(6)}$$

Solving Equation (6) gives:
$u = Ar + B/r$ for $r \neq 0$
For $r = \infty$, $u_\infty = 0$, $\therefore A = 0$, $u = B/r$
At wall of cavity, $\varepsilon_\theta = \varepsilon_o = u_o/r_o$, $\therefore u_o = \varepsilon_o r_o$ and $B = u_o r_o$

$$\mathbf{u = B/r = u_o r_o /r \text{ or } \varepsilon_o r_o^2} \quad \mathbf{(7)}$$

Volume loss, $Vs = \{\pi r_o^2 - \pi(r_o - u_o)^2\}/ \pi r_o^2$
$r_o^2 Vs = r_o^2 - (r_o - u_o)^2$
$$\mathbf{u_o = r_o\{1-\sqrt{(1-Vs)}\}} \quad \mathbf{(8)}$$
Using equation (7) and (8):

At point a, $u_a = u_o r_o /r_a$, where r_a is the distance of point a to the centre of the second tunnel.

At point b, $u_b = u_o r_o /r_b$, where r_a is the distance of point a to the centre of the second tunnel.

The diametrical distortion, δ_d is defined as $\delta_d = u_a - u_b$
The radial distortion is given by:
$$\mathbf{\delta_r = \delta_d /2} \quad \mathbf{(9)}$$

Morgan (1961) showed that the bending moment due to distortion over radius is given by:

$$\mathbf{M = (3EI\delta_r)/ r_o^2} \quad \mathbf{(10)}$$

Where E = the Young's modulus of concrete
I = the second moment of inertia of the segment
δ_r= the radial distortion
r_o= the excavated radius

The induced bending moment due to any distortion on diameter can be estimated by using the above equation.

Based on equations (9) and (10), the additional distortional moment in the first tunnel lining due to the second tunnel construction can be calculated. The total bending moments for structural design of the segments are superimposed by adding the additional distortional moment to the moment due to ground loading, assuming the hoop thrust remains unchanged.

6.0 CONCLUSION

Tunnel lining design is a challenging task, not least because of the variability of the ground. Therefore it should be approached as an iterative process, in which the designer may use a variety of design methods, in order to gain an appreciation of how the ground and lining are likely to interact. From that the support required can be determined to maintain safety both in short and long term and to satisfy project requirements. Sound engineering judgement underpins this process.

Empirical, "closed form" analytical and numerical design methods exist. Each method has its own strengths and limitations. These should be borne in mind when interpreting the results of design calculations. It is recommended that several design methods be used when designing a lining, since the other design methods will provide an independent check on the main design method.

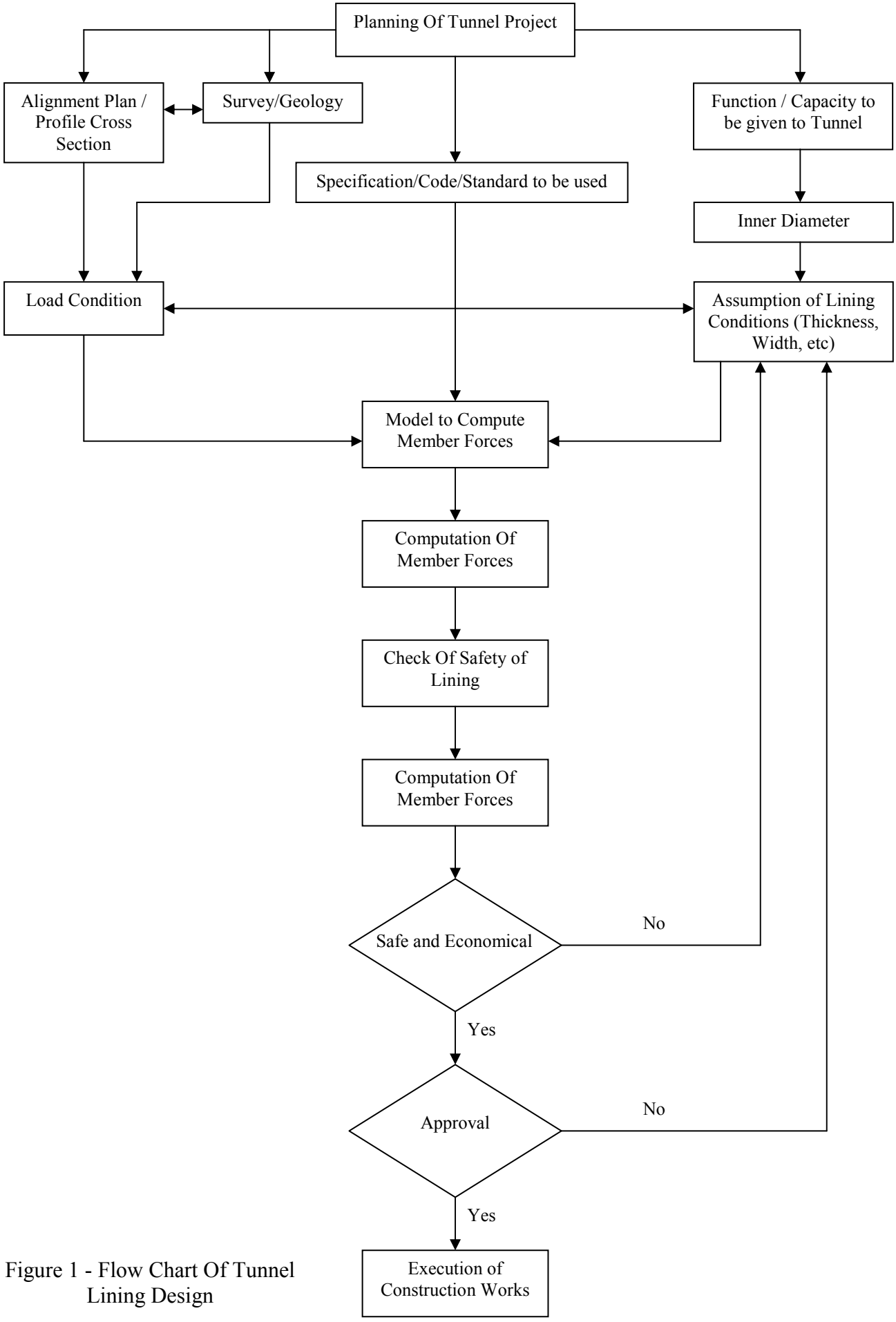


Figure 1 - Flow Chart Of Tunnel Lining Design

Step by Step Design Procedure (Checklist)

Step 1 : Define geometric parameters

Factors to consider are
a) Alignment
b) Excavation diameter
c) Lining diameter
d) Lining thickness
e) Width of lining
f) Segment system
g) Joint connections (radial and circumferential)

Step 2 : Determine Geotechnical Data

Factors to consider are
a) Specific gravity
b) Cohesion (unconfined and effective)
c) Friction angle (unconfined and effective)
d) Modulus of elasticity
e) Modulus of deformation
f) Ko value

Step 3 : Select Critical Sections

Factors to consider are
a) Influence of overburden
b) Surface loads (Surcharges)
c) Water
d) Adjacent structures

Step 4 : Determine Mechanical Data of Tunnel Boring Machine

Factors to consider are
a) Total thrust pressure
b) Number of thrust jacks
c) Number of pads
d) Pad geometry
e) Grouting pressure
f) Space for installation

Step 5 : Define Material Properties

Factors to consider are
a) Concrete grade
b) Compressive strength
c) Modulus of elasticity
d) Steel type
e) Tensile strength
f) Gasket type
g) Gasket width

h) Elastic capacity
i) Allowable gap

Step 6 : Design Loads

Factors to consider are
a) Geostatical loads on lining based on different permutation of load cases
b) Thrust jacking loads
c) Secondary grouting loads
d) Dead loads
e) Temporary loads (storage, lifting, jacking, etc)
f) Effects of adjacent tunnels
g) Effects of settlement
h) Effects of future development
i) Earthquake (if any)
j) Effect of building tolerances like birdmouthing of radial joints

Step 7 : Design Models

The 3-dimensional condition has to be idealised into a 2-dimensional condition through the use of

a) Analytical models like
   - Continuum model proposed by AM Muir Wood modified by D J Curtis
   - Bedded beam model proposed by Duddeck and Erdmann

b) Numerical models like
   - Finite element programmes to compute the stress and strains under elasto-plastic conditions.

Step 8 : Computational Results

In order to define the amount of reinforcement for the segments, the results should include

a) Normal forces
b) Shear forces
c) Bending moment
d) Deflections

Step 9 : Additional Checks

a) Flotation
b) Heave
c) Long term longitudinal settlement

Example 1

a) Geometry

Type of Segment	Precast Segmental Lining
Diameter of Segmental Lining	5800 mm
Width of Segment	1400 mm
Thickness of Segment	275 mm

b) Ground Condition

Soil Type	Parameters	Design Values
Marine Clay, M	Bulk Density, γ	16 kN/m^3
	Undrained Shear Strength, C_u	10 kPa (Depth = 0-5m) 0.25* Effective Overburden (Depth = >5m)
	Young's modulus of ground, E_c	0.8*250*C_u
	Effective cohesion of the ground, c'	0
	Effective friction angle of ground, ϕ'	22°

c) Design Sections

Tunnel Drive	Geological Formation	Section	Approximate Chainage	Tunnel Reference	Borehole Reference	Ground Level	Axis Level	Track Level
OAP - TJK	Marine Clay	1	CH 57 + 127	Deep	M1042	101.925	82.279	80.754
OAP - TJK	Marine Clay	2	CH 57 + 444	Shallow	CC101E	102.077	88.450	86.925

d) Design Method

Continuum method suggested by Muir Wood modified by Curtis was used in the evaluation of the forces.

e) Full Design Calculations are presented in Appendix A

PART 2 – DESIGN OF SPRAYED CONCRETE LINING IN SOFT GROUND

1.0 INTRODUCTION

1.1 NATM Philosophy vs NATM Construction Technique

1.2 Rock Tunnelling or Soft Ground Tunnelling

2.0 ANALYSIS & DESIGN OF SCL TUNNELS

2.1 Components of SCL Design

2.2 Stability Assessment

2.2.1 Ground Stand-up time

2.2.2 Characteristics of ground water conditions

2.2.3 Face Stability

2.2.4 Suitability of proposed excavation and support sequence

2.2.5 Auxiliary support measures

2.3 Methods of Tunnel Analysis

2.3.1 Closed-form solutions

2.3.2 Bedded Beam Models

2.3.3 Finite element methods

2.3.4 Empirical Route to SCL Design

2.4 Prediction of ground settlement

2.5 Planning for contingency

3.0 INSTRUMENTATION & MONITORING FOR SCL TUNNELS

3.1 Instruments for NATM construction

3.2 In-tunnel deformation

3.3 Convergence monitoring

3.4 Tunnel lining forces

3.5 Face monitoring

3.6 Surface settlement

3.7 Frequency of monitoring

4.0 DESIGN OF FINAL LINING

4.1 Analysis of permanent linings

4.2 Flotation check for final lining

LIST OF REFERENCES

Annex A Examples and Characteristics of NATM excavation methods (Tables 4.3 & 4.4 extracted from Japanese Standard for mountain tunnelling)

Annex B Typical Applications of Instrumentation in tunnelling (Figure 8.1 extracted from Tunnel Lining Design Guide, 2004)

1.0 INTRODUCTION

1.1 NATM Philosophy versus NATM Construction Technique

In its original sense, the term NATM (or New Austrian Tunnelling Method) as described by Austrian engineer Rabcewicz, refers to a philosophy of applying a thin, temporary support and allowing deformations so that the rock pressure could be reduced and distributed into the surrounding rock. By doing so, the final support will be less loaded and can be installed even later and as a much thinner structure.

Today, NATM has also been used to refer to a construction technique that uses sprayed concrete as an initial support medium for tunnels. The introduction of NATM into soft ground tunnelling has created much confusion on the application of NATM philosophy versus its application as a construction technique. The ICE Design and Practice Guide (1996) recommends making a distinction between NATM as a tunnelling philosophy and NATM as a set of construction technique.

The key features defined in NATM philosophy are:-

- The strength of the ground around a tunnel should be deliberately mobilised to the maximum extent possible
- Mobilisation of ground strength is achieved by allowing deformation of the ground
- Initial or primary support, having load deformation characteristics appropriate to the ground conditions is installed. Permanent support works are normally carried out at a later stage
- Instrumentation is installed to monitor the deformations of the initial support system and the build-up of load upon it. Where appropriate, the results of this monitoring form the basis for varying the primary and permanent support, and the sequence of excavation

The key features of the set of construction technique referred to as NATM are:

- The tunnel is sequentially excavated and supported, and the excavation sequences and face areas can be varied.
- The primary support is provided by sprayed concrte in combination with some or all of the following: steel mesh, steel arches (such as H-beams, lattice girders, etc.), ground reinforcement (eg. rock bolts, spiling)
- The permanent support is usually (but not always) provided by a cast in-situ concrete lining, which is normally treated separately for design purposes.

1.2 Rock tunnelling or soft ground tunnelling

The NATM philosophy is mostly applied in hard ground or rock tunnelling, and had been mostly developed from experience of tunnels constructed in high mountains. In these situations, the excessive high loads induced on tunnel supports that are too stiff and installed too early, could be reduced by having a delayed installation of a flexible primary support. Where the possibility of excavation collapse can be safely discounted, this delayed support installation mobilises strength of the rock mass, and results in the permanent support experiencing lower loads for a more economic and practical support design.

On the other hand, tunnelling in soft ground or in urban areas would require that deformation be kept to a minimum for stability and support to be installed as soon as possible after excavation. Two essential measures highlighted by the ICE guide are:-

- Excavation stages must be sufficiently short in terms of dimensions and duration
- Completion of primary support (in particular, closure of the sprayed concrete ring) must not be delayed.

Some major differences in the approach to both situations may be tabulated as follows:-

	NATM in hard ground	NATM in soft ground
Ground Deformation	Deliberate ground deformation and mobilisation of ground strength in order to reduce loads acting in the tunnel support system.	Limitation of ground deformation to avoid irreversible shearing of the ground and ensure stability of the excavation, and to limit surface settlement and avoid damage to overlying structures.
Primary support	Just sufficient to prevent immediate collapse but not so stiff to attract excess loading.	Designed to reduce ground settlement to a minimum.
Instrumentation	Instrumentation is installed to monitor the deformation and load build-up on the primary support, with the intention of varying the excavation and support system.	Instrumentation is used to monitor the performance of the primary support and to validate the design, but not to vary the excavation and support design.

As the works undertaken by LTA take place primarily in soil rather than rocks, the ensuing discussions would focus on NATM design and construction in soft ground.

2.0 ANALYSIS & DESIGN OF SCL TUNNELS

2.1 Components of SCL design

Mair and Taylor (1997) commented that the three most important requirements for the successful design and construction of a tunnel can be summarised as follows:-

- Stability Assessment

The choice of excavation and construction technique must be suited to the ground conditions so that it is feasible to build the tunnel safely. This assessment should include the extent to which the ground is able to stand unsupported, the stability of the excavation & support sequence, as well as the size of the face opening and its stability.

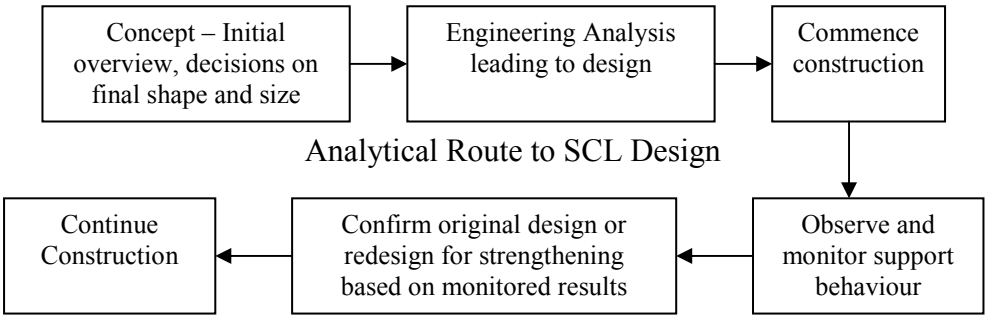
- Ground movements & their effects

Tunnel construction should not cause unacceptable damage to surrounding ground or overlying structure and services. The ground movements should be predicted prior to construction, and their effects on the structures and services assessed. Other than deformation predictions using finite element methods, it is also possible to predict surface settlements based on the volume loss from works of similar nature.

- Lining Performance

The temporary and permanent lining must be capable to withstand all the influences to which it may be subjected during its design life. This requires predictions of the soil loads acting on the lining and of the deformations of the lining, the latter being of particular significance in the case of external influences such as adjacent tunnel construction.

The following flowchart summarises the activities when carrying out the analysis and design of a SCL tunnel.



The ensuing sections will describe the major aspects of analysing and designing for a SCL tunnel constructed by NATM in soft ground.

2.2 Stability Assessment

The assessment on the stability of the NATM works can be attributed to the critical factors of ground stand-up condition, groundwater characteristics, face stability, and

2.2.1 Ground Stand-up Time

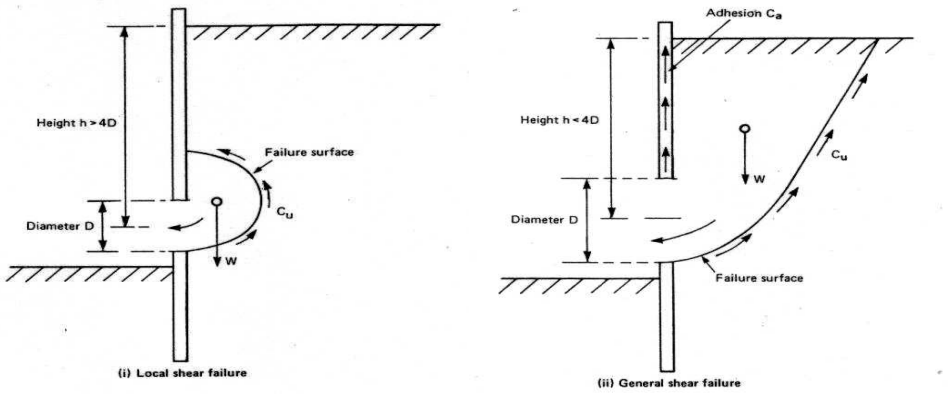
Of prime importance is the stability of the opening prior to installation of the lining. One aspect is to study the ground stand-up time and determine the consequent constraints for construction. Babendererde (1980) stated that “the ground must have a cohesiveness that will allow it to stand safely unsupported for at least 90mins with an advance of 1 metres”, but the actual requirements should be evaluated in conjunction with the size of unsupported face and the duration for which it is unsupported, against the method & duration of the works.

2.2.2 Characteristics of Ground water conditions

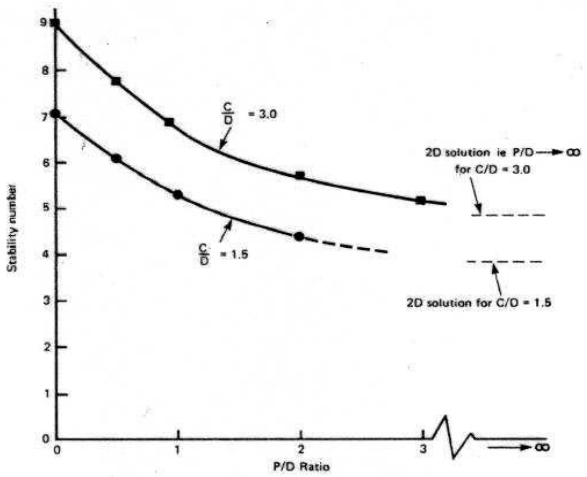
The destabilising effect of ground water on a NATM construction cannot be under-estimated, as this could deteriorate the stand-up time of ground so badly as to affect the safety of a NATM excavation. Other than the permeability characteristics of the soil, it is also important to investigate the site thoroughly for any potential water bearing layers, such as backfill or sand lense. Pre-excavation treatment such as grouting, and contingency planning would be necessary in the areas where there is a significant risk of uncontrollable water ingress that would affect excavation stability.

2.2.3 Face Stability

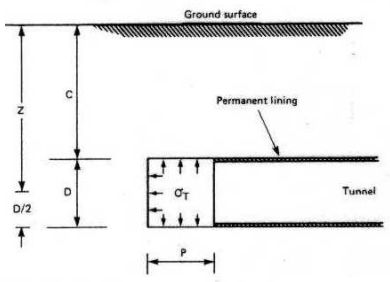
Another important aspect of excavation stability is the Face Stability, especially in the top heading. Broms and Bennermark (1967) were the first to propose the use of a face stability number to analyse tunnel face stability, which is a ratio of the undrained shear strength at tunnel axis and the difference between the overburden pressure at tunnel opening and applied face pressure. ie. $N = (\sigma_z - \sigma_T)/c_u$.

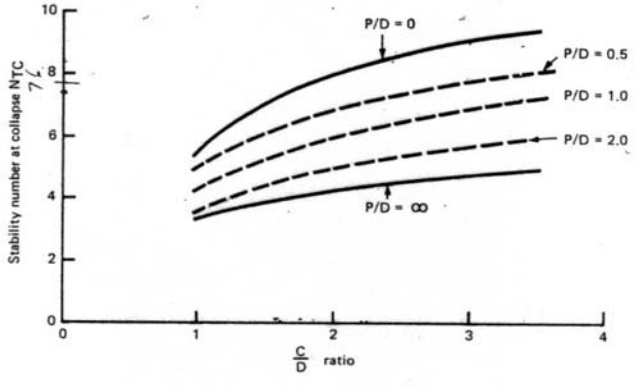


This had been substantiated by researchers, such as Mair (1979) and Kimura and Mair (1981) who carried out several centrifuge model tests and showed that the tunnel heading geometry have a considerable influence on the stability number at collapse.



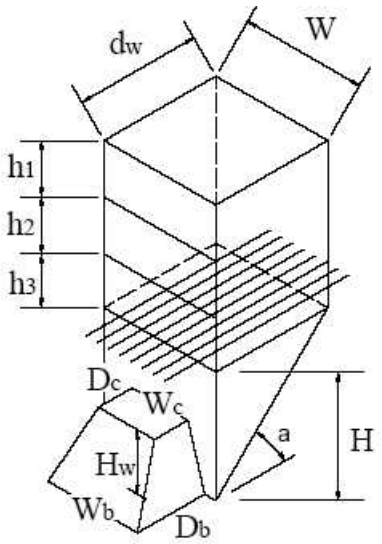
Influence of heading geometry on stability number at collapse from model tests in the centrifuge (Mair 1979)





Influence of heading geometry on stability number at collapse from model tests in the centrifuge (after Kimura and Mair 1981)

Most of the stability charts are developed from an idealised circular tunnel heading which may not be relevant in most NATM excavations. Another technique to assess Face Stability is to consider a failure wedge at the face, and establish the factor of safety corresponding to the face geometry and soil parameters at the limit equilibrium condition. For example, the size of the failure wedge can be determined according to the most likely failure mechanism, and the minimum factor of safety is obtained by adjusting the incline of the sliding wedge. Forepoling, face dowels and central supporting core ("dumpling") could be mobilised in order to enhance the face stability to acceptable minimum factors of safety. The diagram illustrates an example of a failure wedge assumed.

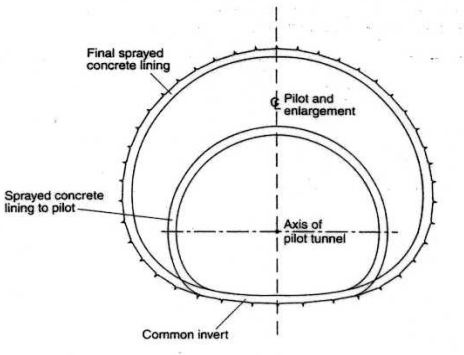


2.2.4 Suitability of proposed Excavation & Support Sequence

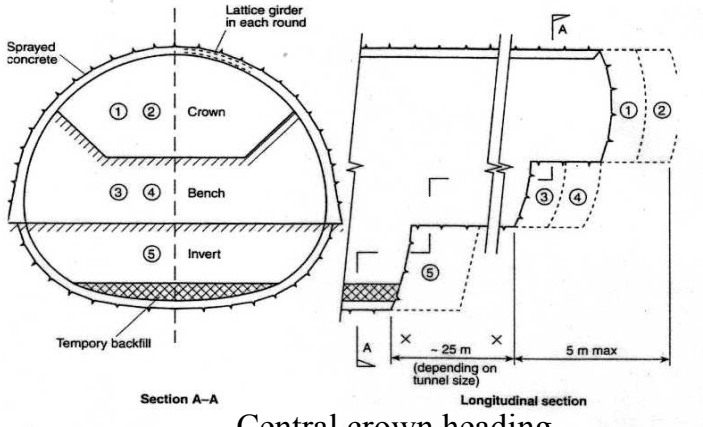
Ideally, the assessment on whether the proposed excavation & support sequence is suitable for the given tunnel geometry & ground conditions, can only be done using a 3D analysis. Although it is possible to model the 3D tunnelling problem using a 2D finite element method, this might involve the introduction of empirical parameters that should be substantiated with experience in similar conditions of geometry & geology. Alternatively, the designer may also demonstrate that the proposed technique of construction sequence had been used in similar jobs elsewhere.

Below are some possible methods of tunnelling sequence as extracted from the ICE Design and Practice Guide (1996):-

A) Full face approach with stepped profile of heading and bench, may be allowed for tunnels up to 30m^2 in cross section;
B) Pilot tunnel driven at full face, which is enlarged into the full size tunnel;
C) Central crown heading followed by full-width bench excavation and invert excavation, with emphasis on immediate tunnel ring closure at various stages (be it temporary invert or final invert);

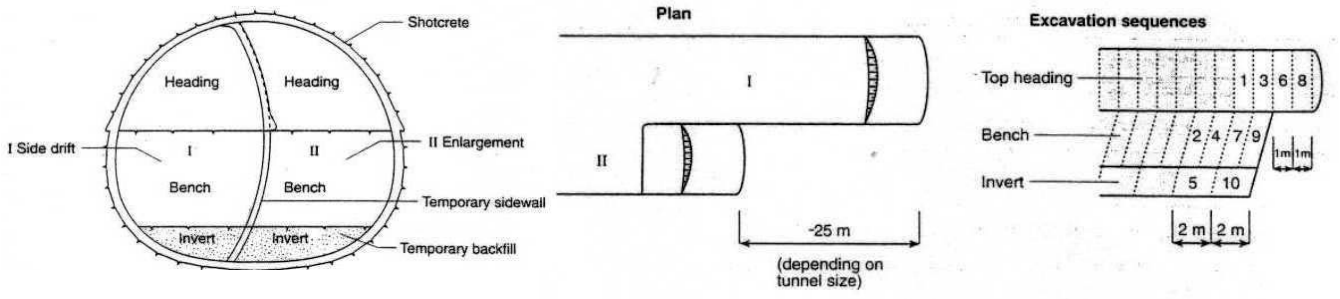


Pilot Tunnel

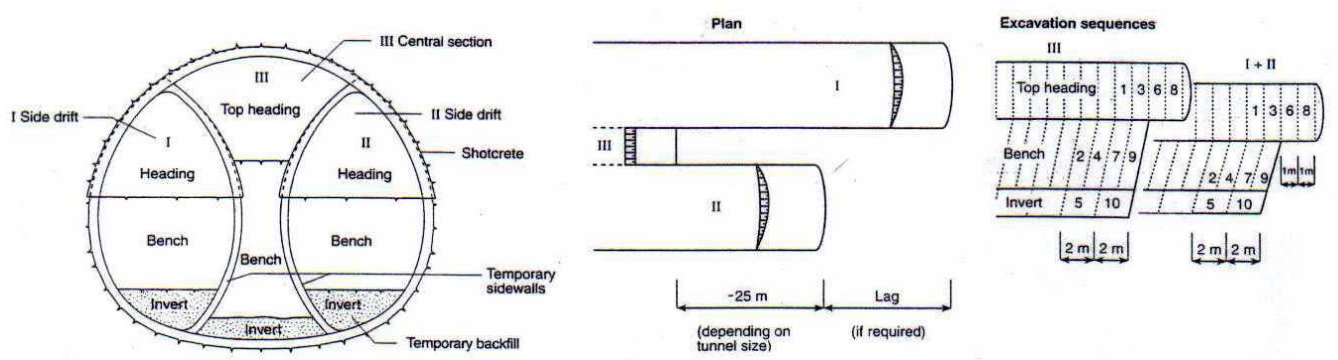


Central crown heading

D) Excavation face advance by the side, with each face stepped at heading, bench and invert as governed by face stability, full ring closure & proper joint continuity near each face, and tunnel enlargement taking place when there is sufficient lag between the two excavation faces.



E) The sidewall drifts separated by the central core can be advanced in parallel, but with sufficient stagger between the excavation faces. Each face may also be stepped at heading, bench and invert with rapid ring closure and proper joint continuity between lattice girders. Central core excavation would commence when there is sufficient lag behind the excavation faces.



2.2.5 Auxiliary Support Measures

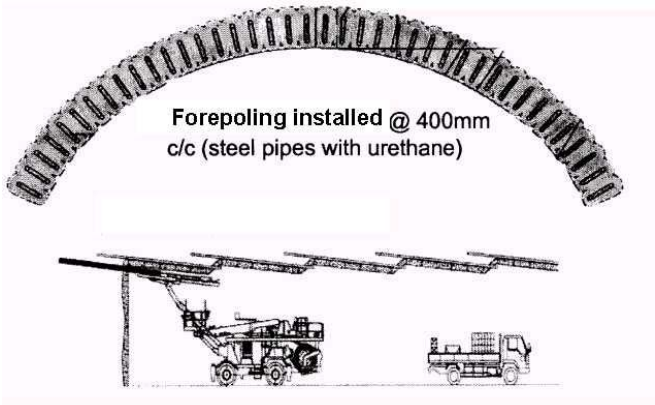
To enhance the stability of the excavation, auxiliary support measures may be initiated as part of the normal sequence of NATM construction, or could be used as a contingency measure during NATM works. The Japanese Standard for Mountain Tunnelling (1996) classifies some of these auxiliary measures according to the stabilisation required. This is as reproduced in the following table.

Stabilisation Objective		Stabilisation measures identified		
Stabilisation of Cutting Face	Crown Stabilisation	Filling type forepoling	Grouting type forepoling	Steel pipe forepoling
	Face Stabilisation	Face Bolting	Grouting	
	Footing Stabilistion	Enlargement of support footing	Top heading temporary invert	Foot reinft bolting & piling
Stabilisation of Water inflow control	Drainage measures	Drainage boring & drainage drift	Well point	Deep well system
	Water Sealing	Grouting Method	Pneumatic method	Cut-off wall method
Environment Preservation	Minimise surface settlement	Pipe-roof method & steel pipe forepoling	Horizontal jet-grouting	Vertical Pre-reinforcement & Chemical grouting
	Protect adjacent structures	Ground reinforcement & improvement	Cut-off Wall	Structural reinforcement and underpinning

Below shows some of the commonly used support measures in soft ground tunnelling.

A) Forepoling

This refers to the insertion of ground supports outside and ahead of the excavated tunnel face, and these ground reinforcement could be in the form of ungrouted spiles, steel pipes injected with grout, or even interlocking steel sheets driven to form an arch ahead of tunnel face. In particularly for tunnels with low soil cover, the use of canopy tube umbrellas as a pre-excavation support measure is extremely effective in controlling deformations and volume losses, through reducing dilation, improving face stability and increasing ground stand-up time.



B) Face Bolting

Face dowels are spiles inserted into the excavation face to enhance the face stability, and have been shown to be very effective in providing stability to allow full-face excavation. These act in tension, and glass fibre dowels generally have the advantage over steel dowels of being easier to cut during excavation. The required number of face dowels could be determined by the minimum factor of safety targeted for face stability using limit equilibrium techniques.



C) Grouting

The grouting method is achieved by injecting the grout into the ground ahead of or near the cutting face, and is extremely effective in achieving ground stability via two means. One application is as a water sealant and to close the fractures or voids in the ground through which water passes, so that the ingress of water affecting ground stability would be controlled. The other application aims to achieve ground improvement by binding the loose ground materials ahead of the excavation and overhead, thereby preventing ravelling that may occur.

2.3 Methods of Tunnel Analysis

Tunnel analysis is a crucial part of the design process, as it gives the loads for designing and checking that the temporary supports are adequate as well as predicting the in-tunnel deformations & convergence that are instrumental in the monitoring of

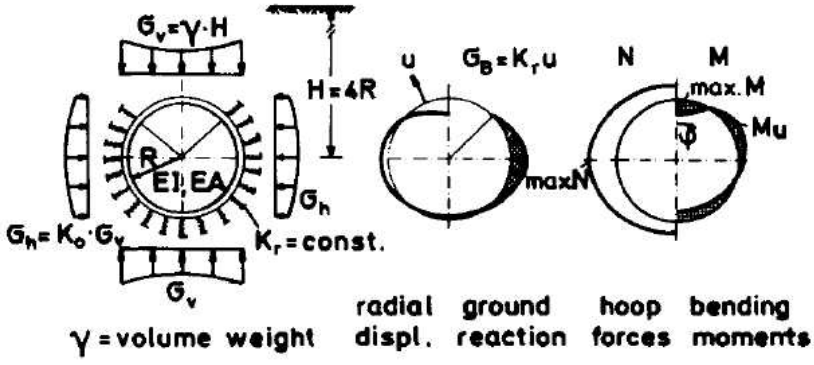
the tunnel performance during NATM works. Where possible, the forces in a tunnel lining should be mitigated by proper rounded geometry, rather than introducing sharp corners and connections in the shotcrete lining. Reinforcements should be kept to a minimum for ease of tunnelling. The following are some of the more common methods of tunnel analysis.

2.3.1 Closed-form solutions

There are several theoretical solutions primarily derived for plane strain circular tunnels in elastic grounds. The soil formation is assumed as an elastic, homogeneous medium surrounding the beam elements that represent the tunnel lining. The most famous solutions are those derived by Muir Wood (1975) and modified by Curtis (1976). As plane strain continuum models usually assume that the ground is a semi-infinite medium, these closed form solutions should only be used for deep tunnels where the axis is deeper than two tunnel diameters below the surface. Furthermore, these simple solutions may be fairly limited in their application to the rarely circular SCL tunnels, other than as a "order of magnitude" check of the more complex analyses.

2.3.2 Bedded beam models

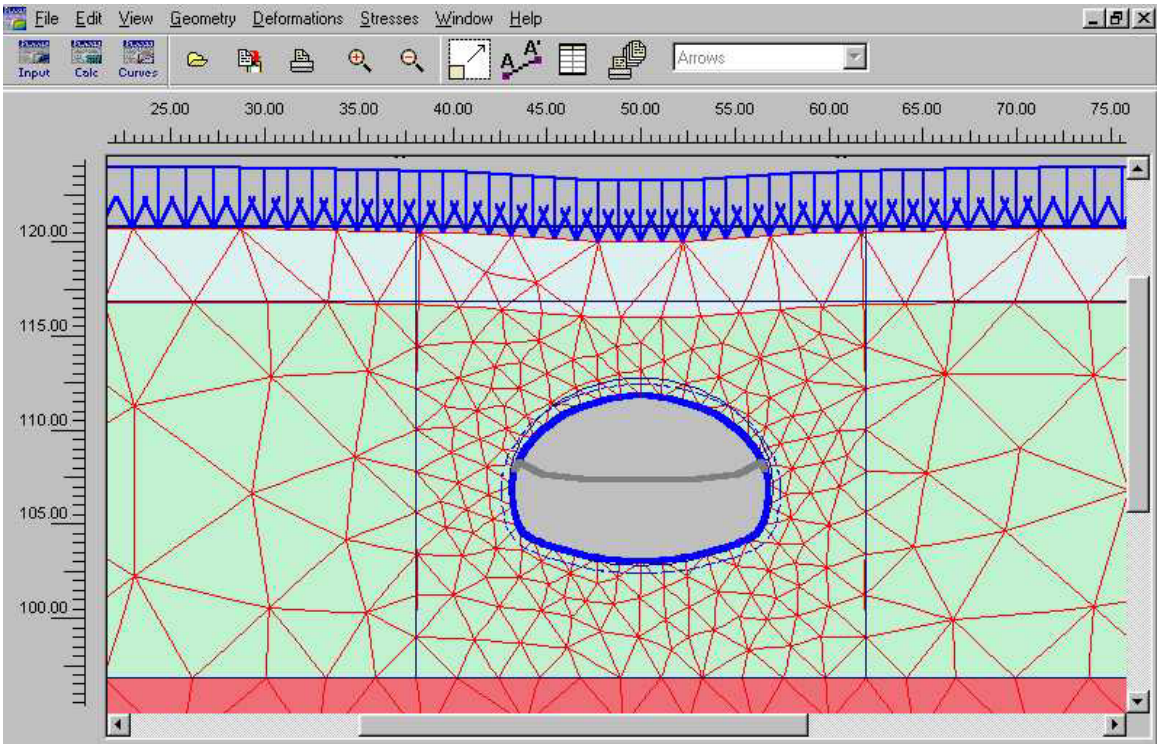
For the bedded beam model, the interaction between the lining and the soil formation is represented by a series of radial springs for normally applied loads and sometimes also by tangential springs for shear embedment at the interface between lining and soil. The soil springs are related to the modulus of subgrade reaction of the ground, and acts only in compression to allow separation of lining from the soil. The bedded beam models may not be widely used during primary support design, but are certainly useful in the design of final linings under the full overburden & ground loading conditions in the long-term.



2.3.3 Finite element methods

Finite element methods are based on the principle of discretising a body into a number of finite elements, whose behaviour is controlled by the fundamental laws of mechanics under external influences such as changed loading conditions.

The primary advantage of using finite element model is that it allows for variations to simulate the complex interaction between the lining and the ground often encountered in SCL and NATM construction. These include the time-dependent material properties of soil & tunnel support, stratified ground with varying properties, variations in boundary conditions such as porewater pressure, the sequence and dimensions of each excavation stage, the non-circular tunnel shape, and other special considerations such as multiple tunnel construction in close proximity.

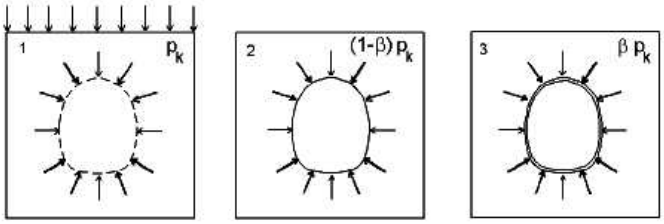


However, this requires a judicious approach on the assumptions to be made in the finite element models, and a sensitivity study on the parameters should always be carried out in the absence of good experience in similar geological & geometrical conditions. The following are some areas where a sensitivity study may be required:-

A) Pre-relief factor of the tunnel excavation advance

The advance of a tunnel excavation induces a reduction in the original primary stress in the undisturbed ground ahead of the tunnel face. The degree of reduction varies with ground conditions, construction method, and speed of the excavation & support installation. Although 3-dimensional elastoplastic finite element analyses would be required in order to model these effects properly, it is usually only practicable to undertake 2-D finite element analyses which make some empirical allowance for stress release ahead of the tunnel face. Two commonly used techniques to simplify the problem, are as follows:-

- To reduce the modulus of elasticity of elements inside the periphery of the tunnel lining to allow the stress reduction, also known as the Progressive Softening Approach (after Swoboda, 1979); and
- To unload or to release a certain percentage of the ground stress prior to installation of the lining, using the principles of the convergence-confinement method (Panet and Guenot, 1982)



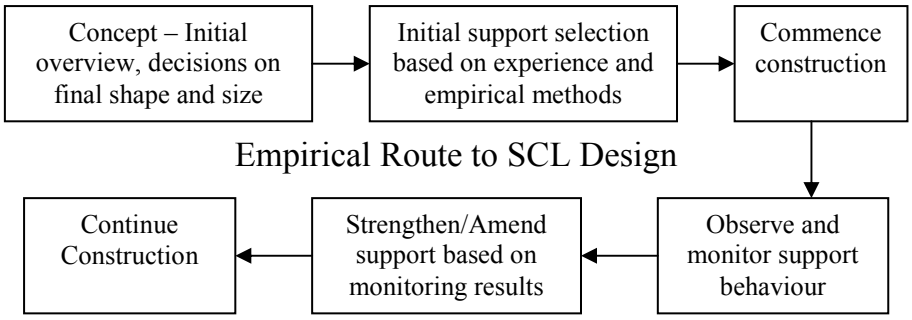
B) Best Estimate vs Worst Credible Soil Parameters

The distinction between soil parameters used for tunnel design against parameters used for tunnel monitoring should be clearly established. The designer should check the sensitivity of his model & design through a reasonable variation of the soil parameters involved. Generally, he should use the worst credible values to design for the allowable deformations, bending moments and

forces, and should use the best estimate prediction for construction monitoring at all stages of excavation.

2.3.4 Empirical route to SCL Design

The above methods of tunnel analysis relate to the analytical route to SCL design which results in SCL dimensions being defined from the foreseeable circumstances at the outset of construction. The ICE Design and Practice Guide (1996) acknowledges the alternative approach to SCL design, via the Empirical Route. See Figure below. Depending on regulatory environment, this approach may be acceptable in other countries but it certainly requires a greater degree of previous experience in similar ground conditions to determine initial lining thickness, and requires an observational method to determine the shotcrete thickness directly from the actual ground conditions and lining performance.



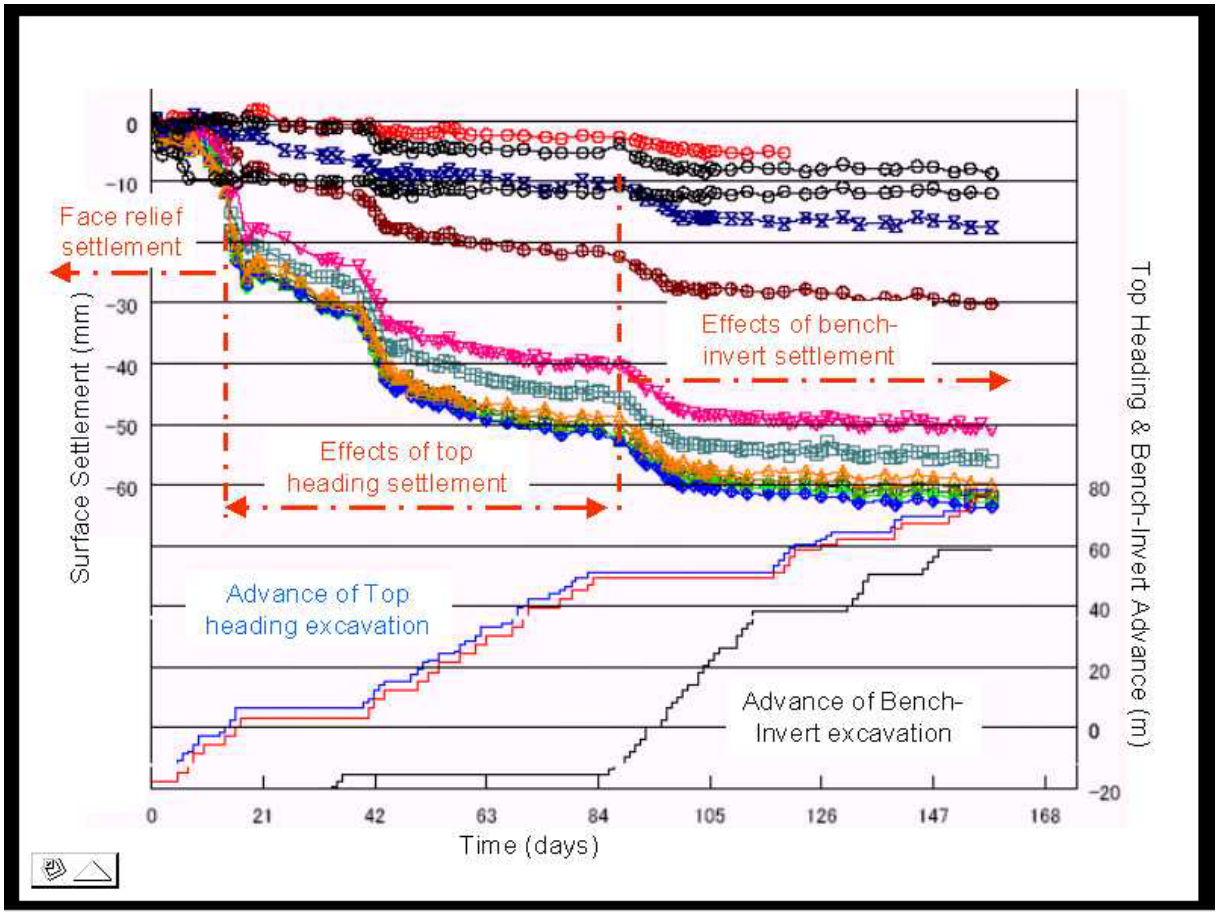
Empirical Route to SCL Design

2.4 Prediction of ground settlement

The components of ground movements associated with NATM construction may be attributed to the following:-

- ground deformation towards the excavation face resulting from stress relief
- ground deformation prior to installation of tunnel lining, above the tunnel opening
- tunnel deformation due to development of ground loading with excavation advance
- Long-term ground deformation due to creep & consolidation effects

An example of such a surface settlement plot is seen below.



Ideally, the prediction of deformation in a NATM construction should be undertaken by a 3D finite element model, which incorporates the tunnel geometry, the ground conditions and geological parameters, the sequence and speed of excavation, and the staged installation of supports and the development of shotcrete stiffness.

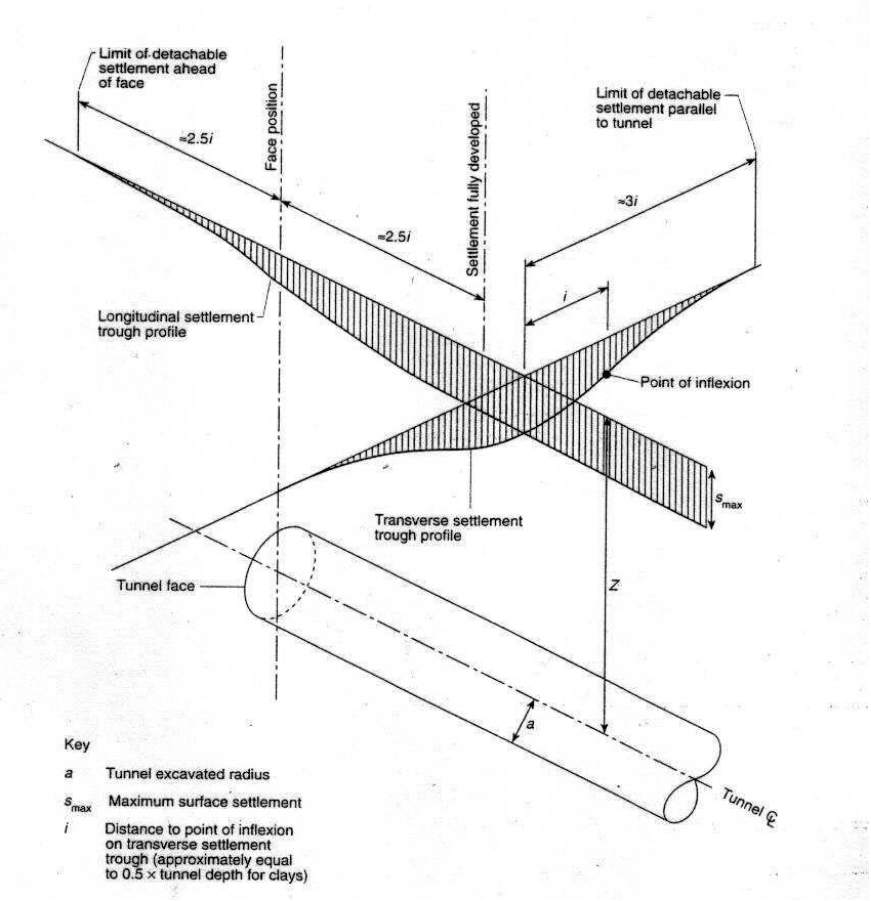
However, an empirical relation may be employed in 2D FE analyses to model the advance stress relief in NATM construction. Due to the variability of the parameters, settlement predictions should always be made in consideration with the sensitivity analyses undertaken in the design, especially in the absence of similar experience.

2.4.1 Empirical estimate from Gaussian Settlement trough

An empirical method to estimate surface settlement would be based on the integration of the Gaussian settlement trough. In the short term, Peck (1969) and O'Reilly and New (1983) have postulated that tunnelling works will generally produce a settlement trough that is Gaussian in nature and described by the trough width parameter i. The maximum settlement can then be obtained by integrating the Gaussian trough and relating this to the loss of ground due to excavation.

i.e $V_l = 2.5* i * Smax / A$, where V_l is the volume loss, $i = Kz_o$ is the trough width parameter, and Smax is the maximum ground settlement.

The volume loss is defined as the amount of ground lost in the region close to the tunnel expressed as a percentage of the excavated area of the tunnel. The magnitude of volume loss depends principally on the type of ground and the method of tunnelling. Mair (1996) reported that the recent NATM construction in London Clay has resulted in volume losses varying from 0.5-1.5%. Incidentally, LTA's Design Criteria suggested that the volume loss could vary from 0.5~1.5% for NATM excavation up to 6.6m diameter in Singapore's Jurong Formation.



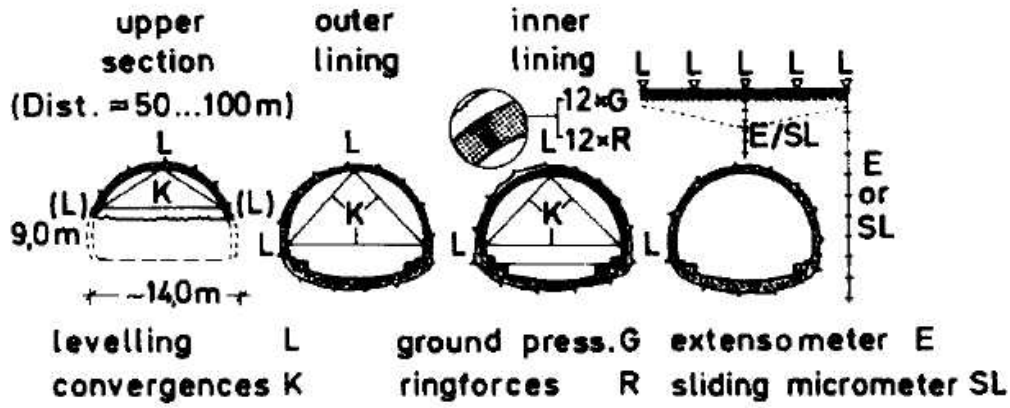
2.5 Planning for Contingency

The design of a NATM construction in soft ground develops the standard support and stabilisation measures based on reasonably anticipated ground conditions. As such, additional support measures and contingency plans should be developed to cope with ground conditions and tunnelling hazards not expected to be encountered during tunnel construction but which cannot be excluded. Prior to the actual excavation, a contingency plan should be developed detailing the additional support and stabilisation measures as well as providing response values or specific observations that trigger a contingency measure. All means and materials required to implement measures outlined should be readily available on site at any time during construction. Such measures could include spiles (either rammed rebars or pre-drilled grouted steel pipes), steel or timber propping and shoring, foot piles, face dowels, well points and drainage drifts, grouting, etc.

3.0 INSTRUMENTATION & MONITORING FOR SCL TUNNELS

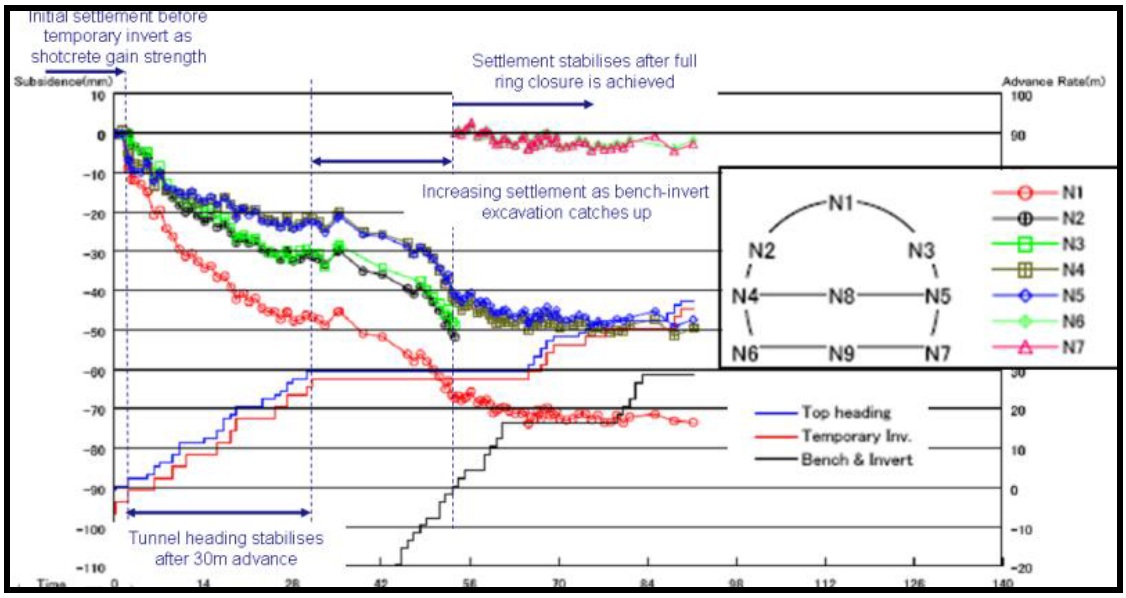
3.1 Instruments for NATM construction

Instrumentation is installed typically to provide control and performance monitoring during construction, and also to verify design parameters. For initial guidance, the Tunnel Lining Design Guide (2004) gives a listing of the instruments that are commonly employed to monitor NATM construction. See Annex B. Furthermore, the ITA Guidelines for the Design of Tunnels (1988) also shows some of the most commonly used instruments in the monitoring of the SCL tunnels.



3.2 In-tunnel deformation

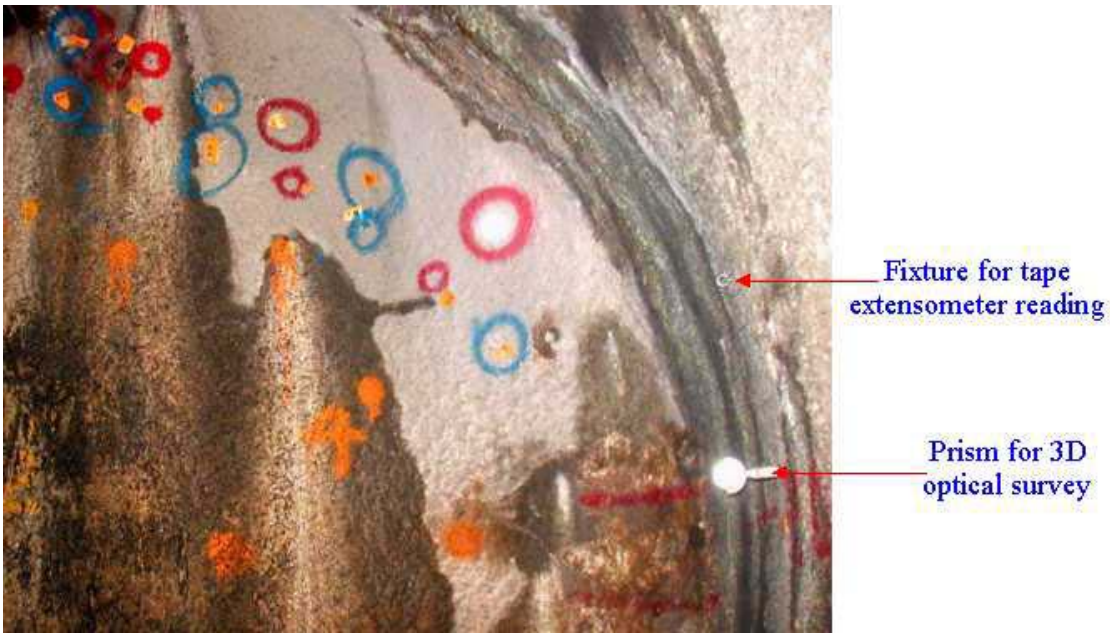
The behaviour of a SCL tunnel is best monitored using levelling points installed in the tunnel crown and other critical locations such as the footing area. This should be installed as soon as practicably possible, because the ground would have started moving once excavation has been initiated. For difficult tunnelling, the distance between two in-tunnel monitoring arrays may be as close as 10~15m. The following shows an example of the development of in-tunnel settlement as a result of increased loading due to tunnelling advance.



3.3 Convergence monitoring

To monitor tunnel integrity, tunnel convergence / divergence can be easily established and monitored as early as possible, and with a good degree of accuracy. This

measures the relative movement across the tunnel lining, and may be monitored using advanced 3D prism survey methods or simply using tape extensomers across fixed chords.



3.4 Tunnel lining forces

The use of strain gauges to monitor lining forces is often riddled with variations in the temperature, shotcrete thickness, concurrent time-development of shotcrete stiffness along with tunnel loads, etc. This makes it challenging to convert the strain values to lining loads, even if the strain gauge is able to survive the rigorous environment during shotcrete spraying. An alternative would be to use total pressure stress cells to monitor the development of stresses in SCL tunnels. For example, the ITA Guidelines for the Design of Tunnels (1988) suggest the use of stress cells to monitor ring forces in the lining, although they cautioned that expectation of reliability for pressure cells may not be met. This is because stresses and strains are very local characteristics, and convergence and deformation readings would be more reliably obtainable as displacements register integrals along a larger section of the ground. As such, the primary use of such cells is limited to tracking changes in the concrete stresses rather than to obtain the absolute stress measurements.

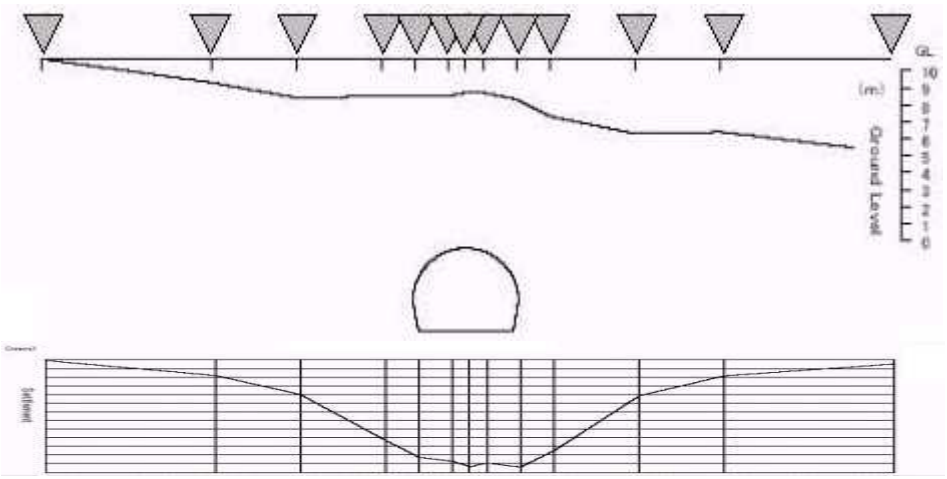
3.5 Face monitoring

The stability of the excavation face can be monitored by installing prisms and measuring out-of-plane face movements over time, especially when the face is left to creep over a period of time.



3.6 Surface Settlement

The monitoring of surface settlement is extremely important in shallow tunnels built using NATM construction. The following shows an example of a settlement marker



array above a shallow NATM tunnel.

The Japanese Standard for Mountain Tunnelling (1996) provides some guidelines on the measurement of surface and ground displacements. This is reproduced and extracted below.

Overburden, h	Necessity of surface monitoring
h < D	Very Important; Necessary to measure
D < h < 2D	Important; preferable to measure
h > 2D	Less important; to be measured if necessary
Measuring interval	Longitudinal direction: 5 to 10m Cross direction: 3 to 5m

Other instruments that can be used to monitor ground movements near to the NATM excavation works include inclinometers to measure lateral movements, and extensometers to measure sub-surface settlements ahead of the face.

3.7 Frequency of monitoring

The frequency of readings depends on how far from the tunnelling face the measurements are taken, and on the results. For example, readings may be performed two times daily when the excavation is near to the monitoring point and the monitored data is near to the alarm levels, or could be reduced gradually to once per month if the time-data curves show that the readings have stabilised and that the instrument is beyond 4 diameters behind the face.

The following table shows another example illustrated in the Japanese Standard for Mountain Tunnelling (1996), where monitoring frequency for the convergence & crown settlement was determined according to the rate of displacement and the distance from the face.

Frequency	Distance of measuring point from face	Rate of displacement
Twice / day	0 to 0.5 D	More than 10mm/day
Once / day	0.5 to 2 D	5 to 10mm/day
Once / 2 days	2 to 5 D	1 to 5mm/day
Once / week	5 D or more	Less than 1mm/day

4.0 DESIGN OF FINAL LINING

4.1 Analysis of permanent linings

The design of final linings is generally carried out using conventional structural design software appropriate to plane frame continuum analysis. Duddeck (1981) reported on an ITA survey on the structural design models for tunnelling. In particularly, the response on tunnel in soft soil supported by steel arches and shotcrete, is reproduced below and re-categorised according to the methods described in this guide:-

	Closed-form solutions	Bedded Ring models	Finite Element methods	Empirical methods
A. J. Neyland Australian Tunnelling Association	X	X		
E. Hackl, J. Golser Geoconsult		X	X	X
E. Eber TU Munich	X	X		
Philipp Holzmann AG		X	X	
Maidl Ruhr-Universitat Bochum		X		
P. Gesta Societe Generale d'Entreprises pour les Traveaux Publics		X		
I. Kitamura Japan Tunnelling Association		X	X	X
Wang Jian-Yu China Civil Engineering Society		X	X	
K. Bulka Budokop, Poland		X		
R.A. Garcia Association Espanola de los Tuneles				X
M. Odier Geotechnique Appliqee P & C Derias et Cie SA Geneve		X		
A.C. Lyons Sir William Halcrow & Partners				X

The analysis of the stresses induced in the final lining shall ignore any possible contribution from support of the imposed loads by the primary support system, but shall take into account of the following:-

- The vertical loading at the maximum and minimum overburden locations, and any asymmetrical loadings if applicable;
- The horizontal ground loading in the long term, and choosing the most critical lateral earth pressure loading coefficient as appropriate to the final tunnel geometry; and
- The ground water loading in the long term in addition to the soil loading, as well as without the effect of soil loading other than for bedding purposes.

Although it is common to represent the horizontal earth pressure as a proportion of the vertical load (i.e. $K_L\sigma_v$), it should be noted that this lateral earth pressure coefficient K_L may not resemble the horizontal earth pressure coefficient at rest K_o. This depends on the bedding of the tunnel, and should be ascertained according to ground characteristics.

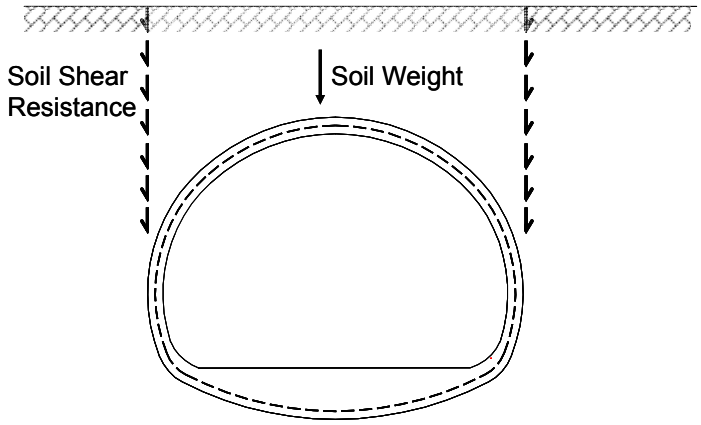
In a two-pass lining, there could be a load case in the intermediate term, where the soil loads were supported by the primary lining and water would seep through the porous shotcrete material and act upon the water-proofing membrane directly. This situation should be considered as a load case for the permanent lining design.

The following table illustrates an example of the load considerations in order to obtain the most adverse combinations in terms of lining design.

Load Case	Vertical Loads	Horizontal Loads
A	Maximum Soil + Water	Maximum Soil + Water
B	Maximum Soil + Water	Minimum Soil + Water
C	Minimum Soil + Water	Maximum Soil + Water
D	Maximum Water Only	Maximum Water Only

4.2 Flotation Check for Final Lining

The final tunnel should be checked for the possibility of flotation throughout the service life of the structure. Design ground water level should be assumed according to the requirements in the contract specifications. The tunnel flotation check would be similar to the flotation check for bored tunnels in LTA Design Criteria Chapter 7.3, i.e. Factor of safety against flotation (= Restraining force / Uplift force) should be at least 1.2, where Uplift force = buoyant weight of tunnel – self-weight of tunnel, and Restraining force = weight of soil above tunnel + shear resistance of soil above tunnel.



LIST OF REFERENCES

Babendererde S. (1980). Application of NATM for metro constructions in the Federal Republic of Germany. *Eurotunnel '80*

Broms, B.B and Bennermark H. (1967) Stability of clay at vertical openings, *Journal of the Soil Mechanics and Foundations Division, ASCE, pp. 71-94*

Copsey, J.P. & Doran, S.R. (1987) Singapore Mass Rapid Transit System Design of the Precast Concrete Segmental Tunnel Linings. *Proceedings of the Singapore Mass Rapid Transit Conference, Singapore 6-9 April1987*

Curtis, D. J. (1976), Discussion, *Geotechnique 26, 231–237*

Duddeck I.H. (1981) Views on Structural Design Models for Tunnelling – Synopsis of Answers to a Questionnaire, *International Tunnelling Association*

ICE design and practice guide (1996), Sprayed Concrete Linings (NATM) for tunnels in soft ground, *The Institution of Civil Engineers, Thomas Telford*

ITA Guidelines for the Design of Tunnels (1988), *International Tunnelling Association Working Group on General Approaches to the Design of Tunnels*

Japanese Standard for Mountain Tunnelling (1996), 5th edition, *Tunnel Engineering Committee, Japan Society of Civil Engineers*

Kimura, T and Mair, R.J (1981) Centrifugal testing of model tunnels in soft clay, *Proceedings of the Tenth International Conference on Soil Mechanics and Foundation Engineering, Stockholm, Balkema, pp. 319-322*

Mair, R.J (1979) Centrifugal modelling of tunnel construction in soft clay, *Ph.D Thesis, Cambridge University*

Mair, R.J (1996) Settlement effects of bored tunnels, *Proceedings of International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground, London, Balkema Rotterdam, pp. 43-53*

Mair, R.J and Taylor, R.N (1997) Theme lecture: Bored tunnelling in the urban environment, *Proceedings of 10th International Conference on Soil Mechanics & Foundation Engineering, Hamburg, Vol. 4, pp. 2353-2385*

Morgan, H. D. (1961), A contribution to the analysis of stresses in a circular tunnel, *Geotechnique, 11, 37-46*

Muir Wood, A. M. (1975) The circular tunnel in elastic ground, *Geotechnique 25, No.1, 115 – 127*

Panet M. and Guenot A. (1982), Analysis of convergence behind the face of a tunnel, *Tunnelling '82, Institution of Mining and Metallurgy, London, pp. 197-204*

Peck (1969) Deep excavations and tunnelling in soft ground, *Proc. 7th Int. Conf. Soil Mech. And Found. Engng, Mexico City, Vol 3, pp. 225-290*

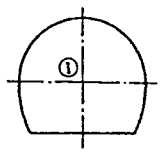
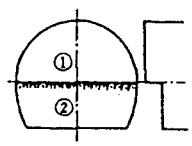
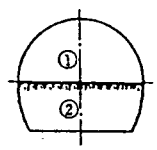
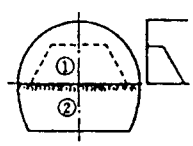
O'Reilly, M.P. and New, B.M. (1983) Settlements above tunnels in the United Kingdom, their magnitude and prediction, *Proc. Tunnelling '82, pp. 173-181 Report of discussion. Trans. Inst. Mining Metallurgy Vol. 92A, pp. A35-A48*

Swoboda, G. (1979), Finite element analysis of the New Austrian tunnelling, *Proceedings of the 3rd International Conference on Numerical Methods In Geomechanics, Aachen, Vol. 2, pp. 581-586*

Tunnel Lining Design Guide (2004), *British Tunnelling Society and The Institution of Civil Engineers, Thomas Telford*

ANNEX A & B

Table* 4.3 Classification and Characteristics of Standard Excavation Method

Excavation Method		Division of Section of Heading	Applicable Ground Conditions	Advantages	Disadvantages
Full Face Method			·Common excavation method for small section tunnel. ·Very stable ground for large section tunnel (A>50m^2) ·Fairly stable ground for medium section tunnel (A≒30m^2) ·Unfit for good grounds interspersed with poor ground that may require the change of the excavation method	·Labor saving by mechanized construction ·Construction Management including safety control is easy because of the single- face excavation.	·Full tunnel length cannot necessarily be excavated by full face alone. Auxiliary bench cut will be adopted as required. ·Fragment rocks from the top of the tunnel may fall down with increased energy & additional safety measures are required.
Full Face Method with Auxiliary Bench Cut		 Bench length ≒ 2~4m	·Comparatively stable ground, but difficult using the Full Face Method. ·Full-face excavation is made difficult during construction. ·Presence of some poor ground in fairly good ground.	·Labor saving due to mechanized construction ·Construction management including safety control is easy because of the single-face excavation.	·Difficult to switch to other excavation methods when the face does not stand up.
Bench Cut Method	Long Bench Cut	 Bench length> 50m	·Ground is fairly stable, but Full-face excavation is difficult.	·Alternate excavation of top heading and lower bench reduces equipment and manpower needs.	·Alternate excavation system elongates the construction period.
	Short Bench Cut	 D<Bench length≦ 50m	·Applicable to various grounds such as soily ground, swelling ground, and medium to hard rock ground. (The most fundamental and popular method.)	·Adaptable to changes in the ground condition.	·Parallel excavation makes difficult the balancing of cycle time for top heading and bench.

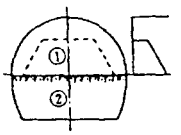
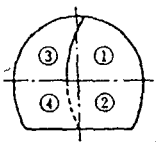
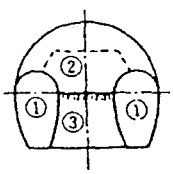
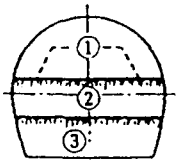
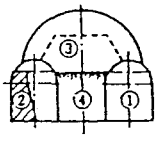
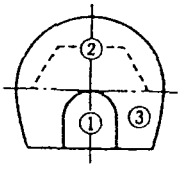
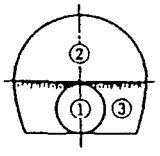
	Mini Bench Cut	 Bench length<D.	・Deformation control of the excavated inner section is more urgently required than in the case of the Short Bench Cut. ・Squeezing ground that require an early closure of the excavated section.	・Easy to make early closure of the invert	・Scaffolding is required for the top heading excavation. ・Selection for construction machines tends to be limited for top heading
Center Diaphragm Method		 One method is to provide a diaphragm only to the top heading, while the other is to provide both a top heading and a bench.	・Ground of shallow overburden where ground surface settlement is required to be kept at a minimum. ・Comparatively poor ground condition for a large section tunnel.	・Face stability is secured by dividing into small sections. ・Ground surface settlement can be significantly reduced. ・Divided sections of heading are larger than those used in the Side Drift Method, and larger machines can be used.	・Displacement or settlement during the removal of the diaphragm shall be checked. ・Time for diaphragm removal is added to the construction period. ・The adoption of a special auxiliary method in the tunnel is difficult.
Side Drift Method			・Bearing capacity of the ground is not sufficient for adopting the Bench Cut Method. ・Ground of shallow overburden where ground surface settlement is required to be kept at a minimum.	・Ground surface settlement can be reduced. ・Temporary diaphragms can be more easily removed than those of center diaphragm method.	・Small machines have to be used for drift excavation.

Table*4.4 Examples and Characteristics of Other Excavation Method

Excavation Method		Division of Section of Heading	Applicable Ground Conditions	Advantages	Disadvantages
Multiple Bench Cut Method			・Fairly good ground for long and large-section tunnel.	・Face stability is readily secured.	・Large deformation may develop if the closure is delayed. ・Each bench length is limited and working space is restricted. ・Careful operation for mucking at each bench is required.
Drift Advancing Method	Side Drift Method		・The bearing capacity of the ground is not sufficient. Improvement of the bearing capacity shall be secured before the excavation of top heading. ・Soft rock with shallow overburden where uneven distribution of geology prevails or landslide is anticipated, or soil-ground.	・Comparatively massive concrete wall for the side drift improves the bearing capacity and strengthens resistance against unsymmetrical pressure.	・Machines for drift excavation have to be smaller in size. ・Loosening of the upper ground by drift excavation may be expected.
	Bottom Drift Advancing Method		・Grounds that require water-table lowering.	・By advancing the drift, geology can be confirmed. ・By cutting up from the drift an additional section and a face, construction period can be reduced.	・Difficult to balance the cycle time for each face. ・Various combinations of machines are required.
	TBM Advancing Method	 A drift may be placed on top as the case may be.	・A drift is advanced by TBM for the confirmation of the geology and drainage effect.		

Objective	Instrumentation	• Range • Resolution • Accuracy	Comments
Relative vertical movement	BRE-type levelling sockets and precise levelling pins installed on structures, settlement monuments, geodetic surveying targets in structures or tunnel linings	• any • 0.1 mm • 0.5–1.0 mm	Includes tunnel crown levelling points; direct measurement of ground response; can be compared to empirical estimates for rapid assessment; automated theodolites can be employed; surface points may be affected by construction of pavement or road – that is, separations and 'bridging' may occur between pavement and underlying ground. When measuring very small movements, closure errors/accuracy may mask initial trends and vary according to surveyor; surface measurements are an indirect measure of tunnelling performance at depth; time consuming – data frequency limited due to manual operation; coverage may be limited due to access restrictions; levelling in some tunnel environments may achieve realistic accuracy of only 2 mm.
	Precise liquid level settlement gauges with LVDTs installed in surface structures	• 100 mm • 0.01–0.02 mm • ±0.25 mm	Direct measurement of ground/structure response; volume changes due to, say, temperature normally affect all gauges equally and can be eliminated during calculation (however, if one gauge is in a warm tunnel, and another is at the portal, for example, temperature can be a factor); risk of vandalism and effects of exposure to weather; require water and air pipes over significant distances and a stable reference gauge pot.
	Borehole magnet extensometer	• any • ±0.1 mm • ±1 mm–5 mm	Includes high precision magnet extensometer probe; simple and robust, utilises inclinometer casing thereby providing dual function in one borehole; accuracy ±0.2 mm with an electronically controlled motor unit; sub-surface data can be obtained; subject to operator variations; manually operated 'dipper' typically used – time consuming and limiting data frequency.
	Borehole rod or invar tape extensometers	• 100 mm • 0.01 mm • ±0.01 mm–0.05 mm	Direct measurement; simple installation; can measure multiple points in one hole; can be data-logged when using VW/LVDT gauges; can measure both settlement and heave; stainless steel rods may be subject to temperature variations; head requires protection; when logging continuously (i.e. in 'real time') actual data will only be at the frequency that the collar is levelled – that is manually; when using a deep datum it is assumed that no movement occurs – may not be the case; rapid changes may cause temporary loss of VW transducer – dynamic transducer may be required; can also be installed in-tunnel to monitor movements normal to tunnel boundary; accuracies with LVDT: ±10 με; VW gauge: ±1 με.
	Satellite geodesy	• Any • to ±50 mm • to ±1 mm	Satellite based levelling techniques include Differential GPS (Global Positioning Satellite) and InSAR (Synthetic Sperture Radar Interferometry). Quality of data can vary with topography, vegetation cover, availability of reflector targets, satellite orbit, and atmospheric effects. Generally applicable to long term monitoring of 'regional' movements at the present time.

Fig. 8.1 Typical applications of instrumentation in tunnelling

Objective	Instrumentation	• Range • Resolution • Accuracy	Comments
Lateral displacement	Surface horizontal BRE invar wire extensometers	• 0.01% • 0.001– 0.05% • 0.01–0.05 mm	Continuous monitoring array possible; direct measure of horizontal strain; require 100 mm diameter telescopic ducting up to 20 m in length to be installed, linked in series between instrument houses; requires substantial installation effort.
Change in inclination	Borehole electrolevels; electrolevel beams on structures and in tunnels; 'tilt meters'	• 50 mm/m (to 175 mm/m) • 0.05 mm/m (to 0.3 mm/m) • to 0.1 mm/m	Data-logged; borehole installations relatively unaffected by temperature variations; additional ground information can be obtained from borehole; can be used to measure longitudinal distortions along tunnels when continuous strings employed; borehole tilt meters and electrolevels can measure tilt in two orthogonal planes; borehole instruments require corrosion protection from groundwater; resolution dependent on beam length. Accuracy can vary with manufacturer.
	Borehole inclinometer probes	• ±53° from vertical • 0.04 mm/m • ±6 mm/25 m	Can be coupled with spider magnet extensometers to obtain the complete movement vector. When interpreting results, can be difficult to pick up small movements.
	Horizontal borehole deflectometer	• ±50 mm • ±0.02 mm • ±0.1 mm	Measures horizontal and vertical deflections. Cannot be used with standard inclinometer casing.
Changes in earth pressure	'Push-in' total pressure cells	• up to 1 MPa • up to 0.1% FS • up to 1.0% FS	Direct measure of changes of pressure in the ground; can be coupled with a piezometer cell to obtain changes in effective stress; can be data-logged using VW transducers; may not be able to obtain actual earth pressures due to installation effects – relative changes only; may require settling-in period of some weeks.
Changes in water pressure	Standpipe piezometers	• any • ±10 mm • ±10–20 mm	Simple to install; robust; rendered ineffective if water table drops below response zone; unable to assess 'real-time' fluctuations in piezometric head due to manual reading and 'lag' in response due to head losses in permeable strata; accuracy depends on operator and condition of 'dip-meter'.
	Pneumatic piezometer (pore pressures are balanced by applied pneumatic pressures)	• 0–20 bar • 0.01 bar • 0.5% FS ± 0.02 bar	Analogue, 'membrane switch' (hydraulic transducer) or digital readout can be used; not affected by very low temperatures; may be pushed into soft soils – minimising disturbance; not effective where suctions occur over sustained periods.
	Vibrating wire piezometer	• up to 35 bar, • 0.025% FS • ±0.1% FS	Can be read using a hand-held digital transducer unit, or remotely using a data-logger; standard sensors can measure suctions up to cavitation (suctions up to −1500 kPa can be measured at shallow depth using the Imperial College Suction Probe); instability in readings may occur for rapidly fluctuating piezometric levels; sensors may require settling-in period of some weeks.

Fig. 8.1 (continued)

Objective	Instrumentation	• Range • Resolution • Accuracy	Comments
Crack or joint movement	Tell-tales	• ±20 mm • 0.5 mm • ±1 mm	Direct measurement of ongoing movement; local point measurement; does not give quantitative measurements of stress and strain; some instruments subject to temperature corrections.
	Calliper pins/ micrometer (DEMEC gauges)	• up to 150 mm • 0.02 mm • ±0.02 mm	DEMEC gauge has a more limited range but resolution to 0.001 mm and accuracy to 0.005 mm. Pins simple and inexpensive to install.
	Vibrating wire joint-meters	• up to 100 mm • up to 0.02% FS • up to 0.15% FS	Can measure three orthogonal directions with triaxial device; built-in temperature correction; can be data-logged; simple surface installation but needs to be protected from vandalism.
Strain in structural member or lining	VW strain gauges	• up to 3000 με • 0.5–1.0 με • ±1–4 με	High accuracy; direct measurement at a point; generally robust and reliable; can be waterproofed for exposed conditions; gauges can be directly installed on rebar or flanges of cast-iron segments, or on rock bolts; provide information on that member only – no indication of overall structure performance; small gauge lengths result in highly localised measurements; may be susceptible to corrosion or damage if not adequately protected; temperature corrections may be required; pattern of strain may be highly variable and difficult to convert into stress; results may be affected by heat of hydration in concrete during curing, cracking and grouting.
	Fibre optics	• to 10,000 με (1% strain) • 5 με • 20 με	Glass cables are light and corrosion resistant; easy to splice cables for long lengths (range from 10 cm to 1 km); can insert many sensor locations along cable length (depending on wavelength of light); can multiplex up to +100 cables; can be embedded in concrete or mounted on a structure; can operate in temperatures between −20 °C and +50 °C.
Tunnel lining diametrical distortion	Tape extensometers across fixed chords	• up to 30 m • 0.001–0.05 mm • ±0.003–0.5 mm	Traditional approach, results 'understood'; simple and portable; direct measurement of relative distortions (only); measurement may disrupt excavation cycle; accuracy may decrease with increasing span; access difficulties may arise in large excavations or shafts; possible interference in construction cycle; results affected by operator experience and temperature fluctuations; cannot be automated; indirect measure of tunnel lining performance.
	3D geodetic optical levelling ('retro' or 'bioflex') targets, levelling diodes or prisms	• any • 0.1–1.0 mm • 0.5–2.0 mm	Rapid monitoring of a large number of points possible; reading can be fully automated and data-logged using motorised instruments; absolute measurements of position obtained; mounting bolts can be used for other measurements such as tape extensometers; in the tunnel environment, usually best to have targets within 100 m of station; monitoring may obstruct construction cycle; indirect measure of tunnel lining performance; probably the most common method used to monitor distortion during construction, at the time of writing.

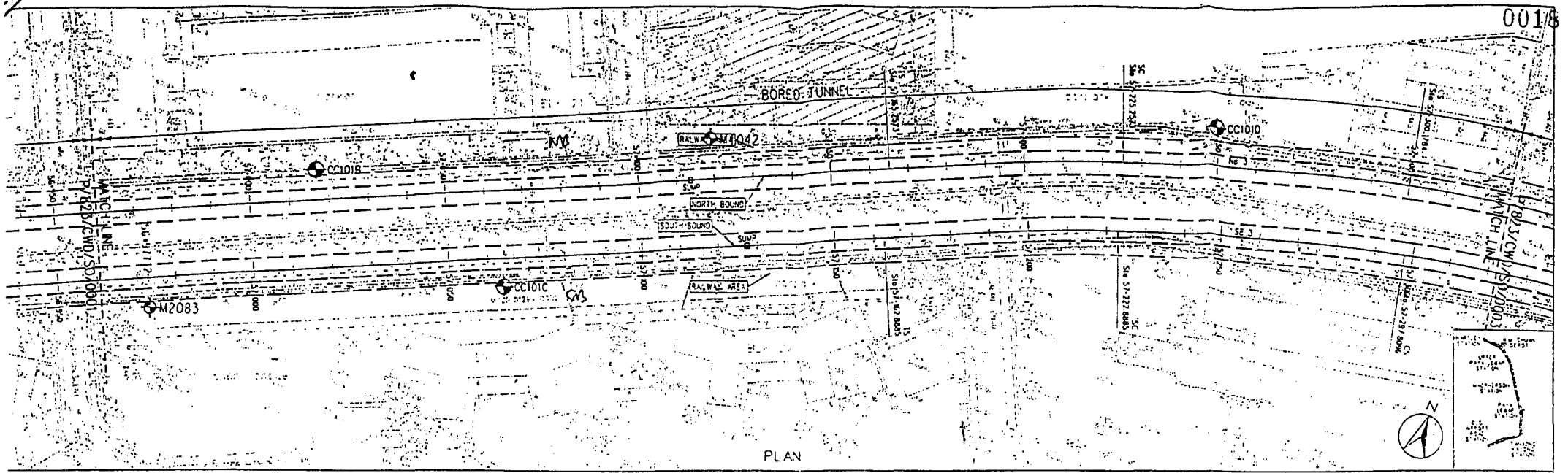
Fig. 8.1 (continued)

Objective	Instrumentation	• Range • Resolution • Accuracy	Comments
Tunnel lining diametrical distortion (cont'd)	Strain gauged borehole extensometers installed from within tunnel	• 100 mm (3000 με) • 0.01 mm (0.5 με) • ±0.01–0.05 mm (±1–10 με)	Direct measurement; simple installation; measure multiple points in one hole; can be data-logged when using VW gauges; accuracy LVDT: ±10 με; micrometer: ±0.01 mm; stainless steel rods may be subject to temperature variations; head requires protection; the deepest anchor is assumed to be beyond the disturbed zone of influence – if not, relative movements may be underestimated.
	Basset Convergence system	• ±50 mm • 0.02 mm • ±0.05 mm	Interlinked tilt sensor array; permits real-time monitoring/data-logging of lining distortion.
Lining stresses	Total pressure (or 'stress') cells	• 2–20 MPa • 0.025–0.25% FS • 0.1%–2.0% FS	Direct measure of subsequent changes in earth pressure at a point; total pressure (or 'stress') cells installed between lining and ground (tangential pressure cells) or cast into lining (radial pressure cells) utilising membrane switch (read using an oil pressure gauge) or VW transducers. Comprise either mercury (high pressure) or oil-filled (low pressure) cells; can be installed between segment joints; better accuracy and resolution obtained from lower range cells; actual pressures not measured due to relative stiffness effects; installation may affect quality of results – requires experience; primary stress state has already been altered by the excavation; may not give realistic estimates due to localised point loads etc.; often need re-pressurising after lining concrete has cured due to concrete shrinkage; a knowledge of concrete creep and deformation characteristics required during interpretation post construction testing such as the flat-jack also possible.
Lining leakage	Flow meter	• any • 1 litre/min • 2 litre/min	Indirect measure of overall inflow; simple apparatus; can be data-logged using a submersible pressure transducer.
Vibration	Triaxial vibration monitor/seismograph	• 250 mm/sec • 0.01–0.1 mm/sec • 3% at 15 Hz	Measures PPV and accelerations in three orthogonal axes; portable equipment.

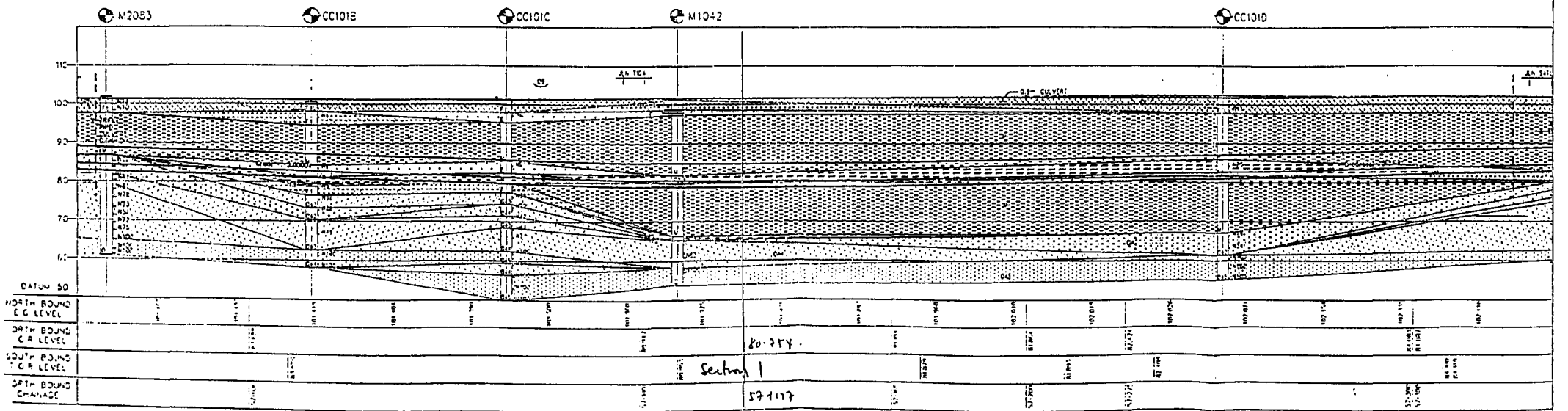
Notes:
1. Quoted range/resolution and accuracy derived from published and trade literature as an indication of relative performance only. May change with ongoing technical development by manufacturers.
2. For borehole installations, additional information can be obtained from logging/in situ testing.
3. Definitions: range = maximum and minimum recordable values for the instrument, resolution = the smallest change that can be recorded by the instrument, accuracy = difference between recorded value and the 'actual' value as quoted by the manufacturers, rather than a measure of field performance; FS = full scale.

Fig. 8.1 (continued)

APPENDIX A



PLAN



LONGITUDINAL SECTION NORTH AND SOUTH BOUND TRACK

PRE-FINAL DESIGN FOR ACCEPTANCE

Land Transport Authority

BOREHOLE PLAN AND SOIL PROFILE SHEET 2

Consultants: Mouchel Consultants (Singapore) Pte Ltd in Association with SAA Partnership/ Ruju Hyder Consulting Pte Ltd Environmental Resources Management (S) Pte Ltd Halpin Consulting Engineers Davis Langdon & Seah

Project Title: CONTRACT 870A - CIRCLE LINE - STAGE 2 MAINLINE

Designed: LWT/HCW | Checked: GVR | Approved: AFC

Scale: 1:500 | Drawn: SHL | Date: 01/07/01

Drawing No.: P/823/CWD/SO/0002 | Rev: A | Sheet: 02 of 23

CAD File Name: P823CWDSO0002A

P:\Soil\P823CWD-SO0002A.dgn 12/07/01 02:37:15 PM

0018

CATION: Old Airport Road

BOREHOLE NO. M 1042
NORTHING : 32203.3
EASTING : 33734.7
REDUCED LEVEL : 101.9m

0019

FIELD & LABORATORY DATA & TESTS REPORTED ELSEWHERE	SPT N VALUE	SPT (N) / cm	WEATHERING GRADE	REDUCED LEVEL (m)	DEPTH (m)	SAMPLE TYPE	GRAPHIC LOG	THICKNESS (m)	GEOLOGICAL CLASSIFICATION	BS SOIL CLASSIFICATION	DESCRIPTION
Trial Pit : 1.00mx1.00mx1.00m				101.6				0.30	FS	SC	(From 0.00 to 0.30m) Clayey fine to coarse SAND with gravels and root remains brown , loose
1 auger : 1.00m-2.00m				101.4				0.20	FS		
@ 2.00m - pH for water sample					1						(From 0.30 to 0.50m) Asphalt and concrete
age = 8.2					2			2.50	FI	SC	
		0		98.9	2.00	UD 1					(From 0.50 to 3.00m) Clayey fine to coarse SAND light yellow and light grey , loose
					2.60						
					3.25	SPT1					
				97.6	4			1.30	E	CO	(From 3.00 to 4.30m) Organic CLAY with decomposed timber and vegetation dark brown , very soft
					5.50						
					6	PS 1					
					6.50						
@ 9.00m- VST 15/2 kPa					7						
					8						
50m- VST 17/4 kPa					9						
					10						
					11						
					11.00			15.50	M	C	(From 4.30 to 19.80m) Marine CLAY with seashell fragments grey very soft to soft
					12	PS 2					
					12.00						
@ 13.50m- VST 24/5 kPa					13						
					14						
@ 15.00m- VST 29/5 kPa					15						
					16						
@ 16.50m- VST 30/5 kPa					17						
@ 18.00m- VST 32/7 kPa					18						
					19						
@ 19.50m- VST 35/10 kPa				82.1	20						(From 19.80 to 22.00m) Silty CLAY with traces of fine sand light grey and brown , firm

ING TYPE
- ROTARY / WASHING BORING

METER OF BOREHOLE
100mm

FRACTURE PER METER | RQD % | SCR % | TCR % | WEATHERING GRADE

SPT SAMPLE (SPT)
UNDISTURBED SAMPLE (UD)
CORE RUN
PISTON SAMPLE (PS)
OPEN DRIVE SAMPLE (OD)
MAZIER SAMPLE (MZ)
Vane Shear Test (VST)

CLIENT: Land Transport Authority

LOG OF BORING
GEOTECHNICAL STUDY - FIELD INVESTIGATIONS

SOIL & FOUNDATION (PTE) LTD 17 1109

PROJECT: C 8201 - SITE INVESTIGATION FOR THE PROPOSED MARINA LINE

SUPERVISOR: JERICK F.C

DATE OF FIELD WORK: 26/06/98 - 02/07/98

CONTRACTOR: Soil & Foundation (Pte) Ltd

PREPARED BY: WAI LENG

SHEET NO: 1 of 3

f63

CATION: Old Airport Road

BOREHOLE NO. M 1042
NORTHING : 32203.3
EASTING : 33734.7
REDUCED LEVEL : 101.9m
0020

FIELD & LABORATORY DATA & TESTS REPORTED ELSEWHERE	SPT N VALUE	SPT (N) / cm	WEATHERING GRADE	REDUCED LEVEL (m)	DEPTH (m)	SAMPLE TYPE	GRAPHIC LOG	THICKNESS (m)	GEOLOGICAL CLASSIFICATION	BS SOIL CLASSIFICATION	DESCRIPTION
		6		79.9	UD 2 20.00 – 20.70 SPT2 21.00 – 21.15			2.70	F2	CM	(From 19.80 to 22.00m) Silty CLAY with traces of fine sand light grey and brown firm
4.00m- VST 25/4 kPa											
5.50m- VST 28/4 kPa											
7.00m- VST 35/13 kPa											
9.50m- VST 37/14 kPa								13.40	M	C	(From 22.00 to 35.40m) Marine CLAY with traces of seashell fragments grey very soft to soft
0.00m- VST 35/13kPa											
11.50m- VST 36/14 kPa											
3.00m- VST 35/11 kPa											
34.50m- VST 38/12 kPa				66.5							
				65.4	36			1.10	F2	CS	(From 35.40 to 36.50m) Fine sandy CLAY light greenish grey , firm
		71			OD 1 36.00 – 36.70 SPT3 37.00 – 37.15			4.00	O	SM	(From 36.50 to 40.50m) Silty fine SAND greenish grey and brown very dense
					Mazier 38.20 – 38.80 MZ						

RING TYPE
ROTARY / WASHING BORING
AMETER OF BOREHOLE
100mm

FRACTURE PER METER | RQD % | SCR % | TCR % | WEATHERING GRADE

SPT SAMPLE (SPT)
UNDISTURBED SAMPLE (UD)
CORE RUN
PISTON SAMPLE (PS)
OPEN DRIVE SAMPLE (OD)
MAZIER SAMPLE (MZ)
Vane Shear Test (VST)

SOIL & FOUNDATION (PTE) LTD

CLIENT: Land Transport Authority

LOG OF BORING
GEOTECHNICAL STUDY - FIELD INVESTIGATIONS

PROJECT: C 8201 - SITE INVESTIGATION FOR THE PROPOSED MARINA LINE

SUPERVISOR: JERICK F.C

DATE OF FIELD WORK: 26/06/98 - 02/07/98

CONTRACTOR: Soil & Foundation (Pte) Ltd

PREPARED BY: WAI LENG

SHEET NO: 2 of 3

C64

LOCATION: Old Airport Road

BOREHOLE NO. M 1042
NORTHING : 32203.3
EASTING : 33734.7
REDUCED LEVEL : 101.9m
0021

FIELD & LABORATORY DATA & TESTS REPORTED ELSEWHERE	SPT N VALUE	SPT (N) / cm	WEATHERING GRADE	REDUCED LEVEL (m)	DEPTH (m)	SAMPLE TYPE	GRAPHIC LOG	THICKNESS (m)	GEOLOGICAL CLASSIFICATION	BS SOIL CLASSIFICATION	DESCRIPTION
		63		61.4	39.80 Alluvial MZ			4.00	O	SM	(From 36.50 to 40.50m) Silty fine SAND greenish grey and brown , very dense
					40.80 SPT4 41.25						
				58.9	43.00			2.50	O	CM	(From 40.50 to 43.00m) Silty CLAY grey and brown very stiff to hard
		100/21		56.0	43.80 OD 2 44.50 SPT5 44.88 CR1 45.88			4.90	O	SC -SM	(From 43.00 to 47.90m) Clayey/silty fine to coarse SAND grey and brown very dense
				55.3	45.88 CR2 46.88						
				54.0	46.88 CR3 47.90						
											End of borehole at 47.90m.

Groundwater Level (measured from ground level)

Date	Time	Borehole Depth (m)	Casing Depth (m)	Water Level (m)
26/06/98	08:50	2.00	Nil	1.00
27/06/98	08:40	5.00	5.00	1.60
28/06/98	08:55	13.50	11.00	0.80
29/06/98	08:50	23.00	23.00	1.90
30/06/98	08:45	34.50	26.00	6.20
01/07/98	08:45	41.25	39.20	4.40
02/07/98	09:00	44.80	43.80	5.50

DRING TYPE
ROTARY / WASHING BORING

DIAMETER OF BOREHOLE
100mm

FRACTURE PER METER | RQD % | SCR % | TCR % | WEATHERING GRADE

SPT SAMPLE (SPT)
UNDISTURBED SAMPLE (UD)
CORE RUN
PISTON SAMPLE (PS)
OPEN DRIVE SAMPLE (OD)
MAZIER SAMPLE (MZ)
Vane Shear Test (VST)

CLIENT: Land Transport Authority

LOG OF BORING
GEOTECHNICAL STUDY - FIELD INVESTIGATIONS

PROJECT: C 8201 - SITE INVESTIGATION FOR THE PROPOSED MARINA LINE

SUPERVISOR: JERICK F.C

DATE OF FIELD WORK: 26/06/98 - 02/07/98

CONTRACTOR: Soil & Foundation (Pte) Ltd

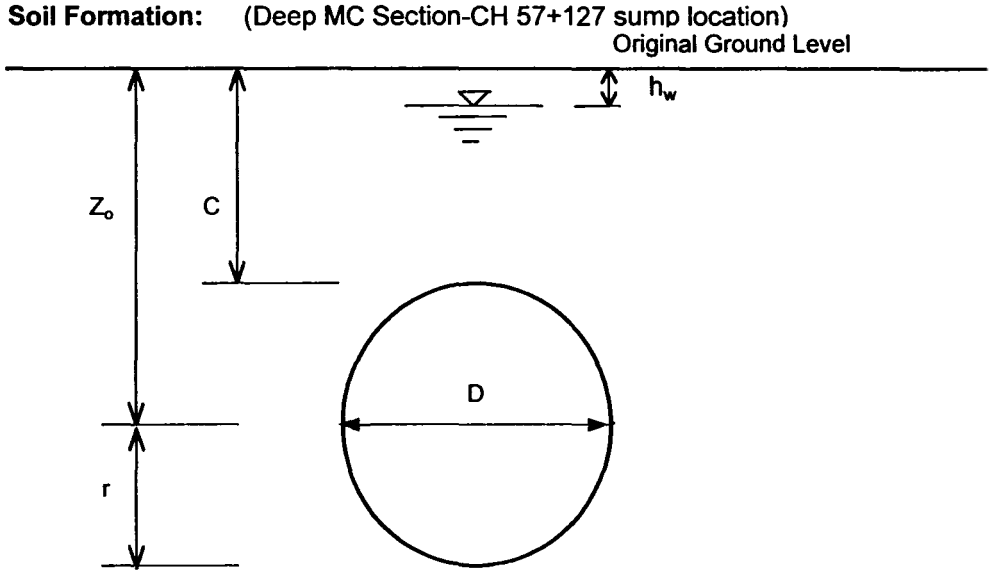
PREPARED BY: WAI LENG

SHEET NO: 3 of 3

C65

TUNNEL LINING DESIGN
[Based on Muir Wood (1975) & Curtis (1976)]

Location: Old Airport to Tanjong Katong (M1042)
Soil Formation: (Deep MC Section-CH 57+127 sump location)



References:

Muir Wood, A. M. (1975) The circular tunnel in elastic groun *Geotechnique 25, No. 1, 115 - 127*

Curtis, D. J. (1976) Discussion on the reference abov *Geotechnique 26, No. 1, 231 - 237*

Duddeck, H., Erdmann, J. (1982) Structural design models for tunnels,
Tunnelling 82, International Symposium organised by Institution of Mining & Metallurgy

Circle Line Contracts, Design Criteria, *Land Transport Authority, Singapore*

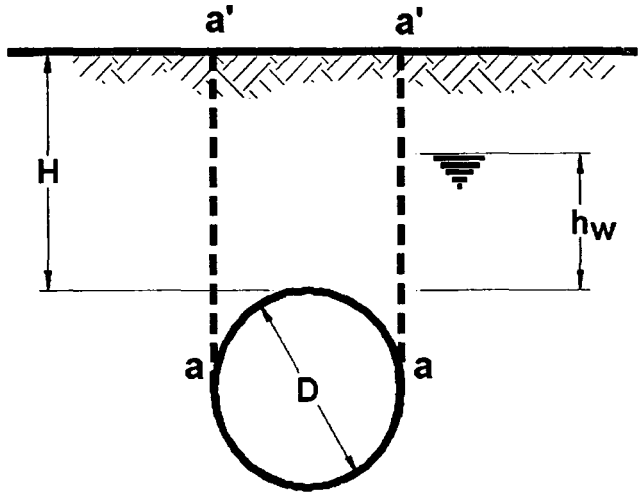
Notation

Symbols	Description
C	cover to tunnel crown
z_o	depth to tunnel axis
D	excavated tunnel diameter
r_o	radius to extrados of tunnel lining
γ	average unit weught of overburden
k	constant
E	Young's modulus for lining (replaced by $E/(1-\nu_1^2)$ where lining continuous along tunnel)
E_c, ν	Young's modulus and Poisson's ratio of ground
I	second moment of initia of lining per unit length of tunnel
I_e	effective value of I for a jointed lining
I_j	effective value of I at joint in a lining
M	bending moment in lining per unit length of tunnel
N	Hoop (circumferential) thrust in lining per unit length of tunnel
η	ratio of radius of lining centroid to that of extrados
u_{max}	maximum radial movement of lining
h_w	water table from ground surface

Location: Old Airport to Tanjong Katong
(Deep MC Section-CH 57+127 sump location)

1. TUNNEL & SOIL PROPERTIES

Nominal Diameter of Tunnel D_n =	5.60	m
Construction Allowance DD =	100.00	mm
Thickness of Lining t =	275.00	mm
Existing ground level GL =	101.925	m
Track level RL1 =	80.754	m
Track Level to Invert of Tunnel d =	1375.00	mm
Excavated Diameter of Tunnel D =	6.3500	m
Internal tunnel radius r_i =	2.9000	m
Radius to lining extrados r_e =	3.1750	m
Radius of lining centroid r_o =	3.0375	m
Depth to tunnel axis z_o =	19.6460	m
Unit weight of water γ_w =	10	kN/m^3
Water table from ground surface =	3.00	m
ie. h_w =	13.47	m



Density of concrete =	24.00	
Weight of 1st stage concrete W_1 =	0.00	kN/m
(Neglect 1st stage concrete)		
Weight of concrete lining W_2 =	125.96	kN/m
Factored self weight of tunnel, W =	$(W_1+W_2)/1.05$	
=	119.96	kN/m
Average shear resistance along a-a' =	29.47	kN/m^2

{ For cohesive soil, $S = c_u$ }
{ For cohesionless soil, $S = \frac{1}{2} K_o \gamma' (H+D/2)$ }

Ave. unit weight of soil above tunnel γ =	16.00	kN/m^3

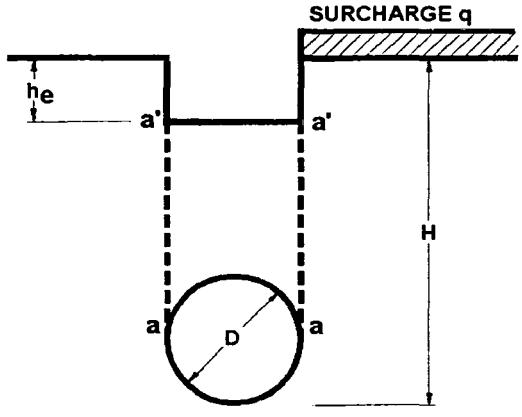
2. FLOTATION

Reference: LTA Civil Design Criteria, section 7.3.3.1

	Uplift $U = \gamma_w (\pi D^2/4) - W =$	196.73	kN/m run
	Depth to tunnel crown H =	16.47	m
	Restraining force R = R1 + R2 + R3		
	$R1 = \gamma' D (h_w + D/2 - \pi D/8) =$	539.20	kN/m run
	$R2 = \gamma_b D (H - h_w) =$	304.80	kN/m run
Shear strength of soil above slip plane	$S (H + D/2) =$	1157.90	kN/m run
	ie Restraining force R =	2001.90	kN/m run
	Overall factor of safety against flotation R/U =	10.18	
		>1.2 --> OK	

3. HEAVE AT TUNNEL INVERT

Reference: LTA Civil Design Criteria, section 7.3.3.2



$$F = \frac{N_c C_u + 2 S (H - D/2 - h_e)/D}{0.25 (\gamma_{b1} \pi D) - W/D + q + \gamma_{b2} h_e}$$

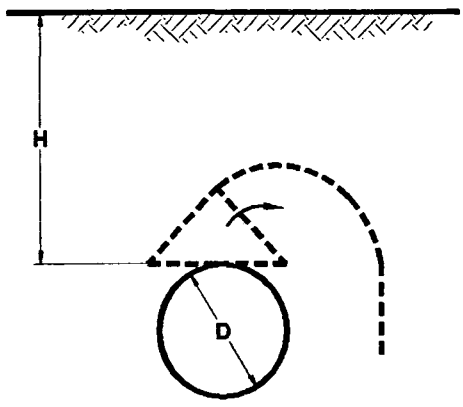
Bearing capacity factor Nc = (after Meyerhoff chart)	7.5	
Factored mean shear strength at tunnel invert Cu =	17.12	kN/m^2
Depth to tunnel invert H =	22.82	m
Depth to excavation above tunnel he =	3	m
Factored soil bulk density in zone of tunnel γ_{b1}=	13.91	kN/m^3
Factored soil bulk density in excavated zone γ_{b2}=	13.91	kN/m^3

Without surcharge,

Overall factor of safety against heave F =	3.07	
	>1.2 --> OK	

With surcharge at ground level beside tunnel, q =	22.5	kN/m^2
Overall factor of safety against heave F =	2.47	
	>1.0 --> OK	

4. HEAVE AT TUNNEL CROWN



Reference: LTA Civil Design Criteria, section 7.3.3.3

Uplift $U = \gamma_b (\pi D^2/4) - W =$	386.74	kN/m run
Restraining force R =	$D.Nc.C_u$	
where Nc =	8.25	(Meyerhoff)
Undrained cohesion at tunnel axis =	29.47	
Factored cohesion at tunnel axis Cu =	14.73	
ie R =	771.90	kN/m run
Overall factor of safety against flotation R/U =	2.00	
	>1.0 --> OK	

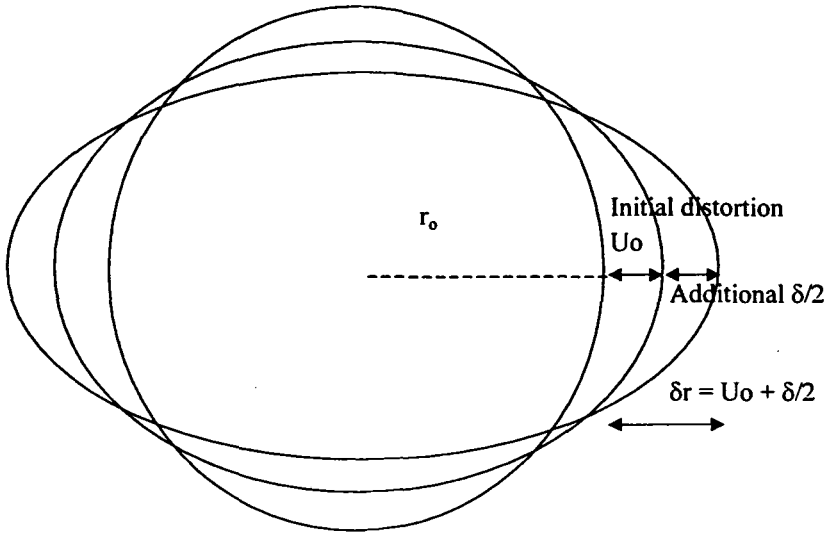
Old Airport to Tanjong Katong
(Deep MC Section-CH 57+127 sump location)

	Load Case	N-axis (kN)	U-axis (mm)	M-axis (kNm)	M-axis, future development	Total M-axis (kNm)
ULS	1	1392.46	3.84	79.05	0	79.05
	2	1769.99	6.84	136.53	0	136.53
	3	1391.24	4.93	99.17	0	99.17
	4	1768.78	7.94	156.65	0	156.65
	5	1757.91	17.37	109.07	55.45	164.52
SLS	6	994.61	2.74	56.46	0	56.46
	7	1230.57	4.62	92.39	0	92.39
	8	993.75	3.52	70.83	0	70.83
	9	1229.70	5.40	106.76	0	106.76
	10	1222.30	11.82	74.33	39.61	113.94
	11	1224.16	10.12	64.33	0	64.33
	12	1222.30	11.82	74.33	0	74.33

	Load Case	N-crown (kN)	U-crown (mm)	M-crown (kNm)	Total M-crown (kNm)
ULS	1	1269.65	-4.73	79.05	79.05
	2	1557.89	-7.96	136.53	136.53
	3	1237.18	-5.82	99.17	99.17
	4	1525.42	-9.05	156.65	156.65
	5	1533.62	-19.59	109.07	164.52
SLS	6	906.89	-3.38	56.46	56.46
	7	1087.04	-5.40	92.39	92.39
	8	883.70	-4.16	70.83	70.83
	9	1063.85	-6.17	106.76	106.76
	10	1069.44	-13.36	74.33	113.94
	11	1091.88	-11.68	64.33	64.33
	12	1069.44	-13.36	74.33	74.33

Location: Old Airport to Tanjong Katong
(Deep MC Section-CH 57+127 sump location)

LOADING DUE TO ADDITIONAL DISTORTION



For 15mm additional distortion on diameter,

Change in radius, δ/2	7.5	mm

Using Morgan's formula, bending moment due to distortion over radius, $M = (3EI/r_o^2)\delta r$

For long term stiffness of concrete, E =		16000	MN/m^2
Excavated radius of tunnel, r_o =		3.175	m
Moment of inertia of flexible lining, I =		0.001109167	m^4
At SLS	M =	39.61	KNm/ m run
At ULS	M =	39.61x1.4	KNm/ m run
	=	55.45	KNm/ m run

Location: Old Airport to Tanjong Katong (ULS for short term - no creep)
(Deep MC Section-CH 57+127 sump location) Rigid linings Load Case 1

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	0.00	m	
Effective overburden pressure	q_1 =	117.8760	kN/m^2	
Surcharge	q_2 =	0.00	kN/m^2	
Load factor for Overburden Load	FS =	1.40		
Load factor for Surcharge	FS =	1.60		
Factored vertical stress	σ_v' =	165.0264	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	123.7698	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	41.2566	kN/m^2	
Load factor for Water	FS_w =	1.40		
Hydrostatic water pressure	p_w =	275.0440	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	103.1415	kN/m^2	
Maximum shear strength of ground	τ =	41.6719	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	41.6719	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3 (2S_n+S_t)/18EI$

$U = -r_e r_o^3 (2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 22.29 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1797	13.4472	22.2855	873.2647	457.7896	-0.2946	-0.1545

N_d (kN)	u_d (mm)	M_d (kN-m)
-61.40	-4.29	-79.05

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1269.65	-4.73	-79.05	**CROWN**
10	1273.35	-4.48	-74.28	
20	1284.02	-3.73	-60.56	
30	1300.35	-2.59	-39.52	
40	1320.39	-1.19	-13.73	
45	1331.05	-0.45	0.00	
50	1341.72	0.29	13.73	
60	1361.76	1.69	39.52	
70	1378.09	2.83	60.56	
80	1388.75	3.58	74.28	
90	1392.46	3.84	79.05	**AXIS**

Location: Old Airport to Tanjong Katong (ULS for short term - no creep)
(Deep MC Section-CH 57+127 sump location) Rigid linings Load Case 2

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	0.00	m	
Effective overburden pressure	q_1 =	117.8760	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Soil Overburden	FS =	1.40		
Load factor for Surcharge	FS =	1.60		
Factored vertical stress	σ_v' =	285.0264	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	213.7698	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	71.2566	kN/m^2	
Load factor for Water	FS_w =	1.40		
Hydrostatic water pressure	p_w =	275.0440	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	103.1415	kN/m^2	
Maximum shear strength of ground	τ =	41.6719	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	41.6719	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60 N/mm^2)	
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	($I_j << I$)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 38.49 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1797	23.2254	38.4905	873.2647	790.6743	-0.2946	-0.2668

N_d (kN)	u_d (mm)	M_d (kN-m)
-106.05	-7.40	-136.53

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1557.89	-7.96	-136.53	**CROWN**
10	1564.28	-7.52	-128.30	
20	1582.70	-6.23	-104.59	
30	1610.91	-4.26	-68.26	
40	1645.52	-1.85	-23.71	
45	1663.94	-0.56	0.00	
50	1682.35	0.72	23.71	
60	1716.96	3.14	68.26	
70	1745.18	5.11	104.59	
80	1763.60	6.39	128.30	
90	1769.99	6.84	136.53	**AXIS**

Location: Old Airport to Tanjong Katong (ULS for short term - no creep)
(Deep MC Section-CH 57+127 sump location) Rigid linings Load Case 3

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	147.8760	kN/m^2	
Surcharge	q_2 =	0.00	kN/m^2	
Load factor for Overburden Load	FS =	1.40		
Load factor for Surcharge	FS =	1.60		
Factored vertical stress	σ_v' =	207.0264	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	155.2698	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	51.7566	kN/m^2	
Load factor for Water	FS_w =	1.40		
Hydrostatic water pressure	p_w =	233.0440	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	129.3915	kN/m^2
Maximum shear strength of ground	τ =	52.2776	kN/m^2 ($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	52.2776	kN/m^2 ($\tau = c' + p_u \tan\phi'$)	
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 27.96 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1797	16.8696	27.9572	739.9147	574.2993	-0.2497	-0.1938

N_d (kN)	u_d (mm)	M_d (kN-m)
-77.03	-5.38	-99.17

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1237.18	-5.82	-99.17	**CROWN**
10	1241.83	-5.49	-93.19	
20	1255.21	-4.56	-75.97	
30	1275.70	-3.13	-49.58	
40	1300.84	-1.38	-17.22	
45	1314.21	-0.44	0.00	
50	1327.59	0.49	17.22	
60	1352.73	2.24	49.58	
70	1373.22	3.67	75.97	
80	1386.60	4.61	93.19	
90	1391.24	4.93	99.17	**AXIS**

Location: Old Airport to Tanjong Katong (ULS for short term - no creep)
(Deep MC Section-CH 57+127 sump location) Rigid linings Load Case 4

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	147.8760	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.40		
Load factor for Surcharge	FS =	1.60		
Factored vertical stress	σ_v' =	327.0264	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	245.2698	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	81.7566	kN/m^2	
Load factor for Water	FS_w =	1.40		
Hydrostatic water pressure	p_w =	233.0440	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	129.3915	kN/m^2	
Maximum shear strength of ground	τ =	52.2776	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	52.2776	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	($I_j << I$)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 44.16 kN

$S_n = \{3(3-4\nu)p_o/2 -[2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1797	26.6478	44.1623	739.9147	907.1839	-0.2497	-0.3061

N_d (kN)	u_d (mm)	M_d (kN-m)
-121.68	-8.49	-156.65

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1525.42	-9.05	-156.65	**CROWN**
10	1532.76	-8.54	-147.20	
20	1553.89	-7.06	-120.00	
30	1586.26	-4.80	-78.32	
40	1625.97	-2.03	-27.20	
45	1647.10	-0.56	0.00	
50	1668.23	0.92	27.20	
60	1707.94	3.69	78.32	
70	1740.31	5.95	120.00	
80	1761.44	7.42	147.20	
90	1768.78	7.94	156.65	**AXIS**

Location: Old Airport to Tanjong Katong (ULS for long term - creep)
(Deep MC Section-CH 57+127 sump location) Flexible lining Load Case 5

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	147.8760	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.40		
Load factor for Surcharge	FS =	1.60		
Factored vertical stress	σ_v' =	327.0264	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	245.2698	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	81.7566	kN/m^2	
Load factor for Water	FS_w =	1.40		
Hydrostatic water pressure	p_w =	233.0440	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	129.3915	kN/m^2	
Maximum shear strength of ground	τ =	52.2776	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	52.2776	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	16000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	16368.2864	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	5		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.1092E-03	m^4	

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3 (2S_n+S_t)/18EI$

$U = -r_e r_o^3 (2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 48.02 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.5616	9.9158	48.0235	739.9147	905.8515	-0.4993	-0.6113

N_d (kN)	u_d (mm)	M_d (kN-m)
-112.14	-18.48	-109.07

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1533.62	-19.59	-109.07	**CROWN**
10	1540.39	-18.47	-102.49	
20	1559.86	-15.26	-83.55	
30	1589.69	-10.35	-54.53	
40	1626.29	-4.32	-18.94	
45	1645.77	-1.11	0.00	
50	1665.24	2.10	18.94	
60	1701.84	8.13	54.53	
70	1731.67	13.04	83.55	
80	1751.15	16.25	102.49	
90	1757.91	17.37	109.07	**AXIS**

Location: Old Airport to Tanjong Katong (SLS for short term - no creep)
(Deep MC Section-CH 57+127 sump location) Rigid linings Load Case 6

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	0.00	m	
Effective overburden pressure	q_1 =	117.8760	kN/m^2	
Surcharge	q_2 =	0.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical stress	σ_v' =	117.8760	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	88.4070	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	29.4690	kN/m^2	
Load factor for Water	FS_w =	1.00		
Hydrostatic water pressure	p_w =	196.4600	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	103.1415	kN/m^2	
Maximum shear strength of ground	τ =	41.6719	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	41.6719	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	($I_j << I$)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3 (2S_n+S_t)/18EI$

$U = -r_e r_o^3 (2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 15.92 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1797	9.6052	15.9182	623.7605	326.9926	-0.2105	-0.1103

N_d (kN)	u_d (mm)	M_d (kN-m)
-43.86	-3.06	-56.46

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	906.89	-3.38	-56.46	**CROWN**
10	909.54	-3.20	-53.06	
20	917.16	-2.67	-43.25	
30	928.82	-1.85	-28.23	
40	943.14	-0.85	-9.80	
45	950.75	-0.32	0.00	
50	958.37	0.21	9.80	
60	972.68	1.21	28.23	
70	984.35	2.02	43.25	
80	991.97	2.56	53.06	
90	994.61	2.74	56.46	**AXIS**

Location: Old Airport to Tanjong Katong (SLS for short term - no creep)
(Deep MC Section-CH 57+127 sump location) Rigid linings Load Case 7

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	0.00	m	
Effective overburden pressure	q_1 =	117.8760	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical stress	σ_v' =	192.8760	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	144.6570	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	48.2190	kN/m^2	
Load factor for Water	FS_w =	1.00		
Hydrostatic water pressure	p_w =	196.4600	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	103.1415	kN/m^2	
Maximum shear strength of ground	τ =	41.6719	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	41.6719	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 26.05 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1797	15.7165	26.0463	623.7605	535.0455	-0.2105	-0.1805

N_d (kN)	u_d (mm)	M_d (kN-m)
-71.76	-5.01	-92.39

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1087.04	-5.40	-92.39	**CROWN**
10	1091.37	-5.10	-86.82	
20	1103.83	-4.23	-70.77	
30	1122.92	-2.90	-46.19	
40	1146.34	-1.26	-16.04	
45	1158.81	-0.39	0.00	
50	1171.27	0.48	16.04	
60	1194.69	2.11	46.19	
70	1213.78	3.45	70.77	
80	1226.24	4.32	86.82	
90	1230.57	4.62	92.39	**AXIS**

Location: Old Airport to Tanjong Katong (SLS for short term - no creep)
(Deep MC Section-CH 57+127 sump location) Rigid linings Load Case 8

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	147.8760	kN/m^2	
Surcharge	q_2 =	0.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical stress	σ_v' =	147.8760	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	110.9070	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	36.9690	kN/m^2	
Load factor for Water	FS_w =	1.00		
Hydrostatic water pressure	p_w =	166.4600	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	129.3915	kN/m^2	
Maximum shear strength of ground	τ =	52.2776	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	52.2776	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3 (2S_n+S_t)/18EI$

$U = -r_e r_o^3 (2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 19.97 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1797	12.0497	19.9695	528.5105	410.2138	-0.1783	-0.1384

N_d (kN)	u_d (mm)	M_d (kN-m)
-55.02	-3.84	-70.83

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	883.70	-4.16	-70.83	**CROWN**
10	887.02	-3.92	-66.56	
20	896.58	-3.26	-54.26	
30	911.21	-2.24	-35.42	
40	929.17	-0.98	-12.30	
45	938.72	-0.32	0.00	
50	948.28	0.35	12.30	
60	966.23	1.60	35.42	
70	980.87	2.62	54.26	
80	990.43	3.29	66.56	
90	993.75	3.52	70.83	**AXIS**

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

Location: Old Airport to Tanjong Katong (SLS for short term - no creep)
(Deep MC Section-CH 57+127 sump location) Rigid linings Load Case 9

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	147.8760	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical stress	σ_v' =	222.8760	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	167.1570	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	55.7190	kN/m^2	
Load factor for Water	FS_w =	1.00		
Hydrostatic water pressure	p_w =	166.4600	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	129.3915	kN/m^2
Maximum shear strength of ground	τ =	52.2776	kN/m^2 ($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	52.2776	kN/m^2 ($\tau = c' + p_u \tan\phi'$)	
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 30.10 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1797	18.1611	30.0976	528.5105	618.2667	-0.1783	-0.2086

N_d (kN)	u_d (mm)	M_d (kN-m)
-82.93	-5.79	-106.76

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1063.85	-6.17	-106.76	**CROWN**
10	1068.85	-5.83	-100.32	
20	1083.25	-4.82	-81.78	
30	1105.31	-3.28	-53.38	
40	1132.38	-1.39	-18.54	
45	1146.78	-0.39	0.00	
50	1161.18	0.62	18.54	
60	1188.24	2.51	53.38	
70	1210.30	4.05	81.78	
80	1224.70	5.05	100.32	
90	1229.70	5.40	106.76	**AXIS**

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date:
Date:
Date:

Location: Old Airport to Tanjong Katong (SLS for long term - creep)
(Deep MC Section-CH 57+127 sump location) Flexible linings Load Case 10

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	147.8760	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical pressure	σ_v' =	222.8760	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	167.1570	kN/m^2	
$p_o = \sigma_v' - \sigma_h'$	p_o =	55.7190	kN/m^2	
Load factor for Water	FS_w =	1.00		
Factored hydrostatic water pressure	p_w =	166.4600	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF GROUND

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	129.3915	kN/m^2
Shear strength $\tau = c' + p_u \tan\phi'$	τ =	52.2776	kN/m^2

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	52.2776	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	16000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	16368.2864	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	5		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.1092E-03	m^4	

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 32.73 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.5616	6.7579	32.7291	528.5105	617.3586	-0.3566	-0.4166

N_d (kN)	u_d (mm)	M_d (kN-m)
-76.43	-12.59	-74.33

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1069.44	-13.36	-74.33	**CROWN**
10	1074.05	-12.61	-69.85	
20	1087.32	-10.42	-56.94	
30	1107.65	-7.07	-37.17	
40	1132.60	-2.96	-12.91	
45	1145.87	-0.77	0.00	
50	1159.14	1.41	12.91	
60	1184.08	5.52	37.17	
70	1204.42	8.87	56.94	
80	1217.69	11.06	69.85	
90	1222.30	11.82	74.33	**AXIS**

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

Location: Old Airport to Tanjong Katong (SLS for long term - creep)
(Deep MC Section-CH 57+127 sump location) Flexible linings Load Case 11

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	0.00	m	
Effective overburden pressure	q_1 =	117.8760	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical pressure	σ_v' =	192.8760	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	144.6570	kN/m^2	
$p_o = \sigma_v' - \sigma_h'$	p_o =	48.2190	kN/m^2	
Load factor for Water	FS_w =	1.00		
Factored hydrostatic water pressure	p_w =	196.4600	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF GROUND

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	103.1415	kN/m^2
Shear strength $\tau = c' + p_u \tan\phi'$	τ =	41.6719	kN/m^2

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	41.6719	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	16000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	16368.2864	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	5		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.1092E-03	m^4	

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 28.32 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.5616	5.8482	28.3236	623.7605	534.2597	-0.4209	-0.3605

N_d (kN)	u_d (mm)	M_d (kN-m)
-66.14	-10.90	-64.33

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1091.88	-11.68	-64.33	**CROWN**
10	1095.87	-11.02	-60.45	
20	1107.35	-9.13	-49.28	
30	1124.95	-6.23	-32.16	
40	1146.53	-2.67	-11.17	
45	1158.02	-0.78	0.00	
50	1169.51	1.11	11.17	
60	1191.09	4.67	32.16	
70	1208.69	7.57	49.28	
80	1220.17	9.46	60.45	
90	1224.16	10.12	64.33	**AXIS**

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

Location: Old Airport to Tanjong Katong (SLS for long term - creep)
(Deep MC Section-CH 57+127 sump location) Flexible linings Load Case 12

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	101.925	
Track Level:	R.L.	80.754	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	19.6460	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	147.8760	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical pressure	σ_v' =	222.8760	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	167.1570	kN/m^2	
$p_o = \sigma_v' - \sigma_h'$	p_o =	55.7190	kN/m^2	
Load factor for Water	FS_w =	1.00		
Factored hydrostatic water pressure	p_w =	166.4600	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF GROUND

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	129.3915	kN/m^2
Shear strength $\tau = c' + p_u \tan\phi'$	τ =	52.2776	kN/m^2

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	5893.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	52.2776	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	16000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	16368.2864	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	5		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.1092E-03	m^4	

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

(Deep MC Section-CH 57+127 sump location)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3 (2S_n+S_t)/18EI$

$U = -r_e r_o^3 (2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 32.73 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

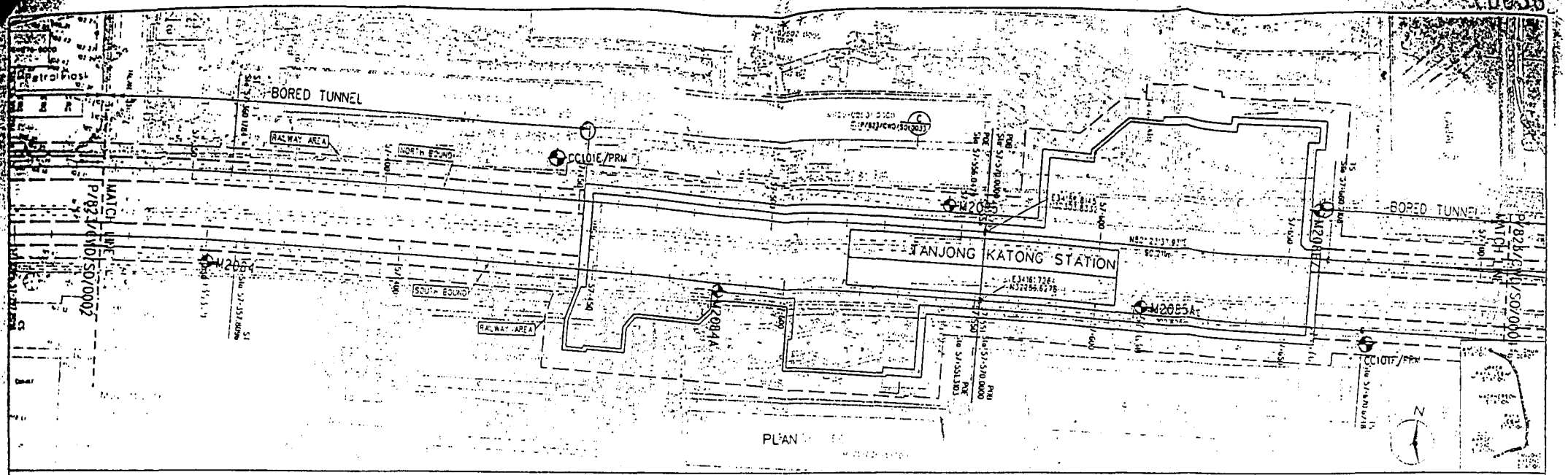
$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

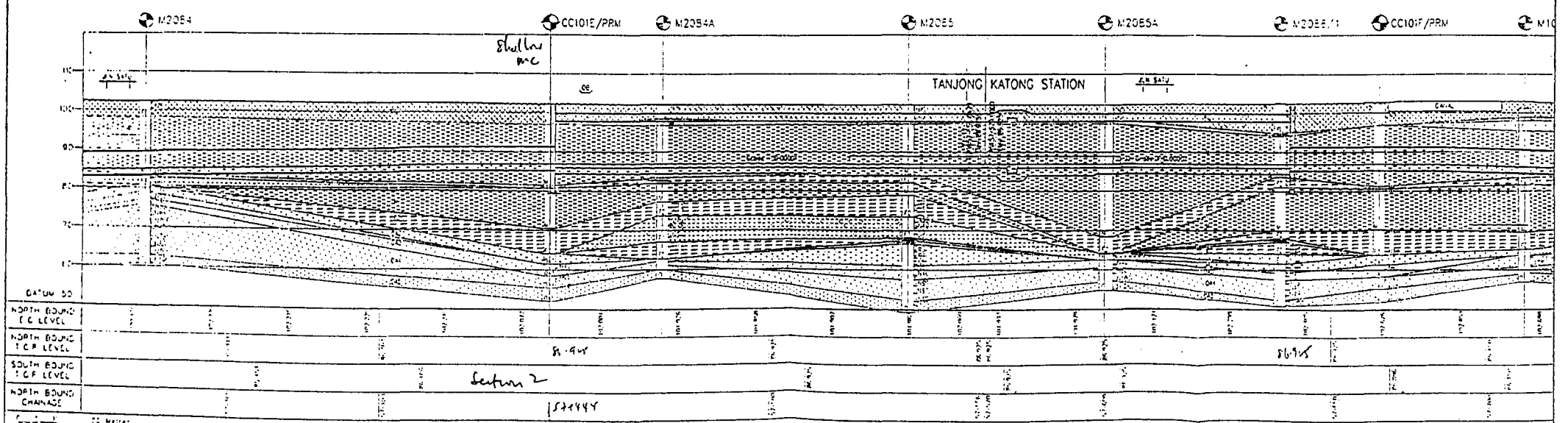
$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.5616	6.7579	32.7291	528.5105	617.3586	-0.3566	-0.4166

N_d (kN)	u_d (mm)	M_d (kN-m)
-76.43	-12.59	-74.33

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1069.44	-13.36	-74.33	**CROWN**
10	1074.05	-12.61	-69.85	
20	1087.32	-10.42	-56.94	
30	1107.65	-7.07	-37.17	
40	1132.60	-2.96	-12.91	
45	1145.87	-0.77	0.00	
50	1159.14	1.41	12.91	
60	1184.08	5.52	37.17	
70	1204.42	8.87	56.94	
80	1217.69	11.06	69.85	
90	1222.30	11.82	74.33	**AXIS**





PRE-FINAL DESIGN FOR ACCEPTANCE

Consultants: Maunsell Consultants (Singapore) Pte Ltd in Association with SAA Partnership Pte Ltd, Hyder Consulting Pte Ltd, Environmental Resources Management (S) Pte Ltd, Meinhardt Consulting Engineers, Davis Langdon & Seah

Land Transport Authority

CONTRACT 870A - CIRCLE LINE STAGE 2 MAINLINE

BOREHOLE PLAN AND SOIL PROFILE SHEET 3

Designed: LW/HCH | Checked: CVP | Approved: APC

Scale: 1:500 | Drawn: SHL | Date: 07/07/01

Drawing No: P/823/CWD/SO/0003 | Rev: A | Sheet: 53 of 77

CAD File Name: P823SHC-SO003A

P:\SO\P823CWD-SO0003A.dgn 12/07/01 02:49:17 PM

0053

LOCATION: DUNMAN ROAD

BOREHOLE NO. CC101
NORTHING: 32361.23(m)
EASTING: 34388.07(m)
REDUCED LEVEL: 101.11m

FIELD & LABORATORY DATA & TESTS REPORTED ELSEWHERE

DAILY WATER TABLE READING

DATE	TIME (Hrs.)	BOR. DEPTH (m BGL)	WATER LEVEL (m BGL)
16.10.00	18:30	4.55	1.8
17.10.00	8:10	4.55	1.2
17.10.00	18:40	18.50	2.8
18.10.00	8:10	18.50	0.7
18.10.00	18:40	28.25	0.9
19.10.00	8:10	28.25	0.6
19.10.00	18:45	39.45	0.8
20.10.00	8:10	39.45	2.1

SPT N VALUE: 0 10 20 30 40 50 60 70 80 90 100

SPT (N) / mm: 11/300

WEATHERING GRADE: VI VII VIII I

REDUCED LEVEL(m)	DEPTH (m)	SAMPLE TYPE	GRAPHIC LOG	THICKNESS(m)	GEOLOGICAL CLASSIFICATION	BRITISH SOIL CLASSIFICATION	DESCRIPTION
	0						BACKFILL (Soft, dark brown, grey, yellowish brown Silty CLAY with sand and rock fragments) Trial Pit 0.7x0.8x2.5m
98.21	3			2.9	FILL		
	3.00	U1					UD1 [MC=96, BD=1.36, LL=123, PL=62, PD=2.45, Cuu=14] Soft, dark brown Peaty CLAY with partially decayed wood pieces
	3.80	V1					FVT1(4.55m): Su(U)=12.4 Su(R)=5.6
95.11	6			3.1	E	OE	
	6.00	U2					Very soft to soft, grey Marine CLAY with few shell fragments UD2 [MC=72, BD=1.54, LL=76, PL=32, PD=2.70, Cuu=10, C'=0, ø'=23°]
	6.90	V2					FVT2(7.65m): Su(U)=15.9 Su(R)=2.1
	9.00	U3					UD3 [MC=64, BD=1.56, LL=74, PL=31, SILT=43, CLAY=57, PD=2.62, Cuu=8]
	9.80	V3					FVT3(10.55m): Su(U)=21.5 Su(R)=7.9
	12.00	U4					UD4 [MC=71, BD=1.52, LL=78, PL=32, PD=2.61, Cuu=6]
	12.90	V4					FVT4(13.65m): Su(U)=24.7 Su(R)=8.8
86.11	15			9.0	M	CV	
	15.00	U5				CI	Stiff, light grey, yellowish-reddish brown Silty CLAY with traces of sand UD5 [MC=28, BD=1.92, LL=49, PL=23, SAND=4, SILT=41, CLAY=55, PD=2.68, Cuu=63]
	15.60	P1					
	16.05						
	18.00	U6				CH	Very soft, grey Silty CLAY with traces of sand UD6 [MC=49, BD=1.69, LL=58, PL=25, PD=2.67, Cuu=9]
	18.50						
	20	V5					FVT5(19.25m): Su(U)=25.0 Su(R)=8.5

BORING TYPE: ROTARY
METER(mm): 100

FRACTURE PER METRE: 16 8 4 2
RQD %: 20 40 60 80
SCR %: 20 40 60 80
TCR %: 20 40 60 80
WEATHERING GRADE: VI VII VIII I

SPT SAMPLE
UNDISTURBED SAMPLE
CORE RUN

MC=MOISTURE CONTENT (%)
BD=BULK DENSITY
SG=SPECIFIC GRAVITY
LL=LIQUID LIMIT(%)
PL=PLASTIC LIMIT (%)
UU=UNCONSOLIDATED UNDRAINED TEST (kPa)

FVT = FIELD VANE TEST
Su(U) = Undisturbed Test (kPa)
Su(R) = Remolded Test (kPa)
PMT = PRESSURE METER TEST
PKT = PACKER TEST(Lugeon)
UCT = Unified Compression Test(MPa)
STT = Splitting Tensile Test(MPa)

LAND TRANSPORT AUTHORITY

LOG OF BORING
GEOTECHNICAL STUDY – FIELD INVESTIGATIONS

PROJECT: SITE INVESTIGATION BETWEEN DUNMAN ROAD AND PAYA LEBAR ROAD/ KIM CHUAN ROAD

PREPARED BY: CHEN

DATE OF FIELD WORK: 16~20/10/00

ECON GEOTECH PTE LTD

CHECKED BY: KUNDU

SHEET NO. Page 1/3

LOCATION: DUNMAN ROAD

BOREHOLE NO. CC101
NORTHING: 32361.23(m)
EASTING: 34388.07(m)
REDUCED LEVEL: 101.11m

FIELD & LABORATORY DATA & TESTS REPORTED ELSEWHERE	SPT N VALUE (0 10 20 30 40 50 60 70 80 90 100)	SPT (N) / mm	WEATHERING GRADE (W VI V IV III II I)	REDUCED LEVEL(m)	DEPTH (m)	SAMPLE TYPE	GRAPHIC LOG	THICKNESS(m)	GEOLOGICAL CLASSIFICATION	BRITISH SOIL CLASSIFICATION	DESCRIPTION
				79.51	21.00	U7 21.60		6.6	F2	CI	Firm to stiff, light grey-brownish red Silty CLAY with sand UD7 [MC=25, BD=1.99, LL=42, PL=19, SAND=16, SILT=39, CLAY=45, Cuu=58]
		6/300			22.00	P2 22.05					Firm, dark brown to black Peaty CLAY with decayed wood pieces
				77.11	24.00			2.4	E	OH	
		8/300		76.31		U8 24.80		0.8	F2	CH	Very soft, grey Silty CLAY with traces of sand UD8 [MC=48, BD=1.7, LL=56, PL=25, SAND=1, SILT=44, CLAY=55, PD=2.72, Cuu=11]
				75.11	25.00	P3 25.25		1.2	E	OH	Firm, dark brown to black Peaty CLAY with decayed wood pieces and a few sand
					27.00	U9 27.80		1.8	F2	CH	Very soft, dark brownish grey-pale brown Silty CLAY with peat and traces of sand UD9 [MC=53, BD=1.65, LL=68, PL=28, SAND=3, SILT=42, CLAY=55, Cuu=10]
		25/300			28.00	P4 28.25					Medium dense, yellowish brown-light grey Clayey SAND with traces of gravels
		26/300		70.11	30.00	U10 30.60 P5 31.05		3.2	O(W)	SCL	UD10(Ir) [MC=16, BD=2.07, LL=28, PL=17, PI=11, GRAVEL=4, SAND=65, SILT+CLAY=31, PD=2.66, Cuu=159, Suu=5]
					33.00	U11 33.60		2.5	O(W)	CL	Firm to stiff, grey Sandy CLAY UD11 [MC=17, BD=2.02, LL=28, PL=13, SAND=52, SILT=25, CLAY=23, Cuu=44]
		50/300		67.61	34.00	P6 34.05					Dense to very dense, light green, mottled yellowish brown Clayey SAND
				65.61	36.00	U12 36.60		2.0	O(SW2)	SC	
		90/300		64.21	37.00	P7 37.05		1.4	O(SW1)	CI	Hard, greenish brown-greenish grey Silty CLAY with sand UD12 [MC=20, BD=2.07, LL=43, PL=22, SAND=24, SILT=56, CLAY=20, Cuu=343]
											Very dense, light green, yellowish brown Silty fine SAND
		76/300			39.00	P8 39.45					

BORING TYPE: ROTARY
DIAMETER(mm): 100

FRACTURE PER METRE (16 8 4 2) | RQD % (20 40 60 80) | SCR % (20 40 60 80) | TCR % (20 40 60 80) | WEATHERING GRADE (W VI V IV III II I)

SPT SAMPLE
UNDISTURBED SAMPLE
CORE RUN

MC=MOISTURE CONTENT (%)
BD=BULK DENSITY
SG=SPECIFIC GRAVITY
LL=LIQUID LIMIT(%)
PL=PLASTIC LIMIT (%)
UU=UNCONSOLIDATED UNDRAINED TEST (kPa)

FVT = FIELD VANE TEST
Su(U) = Undisturbed Test (kPa)
Su(R) = Remolded Test (kPa)
PMT = PRESSURE METER TEST
PKT_i = PACKER TEST(Lugeon)
UCT = Unified Compression Test(MPa)
STT = Splitting Tensile Test(MPa)

CLIENT: LAND TRANSPORT AUTHORITY

LOG OF BORING
GEOTECHNICAL STUDY – FIELD INVESTIGATIONS

PROJECT: SITE INVESTIGATION BETWEEN DUNMAN ROAD AND PAYA LEBAR ROAD/ KIM CHUAN ROAD

PREPARED BY: CHEN

DATE OF FIELD WORK: 16~20/10/00

ECON GEOTECH PTE LTD

CHECKED BY: KUNDU

SHEET NO. Page 2/3

LOCATION: DUNMAN ROAD

BOREHOLE NO. CC101
NORTHING: 32361.23(m)
EASTING: 34388.07(m)
REDUCED LEVEL: 101.11m

FIELD & LABORATORY DATA & TESTS REPORTED ELSEWHERE	SPT N VALUE (0 10 20 30 40 50 60 70 80 90 100)	SPT (N) / mm	WEATHERING GRADE (VI V IV III II I)	REDUCED LEVEL(m)	DEPTH (m)	SAMPLE TYPE	GRAPHIC LOG	THICKNESS(m)	GEOLOGICAL CLASSIFICATION	BRITISH SOIL CLASSIFICATION	DESCRIPTION
		76/300			42.00 P9 42.45						Very dense, light green, yellowish brown Silty f-c SAND with some gravels
		98/300			45.00 P10 45.45						
		100/100		52.86	48.00 P11 48.25			11.35	o(swi)	SM	
											Borehole terminated at 48.25m and backfilled with bentonite cement grout as instructed by Client.

BORING TYPE: ROTARY
DIAMETER(mm): 100

FRACTURE PER METRE (16 8 4 2) | RQD % (20 40 60 80) | SCR % (20 40 60 80) | TCR % (20 40 60 80) | WEATHERING GRADE (VI V IV III II I)

SPT, SAMPLE
UNDISTURBED SAMPLE
CORE RUN

MC=MOISTURE CONTENT (%)
BD=BULK DENSITY
SG=SPECIFIC GRAVITY
LL=LIQUID LIMIT(%)
PL=PLASTIC LIMIT (%)
UU=UNCONSOLIDATED UNDRAINED TEST (kPa)

FVT = FIELD VANE TEST
Su(U) = Undisturbed Test (kPa)
Su(R) = Remolded Test (kPa)
PMT = PRESSURE METER TEST
PKT = PACKER TEST(Lugeon)
UCT = Unified Compression Test(MPa)
STT = Splitting Tensile Test(MPa)

CLIENT: LAND TRANSPORT AUTHORITY

LOG OF BORING
GEOTECHNICAL STUDY – FIELD INVESTIGATIONS

PROJECT: SITE INVESTIGATION BETWEEN DUNMAN ROAD AND PAYA LEBAR ROAD/ KIM CHUAN ROAD

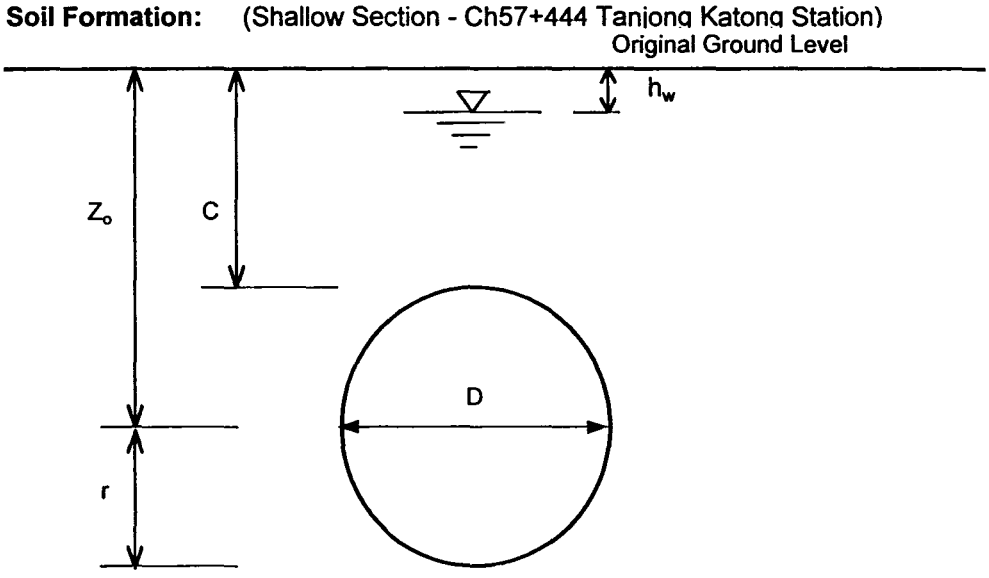
PREPARED BY: CHEN
DATE OF FIELD WORK: 16~20/10/00

ECON GEOTECH PTE LTD

CHECKED BY: KUNDU
SHEET NO. Page 3/3

TUNNEL LINING DESIGN
[Based on Muir Wood (1975) & Curtis (1976)]

Location: Old Airport to Tanjong Katong (CC101)
Soil Formation: (Shallow Section - Ch57+444 Tanjong Katong Station)



References:

Muir Wood, A. M. (1975) The circular tunnel in elastic groun *Geotechnique 25, No. 1, 115 - 127*

Curtis, D. J. (1976) Discussion on the reference abov *Geotechnique 26, No. 1, 231 - 237*

Duddeck, H., Erdmann, J. (1982) Structural design models for tunnels,
Tunnelling 82, International Symposium organised by Institution of Mining & Metallurgy

Circle Line Contracts, Design Criteria, *Land Transport Authority, Singapore*

Notation

Symbols	Description
C	cover to tunnel crown
z_o	depth to tunnel axis
D	excavated tunnel diameter
r_o	radius to extrados of tunnel lining
γ	average unit weught of overburden
k	constant
E	Young's modulus for lining (replaced by $E/(1-\nu_1^2)$ where lining continuous along tunnel)
E_c, ν	Young's modulus and Poisson's ratio of ground
I	second moment of initia of lining per unit length of tunnel
I_e	effective value of I for a jointed lining
I_j	effective value of I at joint in a lining
M	bending moment in lining per unit length of tunnel
N	Hoop (circumferential) thrust in lining per unit length of tunnel
η	ratio of radius of lining centroid to that of extrados
u_{max}	maximum radial movement of lining
h_w	water table from ground surface

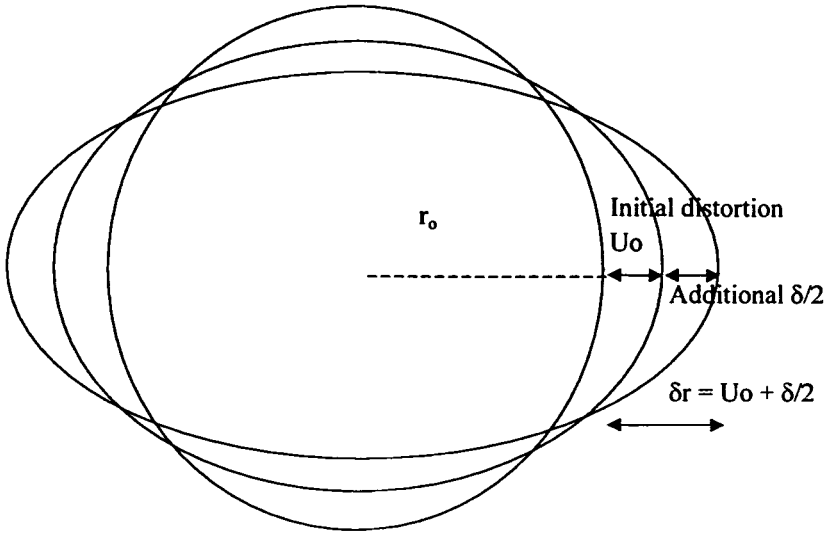
Old Airport to Tanjong Katong
(Shallow Section - Ch57+444 Tanjong Katong Station)

	Load Case	N-axis (kN)	U-axis (mm)	M-axis (kNm)	M-axis, future development	Total M-axis (kNm)
ULS	1	966.73	2.86	58.51	0	58.51
	2	1342.08	6.12	120.76	0	120.76
	3	965.84	4.03	79.98	0	79.98
	4	1344.29	7.24	141.32	0	141.32
	5	1336.60	17.26	106.82	55.45	162.28
SLS	6	690.52	2.04	41.79	0	41.79
	7	927.05	4.05	80.13	0	80.13
	8	689.88	2.88	57.13	0	57.13
	9	926.42	4.89	95.47	0	95.47
	10	921.22	11.65	72.16	39.61	111.77
	11	922.69	9.68	60.57	0	60.57
	12	921.22	11.65	72.16	0	72.16

	Load Case	N-crown (kN)	U-crown (mm)	M-crown (kNm)	Total M-crown (kNm)
ULS	1	880.07	-3.48	58.51	58.51
	2	1170.79	-6.97	120.76	120.76
	3	847.38	-4.64	79.98	79.98
	4	1135.00	-8.08	141.32	141.32
	5	1141.13	-18.93	106.82	162.28
SLS	6	628.62	-2.49	41.79	41.79
	7	808.38	-4.64	80.13	80.13
	8	605.27	-3.32	57.13	57.13
	9	785.03	-5.46	95.47	95.47
	10	789.17	-12.80	72.16	111.77
	11	811.86	-10.85	60.57	60.57
	12	789.17	-12.80	72.16	72.16

Location: Old Airport to Tanjong Katong
(Shallow Section - Ch57+444 Tanjong Katong Station)

LOADING DUE TO ADDITIONAL DISTORTION



For 15mm additional distortion on diameter,

Change in radius, δ/2	7.5	mm

Using Morgan's formula, bending moment due to distortion over radius, $M = (3EI/r_o^2)\delta r$

For long term stiffness of concrete, E =	16000	MN/m^2
Excavated radius of tunnel, r_o =	3.175	m
Moment of inertia of flexible lining, I =	1.11E-03	m^4
At SLS M =	39.61	KNm/m run
At ULS M =	39.61x1.4	KNm/m run
=	55.45	KNm/m run

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

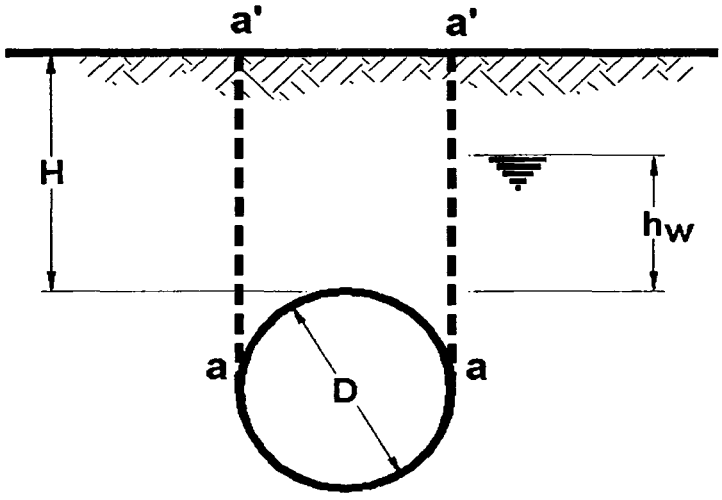
LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

Location: Old Airport to Tanjong Katong
(Shallow Section - Ch57+444 Tanjong Katong Station)

1. TUNNEL & SOIL PROPERTIES

Nominal Diameter of Tunnel D_n =	5.60	m
Construction Allowance DD =	100.00	mm
Thickness of Lining t =	275.00	mm
Existing ground level GL =	102.077	m
Track level RL1 =	86.925	m
Track Level to Invert of Tunnel d =	1375.00	mm
Excavated Diameter of Tunnel D =	6.3500	m
Internal tunnel radius r_i =	2.9000	m
Radius to lining extrados r_e =	3.1750	m
Radius of lining centroid r_o =	3.0375	m
Depth to tunnel axis z_o =	13.6270	m
Unit weight of water γ_w =	10	kN/m^3
Water table from ground surface =	3.00	m
ie. h_w =	7.45	m



Density of concrete =	24.00	
Weight of 1st stage concrete W_1 =	0.00	kN/m
(Neglect 1st stage concrete)		
Weight of concrete lining W_2 =	125.96	kN/m
Factored self weight of tunnel, W =	$(W_1+W_2)/1.05$	
=	119.96	kN/m
Average shear resistance along a-a' =	20.44	kN/m^2

{ For cohesive soil, $S = c_u$ }
{ For cohesionless soil, $S = ½ Ko \gamma' (H+D/2)$ }

Ave. unit weight of soil above tunnel γ =	16.00	kN/m^3

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

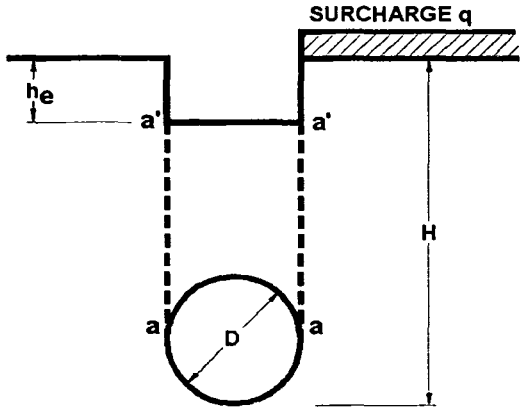
2. FLOTATION

Reference: LTA Civil Design Criteria, section 7.3.3.1

	Uplift $U = \gamma_w (\pi D^2/4) - W =$	196.73	kN/m run
	Depth to tunnel crown H =	10.45	m
	Restraining force R = R1 + R2 + R3		
	$R1 = \gamma' D (h_w + D/2 - \pi D/8) =$	309.88	kN/m run
	$R2 = \gamma_b D (H - h_w) =$	304.80	kN/m run
Shear strength of soil above slip plane	$S (H + D/2) =$	557.09	kN/m run
	ie Restraining force R =	1171.77	kN/m run
	Overall factor of safety against flotation R/U =	5.96	
		>1.2 --> OK	

3. HEAVE AT TUNNEL INVERT

Reference: LTA Civil Design Criteria, section 7.3.3.2



$$F = \frac{N_c C_u + 2 S (H - D/2 - h_e)/D}{0.25 (\gamma_{b1} \pi D) - W/D + q + \gamma_{b2} h_e}$$

Bearing capacity factor Nc = (after Meyerhoff chart)	7.5	
Factored mean shear strength at tunnel invert Cu =	12.60	kN/m^2
Depth to tunnel invert H =	16.80	m
Depth to excavation above tunnel he =	3	m
Factored soil bulk density in zone of tunnel γ_{b1}=	13.91	kN/m^3
Factored soil bulk density in excavated zone γ_{b2}=	13.91	kN/m^3

Without surcharge,

Overall factor of safety against heave F =	1.77	
	>1.2 --> OK	

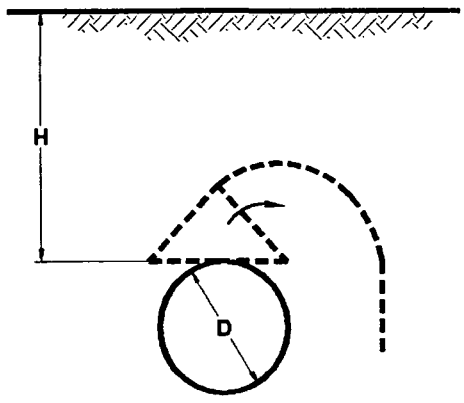
With surcharge at ground level beside tunnel, q =	22.5	kN/m^2
Overall factor of safety against heave F =	1.42	
	>1.0 --> OK	

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

4. HEAVE AT TUNNEL CROWN



Reference: LTA Civil Design Criteria, section 7.3.3.3

Uplift $U = \gamma_b (\pi D^2/4) - W =$	386.74	kN/m run
Restraining force R =	$D.Nc.C_u$	
where Nc =	8.25	(Meyerhoff)
Undrained cohesion at tunnel axis =	20.44	
Factored cohesion at tunnel axis Cu =	10.22	
ie R =	535.41	kN/m run
Overall factor of safety against flotation R/U =	1.38	
	>1.0 --> OK	

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

Location: Old Airport to Tanjong Katong (ULS for short term - no creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Rigid linings Load Case 1

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	0.00	m	
Effective overburden pressure	q_1 =	81.7620	kN/m^2	
Surcharge	q_2 =	0.00	kN/m^2	
Load factor for Overburden Load	FS =	1.40		
Load factor for Surcharge	FS =	1.60		
Factored vertical stress	σ_v' =	114.4668	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	85.8501	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	28.6167	kN/m^2	
Load factor for Water	FS_w =	1.40		
Hydrostatic water pressure	p_w =	190.7780	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	71.5418	kN/m^2	
Maximum shear strength of ground	τ =	28.9047	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	28.9047	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ = 15.16 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1247	10.6215	15.1592	605.7202	317.6785	-0.2044	-0.1072

N_d (kN)	u_d (mm)	M_d (kN-m)
-43.33	-3.17	-58.51

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	880.07	-3.48	-58.51	**CROWN**
10	882.68	-3.29	-54.98	
20	890.21	-2.74	-44.82	
30	901.73	-1.90	-29.26	
40	915.87	-0.86	-10.16	
45	923.40	-0.31	0.00	
50	930.92	0.24	10.16	
60	945.06	1.27	29.26	
70	956.59	2.12	44.82	
80	964.11	2.67	54.98	
90	966.73	2.86	58.51	**AXIS**

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date:
Date:
Date:

Location: Old Airport to Tanjong Katong (ULS for short term - no creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Rigid linings Load Case 2

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	0.00	m	
Effective overburden pressure	q_1 =	81.7620	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Soil Overburden	FS =	1.40		
Load factor for Surcharge	FS =	1.60		
Factored vertical stress	σ_v' =	234.4668	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	175.8501	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	58.6167	kN/m^2	
Load factor for Water	FS_w =	1.40		
Hydrostatic water pressure	p_w =	190.7780	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	71.5418	kN/m^2	
Maximum shear strength of ground	τ =	28.9047	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	28.9047	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60 N/mm^2)	
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	($I_j << I$)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3 (2S_n+S_t)/18EI$

$U = -r_e r_o^3 (2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 31.05 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1247	23.1138	28.9047	605.7202	650.7132	-0.2044	-0.2196

N_d (kN)	u_d (mm)	M_d (kN-m)
-85.64	-6.55	-120.76

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1170.79	-6.97	-120.76	**CROWN**
10	1175.95	-6.58	-113.48	
20	1190.83	-5.44	-92.51	
30	1213.61	-3.70	-60.38	
40	1241.56	-1.56	-20.97	
45	1256.43	-0.42	0.00	
50	1271.31	0.71	20.97	
60	1299.26	2.85	60.38	
70	1322.04	4.59	92.51	
80	1336.91	5.73	113.48	
90	1342.08	6.12	120.76	**AXIS**

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

Location: Old Airport to Tanjong Katong (ULS for short term - no creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Rigid linings Load Case 3

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	111.7620	kN/m^2	
Surcharge	q_2 =	0.00	kN/m^2	
Load factor for Overburden Load	FS =	1.40		
Load factor for Surcharge	FS =	1.60		
Factored vertical stress	σ_v' =	156.4668	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	117.3501	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	39.1167	kN/m^2	
Load factor for Water	FS_w =	1.40		
Hydrostatic water pressure	p_w =	148.7780	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	97.7918	kN/m^2	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	($I_j << I$)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 20.72 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1247	14.5187	20.7213	472.3702	434.2407	-0.1594	-0.1465

N_d (kN)	u_d (mm)	M_d (kN-m)
-59.23	-4.34	-79.98

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	847.38	-4.64	-79.98	**CROWN**
10	850.96	-4.38	-75.16	
20	861.24	-3.63	-61.27	
30	877.00	-2.47	-39.99	
40	896.33	-1.06	-13.89	
45	906.61	-0.31	0.00	
50	916.90	0.45	13.89	
60	936.22	1.86	39.99	
70	951.98	3.02	61.27	
80	962.26	3.77	75.16	
90	965.84	4.03	79.98	**AXIS**

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

Location: Old Airport to Tanjong Katong (ULS for short term - no creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Rigid linings Load Case 4

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	111.7620	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.40		
Load factor for Surcharge	FS =	1.60		
Factored vertical stress	σ_v' =	276.4668	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	207.3501	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	69.1167	kN/m^2	
Load factor for Water	FS_w =	1.40		
Hydrostatic water pressure	p_w =	148.7780	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	97.7918	kN/m^2	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	($I_j << I$)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3 (2S_n+S_t)/18EI$

$U = -r_e r_o^3 (2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 36.61 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1247	25.6537	36.6133	472.3702	767.2754	-0.1594	-0.2589

N_d (kN)	u_d (mm)	M_d (kN-m)
-104.65	-7.66	-141.32

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1135.00	-8.08	-141.32	**CROWN**
10	1141.31	-7.62	-132.80	
20	1159.48	-6.29	-108.26	
30	1187.32	-4.25	-70.66	
40	1221.47	-1.75	-24.54	
45	1239.65	-0.42	0.00	
50	1257.82	0.91	24.54	
60	1291.97	3.41	70.66	
70	1319.81	5.45	108.26	
80	1337.98	6.78	132.80	
90	1344.29	7.24	141.32	**AXIS**

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

Location: Old Airport to Tanjong Katong (ULS for long term - creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Flexible lining Load Case 5

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	111.7620	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.40		
Load factor for Surcharge	FS =	1.60		
Factored vertical stress	σ_v' =	276.4668	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	207.3501	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	69.1167	kN/m^2	
Load factor for Water	FS_w =	1.40		
Hydrostatic water pressure	p_w =	148.7780	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	97.7918	kN/m^2	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	16000.0	MN/m^2, (f_{cu} = 60 N/mm^2)	
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	16368.2864	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	5		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.1092E-03	m^4	

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3 (2S_n+S_t)/18EI$

$U = -r_e r_o^3 (2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 39.41 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.3895	13.5236	39.4125	472.3702	766.4930	-0.3188	-0.5172

N_d (kN)	u_d (mm)	M_d (kN-m)
-97.74	-18.10	-106.82

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1141.13	-18.93	-106.82	**CROWN**
10	1147.02	-17.84	-100.38	
20	1163.99	-14.70	-81.83	
30	1190.00	-9.88	-53.41	
40	1221.89	-3.98	-18.55	
45	1238.86	-0.84	0.00	
50	1255.83	2.31	18.55	
60	1287.73	8.21	53.41	
70	1313.73	13.03	81.83	
80	1330.70	16.17	100.38	
90	1336.60	17.26	106.82	**AXIS**

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

Location: Old Airport to Tanjong Katong (SLS for short term - no creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Rigid linings Load Case 6

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	0.00	m	
Effective overburden pressure	q_1 =	81.7620	kN/m^2	
Surcharge	q_2 =	0.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical stress	σ_v' =	81.7620	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	61.3215	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	20.4405	kN/m^2	
Load factor for Water	FS_w =	1.00		
Hydrostatic water pressure	p_w =	136.2700	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	71.5418	kN/m^2	
Maximum shear strength of ground	τ =	28.9047	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	28.9047	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60 N/mm^2)	
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 10.83 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1247	7.5868	10.8280	432.6573	226.9132	-0.1460	-0.0766

N_d (kN)	u_d (mm)	M_d (kN-m)
-30.95	-2.27	-41.79

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	628.62	-2.49	-41.79	**CROWN**
10	630.49	-2.35	-39.27	
20	635.86	-1.96	-32.02	
30	644.10	-1.36	-20.90	
40	654.20	-0.62	-7.26	
45	659.57	-0.22	0.00	
50	664.94	0.17	7.26	
60	675.04	0.91	20.90	
70	683.28	1.51	32.02	
80	688.65	1.91	39.27	
90	690.52	2.04	41.79	**AXIS**

Location: Old Airport to Tanjong Katong (SLS for short term - no creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Rigid linings Load Case 7

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	0.00	m	
Effective overburden pressure	q_1 =	81.7620	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical stress	σ_v' =	156.7620	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	117.5715	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	39.1905	kN/m^2	
Load factor for Water	FS_w =	1.00		
Hydrostatic water pressure	p_w =	136.2700	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	71.5418	kN/m^2	
Maximum shear strength of ground	τ =	28.9047	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	28.9047	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	($I_j << I$)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 20.76 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1247	14.5461	20.7604	432.6573	435.0599	-0.1460	-0.1468

N_d (kN)	u_d (mm)	M_d (kN-m)
-59.34	-4.34	-80.13

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	808.38	-4.64	-80.13	**CROWN**
10	811.96	-4.37	-75.30	
20	822.26	-3.62	-61.38	
30	838.05	-2.46	-40.07	
40	857.41	-1.05	-13.91	
45	867.72	-0.29	0.00	
50	878.02	0.46	13.91	
60	897.39	1.88	40.07	
70	913.17	3.03	61.38	
80	923.48	3.79	75.30	
90	927.05	4.05	80.13	**AXIS**

Calculated by:John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

Location: Old Airport to Tanjong Katong (SLS for short term - no creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Rigid linings Load Case 8

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	111.7620	kN/m^2	
Surcharge	q_2 =	0.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical stress	σ_v' =	111.7620	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	83.8215	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	27.9405	kN/m^2	
Load factor for Water	FS_w =	1.00		
Hydrostatic water pressure	p_w =	106.2700	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	97.7918	kN/m^2	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

Calculated by:John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 14.80 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1247	10.3705	14.8010	337.4073	310.1719	-0.1138	-0.1047

N_d (kN)	u_d (mm)	M_d (kN-m)
-42.30	-3.10	-57.13

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	605.27	-3.32	-57.13	**CROWN**
10	607.83	-3.13	-53.68	
20	615.17	-2.59	-43.76	
30	626.43	-1.77	-28.56	
40	640.23	-0.76	-9.92	
45	647.58	-0.22	0.00	
50	654.93	0.32	9.92	
60	668.73	1.33	28.56	
70	679.99	2.15	43.76	
80	687.33	2.69	53.68	
90	689.88	2.88	57.13	**AXIS**

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date: .
Date: .
Date: .

Location: Old Airport to Tanjong Katong (SLS for short term - no creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Rigid linings Load Case 9

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	111.7620	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical stress	σ_v' =	186.7620	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	140.0715	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	46.6905	kN/m^2	
Load factor for Water	FS_w =	1.00		
Hydrostatic water pressure	p_w =	106.2700	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	97.7918	kN/m^2	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o\, r_e\, (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o\,(S_n+2S_t)/3$

$M = -r_o\, r_e\, (2S_n + S_t)\cos 2\theta/6$ $N = -r_o\,(S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 24.73 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.1247	17.3298	24.7334	337.4073	518.3186	-0.1138	-0.1749

N_d (kN)	u_d (mm)	M_d (kN-m)
-70.69	-5.17	-95.47

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	785.03	-5.46	-95.47	**CROWN**
10	789.30	-5.15	-89.71	
20	801.57	-4.25	-73.13	
30	820.38	-2.88	-47.73	
40	843.45	-1.19	-16.58	
45	855.73	-0.29	0.00	
50	868.00	0.61	16.58	
60	891.07	2.30	47.73	
70	909.88	3.68	73.13	
80	922.16	4.57	89.71	
90	926.42	4.89	95.47	**AXIS**

Location: Old Airport to Tanjong Katong (SLS for long term - creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Flexible linings Load Case 10

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	111.7620	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical pressure	σ_v' =	186.7620	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	140.0715	kN/m^2	
$p_o = \sigma_v' - \sigma_h'$	p_o =	46.6905	kN/m^2	
Load factor for Water	FS_w =	1.00		
Factored hydrostatic water pressure	p_w =	106.2700	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF GROUND

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	97.7918	kN/m^2
Shear strength $\tau = c' + p_u \tan\phi'$	τ =	39.5104	kN/m^2

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	16000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	16368.2864	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	5		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.1092E-03	m^4	

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 26.62 kN

$S_n = \{3(3-4\nu)p_o/2 -[2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.3895	9.1356	26.6244	337.4073	517.7901	-0.2277	-0.3494

N_d (kN)	u_d (mm)	M_d (kN-m)
-66.02	-12.22	-72.16

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	789.17	-12.80	-72.16	**CROWN**
10	793.16	-12.06	-67.81	
20	804.62	-9.94	-55.28	
30	822.19	-6.69	-36.08	
40	843.73	-2.70	-12.53	
45	855.20	-0.58	0.00	
50	866.66	1.55	12.53	
60	888.21	5.54	36.08	
70	905.77	8.79	55.28	
80	917.24	10.91	67.81	
90	921.22	11.65	72.16	**AXIS**

Location: Old Airport to Tanjong Katong (SLS for long term - creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Flexible linings Load Case 11

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	0.00	m	
Effective overburden pressure	q_1 =	81.7620	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical pressure	σ_v' =	156.7620	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	117.5715	kN/m^2	
$p_o = \sigma_v' - \sigma_h'$	p_o =	39.1905	kN/m^2	
Load factor for Water	FS_w =	1.00		
Factored hydrostatic water pressure	p_w =	136.2700	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF GROUND

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	71.5418	kN/m^2	
Shear strength $\tau = c' + p_u \tan\phi'$	τ =	28.9047	kN/m^2	

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	28.9047	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	16000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	16368.2864	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	5		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.1092E-03	m^4	

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3 (2S_n+S_t)/18EI$

$U = -r_e r_o^3 (2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 22.35 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.3895	7.6681	22.3477	432.6573	434.6163	-0.2920	-0.2933

N_d (kN)	u_d (mm)	M_d (kN-m)
-55.42	-10.26	-60.57

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	811.86	-10.85	-60.57	**CROWN**
10	815.20	-10.23	-56.92	
20	824.82	-8.45	-46.40	
30	839.56	-5.72	-30.29	
40	857.65	-2.37	-10.52	
45	867.27	-0.59	0.00	
50	876.90	1.20	10.52	
60	894.98	4.55	30.29	
70	909.73	7.27	46.40	
80	919.35	9.06	56.92	
90	922.69	9.68	60.57	**AXIS**

Location: Old Airport to Tanjong Katong (SLS for long term - creep)
(Shallow Section - Ch57+444 Tanjong Katong Station) Flexible linings Load Case 12

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.077	
Track Level:	R.L.	86.925	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	13.6270	m

3. LOADING

Ave. unit weight of soil	γ =	16.00	kN/m^3	
Water table from ground surface	h_w =	3.00	m	
Effective overburden pressure	q_1 =	111.7620	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Overburden Load	FS =	1.00		
Load factor for Surcharge	FS =	1.00		
Factored vertical pressure	σ_v' =	186.7620	kN/m^2	
k value	k =	0.75	***Marine Clay***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	140.0715	kN/m^2	
$p_o = \sigma_v' - \sigma_h'$	p_o =	46.6905	kN/m^2	
Load factor for Water	FS_w =	1.00		
Factored hydrostatic water pressure	p_w =	106.2700	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF GROUND

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	97.7918	kN/m^2
Shear strength $\tau = c' + p_u \tan\phi'$	τ =	39.5104	kN/m^2

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	4088.1	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	22.0	Degree	
Maximum shear strength of ground	τ =	39.5104	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	16000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	16368.2864	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	(I_j<<I)
Total no. of segments	n =	5		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.1092E-03	m^4	

(Shallow Section - Ch57+444 Tanjong Katong Station)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ = 26.62 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.3895	9.1356	26.6244	337.4073	517.7901	-0.2277	-0.3494

N_d (kN)	u_d (mm)	M_d (kN-m)
-66.02	-12.22	-72.16

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	789.17	-12.80	-72.16	**CROWN**
10	793.16	-12.06	-67.81	
20	804.62	-9.94	-55.28	
30	822.19	-6.69	-36.08	
40	843.73	-2.70	-12.53	
45	855.20	-0.58	0.00	
50	866.66	1.55	12.53	
60	888.21	5.54	36.08	
70	905.77	8.79	55.28	
80	917.24	10.91	67.81	
90	921.22	11.65	72.16	**AXIS**

Calculated by: John Poh
Checked by: Wen Dazhi
Approved by: Fred Lee

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2

Date:
Date:
Date:

Location: Tanjong Katong to Paya Lebar (ULS for short term - no creep)
(F2 Section - CH57+953) Rigid linings Load Case 2

1. ALIGNMENT DATA

Nominal Diameter of Tunnel	D_n =	5.60	m
Construction Allowance	ΔD =	100.00	mm
Thickness of Lining	t =	275.00	mm
Existing Ground Level:	R.L.	102.081	
Track Level:	R.L.	80.007	
Track Level to Invert of Tunnel	d =	1375.00	mm

2. TUNNEL GEOMETRY

Excavated Diameter of Tunnel	D =	6.3500	m
Internal radius of tunnel	r_i =	2.9000	m
Radius to extrados of lining	r_e =	3.1750	m
Radius of lining centroid	r_o =	3.0375	m
Depth to Tunnel Axis	z_o =	20.5490	m

3. LOADING

Ave. unit weight of soil	γ =	19.00	kN/m^3	
Water table from ground surface	h_w =	0.00	m	
Effective overburden pressure	q_1 =	184.9410	kN/m^2	
Surcharge	q_2 =	75.00	kN/m^2	
Load factor for Soil Overburden	FS =	1.40		
Load factor for Surcharge	FS =	1.60		
Factored vertical stress	σ_v' =	378.9174	kN/m^2	
k value	k =	0.75	***F2***	
Factored horizontal stress, $\sigma_h' = k\sigma_v'$	σ_h' =	284.1881	kN/m^2	
$p_o = \sigma_v - \sigma_h$	p_o =	94.7294	kN/m^2	
Load factor for Water	FS_w =	1.40		
Hydrostatic water pressure	p_w =	287.6860	kN/m^2	(γ_w = 10 kN/m^3)

4. SHEAR STRENGTH OF SOIL

Uniform loading, $p_u = (q_1 + kq_1) / 2$	p_u =	161.8234	kN/m^2	
Maximum shear strength of ground	τ =	68.6899	kN/m^2	($\tau = c' + p_u \tan\phi'$)

5. PROPERTIES OF GROUND AND LINING

Young's modulus of ground	E_c =	13315.8	kN/m^2	
Poisson's ratio of ground	ν =	0.35		
Effective cohesion of the ground	c' =	0.0	kN/m^2	
Effective friction angle of ground	ϕ' =	23.0	Degree	
Maximum shear strength of ground	τ =	68.6899	kN/m^2	($\tau = c' + p_u \tan\phi'$)
Young's modulus of lining	E_l =	32000.0	MN/m^2, (f_{cu} = 60	N/mm^2)
Poisson's ratio of lining	ν_l =	0.15		
E of lining in plane strain condition	E_l =	32736.5729	MN/m^2	
Area of lining	A =	0.2750	m^2	
Second moment of area of lining	I =	1.7331E-03	m^4	
I_j at a joint of lining	I_j =	0.0000	m^4	($I_j << I$)
Total no. of segments	n =	1		
Effective I , $I_e = I_j + (4/n)^2 I$, (n>4)	I_e =	1.7331E-03	m^4	

(F2 Section - CH57+953)

6. BENDING MOMENT, HOOP TRUST AND RADIAL MOVEMENT OF LINING

Based on Muir Wood (1975) and Curtis (1976), moments and forces can be calculated as:

$M_d = -r_o r_e (2S_n + S_t)/6$ (hogging moment positive) $N_d = -r_o (S_n+2S_t)/3$

$M = -r_o r_e (2S_n + S_t) \cos 2\theta/6$ $N = -r_o (S_n+2S_t)\cos 2\theta/3 + p_w r_e + N_o$ $u_d = -r_e r_o^3(2S_n+S_t)/18EI$

$U = -r_e r_o^3(2S_n+S_t)\cos 2\theta/18EI + U_w + U_u$ where S_n and S_t are the normal and shear stresses

$S_n = (1-Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)]$ (if $S_t<\tau$) $S_t = (1+2Q_2)p_o/2[1+Q_2(3-2\nu/3-4\nu)] =$ 54.20 kN

$S_n = \{3(3-4\nu)p_o/2 - [2Q_2+(4-6\nu)]\tau\}/[4Q_2+5-6\nu]$ (if $S_t>\tau$)

$Q_2 = E_c r_o^3/12EI(1+\nu)$ $N_o = \sigma_v'(1+k)r_e/(2+2E_c r_o/EA(1+\nu))$

$u_w = -p_w r_e r_o/EA$ $u_u = -N_o r_o/EA$

Q_2	S_n	S_t (<=τ)	$p_w r_e$	N_o	u_w (mm)	u_u (mm)
0.4060	17.7649	54.1954	913.4031	1049.1882	-0.3082	-0.3540

N_d (kN)	u_d (mm)	M_d (kN-m)
-133.51	-7.82	-144.22

θ (Deg.)	N (kN)	U (mm)	M (kN-m)	
0	1829.08	-8.48	-144.22	**CROWN**
10	1837.13	-8.01	-135.52	
20	1860.31	-6.65	-110.48	
30	1895.83	-4.57	-72.11	
40	1939.41	-2.02	-25.04	
45	1962.59	-0.66	0.00	
50	1985.78	0.70	25.04	
60	2029.35	3.25	72.11	
70	2064.87	5.33	110.48	
80	2088.05	6.68	135.52	
90	2096.11	7.16	144.22	**. AXIS**

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2 PROJECT
Design Sheet

File No : Interaction_Diagram_OAP-TJK.xls
Drawing No :

Sheet 1 of 1
Calculated By: John Poh Date:
Checked By: Wen Dazhi Date:

OLD AIRPORT ROAD - TANJONG KATONG

Structural Design

This section checks the capacity of the tunnel segments assumming as a short column (Design Criteria 7.5.1.6)

Ultimate Limit State (ULS)

Forces calculated from the Curtis Formula at both axis and crown are plotted against the interaction diagram for the tunnel segment derived in accordance with the CP 65 Part 1 : 1999

Section	Old Airport Road - Tanjong Katong (OAP to TKJ)	Axis		Crown		Load Case
		N (kN)	M (kNm)	N (kN)	M (kNm)	
1	ULS-ST-Rigid (Deep)	1392.46	79.05	1269.65	79.05	1
	ULS-ST-Rigid-SC (Deep)	1769.99	136.53	1557.89	136.53	2
	ULS-ST-Rigid-BGL (Deep)	1391.24	99.17	1237.18	99.17	3
	ULS-ST-Rigid-BGL-SC (Deep)	1768.78	156.65	1525.42	156.65	4
	ULS-LT-Flexible-BGL-SC (Deep)	1757.91	164.52	1533.62	164.52	5
2	ULS-ST-Rigid (Shallow)	966.73	58.51	880.07	58.51	1
	ULS-ST-Rigid-SC (Shallow)	1342.08	120.76	1170.79	120.76	2
	ULS-ST-Rigid-BGL (Shallow)	965.84	79.98	847.38	79.98	3
	ULS-ST-Rigid-BGL-SC (Shallow)	1344.29	141.32	1135.00	141.32	4
	ULS-LT-Flexible-BGL-SC (Shallow)	1336.60	162.28	1141.13	162.28	5

From the interaction diagram, it can be seen that all the points of the above load cases are within 0.69% reinforcement (Type A).

LAND TRANSPORT AUTHORITY
CIRCLE LINE STAGE 2 PROJECT
Design Sheet

File No : Interaction_Diagram_OAP-TJK.xls
Drawing No :

Calculated by: John Poh Date:
Checked by : Wen Dazhi Date :

OLD AIRPORT ROAD - TANJONG KATONG

Structural Design

This section checks the capacity of the tunnel segments assumming as a short column (Design Criteria 7.5.1.6)

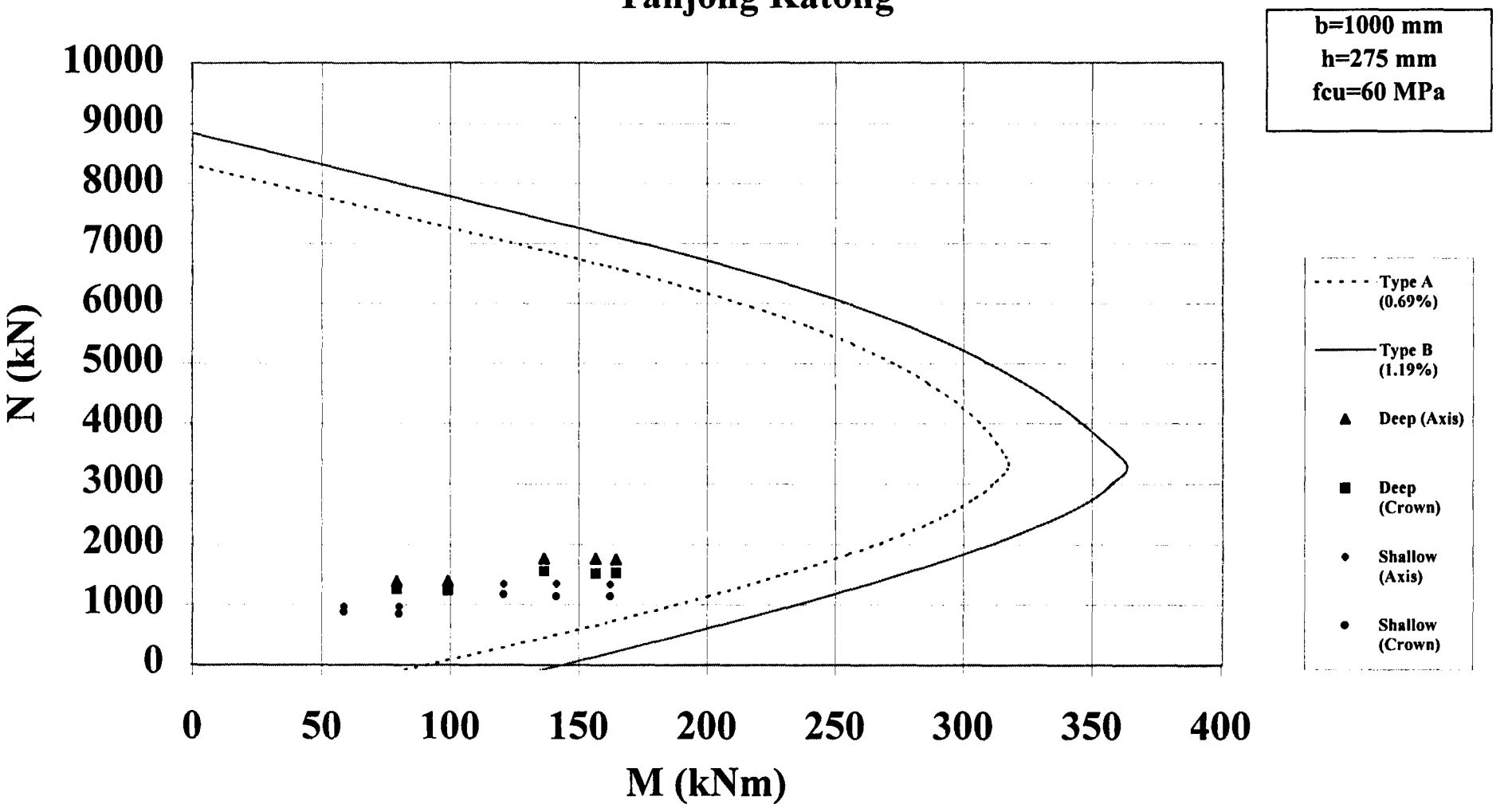
Serviceability Limit State (SLS)

The crackwidth calculations are carried out in accordance with CP 65 : Part 2 : 1999 Section 3.8. the results are tabulated in the following tables for all the load cases considered in the design at both the crown and axis level.

Section	Old Airport Road - Tanjong Katong (OAP to TKJ)	Axis		Crown		Load Case
		N (kN)	M (kNm)	N (kN)	M (kNm)	
1	SLS-ST-Rigid (Deep)	994.61	56.46	906.89	56.46	6
	SLS-ST-Rigid-SC (Deep)	1230.57	92.39	1087.04	92.39	7
	SLS-ST-Rigid-BGL (Deep)	993.75	70.83	883.70	70.83	8
	SLS-ST-Rigid-BGL-SC (Deep)	1229.70	106.76	1063.85	106.76	9
	SLS-LT-Flexible-BGL-SC (Deep)	1222.30	113.94	1069.44	113.94	10
2	SLS-ST-Rigid (Shallow)	690.52	41.79	628.62	41.79	6
	SLS-ST-Rigid-SC (Shallow)	927.05	80.13	808.38	80.13	7
	SLS-ST-Rigid-BGL (Shallow)	689.88	57.13	605.27	57.13	8
	SLS-ST-Rigid-BGL-SC (Shallow)	926.42	95.47	785.03	95.47	9
	SLS-LT-Flexible-BGL-SC (Shallow)	921.22	111.77	789.17	111.77	10

Section	Old Airport Road - Tanjong Katong (OAP to TKJ)	Axis	Crown	Load Case
		Crackwidth (mm)	Crackwidth (mm)	
1	SLS-ST-Rigid (Deep)	0.00	0.03	6
	SLS-ST-Rigid-SC (Deep)	0.00	0.00	7
	SLS-ST-Rigid-BGL (Deep)	0.00	0.00	8
	SLS-ST-Rigid-BGL-SC (Deep)	0.02	0.06	9
	SLS-LT-Flexible-BGL-SC (Deep)	0.04	0.08	10
2	SLS-ST-Rigid (Shallow)	0.02	0.00	6
	SLS-ST-Rigid-SC (Shallow)	0.00	0.03	7
	SLS-ST-Rigid-BGL (Shallow)	0.00	0.00	8
	SLS-ST-Rigid-BGL-SC (Shallow)	0.05	0.10	9
	SLS-LT-Flexible-BGL-SC (Shallow)	0.13	0.18	10

Interaction Diagram For Bored Tunnel Segment From Old Airport To Tanjong Katong



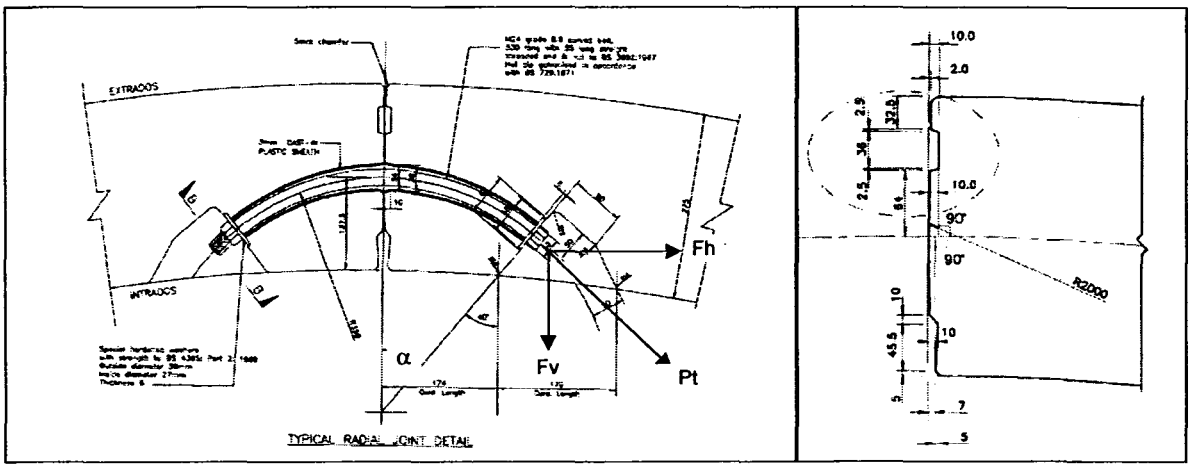
0197

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: RadialBolts.xls Calculated by: John Poh Date: _____

Drawing No.: _____ Checked by: Wen Dazhi Date: _____

Radial Bolts Design



Geometry

External diameter of tunnel,	D_E	=	6350	mm
Internal diameter of tunnel,	D_I	=	5800	mm
Nominal diameter of tunnel,	D	=	6075	mm
Nominal radius of tunnel,	R	=	3037.5	mm
Width of segment,	W	=	1400	mm
Angle of ordinary segments,	θ	=	67.5	°
Angle of rear face bolt,	α	=	40	°
Specific gravity of concrete,	γ	=	24	kNm^{-3}
Unfactored gasket force assumed	Fg	=	45	KN/m
Load factor	L.F	=	1.4	

Factored gasket force assumed F'g = L.F x Fg x W
88.2 KN per segment

Bolts provided are 2 no. of M24 grade 8.8,

Tensile stress area	A_t	=	353	mm^2	From "Steel Designers' Manual (Fifth Edition)" See Attached
Tensile strength	p_t	=	450	N/mm^2	

(Ref. Steel Designers' Manual -Fifth Edition, pg. 1165 & 1170)

Tensile Capacity per bolt $P_t = 450*353*10^{-3}$
= 158.85 kN

Tensile capacity of 2 bolts = $2P_t$
= 317.7 kN

Resolve bolt force at joint,

Vertical Force Fv = $P_t \sin\alpha$

Horizontal force Fh = $P_t \cos\alpha$

Fv = 102.107 kN

Fh = 243.37 kN > F'g (ok)

Therefore bolts provided at the radial joint are capable of compressing the gaskets.

0202

Hole sizes – for ordinary bolts and friction grip connections

Nominal diameter (mm)	Clearance hole diameter[b] (mm)	Oversize hole diameter[a] (mm)	Short slotted holes[a] (mm)		Long slotted holes[a] (mm)	
			Narrow dimension	Slot dimension	Narrow dimension	Maximum dimension
M12[a]	14	17	14	18	14	30
M16	18	21	18	22	18	40
M20	22	25	22	26	22	50
M22	24	27	24	28	24	55
M24	26	30	26	32	26	60
M27	30	35	30	37	30	67
M30	33	38	33	40	33	75

[a] Hardened washers to be used
[b] In cases where there are more than three plies in joint the holes in the inner plies should be one millimetre larger than those in the outer plies

Bolt strengths

	Bolt grade		Steel to BS 4360		
	4.6	8.8	43	50	55
Shear strength, p_s (N/mm^2)	160	375			
Bearing strength, p_{bb} (N/mm^2)	460	1035[a]	460	550	650
Tension strength, p_t (N/mm^2)	195	450			

[a] The bearing value of the connected part is critical

Steel Designers' Manual
(Fifth Edition)
The Steel Construction Institute

0203

Bolt capacities in tension

Nominal diameter (mm)	Tensile stress area[b] (mm^2)	Bolts grade 4.6 @ 195 N/mm^2 (kN)	Bolts grade 8.8 @ 450 N/mm^2 (kN)
M12	84.3	16.43	37.93
M16	157	30.61	70.65
M20	245	47.77	110.25
M22[a]	303	59.08	136.35
M24	353	68.83	158.85
M27[a]	459	89.50	206.55
M30	561	109.39	252.45

[a] Non-preferred sizes
[b] Tensile stress areas are taken from BS 4190 and BS 3692

Spacing, end and edge distances – minimum values (see Fig. 23.1)

Nominal diameter of fastener (mm)	Diameter of clearance hole (mm)	Minimum spacing (mm)	Edge distance to rolled, sawn, planed, or machine flame cut edge (mm)	Edge distance to sheared edge or hand flame cut edge and end distance (mm)
M12	14	30	18	20
M16	18	40	23	26
M20	22	50	28	31
M22[a]	24	55	30	34
M24	26	60	33	37
M27[a]	30	68	38	42
M30	33	75	42	47

[a] Non-preferred size

Maximum centres of fasteners

Thickness of element (mm)	Spacing in the direction of stress (mm)	Spacing in any direction in corrosive environments (mm)
5	70	80
6	84	96
7	98	112
8	112	128
9	126	144
10	140	160
11	154	176
12	168	192
13	182	200
14	196	200
15	210	200

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: ConvexRadialJoint_Design823.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

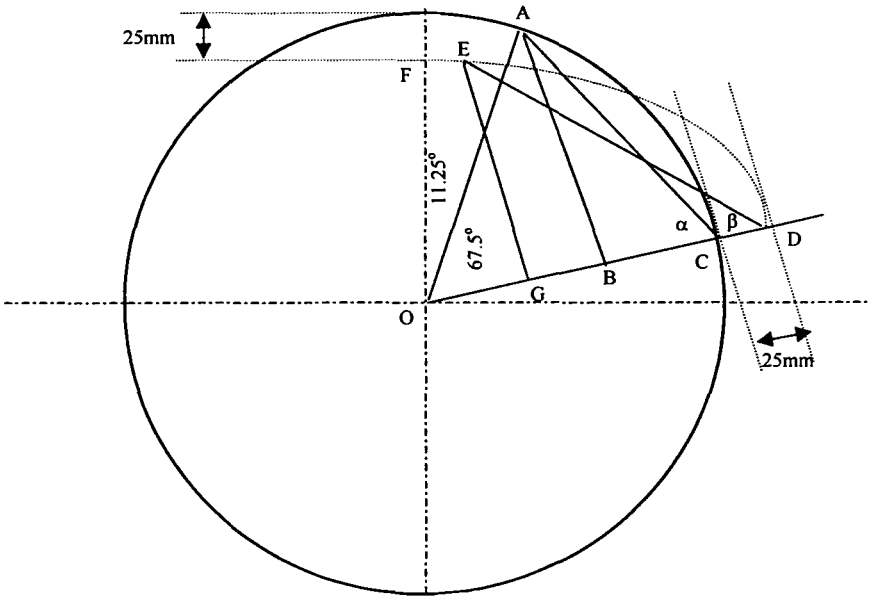
Checked by: Wen Dazhi Date: _____

JOINT ROTATION

Data

External diameter of tunnel,	D_E	=	6350	mm
Internal diameter of tunnel,	D_I	=	5800	mm
Radius of tunnel,	R	=	2900	mm
Norminal diameter of tunnel,	D	=	6075	mm
Nominal radius of tunnel,	R	=	3037.5	mm
Width of segment,	b	=	1400	mm
Segment thickness,	t	=	275	mm
Angle of ordinary segments,	ϕ	=	67.5	°
Angle of key segments,	θ	=	22.5	°
Change in diameter	δD_E	=	50	mm
Change in radius	δR	=	25	mm
% change in diameter	k	=	$(\delta D_E/D)*100$	
		=	0.82305	

Rotation of radial joint due to ground deformation and building tolerances will not be greater than that caused by an ellipse whose maximum and minimum diameters



File No.: ConvexRadialJoint_Design823.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

Assume that the segments rotate as a rigid body

$OC = OA$
$= 3037.5$

$AB = R\sin\phi$
$= 2806.28$ mm

$OB = R\cos\phi$
$= 1162.4$ mm

$BC = R - OB$
$= 1875.1$ mm

$\alpha = \tan^{-1}(AB/BC)$
$= 56.25$ °

$OD = OC + \delta R$
$= OB + BC + \delta R$
$= 3062.5$ mm

$OF = R - \delta R$
$= 3012.50$ mm

Length of chord AC and ED

$L = (AB^2 + BC^2)^{0.5}$
$= 3375.09$ mm

Using ellipse equation

$EG^2/OF^2 + OG^2/OD^2 = 1.00$

$EG^2 = (OF^2)/(1 - OG^2/OD^2)$ EQ 1

Using Pythagoras' theorem in triangle EGD

$EG^2 = EG^2 + GD^2$
$= ED^2 - GD^2$ EQ 2

Substituting EQ 1 into EQ 2

$ED^2 - GD^2 = (OF^2)/(1 - OG^2/OD^2)$
$ED^2 - (OD - OG)^2 = (OF^2)/(1 - OG^2/OD^2)$
$ED^2 - OD^2 + 2(OD)(OG) - OG^2 = (OF^2)/(1 - OG^2/OD^2)$
$((OF^2/OD^2) - 1)OG^2 + 2(OD)(OG) + ED^2 - OD^2 - OF^2 = 0$

Let

$A = (OF^2/OD^2) - 1$
$= -0.03$ mm

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: ConvexRadialJoint_Design823.xls Calculated by: John Poh Date: _____

Drawing No.: _____ Checked by: Wen Dazhi Date: _____

$B = 2\,(OD)$
$= 6125$ mm

$C = ED^2 - OD^2 - OF^2$
$= -7.06E+06$ mm

Therefore $OG = 1160.23$ mm

Substituting OG into EQ 1

$EG = 2787.94$ mm

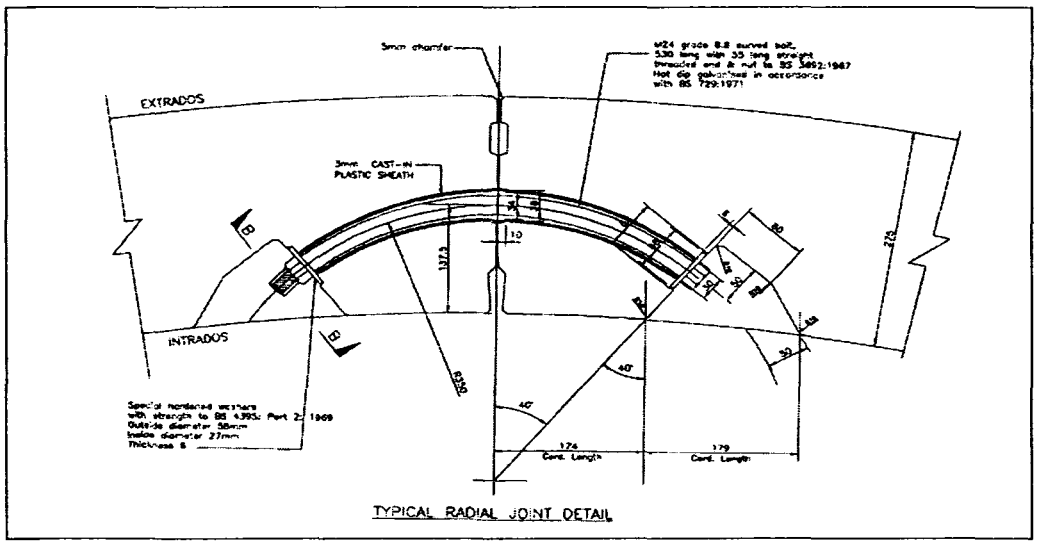
$\beta = \sin^{-1}(EG/ED)$
$= 55.6935\ °$

Angular rotation $\gamma = \alpha - \beta$
$= 0.55648\ °$

Total rotation at joint C $\gamma_c = 2*\gamma$
$= 1.11\ °$

Radius of convex joint $r = 2000$

Eccentricity due to joint rotation $e_1 = (r_1 r_2 \gamma_c)/(r_1 + r_2)$
$= r\gamma_c/2 \qquad r_1 = r_2$
$= 19.4249$ mm



TYPICAL RADIAL JOINT DETAIL

File No.: ConvexRadialJoint_Design823.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

Misalignment				
Bolt size	d_1	=	24	mm
Bolt gap	d_2	=	34	mm
Tolerance	tol	=	$(d_2 - d_1)/2$	
		=	5	mm
Max possible misalignment	S	=	10	mm
Eccentricity due to misalignment	e_2	=	$r_1 S/r_1 + r_2$	
		=	S/2	
		=	5	mm
Total eccentricity	e	=	$e_1 + e_2$	
		=	24.4249	mm

File No.: ConvexRadialJoint_Design823.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

RADIAL JOINT DESIGN

Data

Concrete characteristic strength,	f_{cu}	=	60	MPa
Partial factor of safety, dead load	γ_{DL}	=	1.4	
Partial factor of safety, live load	γ_{LL}	=	1.6	
Partial factor of safety, concrete	γ_m	=	1.5	
Partial factor of safety, steel	γ_s	=	1.15	
Young's Modulus (long term)	E	=	16	$kNmm^{-2}$
Segment geometry				
Width of tunnel segment,	W	=	1400	mm
Recess length,	l_C	=	100	mm
Length,	b_y	=	$W - 2l_C$	
		=	1200	mm
Thickness,	t_s	=	275	mm
Radius of convex joint surface,	R	=	2000	mm
External diameter of tunnel,	D_E	=	6.35	
External radius of tunnel,	R_E	=	3.175	m
Poisson's ratio of lining,	υ	=	0.2	

Determining Critical Design Section,

From output of the Muir Wood/Curtis analysis, maximum hoop load observed is in design section case 4 of F2 section for tunnel from Tanjong Katong Station To Paya Lebar Station.

Refer to hoop load from Muir Wood/Curtis analysis (F2 Case 4)

Maximum factored N per m		=	2093.85	KN
Maximum factored N per 1.4 m (width of segment)	N	=	2931.39	KN

File No.: ConvexRadialJoint_Design823.xls Calculated by: John Poh Date: _____

Drawing No.: _____ Checked by: Wen Dazhi Date: _____

BEARING STRESS CHECK

Load per unit length of segment, $\rho = N/b_y$

$= 2.44$ kNmm^{-1}

Width of bearing area is determined to formula in "Roark's formula for stress & strain" (see attached)

Constant, $K_D = \frac{D_1 D_2}{D_1 + D_2}$ $(D_1 = D_2 = 2R)$

$= 2000$ mm

$C_E = \frac{1 - \upsilon_1^2}{E_1} + \frac{1 - \upsilon_2^2}{E_2}$ $(\upsilon_1 = \upsilon_2 = \upsilon)$ $(E_1 = E_2 = E)$

$= 0.120$ mm^{2}kN^{-1}

Width of bearing area, $b = 1.60(\rho K_D C_E)^{1/2}$

$= 38.74$ mm

Allowable bearing stress, $f_b = 2f_{cu}$

$= 120$ Mpa

(or 105 Mpa whichever is lesser)

(DTP Highways and Traffic Technical Memorandum (Bridges) BE5/75 Cl.303(a) allows compressive stress at the throat of a Freyssinet Hinge to be twice the characteristic strength, fcu, but limited to a maximum of 105 Mpa - See Appendix)

Design bearing stress, $f_{bc} = \frac{N}{b.b_y}$

$= 63.06$ MPa

$f_{bc} < f_b$ OK

Eccentricity

Total rotation at convex radial joint, $\theta = 1.11$ °

$= 0.02$ rad

Eccentricity due to joint rotation for each segment, $e_1 = R\theta/2$

$= 19.38$ mm

Bolt size used, $d_1 = 24$ mm

Bolt gap, $d_2 = 34$ mm

Tolerance, $tol = (d_2 - d_1)/2$

$= 5$ mm

Maximum possible misalignment, $S = 10$ mm

Eccentricity due to misalignment, $e_2 = S/2$

$= 5$ mm

Total eccentricity, $e = e_1 + e_2$

$= 24.38$ mm

Eccentric moment (ULS), $M_{ecc} = Ne/1000$

$= 71.47$ KNm/segment

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: ConvexRadialJoint_Design823.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

Reinforcement Check for eccentric moment at radial joint

Type A

Characteristic strength of reinforcement,	f_y	=	460 MPa	
Concrete cover,	c	=	40 mm	
Shear links diameter,	d_s	=	10 mm	
Re-bar (U bar at radial joint) diameter,	d_U	=	13 mm	
Effective Depth,	d	=	$t_s - c - d_s - d_U/2$	
		=	218.5 mm	
Ref. Cl.3.4.4.4, CP65 Part 1:1999,	K	=	$M_{ecc}/b_y d^2 fcu$	
		=	0.02079 < 0.156	
			Compression steel is not needed	
Lever arm,	z	=	$d[0.5+(0.25-K/0.9]^{1/2}$	
		=	213.33	
Eccentricity moment,	M_{ecc}	=	$0.87 f_y A_s z$	
Area of tension reinforcement,	A_s	=	$\frac{M_{ecc}}{0.87 f_y z}$	
		=	837 mm^2	
Provision (9 U-bars dia. 13 mm),				
Number of bars,	n	=	9	
Diameter of bar,	d	=	13 mm	
Area of reinforcement provided at each face,	A	=	1195 mm^2	Asprov is ok

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: ConvexRadialJoint_Design823.xls Calculated by: John Poh Date: _____

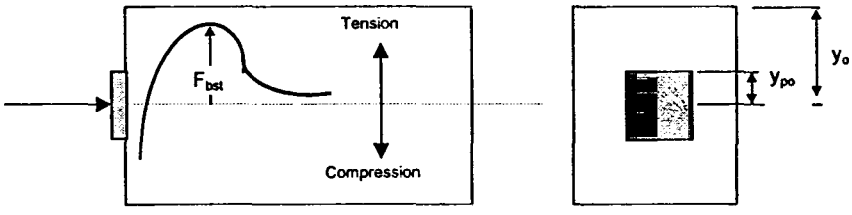
Drawing No.: _____ Checked by: Wen Dazhi Date: _____

Reinforcement Type A

Radial joints are checked for splitting force due to hoop force.
Closed links are provided at the circumferential joints to resist this force.
Type A segments are checked.

All types are checked that the links provided are sufficient to resist the splitting force at both ultimate and serviceability limits.

Checking Bursting (Adopt the End block design in CP65 Cl.4.11.2)
(Check at Ultimate)



Hoop force under normal operation is assumed to be = 2093.85 KN

At ultimate, Po = 1.4 x 2094 KN
= 2931.39 KN

y_{po} = 19.3705 mm
y_o = 137.5 mm
y_{po}/y_o = 0.14

y_{po}/y_o	0.2	0.3	0.4	0.5	0.6	0.7
F_{bst}/P_o	0.23	0.23	0.2	0.17	0.14	0.11

Table 4.7 Design Bursting Tensile Strength In End Blocks

From Table 4.7,
F_{bst}/P_o = 0.23 (conservative)

Checking Of Joint Bursting
Fbst = 674 KN (Bursting Force)

Min Area of links required
Asv = 1685 mm^2 (Stress of steel = 0.87*460N/mm^2)

Provide 9 Legs T 13 (U bars)
16 Legs T 13 (Ladder bars)
8 Legs T 10 (Links)
= 3947 mm^2

Links to be distributed from 0.2yo to 2yo
27.5 to 275 mm

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: ConvexRadialJoint_Design823.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

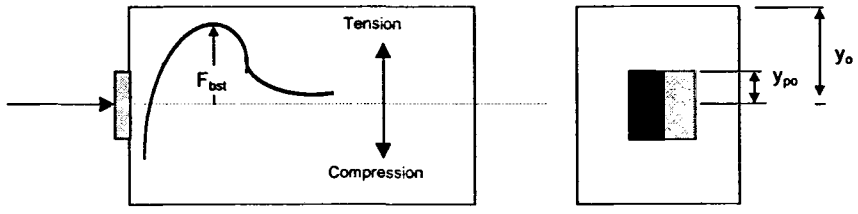
Checked by: Wen Dazhi Date: _____

Reinforcement Type A

Radial joints are checked for splitting force due to hoop force.
Closed links are provided at the circumferential joints to resist this force.
Type A segments are checked.

All types are checked that the links provided are sufficient to resist the splitting force at both ultimate and serviceability limits.

Checking Bursting (Adopt the End block design in CP65 Cl.4.11.2)
(Check at Service)



Hoop force under normal operation is assumed to be = 2093.85 KN

At service, Po = 2093.85 KN
= 2093.85 KN

y_{po} = 19.3705 mm
y_o = 137.5 mm
y_{po}/y_o = 0.14

y_{po}/y_o	0.2	0.3	0.4	0.5	0.6	0.7
F_{bst}/P_o	0.23	0.23	0.2	0.17	0.14	0.11

Table 4.7 Design Bursting Tensile Strength In End Blocks

From Table 4.7,
F_{bst}/P_o = 0.23

Checking Of Joint Bursting
Fbst = 482 KN (Bursting Force)

Min Area of links required
Asv = 2408 mm^2 (Stress of steel = 200N/mm2)

Provide **9 Legs T 13** (U bars)
16 Legs T 13 (Ladder bars)
8 Legs T 10 (Links)
= 3947 mm^2

Links to be distributed from 0.2yo to 2yo
27.5 to 275 mm

Department of Transport
Highways & Traffic
Technical Memorandum
BE 5/75

0213

SECTION THREE. DESIGN

301. Basic assumptions

(a) The effect of any reinforcing steel which may be incorporated in the throat of the hinge for ease of handling is neglected.

(b) The effect of shrinkage cracks in the throat is neglected.

(c) For short term loading the behaviour of the concrete is elastic.

(d) For long term loading the creep is proportional to the initial stress.

(e) In considering the transverse tensile forces on either side of the throat the tensile strength of the concrete is neglected.

302. Loadings

The loadings shall be as specified in British Standard 153: Part 3A; 1972 and in Department of the Environment Technical Memorandum (Bridges) BE5/73 'Standard Highway Loadings'. In addition the hinges shall be designed to withstand all loadings which may be applied during construction. See Clause 403.

303. Permissible stresses

(a) Concrete

The average compressive stress in the concrete in the throat shall not exceed $2u_w$ or 105 N/mm^2 whichever is the lesser. Tensile stresses in the throat shall not be permitted except for shrinkage stresses which may arise during construction.

(b) Steel

The stresses in the transverse mat reinforcement shall not exceed 105 N/mm^2

304. Design of throat

(a) The design of the throat is dependent on:

(i) the maximum load to be carried, and

(ii) the maximum value of rotation per unit load.

(b) The basis of calculation is given in Appendix A and shall be used for the design of the width of throat, which shall be not less than 50mm or such value as will provide a minimum cover of 25mm to any reinforcement in the throat.

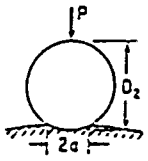
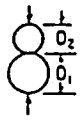
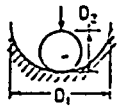
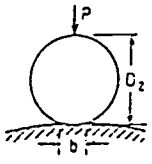
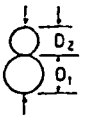
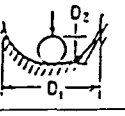
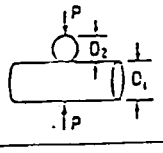
(c) The values given in Tables 1 and 2 have been calculated by the method given in Appendix A. These cover all normal cases and enable the designer to see whether a throat of given width can accommodate a given loading and the rotations due to long and short term causes.

For simplicity of use the short term value of E has been taken for both long and short term effects in these tables. The rotations due to shrinkage, creep, elastic shortening and permanent loading must therefore be halved before being added to the rotations due to temperature and transient loading (See Appendix A).

TABLE 33 Formulas for stress and strain due to pressure on or between elastic bodies

NOTATION: P = total load; p = load per unit length; a = radius of circular contact area for case 1; b = width of rectangular contact area for case 2; c = major semiaxis and d = minor semiaxis of elliptical contact area for cases 3 and 4; y = relative motion of approach along the axis of loading of two points, one in each of the two contact bodies, remote from the contact zone; ν = Poisson's ratio; E = modulus of elasticity. Subscripts 1 and 2 refer to bodies 1 and 2, respectively. To simplify expressions let

$$C_E = \frac{1-\nu_1^2}{E_1} + \frac{1-\nu_2^2}{E_2}$$

Conditions and case no.	Formulas	
1. Sphere  	$a = 0.721\sqrt[3]{PK_DC_E}$ Max $\sigma_c = 1.5\frac{P}{\pi a^2}$ $= 0.918\sqrt[3]{\frac{P}{K_D^2C_E^2}}$ $y = 1.040\sqrt[3]{\frac{P^2C_E^2}{K_D}}$ If $E_1 = E_2 = E$ and $\nu_1 = \nu_2 = 0.3$, then $a = 0.881\sqrt[3]{\frac{PK_D}{E}}$ (*Note:* 50% of y occurs within a distance of 1.2 times the contact radius a and 90% within a distance $7a$ from the contact zone) Max $\sigma_c = 0.616\sqrt[3]{\frac{PE^2}{K_D^2}}$ $y = 1.55\sqrt[3]{\frac{P^2}{E^2K_D}}$ Max $\sigma_t \simeq 0.133$ (max σ_c) radially at the edge of contact area Max $\tau \simeq \frac{1}{3}$ (max σ_c) at a point on the load line a distance $a/2$ below the contact surface (Approximate stresses from Refs. 3 and 6) For graphs of subsurface stress variations see Refs. 6 and 57	1a. Sphere on a flat plate  $K_D = D_2$ 1b. Sphere on a sphere  $K_D = \frac{D_1D_2}{D_1 + D_2}$ 1c. Sphere in a spherical socket  $K_D = \frac{D_1D_2}{D_1 - D_2}$
2. Cylinder of length L large as compared with D; p = load per unit length = P/L  	$b = 1.60\sqrt{pK_DC_E}$ Max $\sigma_c = 0.798\sqrt{\frac{p}{K_DC_E}}$ If $E_1 = E_2 = E$ and $\nu_1 = \nu_2 = 0.3$, then $b = 2.15\sqrt{\frac{pK_D}{E}}$ Max $\sigma_c = 0.591\sqrt{\frac{pE}{K_D}}$ For a cylinder between two flat plates $\Delta D_2 = \frac{4p(1-\nu^2)}{\pi E}\left(\frac{1}{3} + \ln\frac{2D}{b}\right)$ Refs. 5 and 44 For a cylinder on a cylinder the distance between centers is reduced by $\frac{2p(1-\nu^2)}{\pi E}\left(\frac{2}{3} + \ln\frac{2D_1}{b} + \ln\frac{2D_2}{b}\right)$ Ref. 31 For graphs of subsurface stress variations see Refs. 6 and 56	2a. Cylinder on a flat plate  $K_D = D_2$ Max $\tau \simeq \frac{1}{3}$(max σ_c) at a depth of 0.48 below the surface of the plane 2b. Cylinder on a cylinder  $K_D = \frac{D_1D_2}{D_1 + D_2}$ 2c. Cylinder in a cylindrical socket  $K_D = \frac{D_1D_2}{D_1 - D_2}$
3. Cylinder on a cylinder; axes at right angles  	$c = \alpha\sqrt[3]{PK_DC_E}$ $d = \beta\sqrt[3]{PK_DC_E}$ Max $\sigma_c = \frac{1.5P}{\pi cd}$ $y = \lambda\sqrt[3]{\frac{P^2C_E^2}{K_D}}$ Max $\tau \simeq \frac{1}{3}$(max σ_c)	$K_D = \frac{D_1D_2}{D_1 + D_2}$ and α, β, and λ depend upon $\frac{D_1}{D_2}$ as shown

D_1/D_2	1	1.5	2	3	4	6	10
α	0.908	1.045	1.153	1.350	1.505	1.767	2.175
β	0.908	0.799	0.732	0.651	0.602	0.544	0.481
λ	0.825	0.813	0.804	0.774	0.747	0.702	0.641

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: CircumferentialJoint_Design_TypeA&B_823.xls Calculated by: _____ Date: _____

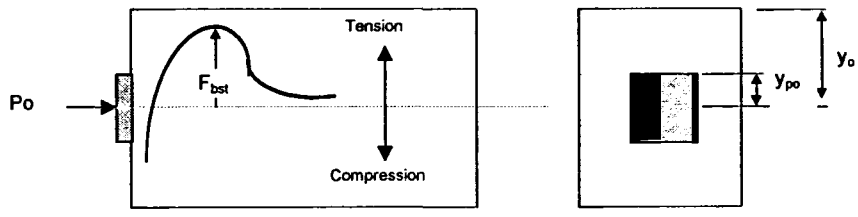
Drawing No.: _____ Checked by: _____ Date: _____

Reinforcement Type A

Circumferential joints are checked for splitting force due to jacking force from the TBM during shoving.
Closed links are provided at the circumferential joints to resist this force.
Type A segments are checked.

All types are checked that the links provided are sufficient to resist the splitting force at both ultimate and serviceability limits.

Checking Bursting (Adopt the End block design in CP65 Cl.4.11.2)
(Check at Serviceability)



Jacking force under normal operation is assumed to be = 1250 KN
(Contractor is required to check against their selected TBM specification accordingly)

No of jack per segment , n = 3

At ultimate, Po = 3 x 1.4 x 1250 KN (assuming 3 jack per segment)
= 5250 KN

y_{po} = 64 mm
y_o = 137.5 mm
y_{po}/y_o = 0.47

y_{po}/y_o	0.2	0.3	0.4	0.5	0.6	0.7
F_{bst}/P_o	0.23	0.23	0.2	0.17	0.14	0.11

Table 4.7 Design Bursting Tensile Strength In End Blocks

From Table 4.7,
F_{bst}/P_o = 0.23 (conservative)

Checking Of Joint Bursting
Fbst = 1208 KN (Bursting Force)

Min Area of links required
Asv = 3017.2 mm^2 (Stress of steel = 0.87*460N/mm^2)

Provide **40 Legs T 10**
= 6284 mm^2

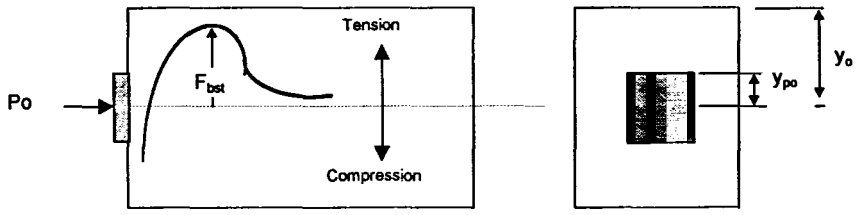
Links to be distributed from 0.2yo to 2yo
27.5 to 275 mm

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: CircumferentialJoint_Design_TypeA&B_823.xls Calculated by: _____ Date: _____

Drawing No.: _____ Checked by: _____ Date: _____

Checking Bursting (Adopt the End block design in CP65 Cl.4.11.2)
(Check at Serviceability)



Jacking force under normal operation is assumed to be = 1250 KN
(Contractor is required to check against their selected TBM specification accordingly)

No of jack per segment , n = 3

At ultimate, Po = 3 x 1250 KN (assuming 3 jack per segment)
= 3750 KN

y_{po} = 64 mm
y_o = 137.5 mm
y_{po}/y_o = 0.47

y_{po}/y_o	0.2	0.3	0.4	0.5	0.6	0.7
F_{bst}/P_o	0.23	0.23	0.2	0.17	0.14	0.11

Table 4.7 Design Bursting Tensile Strength In End Blocks

From Table 4.7,
F_{bst}/P_o = 0.23 (conservative)

Checking Of Joint Bursting
Fbst = 863 KN (Bursting Force)

Min Area of links required
Asv = 4312.5 mm^2 (Stress of steel = 200N/mm2)

Provide **40 Legs T 10**
= 6284 mm^2

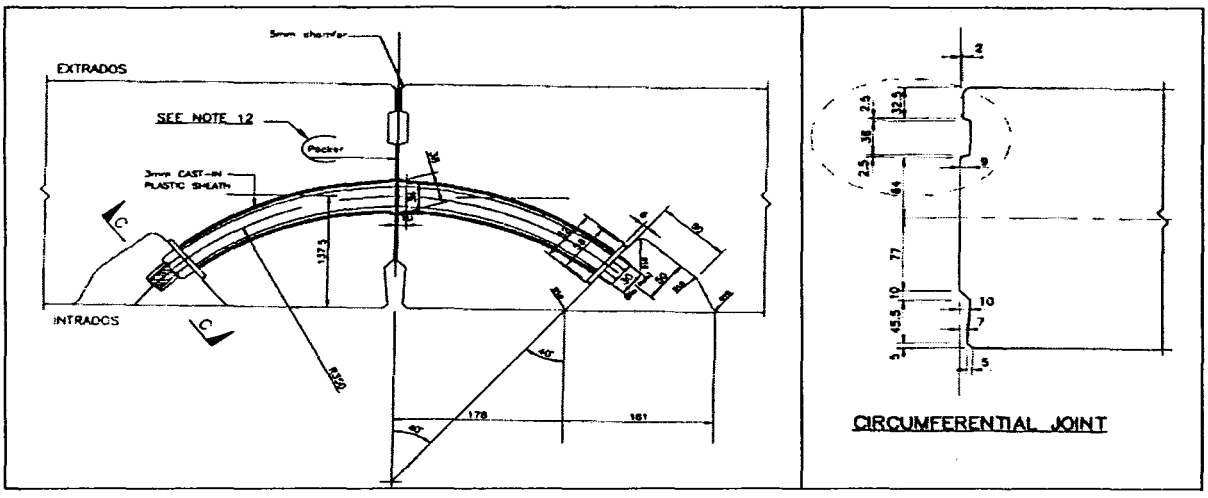
Links to be distributed from 0.2yo to 2yo
27.5 to 275 mm

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: CircumferentialBolts_Design_823.xls Calculated by: John Poh Date: _____

Drawing No.: _____ Checked by: Wen Dazhi Date: _____

Checking Of Bolts At Circumferential Joint



Load Cases

LC1: The bolts in the circumferential joint are required to maintain the compression of the gasket for a complete ring of segments, should the TBM rams be removed. (If the TBM rams are acting, then the bolts are not required to compress the gasket).
LC2: This load case checks that in the event the TBM ram loads are removed from an incomplete ring of segments, the bolts can withstand the force due to the self-weight of the segment. (accidental loadcase)

Data

External diameter of tunnel,	D_E	=	6350	mm
Internal diameter of tunnel,	D_I	=	5800	mm
Norminal diameter of tunnel,	D	=	6075	mm
Nominal radius of tunnel,	R	=	3037.5	mm
Width of segment,	W	=	1400	mm
Angle of ordinary segments,	θ	=	67.5	°
Angle of rear face bolt,	α	=	40	°
Specific gravity of concrete,	γ	=	24	kNm^{-3}

(i) LC1 - To check bolts provided are sufficient to compress the gasket

Length of arc of segment,	S	=	$R\theta$	
		=	3.58	m
Gasket force assumed		=	45	kNm^{-1}
Factored gasket force	Fg	=	1.4 * 45	
		=	63	kNm^{-1}
Gasket force per segment		=	S*Fg	
		=	225.4436	KN

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: CircumferentialBolts_Design_823.xls Calculated by: John Poh Date: _____

Drawing No.: _____ Checked by: Wen Dazhi Date: _____

Bolts are provided are 3 no. of M24 grade 8.8,

Tensile stress area	A_t	=	353	
Tensile strength	p_t	=	450	
Tensile capacity per bolt	P_t	=	$450*353*10^{-3}$	
		=	158.85 kN	
Tensile capacity of 3 bolts		=	$3P_t$	
		=	476.55 kN	

Resolve bolt force at joint,

Vertical Force	Fv	=	Ptsinβ		
Horizontal force	Fh	=	Ptcosβ		
	Fv	=	306.3204 kN		
	Fh	=	365.06 kN	>	225.44 KN
					OK

Therefore bolts provide at the circumferential joint is capable of compressing the gaskets.

(ii)LC2-To check for worst case when TBM removed from incomplete ring of segment

Conservatively assume that segment supported by circumferential bolts,and ignore any support from adjacent radial joint bolts. This is a highly unlikely case. The design check considers the segment in the crown would be the most critical case.

Self weight of segment,	w	=	$(\theta/360)(\pi(D_E^2-D_I^2)/4)(W\gamma)$
		=	33.07 kN
		=	
Factored Self Weight	W	=	1.2*w = 39.68 kN
Factored Compressive force of gasket,	Fg	=	$1.2*45$ kNm^{-1}
		=	54 kNm^{-1}

(Load factor of 1.2 applied for this temporary load case.)

Considering 2 effective bolts per ordinary segment,

Compressive force of gasket per bolt,	F_C	=	S*F/3
		=	64.41 kN

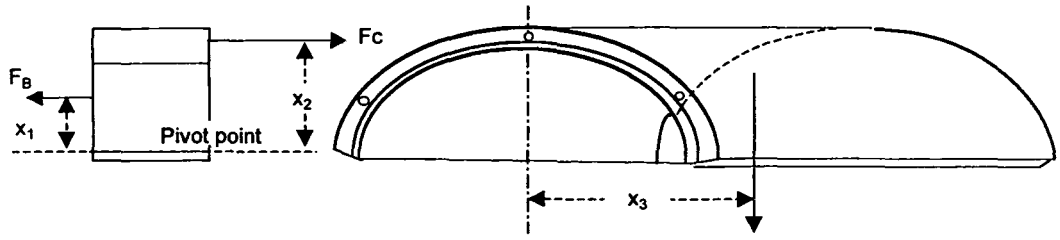
LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: CircumferentialBolts_Design_823.xls Calculated by: John Poh Date: _____

Drawing No.: _____ Checked by: Wen Dazhi Date: _____

Reference to sketch 'A' attached,

Distance from centroid of bolt to pivot point,	x_1	=	345	mm
Distance from centroid of gasket to packer force,	x_2	=	681	mm
Distance from centroid of segment to pivot point,	x_3	=	700	mm



Considering equilibrium about packer force, $F_B(x_1) = F_C(x_2) + (w/2)(x_3)$

Tensile Force per bolt $F_B = (F_C(x_2) + (w/2)(x_3))/x_1$

$= 160.6889$ kN

Shear force per bolt, $V = w/3$

$= 13.23$ kN

For combined shear and tension on 1 bolt, $\frac{F_S}{P_S} + \frac{F_t}{P_t} \leq 1.40$

(Cl.6.3.6.3 BS 5950:Part 1:1990)

Bolts are provided are M24 grade 8.8,

Effective shear area $A_s = 353$ mm^2

Shear strength $p_s = 375$ N/mm^2

(Ref. Steel Designers' Manual -Fifth Edition, pg. 1165 & 1170)

Shear Capacity per bolt $P_s = 375*353*10^{-3}$

$= 132.38$ kN

For a 24 mm bolt, assume grade 8.8 bolt, $\frac{F_S}{P_S} + \frac{F_t}{F_h} = 1.11 < 1.40$ **(OK)**

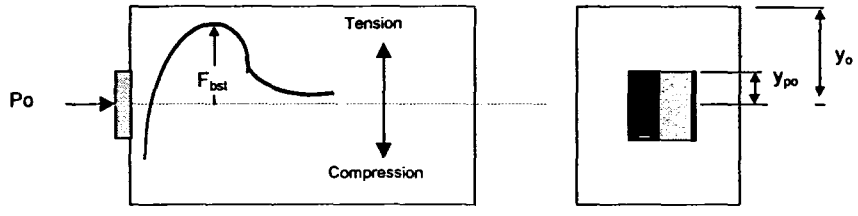
File No.: CircumferentialJoint_Design_TypeA&B_823.xls Calculated by: _____ Date: _____

Drawing No.: _____ Checked by: _____ Date: _____

Reinforcement Type A

Circumferential joints are checked for splitting force due to jacking force from the TBM during shoving.
Closed links are provided at the circumferential joints to resist this force.
Type A, C segments are checked.
Type C has lesser links due to provision of a fine mesh at the intrados.
All 2 types are checked that the links provided are sufficient to resist the splitting force at both ultimate and serviceability limits.

Checking Bursting (Adopt the End block design in CP65 Cl.4.11.2)
(Check at Serviceability)



Jacking force under normal operation is assumed to be = 1250 KN
(Contractor is required to check against their selected TBM specification accordingly)

No of jack per segment , n = 3

At ultimate, Po = 3 x 1.4 x 1250 KN (assuming 3 jack per segment)
= 5250 KN

y_{po} = 64 mm
y_o = 137.5 mm
y_{po}/y_o = 0.47

y_{po}/y_o	0.2	0.3	0.4	0.5	0.6	0.7
F_{bst}/P_o	0.23	0.23	0.2	0.17	0.14	0.11

Table 4.7 Design Bursting Tensile Strength In End Blocks

From Table 4.7,
F_{bst}/P_o = 0.23 (conservative)

Checking Of Joint Bursting
Fbst = 1208 KN (Bursting Force)

Min Area of links required
Asv = 3017.2 mm^2 (Stress of steel = 0.87*460N/mm^2)

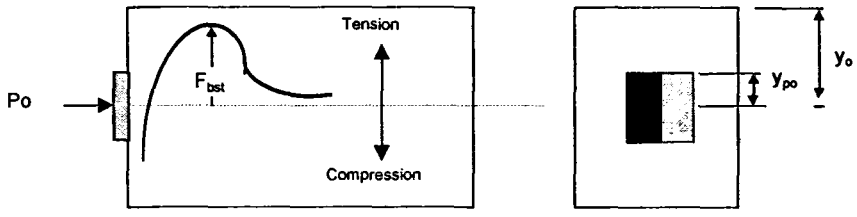
Provide **40 Legs T 10**
= 6284 mm^2

Links to be distributed from 0.2yo to 2yo
27.5 to 275 mm

File No.: CircumferentialJoint_Design_TypeA&B_823.xls Calculated by: _____ Date: _____

Drawing No.: _____ Checked by: _____ Date: _____

Checking Bursting (Adopt the End block design in CP65 Cl.4.11.2)
(Check at Serviceability)



Jacking force under normal operation is assumed to be = 1250 KN
(Contractor is required to check against their selected TBM specification accordingly)

No of jack per segment , n = 3

At ultimate, Po = 3 x 1250 KN (assuming 3 jack per segment)
= 3750 KN

y_{po} = 64 mm
y_o = 137.5 mm
y_{po}/y_o = 0.47

y_{po}/y_o	0.2	0.3	0.4	0.5	0.6	0.7
F_{bst}/P_o	0.23	0.23	0.2	0.17	0.14	0.11

Table 4.7 Design Bursting Tensile Strength In End Blocks

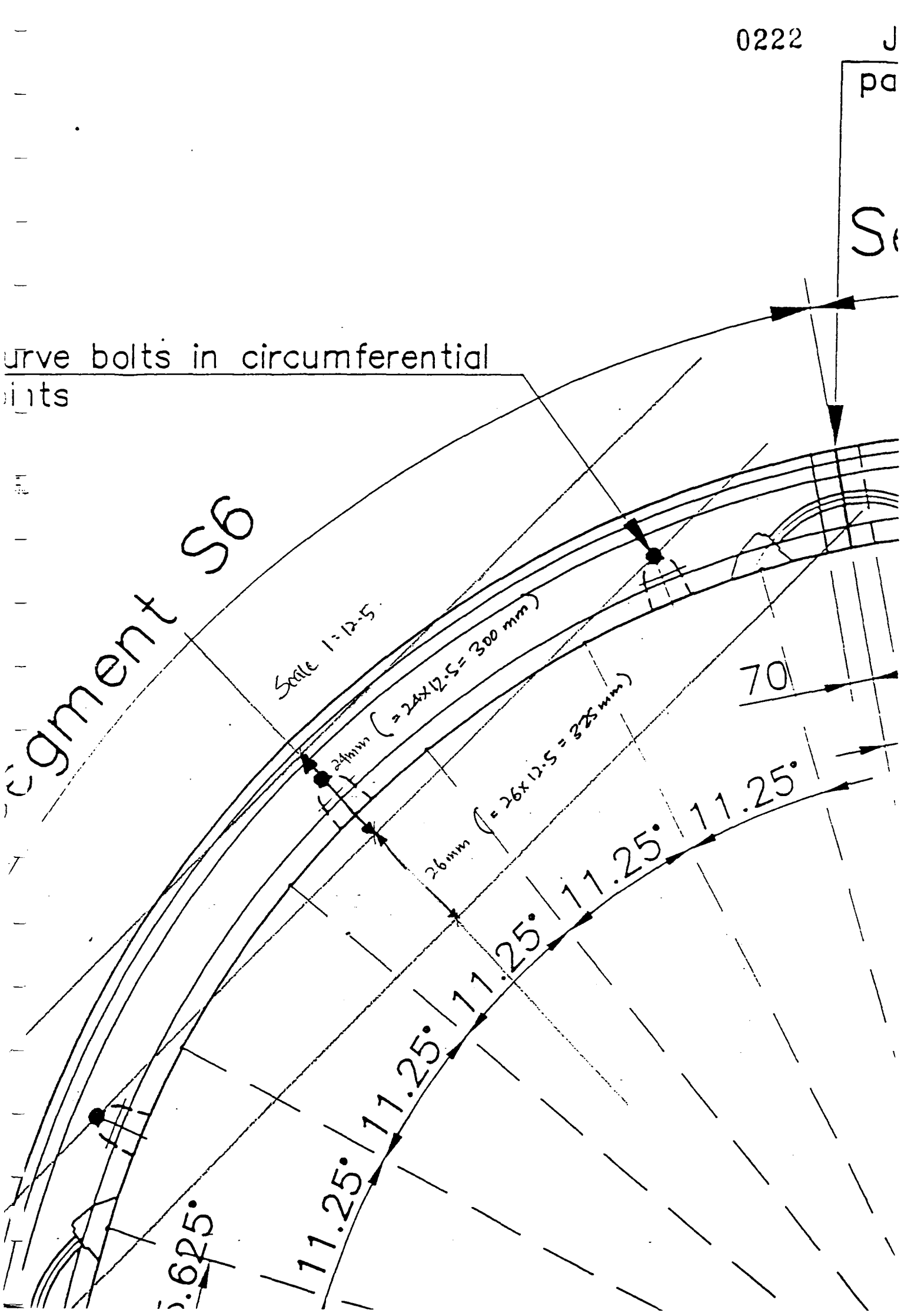
From Table 4.7,
F_{bst}/P_o = 0.23 (conservative)

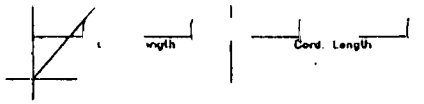
Checking Of Joint Bursting
Fbst = 863 KN (Bursting Force)

Min Area of links required
Asv = 4312.5 mm^2 (Stress of steel = 200N/mm2)

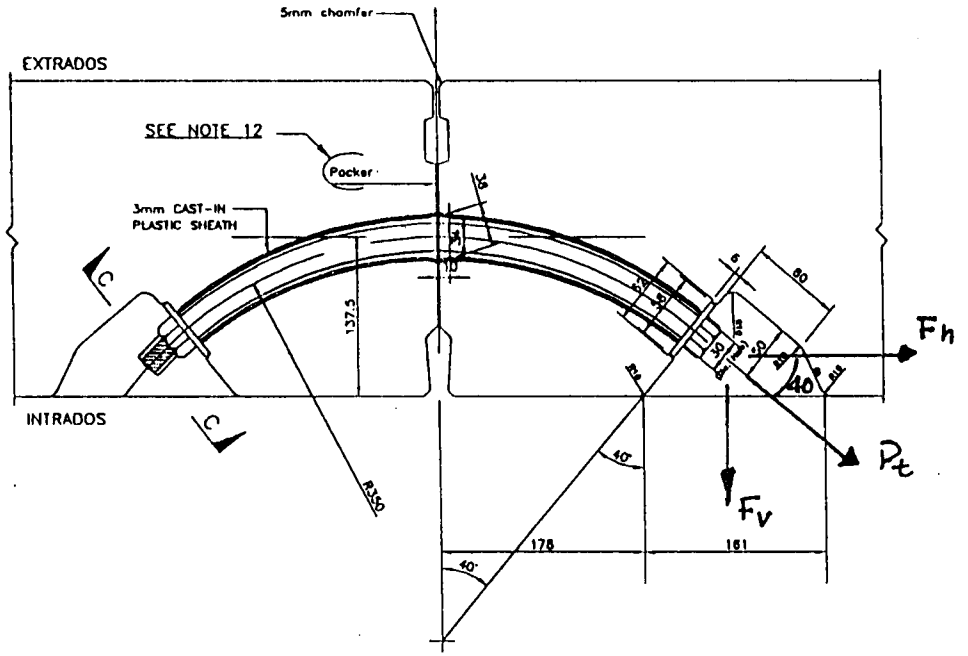
Provide **40 Legs T 10**
= 6284 mm^2

Links to be distributed from 0.2yo to 2yo
27.5 to 275 mm

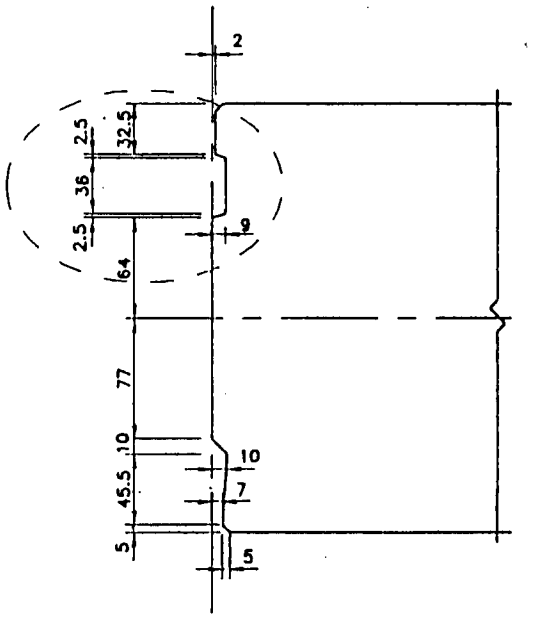

urve bolts in circumferential
ints
egment S6
Scale 1:12.5
24mm (=24x12.5=300 mm)
26mm (=26x12.5=325 mm)
70
11.25°
11.25°
11.25°
11.25°
11.25°
.625°



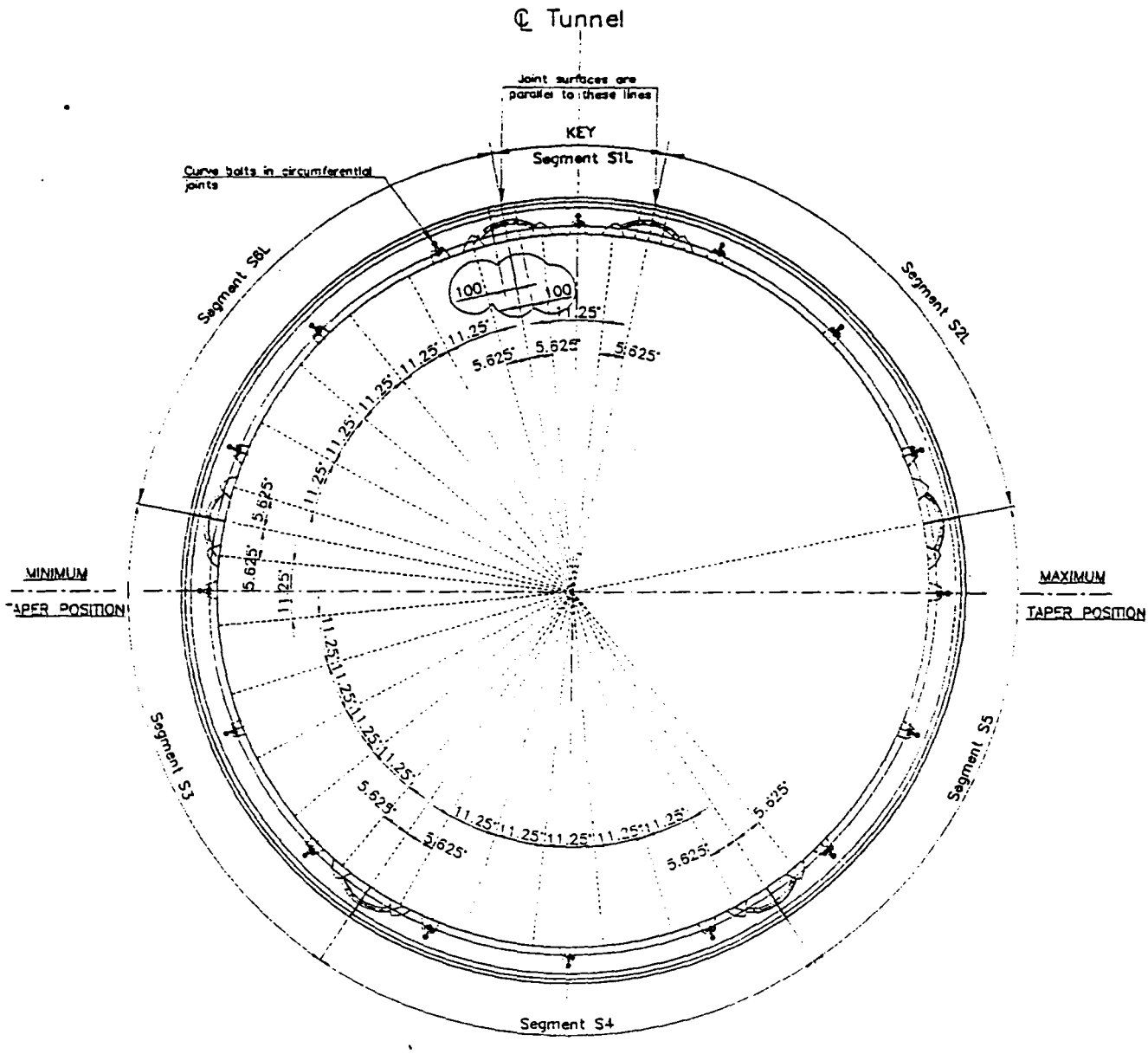
TYPICAL RADIAL JOINT DETAIL



TYPICAL CIRCUMFERENTIAL JOINT DETAIL



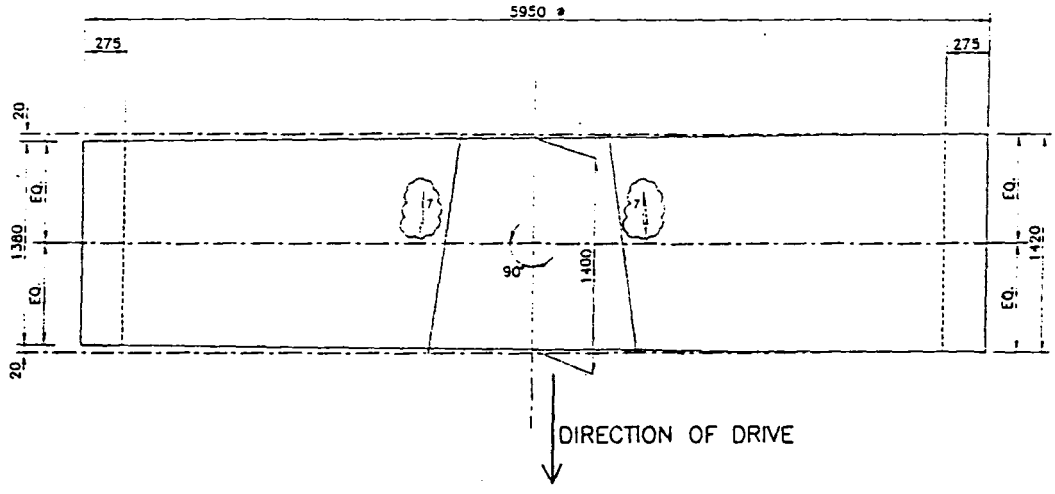
CIRCUMFERENTIAL JOINT



TYPICAL ELEVATION OF LEFT HAND TAPERED RING

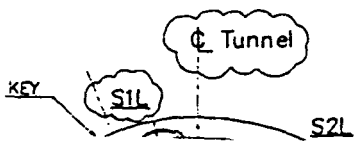
scale 1:25

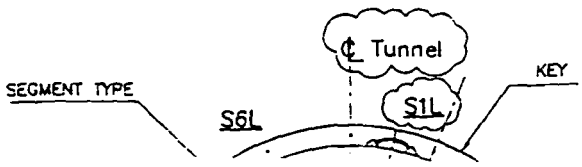
(Looking from Tunnel Boring Machine)



DIAGRAMMATIC PLAN VIEW OF LEFT HAND TAPERED RING

(RIGHT HAND TAPERED RING SIMILAR BUT HANDED)





0225

Hole sizes – for ordinary bolts and friction grip connections

Nominal diameter (mm)	Clearance hole diameter[b] (mm)	Oversize hole diameter[a] (mm)	Short slotted holes[a] (mm)		Long slotted holes[a] (mm)	
			Narrow dimension	Slot dimension	Narrow dimension	Maximum dimension
M12[a]	14	17	14	18	14	30
M16	18	21	18	22	18	40
M20	22	25	22	26	22	50
M22	24	27	24	28	24	55
M24	26	30	26	32	26	60
M27	30	35	30	37	30	67
M30	33	38	33	40	33	75

[a] Hardened washers to be used
[b] In cases where there are more than three plies in joint the holes in the inner plies should be one millimetre larger than those in the outer plies

Bolt strengths

	Bolt grade		Steel to BS 4360		
	4.6	8.8	43	50	55
Shear strength, p_s (N/mm^2)	160	375			
Bearing strength, p_{bb} (N/mm^2)	460	1035[a]	460	550	650
Tension strength, p_t (N/mm^2)	195	450			

[a] The bearing value of the connected part is critical

Steel Designers' Manual
(Fifth Edition)
The Steel Construction Institute

Bolt capacities in tension

Nominal diameter (mm)	Tensile stress area[b] (mm^2)	Bolts grade 4.6 @ 195 N/mm^2 (kN)	Bolts grade 8.8 @ 450 N/mm^2 (kN)
M12	84.3	16.43	37.93
M16	157	30.61	70.65
M20	245	47.77	110.25
M22[a]	303	59.08	136.35
M24	353	68.83	158.85
M27[a]	459	89.50	206.55
M30	561	109.39	252.45

[a] Non-preferred sizes
[b] Tensile stress areas are taken from BS 4190 and BS 3692

Spacing, end and edge distances – minimum values (see Fig. 23.1)

Nominal diameter of fastener (mm)	Diameter of clearance hole (mm)	Minimum spacing (mm)	Edge distance to rolled, sawn, planed, or machine flame cut edge (mm)	Edge distance to sheared edge or hand flame cut edge and end distance (mm)
M12	14	30	18	20
M16	18	40	23	26
M20	22	50	28	31
M22[a]	24	55	30	34
M24	26	60	33	37
M27[a]	30	68	38	42
M30	33	75	42	47

[a] Non-preferred size

Maximum centres of fasteners

Thickness of element (mm)	Spacing in the direction of stress (mm)	Spacing in any direction in corrosive environments (mm)
5	70	80
6	84	96
7	98	112
8	112	128
9	126	144
10	140	160
11	154	176
12	168	192
13	182	200
14	196	200
15	210	200

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

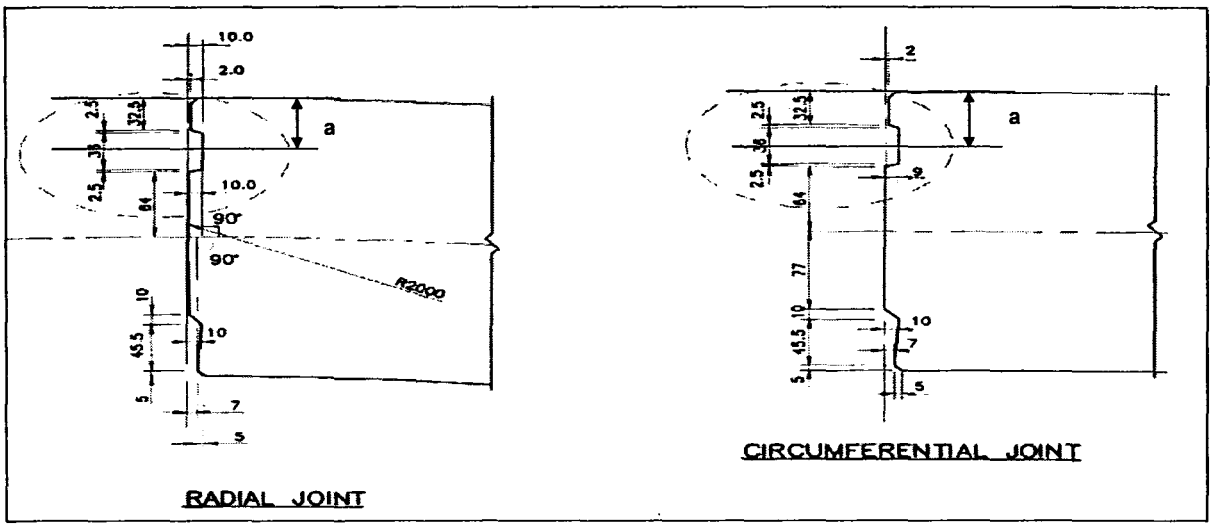
File No.: SpallingJoint_Design_823.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

Spalling of joints



Data

External diameter of tunnel,	D_E	=	6350	mm
Internal diameter of tunnel,	D_I	=	5800	mm
Norminal diameter of tunnel,	D	=	6075	mm
Nominal radius of tunnel,	R	=	3037.5	mm
Angle of ordinary segments,	θ	=	67.5	°
Angle of rear face bolt,	α	=	40	°
Specific gravity of concrete,	γ	=	24	kNm^{-3}
Depth to failure plane	a	=	32.5 + 2.5 + 36	
		=	71	mm
Maximum pressure to close gasket per m	F_g	=	45	kN/m
Characteristic strength of concrete for shear check	f_{cu}	=	40	N/mm^2

(Limit to 40N/mm2 in CP65 Part 1 1999, table 3.9)

File No.: SpallingJoint_Design_823.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

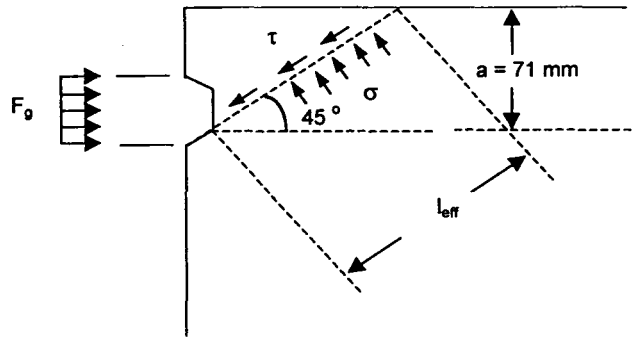
Partial safety factor for concrete, $\gamma_m = 1.25$

Partial factor for loading $\gamma_l = 1.4$

Failure angle assumed $\theta = 45\ ^\circ$

Concrete design shear strength $V_c = 0.45(fcu/30)^{1/3}$

(CP65:1999, Part 1,Table 3.9) $= 0.50\ N/mm^2$



$l_{eff} = a/\sin 45$

$= 100.4092$ mm

Resolve forces, $\Sigma F_v = 0$

$\tau \sin 45 = \sigma \cos 45$

$\tau = \sigma$

$\tau = \gamma_l \times Fg$

$\Sigma F_h = 0$

$\gamma_l \times F_g = (\tau \cos 45 + \sigma \sin 45) l_{eff}$

$= (\tau \cos 45 + \sigma \sin 45)(a/\sin 45)$

$= \tau(1/\tan 45 + 1)(a)$

$\tau = (\gamma l \times Fg)/[(1/\tan 45 + 1)(a)]$

$= 0.44\ N/mm^2$

$< Vc$, OK

File No.: Segment_Handling.xls

Calculated by: John Poh Date:_____

Drawing No.: _____

Checked by:Wen Dazhi Date:_____

Segment Handling

2 stages of segment handling will be checked. However, contractor need to carry out their own check to suit their own methods of handling and erection.

(i) Demoulding of segments
(ii) Stacking of segments in storage

(i) Demoulding

The demoulding of stacking of precast segments is analysed at SLS using elastic method to ensure extreme fibre stresses do not exceed the allowable tensile stresses

Data

External diameter of tunnel,	D_E	=	6350	mm
Internal diameter of tunnel,	D_I	=	5800	mm
Norminal diameter of tunnel,	D	=	6075	mm
Nominal radius of tunnel,	R	=	3037.5	mm
Width of segment,	B	=	1400	mm
Thickness of segment	T	=	275	mm
Angle of ordinary segments,	θ	=	67.5	°
Specific gravity of concrete,	γ	=	24	kNm^{-3}
Dynamic load factor	γ_{dyn}	=	2	
Self Weight load factor	γ_g	=	1.2	
Arc length of segment,	S	=	$R\theta$	
		=	3.58	m

Self weight of segment, $w = (\theta/360)(\pi(D_E^2-D_I^2)/4)(B*\gamma)$

$= 33.07$ kN

$=$

Factored Self Weight $W = \gamma_g*\gamma_{dyn}*w = 79.36$ kN

Assume a compressive stress can be attained at demoulding.

Compressive stress required $f_{cud} = 15$ N/mm^2

Allowable stressess

Characteristic compressive stress of concrete, $f_{cu} = 15$ N/mm^2

Design tensile strength, $f_{ct} = 0.36*(f_{cu})^{1/2}$ (CP65:Part 1:1999

$= 1.39$ N/mm^2 Table 4.1)

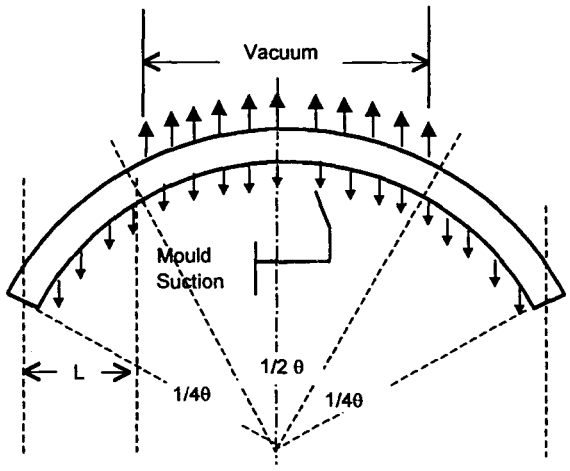
$z = B*T^2/6$

$= 17645833$ mm^3

Extreme fibre stress $\sigma = M/z$

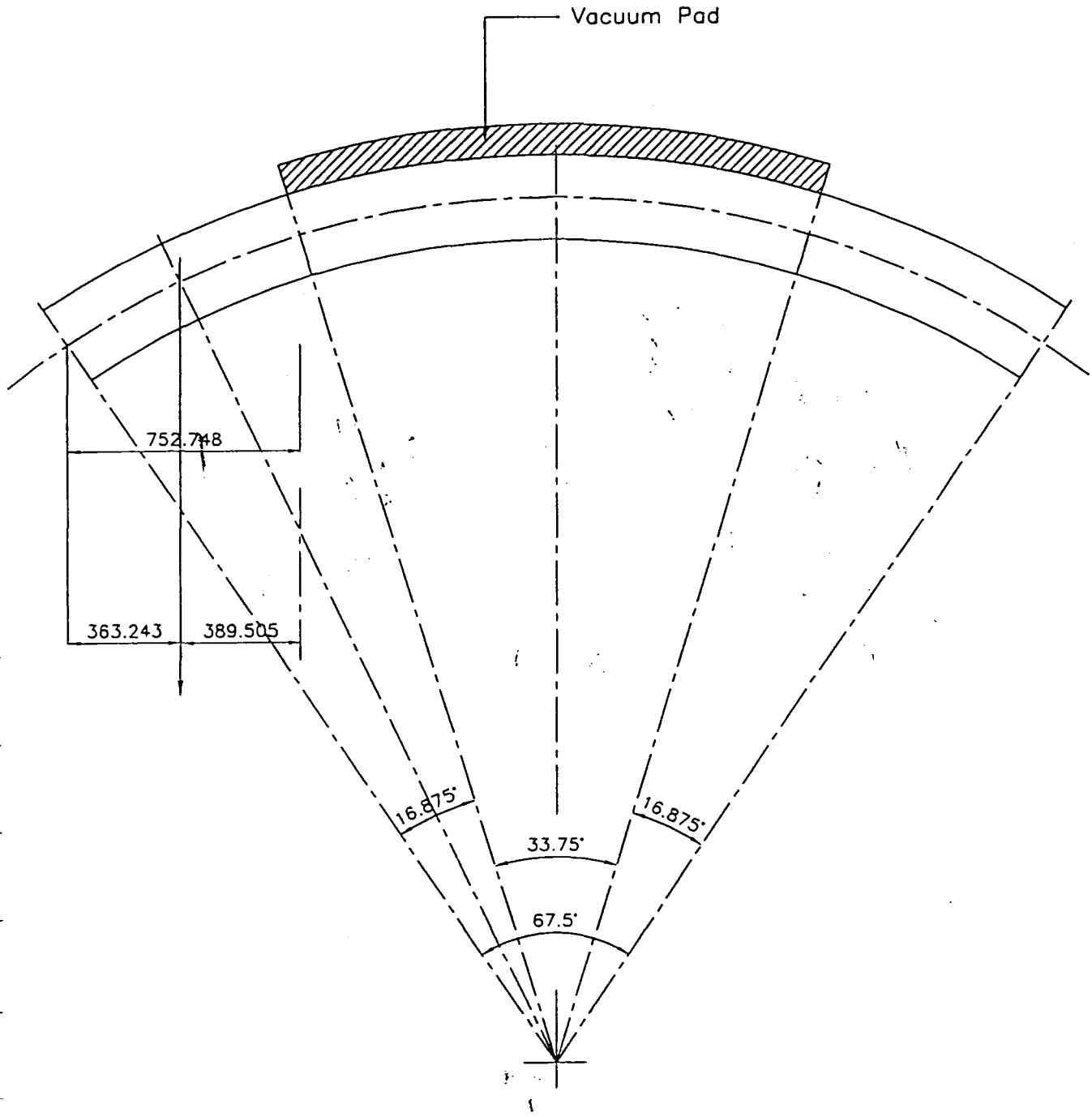
if $\sigma < f_{ct}$ ok.

Assume segments are demoulded by means of vacuum lifting device. Segment is supported within the vacuum area of the device. Suction at the bottom of the mould is also taken into accound in the calculation. The loading caused by the suction as a uniform load acting in radial direction. The resultant maximum suction load is assumed to be equal is considered to the self weight of the segment.



Area of intrados surface of segment	A_{int}	=	$\pi*D_I*B*\theta/360$	
		=	3.42	m2
Radial uniform suction load	F_{suc}	=	w/A_{int}	
		=	9.68	kN/m^2
Line load due to suction	q_{suc}	=	$F_{suc}*B$	
		=	13.55	kN/m
Bending moment at edge of vacuum area:				
Lever arm to edge of vacuum pad (see sketch 1).				
	L	=	805.81	mm
Due to suction	Msuc	=	$q_{suc}*R*(\theta/4)*(L/2)$	
		=	4.88	KNm
Distance of centroid of extending part of segment to edge of vacuum pad (see sketch 1)				
	L'	=	389.51	mm
Due to self weight	Mw	=	w/4*L'	
		=	7.73	KNm
Tensile stress due to both suction and self weight		=	(Msuc + Mw)/z	
		=	0.71	N/mm^2 **< fct**

ok, caculated tensile stress < allowable

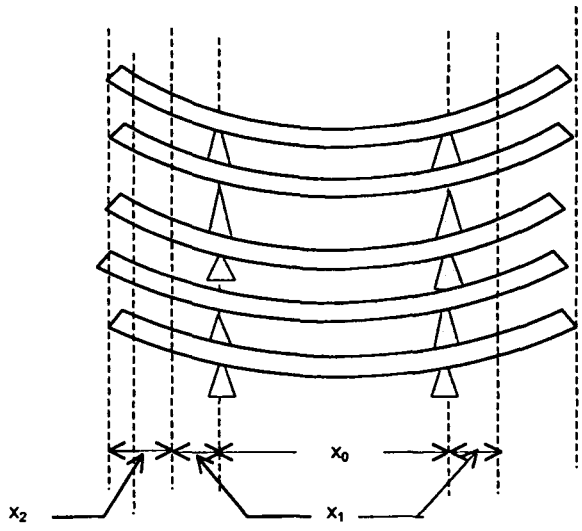


SKETCH 1 : DEMOULDING OF SEGMENT

(SCALE : 1 = 200)

(ii) Stacking

Most critical case is temporary stacking after demoulding with regard to the lower concrete strength.



Assume a compressive stress can be attained at stacking.

Compressive stress required,	fcud	=	15	N/mm^2	
Allowable stressess					
Characteristic compressive stress of concrete,	fcu	=	15	N/mm^2	
Design tensile strength,	fct	=	$0.36*(fcu)^{1/2}$		(CP65:Part 1:1999:
		=	1.39	N/mm^2	Table 4.1)
Assume horizontal span between supports,	x_0	=	1763.48	mm	(see sketch 2)
Horizontal projected length of segment,	L_{hor}	=	3375.10	mm	(see sketch 2)
Self weight per m run	w_{self}	=	9.80	kN/m	
Dynamic load factor	γ_{dyn}	=	2		
Self Weight load factor	γ_g	=	1.2		

File No.: Segment_Handling.xls

Calculated by: John Poh Date:_____

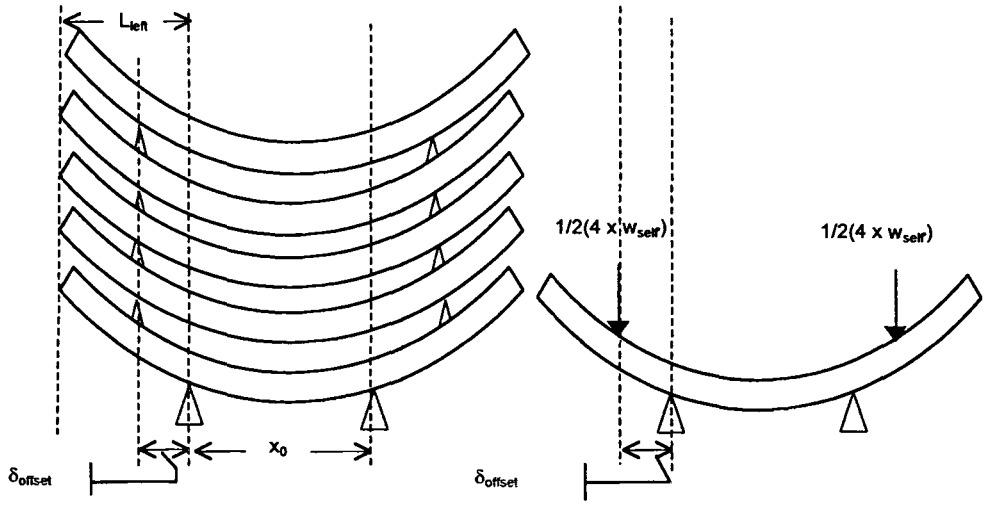
Drawing No.: _____

Checked by:Wen Dazhi Date:_____

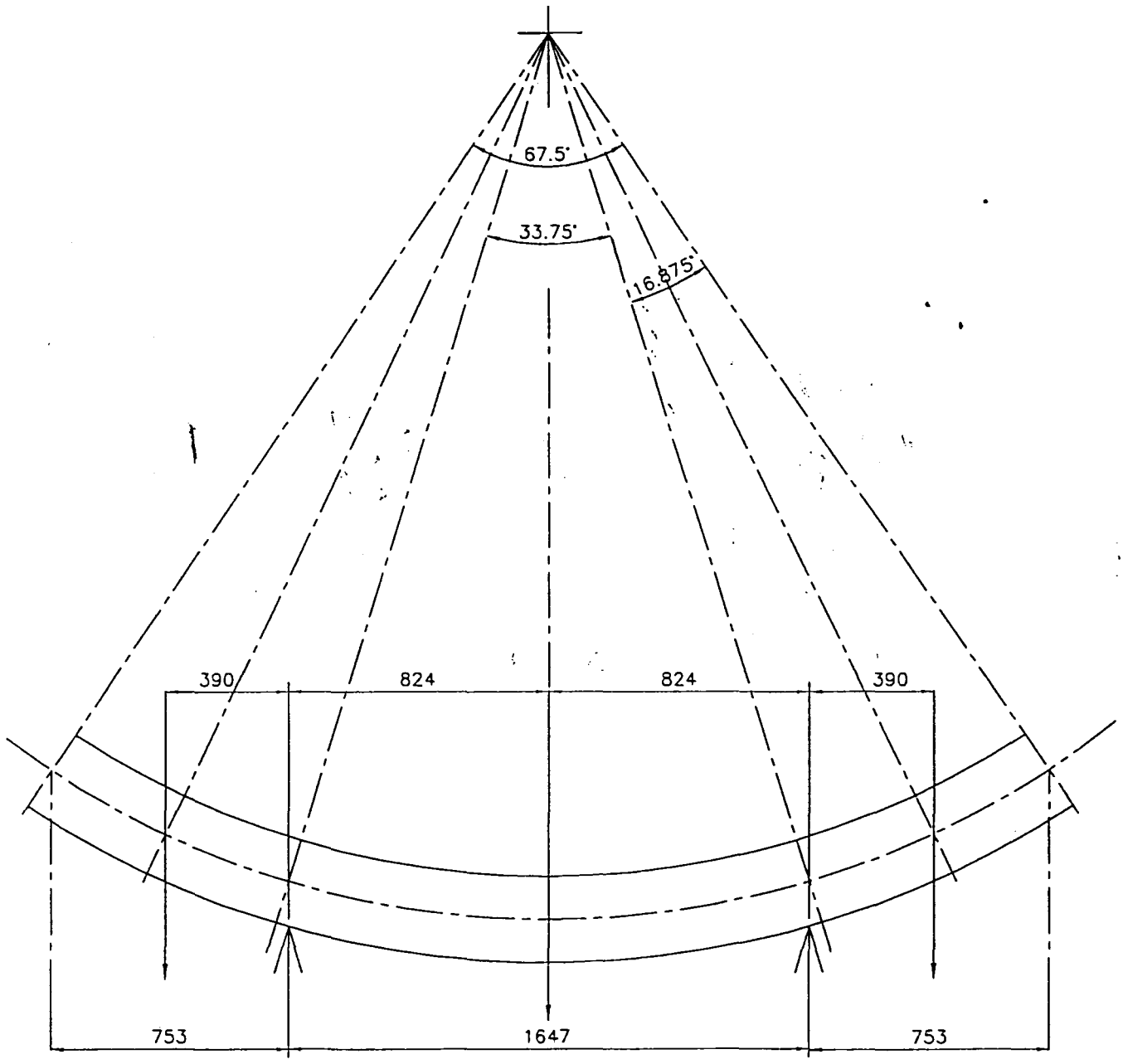
Factored self weight per m	W_{self}	=	23.51 kN/m
Length of edge to support	L_{left}	=	$1/2*(L_{hor}-x_0)$
		=	0.81 m
Lever arm of overhang part of segment to edge	L_{cg}	=	390 mm
Bending moment at support	$M_{support}$	=	$W_{self}*L_{left}*L_{left}/2$
		=	7.63 KNm (+ve hogging)
Bending moment at mid span	M_{mid}	=	$M_{support} - (W_{self})*(x_0^2/8)$
		=	-1.51 KNm (-ve sagging)
Max tensile stress	$\sigma_{support}$	=	$M_{support}/z$
		=	0.43 N/mm^2 **< fct**

Consider multiple stacking with lateral offset of supports.

Conside storage of 5 segments in one stack. Additional moment is due to offset of supports.



Assumed offset	δ_{offset}	=	50 mm
Bending moment at support	$M_{support}$	=	$W_{self}*(L_{left} - \delta_{offset})^2/2 + 1/2[\{W_{self}(L_{left}-\delta_{offset})+1/2(4*W_{self})\}$ $+\{W_{self}*(L_{left}-\delta_{offset})+1/2(4*W_{self})+ W_{self}*\delta_{offset}\}]*\delta_{offset}$
		=	9.98 KNm (+ve hogging)
Bending moment at mid span	M_{mid}	=	$M_{support} - (W_{self})*(x_0^2/8)$
		=	0.84 KNm (-ve sagging)
Max tensile stress	$\sigma_{support}$	=	$M_{support}/z$
		=	0.57 N/mm^2 **< fct**



SKETCH 2 : STACKING OF SEGMENT

(SCALE : 1 = 200)

LAND TRANSPORT AUTHORITY
CCL2 Project
Design Sheet

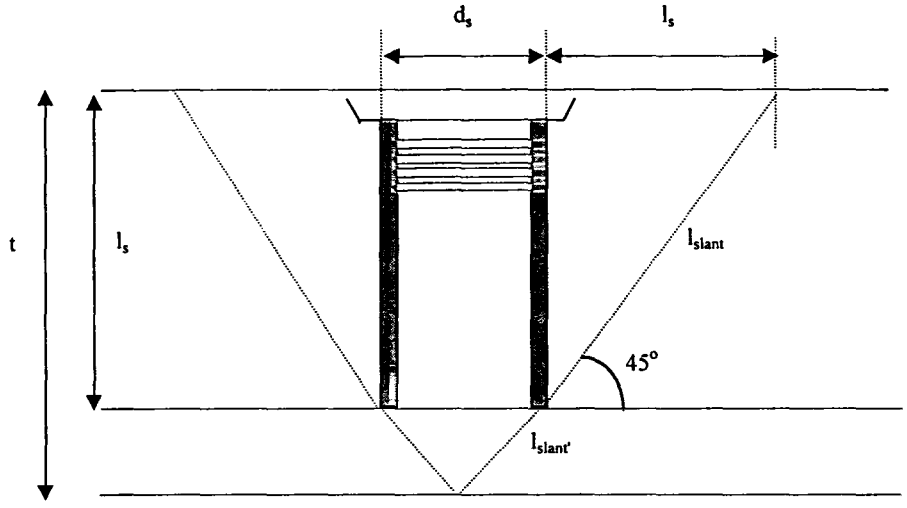
File No.: Segment_Handling.xls

Calculated by: John Poh Date:

Drawing No.:

Checked by: Wen Dazhi Date:

Typical Grout/Lifting Socket



Note : The contractor will need to carry out design check based on their grout lifting socket

Data

External diameter of tunnel,	D_E	=	6350	mm
Internal diameter of tunnel,	D_I	=	5800	mm
Norminal diameter of tunnel,	D	=	6075	mm
Nominal radius of tunnel,	R	=	3037.5	mm
Width of segment,	B	=	1400	mm
Thickness of segment	t	=	275	mm
Angle of ordinary segments,	θ	=	67.5	°
Specific gravity of concrete,	γ	=	24	kNm^{-3}
Length of socket	l_s	=	185	mm
Diameter of socket	d_s	=	70	mm
Partial safety factor, material	γ_m	=	1.5	
Partial safety factor, loads	γ_l	=	1.4	
Dynamic load factor	γ_{dyn}	=	2	
Concrete charactoeristic strength	f_{cu}	=	60	N/mm^2
	l_{slant}	=	261.63	mm
	$l_{slant'}$	=	127.28	mm

Weight of segment $w = (\theta/360)(\pi(D_E^2-D_I^2)/4)(B*\gamma)$

33.07 kN

Load on socket $W = \gamma_g*\gamma_{dyn}*w$

$= 92.58$ kN

LAND TRANSPORT AUTHORITY
CCL2 Project
Design Sheet

File No.: Segment_Handling.xls

Calculated by: John Poh Date:_____

Drawing No.: _____

Checked by: Wen Dazhi Date:_____

a Check bonding

Reference clause 3.12.8.4 and table 3.28 CP65:Part 1: 1999

Design ultimate anchorage bond stress f_{bu} = $\beta(f_{cu})^{0.5}$
= $0.4*(60)^{0.5}$ N/mm^2
= 3.10 N/mm^2

Bond capacity F_s = $\pi*d_s*l_s*f_{bu}$
= 126.05 OK,>W

b Check for concrete rupture

Area of failure plane A = $(\pi*(l_s + d_s/2)*l_{slant}) - (\pi*d_s/2*l_{slant})$
= 166830.25 mm

Allowable tensile stress for concrete f_t = $0.36*(f_{cu})^{0.5}$
= 2.79 N/mm^2

Factor of safety for concrete failure FOS = 1.5

Allowable design load = $A*f_t/FOS$
= 310.14 kN
OK, >W

c Check shear

Shear area A = $(\pi*(l_s + d_s/2)*l_{slant}) - (\pi*d_s/2*l_{slant})$
= 166830.25 mm

From table 3.9 of CP65: Part 1 : 1999, shear capacity for 275mm thick section

v_c = $0.84(100As/(b_v d))^{1/3}(400/d)^{1/4}/\gamma m$ x $(40/30)^{1/3}$
= 0.43

Design shear stress along failure cone v = W/bd
= 0.34
OK, < vc

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: Grout_Pressure_Checking.xls

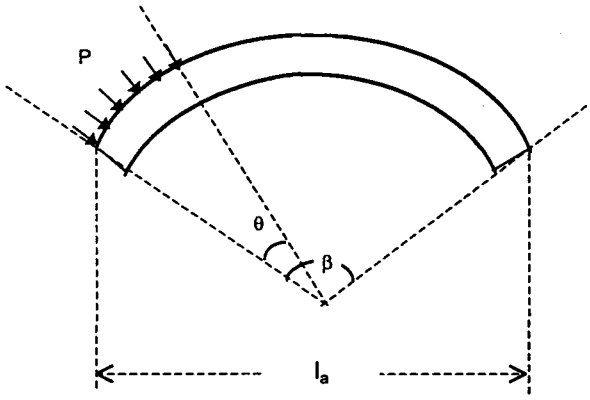
Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

CHECKING OF GROUT PRESSURE EFFECT ON SEGMENT LINING

Check is done on a scenario where only part of the segment is subjected to grout pressure due to uneven distribution of grout at the back of the segment. Situation occur before ground loads exerted on the segment and thus no hoop thrust induced due to ground loading. A pressure differential of 5 bar has been assumed



Data

External diameter of tunnel,	D_E	=	6350	mm
Internal diameter of tunnel,	D_I	=	5800	mm
Norminal diameter of tunnel,	D	=	6075	mm
Nominal radius of tunnel,	R	=	3037.5	mm
Width of segment,	b	=	1400	mm
Segment thickness,	t	=	275	mm
Angle of ordinary segments,	β	=	67.5	°
Specific gravity of concrete,	γ	=	24	kNm^{-3}
Grout pressure applied	P	=	5	bar
		=	0.5	MPa
Grade of concrete	fcu	=	60	MPa
Partial factor of safety, load	γ_L	=	1.2	
Partial factor of safety, Concrete	γ_{conc}	=	1.25	(for shear only)
Partial Factor of safety, Steel	γ_{steel}	=	1.15	
Assumed length of segment subject to grout pressure	l_g	=	1000.00	mm
Arc length subtended by 1 segment	l_a	=	(β/360) x pi x D_E	
		=	3741.96	mm
	θ	=	l_g/l_a x (β)	
		=	18.04	°

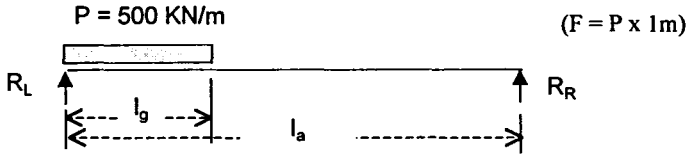
File No.: Grout_Pressure_Checking.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

Simplifying into a beam model,

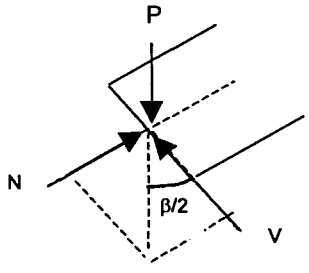


R_L = $0.5x\gamma_L x\ (F)(1 - 2.504/3.504)$

R_R = $0.5x\gamma_L x\ (F)(1 + 2.504/3.504)$

Therefore R_L = 519.83 KN

R_R = 80.17 KN



Resolve forces ,
Left support,

Hoop force	N_L	$= R_L\sin(\beta/2)$	kN
		= 288.90	kN
Shear force	V_L	$= R_L\cos(\beta/2)$	kN
		= 432.15	kN

Right Support,

Hoop force	N_R	$= R_R\sin(\beta/2)$	kN
		= 44.56	kN
Shear force	V_R	$= R_R\cos(\beta/2)$	kN
		= 66.65	kN

Effective depth	d	= 275 - 40 - 10 - 16/2	
		217	mm
Design shear stress	v	$= V_L/bd$	
		= 1.99	N/mm2

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: Grout_Pressure_Checking.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

Main tension reinforcement area per segment (Type A - lighter segment)
As = 4 T16 + 4 T13
= 1335.71 mm2

As per m width
= (1000/1400) x As
= 954.08 mm2

From table 3.9, SS65:Part 1:1999,

Design Conc Shear capacity, vc = $0.84(100As/(b_v d))^{1/3}(400/d)^{1/4}/\gamma m$ x $(60/30)^{1/3}$
= 0.75 N/mm^2

Considering Cl.3.4.5.12, SS65:Part 1:1999

$v'_c = vc + 0.6\ NVh/A_cM$
$= vc + 0.6\ N/A_c(1)$
$= 0.70+0.6\ (N_L)/(1000x275)$
= 1.38

Asv/Sv = (v-v'_c)x1000/0.87(460)
= 1.53

Near the joints, largest link spacing (Type A) = 190.00 mm

$(Asv/Sv)_{prov}$ = (no.of legs per m x link cross-sect area/spacing)
= 2.48 > Asv/Sv OK

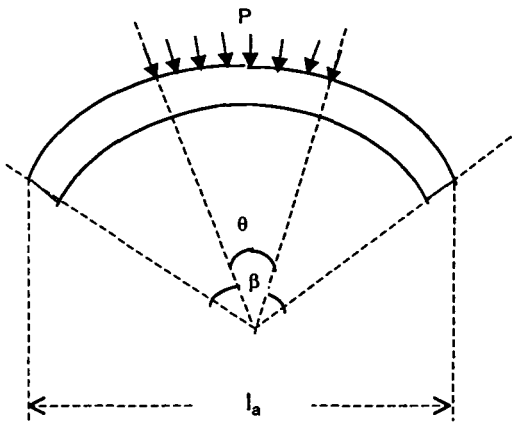
LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: Grout_Pressure_Checking.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____



Data

External diameter of tunnel,	D_E	=	6350	mm
Internal diameter of tunnel,	D_I	=	5800	mm
Norminal diameter of tunnel,	D	=	6075	mm
Nominal radius of tunnel,	R	=	3037.5	mm
Width of segment,	b	=	1400	mm
Segment thickness,	t	=	275	mm
Angle of ordinary segments,	β	=	67.5	°
Specific gravity of concrete,	γ	=	24	kNm^{-3}
Grout pressure applied	P	=	5	bar
		=	0.5	MPa
Grade of concrete	fcu	=	60	MPa
Partial factor of safety, load	γ_L	=	1.2	
Partial factor of safety, Concrete	γ_{conc}	=	1.25	(for shear only)
Partial Factor of safety, Steel	γ_{steel}	=	1.15	
Assumed length of segment subject to grout pressure	l_g	=	1000.00	mm
Arc length subtended by 1 segment	l_a	=	(β/360) x pi x D_E	
		=	3741.96	mm
	θ	=	l_g/l_a x (β)	
		=	18.04	°

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

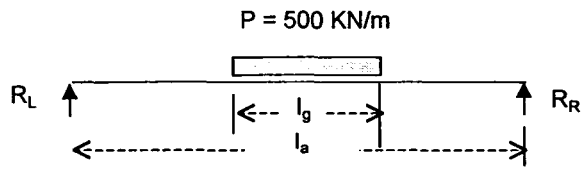
File No.: Grout_Pressure_Checking.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

Simplifying into a beam model,

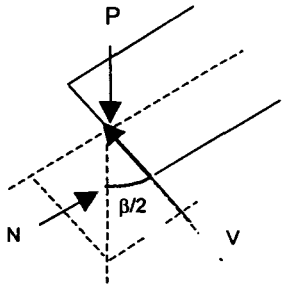


R_L = $0.5 \times \gamma_L \times (F)$

R_R = $0.5 \times \gamma_L \times (F)$

Therefore R_L = 300 KN

R_R = 300 KN



Resolve forces ,
Right support,

Hoop force	N_R	$= R_L \sin(\beta/2)$	kN
		= 166.73	kN
Shear force	V_R	$= R_L \cos(\beta/2)$	kN
		= 249.40	kN

Left Support,

Hoop force	N_L	$= R_L \sin(\beta/2)$	kN
		= 166.73	kN
Shear force	V_L	$= R_L \cos(\beta/2)$	kN
		= 249.40	kN

Effective depth	d	= 275 - 40 - 10 - 16/2	
		217	mm
Design shear stress	v	= V_L/bd	
		= 1.15	N/mm2

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: Grout_Pressure_Checking.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

Main tension reinforcement area per segment (Type A - lighter segment)

As = 4 T16 + 4 T13
= 1335.71 mm2

As per m width

= (1000/1400) x As
= 954.08 mm2

From table 3.9, SS CP65:Part 1:1999,

Design Conc Shear capacity, vc = 0.84(100As/(bvd))1/3(400/d)1/4/gm x (40/25)1/3
= 0.70 N/mm^2

Considering Cl.3.4.5.12, SS CP65:Part 1:1999

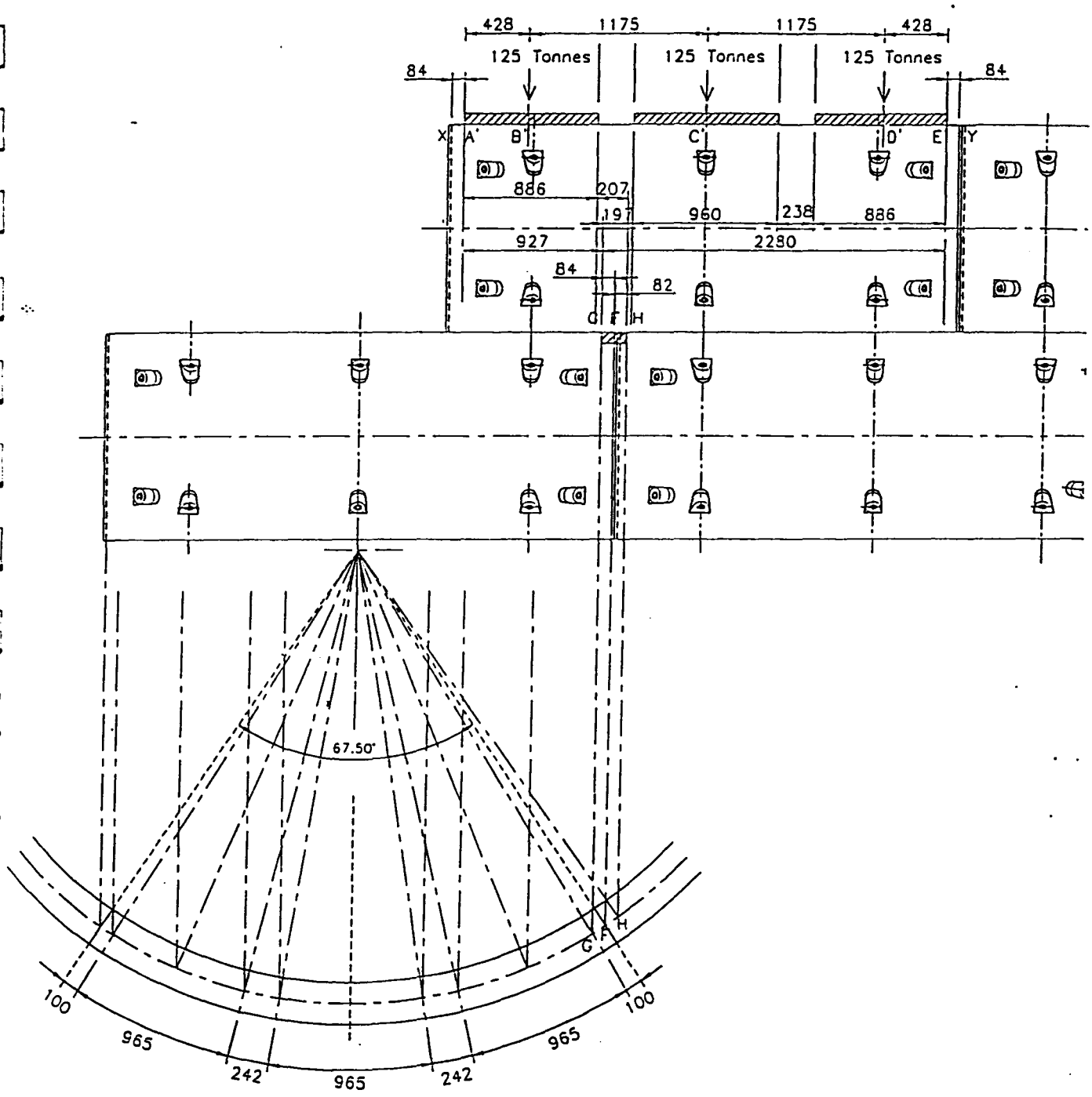
$v'_c = vc + 0.6\ NVh/A_cM$
$= vc + 0.6\ N/A_c(1)$
$= 0.7+0.6\ (N_L)/(1000x275)$
= 1.06

Asv/Sv = (v-v'c)x1000/0.87(460)
= 0.22

Link spacing at body of segment (Type A) = 150.00 mm

$(Asv/Sv)_{prov}$ = (no.of legs per m x link cross-sect area/spacing)
= 3.14 > Asv/Sv OK

0242A

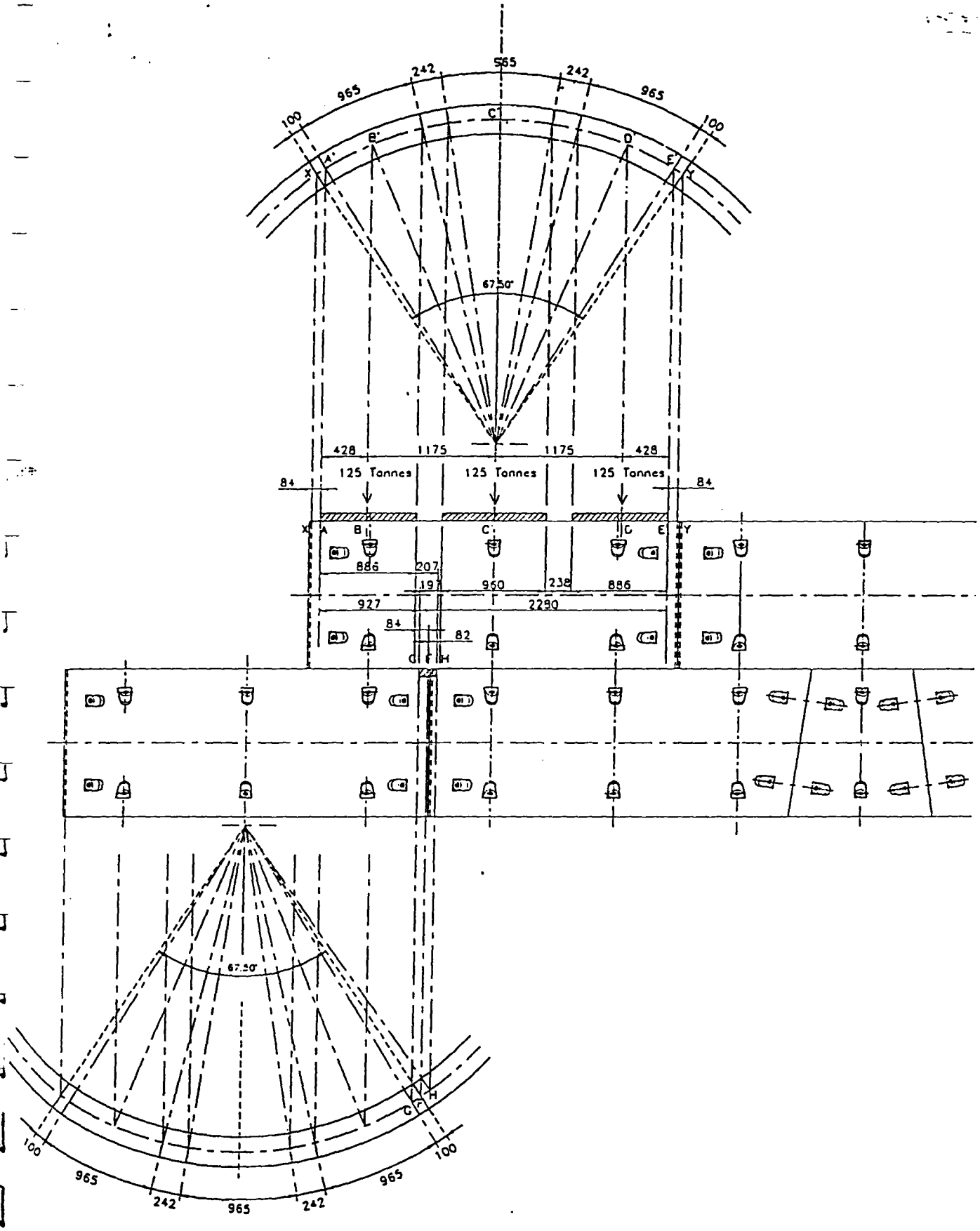


250b

SKETCH 'B'

SCALE: 1 = 40

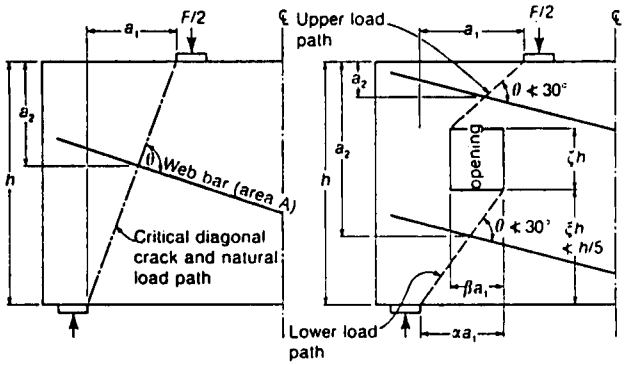
0242B



250a

SKETCH 'A'

1. Calculate ...ate b...g mon... ... acti... ... beam.
2. Calculate area of main reinforcement required from formula A.
3. Calculate ultimate shearing force V acting on beam.
4. Calculate suitable minimum breadth of beam (or check, if breadth is specified) from formula B.
5. ...steel.
6. Calculate shearing resistance V_1 for beam with main reinforcement only from formula C: thus determine shearing resistance $(V - V_1)$ to be provided by web reinforcement.
7. From sketch of beam, measure values of θ and a_2 for each individual web bar.
8. Calculate area of web bars required from formula D.



Design formula	Without openings in beam	With openings in beam
A	$A_{s\,req} \not< \frac{1.9M}{f_y l}$ or $\frac{1.9M}{f_y h}$	$A_{s\,req} \not< \frac{1.9M}{f_y l}$ or $\frac{1.55M}{f_y \xi h}$
B	$b \not< \frac{0.65V}{k_1(h - 0.35a_1) f_t}$	$b \not< \frac{0.55V}{k_1(\xi h - 0.35\alpha a_1) f_t}$
C	$V_1 = k_1(h - 0.35a_1) f_t b + k_2 A_{s\,prov} d \sin^2\theta / h$	$V_1 = k_1(\xi h - 0.35\alpha a_1) f_t b + k_2 A_{s\,prov} d \sin^2\theta / h$
D	$V - V_1 = k_2 \sum A a_2 \sin^2\theta / h$	$V - V_1 = 1.5 k_2 \sum A a_2 \sin^2\theta / h$

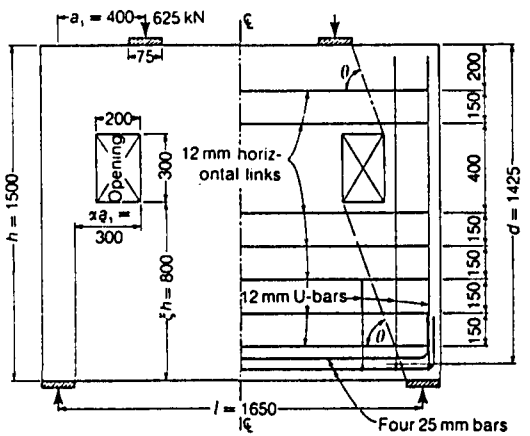
Notes

1. The formulae are only known to be applicable if the following conditions apply: $l/h \geq 2$. Static loads only occur and these are applied to top of beam only. a_1/h is not greatly outside range of 0.23 to 0.70. Positive anchorage is provided to main reinforcement.
2. Restrictions to θ and ξh shown in diagrams only apply when opening intersects line of critical diagonal crack. If opening is reasonably clear of this line, the effect of the opening may be disregarded completely when considering shearing resistence.
3. For distributed loads, substitute statically equivalent twin concentrated loads (i.e. replace uniform load F by two concentrated loads of $F/2$ at distances of $l/4$ from supports.
4. The more nearly perpendicular a web bar is to the principal diagonal crack, the more effective it is in resisting shearing and limiting cracking: its effectiveness also increases with increasing depth a_2. However, inclined web reinforcement may be more expensive to bend and fix.
5. If openings are present, web reinforcement must pass both above and below them.

If cylinder splitting tensile strength is not known, estimate as follows:

cube strength f_{cu} (N/mm²)	cylinder splitting tensile strength f_t (N/mm²)
20	2.24
25	2.50
30	2.74
40	3.16
50	3.54

area ... lividu... bar	
$A_{s\,req}, A_{s\,prov}$	minimum area of main steel required and actual area provided
a_1	clear distance from edge of load to face of support
αa_1	distance from inner edge of opening to face of support
βa_1	width of opening
a_2	depth at which web bar intersects critical diagonal crack
b	breadth of beam
d	effective depth to main steel
f_t	cylinder splitting tensile strength of concrete (see table on left below)
f_y	yield strength of reinforcement
h	overall depth of beam
k_1, k_2	empirical coefficients for concrete and reinforcement. Take k_1 as 0.7 for normal-weight concrete and 0.5 for light-weight concrete: take k_2 as 100 for plain round bars and 225 for deformed bars
l	span of beam between centres of supports
M	ultimate moment
V	ultimate shearing force
V_1	shearing force resisted by concrete and main reinforcement only
θ	angle between bar being considered and critical diagonal crack
ξ	distance of bottom of opening from beam soffit expressed as proportion of total depth of beam
ζ	depth of opening expressed as proportion of total depth of beam



Reinforced Concrete Designer, Handbook
Tenth edition
Dunn & Reynolds
James C Steelman

02420

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: LateralBending_Checking.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

Lateral Bending Of Segments During Shoving

Geometry

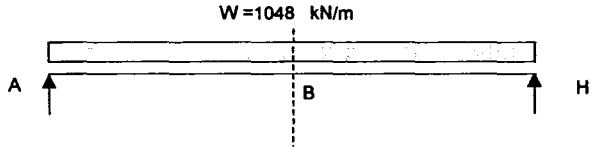
External diameter of tunnel,	D_E	=	6350	mm
Internal diameter of tunnel,	D_I	=	5800	mm
Norminal diameter of tunnel,	D	=	6075	mm
Norminal radius of tunnel,	R	=	3037.5	mm
Angle of key segment,	ϕ	=	22.5	°
Angle of ordinary segment,	θ	=	67.5	°
Width of segment,	W	=	1400	mm
Length of ordinary segment,	L'	=	$(\theta/360)\pi D$	
		=	3578.47	mm
Length of chamfer,	L_C	=	100	mm
Length of each packer,	L_P	=	965	mm
Length of gap in between each packer,	L_G	=	$(L - 2L_C - 3L_P)/2$	
		=	241.7	mm

Pls refer to sketch "A" and "B" attached.

Assume 16 rams per ring of segment, ram force evenly distributed along the circumference via the spreader and with the use of packers to cushion the load.

Assume ram force per jack	F_{ram}	=	1250	←	(contractor to confirm and re-check
Total no. of ram per ring	N_{ram}	=	16	←	if necessary)
Total jacking force per ring	F_{tot}	=	$N_{ram} \times F_{ram}$	kN	
			20000	kN	
Distributed load intensity	W	=	Ftot/(π x D)		
		=	1047.93	kN/m	

Simply Supported Case over approximately 1/3 of segment.



Consider the case where a single ram force is exerted between point A and H (refer sketch A & B) due to construction inaccuracy or surface unevenness.

Consider span AH,	L_{AH}	=	886 +207	mm
			1093.00	
Design distributed ram load	W	=	1048	kN/m

Assume simply supported between point A and H,

Max moment	M_{max}	=	$WL^2/8$	
		=	156.49	KNm

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: LateralBending_Checking.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

Only the lighter weight segment, **Type A** is considered (conservative).

(i) Ultimate Limit State Check

Segment is checked that reinforcement provided is able to resist the lateral bending effect due to uneven support or construction inaccuracy.

Data

Concrete strength	f_{cu}	=	60	Nmm^{-2}	
Yield strength of steel	f_y	=	460	Nmm^{-2}	
Total Tensile Reinforcement Area provided	As, prov	=	1070	mm2	(Consider only 4 T16 Edge bar and 2 T13)
Average cover to A_s		=	91	mm	
Depth of section	h	=	1400	mm	
Average Effective depth	d_{avg}	=	1309.00	mm	
Design moment	M	=	156.49	KNm	
Load factor for temporary load case	γ_L	=	1.20		
Factored Design moment	M_f	=	187.79	KNm	
Breath of section	b_v	=	275.00	mm	
Span between support	L_{AH}	=	1093.00	mm	
Overall depth of section	h	=	1400.00	mm	
Ratio, l/h		=	0.78	< 2	Consider as deep beam
Active height	h_a	=	L_{AH}	since h > l	
Lever Arm	z	=	$0.2l + 0.4h_a$		
		=	655.80	mm	

a. Check bending

As required	As, req	=	$M_f/0.87f_yz$		
		=	715.51	mm^2	**ok Asreq < Asprov**

File No.: LateralBending_Checking.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

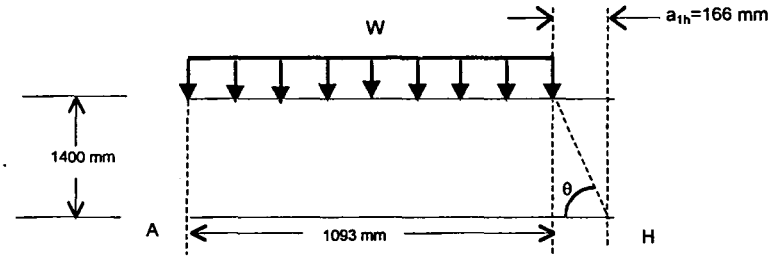
b. Check shear at support

Ultimate Shear strength $V = K_1(h-0.35a_1)f_tb + K_2(Asprov*d*\sin^2\theta/h)$

(Reference: Reynolds and Steedman's reinforced concrete designer's handbook, table 148)

Refer to attached table 148 for definition of terms in formula

	K_1	= 1	
	K_2	= 225	
	a_{1H}	= 166	mm
	a_{1A}	= 0	mm
	b	= 275	mm
	d	= 1309	mm
Estimated cylinder splitting tensile strengh	f_t	$= 0.5(f_{cu})^{1/2}$	
		= 4	N/mm^2



Total Ram force	P_B	=	$W*L_{AH}$	
			1145.4 kN	
Support reaction at A,	R_A	=	0.5(1+166/1093)1145	
		=	659.67 kN	
Support reaction at H,	R_H	=	485.72 kN	

At support A,

	θ	= 1.57	rad	
	sinθ	= 1.00		
	As_{prov}	= 1070.00	mm^2	(Consider only 4 T16 Edge bar 2T13 bars)
Ultimate Shear strength	V	= 1225.55	KN	
Design Shear Force	R_A	= 659.67	KN	**ok design shear force < V**

At support H,

	θ	= 1.45	rad	
	sinθ	= 0.99		
	As_{prov}	= 1070.00	mm^2	(Consider only 4 T16 Edge bar
Ultimate Shear strength	V	= 1222.43	KN	
Design Shear Force	R_A	= 485.72	KN	**ok design shear force < V**

LAND TRANSPORT AUTHORITY
CCL2 PROJECT
Design Sheet

File No.: LateralBending_Checking.xls

Calculated by: John Poh Date: _____

Drawing No.: _____

Checked by: Wen Dazhi Date: _____

(ii) Serviceablity Limit Check

Section is first checked to determine whether the concrete tensile strength is exceeded. If allowable stress is exceeded, design proceed to check that the magnitude of the crackwidth is less than according to SS65.

Allowable stressess

Characteristic compressive stress of concrete,	fcu	=	60	N/mm^2	
Design tensile strength,	fct	=	$0.36*(fcu)^{1/2}$		(SS65:Part 1:1999
		=	2.79	N/mm^2	: Table 4.1)
Thickness	t	=	275.00	mm	
Width	B	=	1400.00	mm	
	z	=	$B*T^2/6$		
		=	8.98E+07	mm^3	
Extreme fibre stress	σ	=	**M/z**		
Check	σ	<	2.79	ok.	
Allowable crackwidth	ω	=	0.30	mm	

Load Cases	Moment (KNm)	Extreme fibre stress (N/mm2)	Check
Simply Supported Case 1	156.49	1.74	ok

Allowable concrete tensile stress is not exceeded.

Serviceability check is satisfactory, segment not expected to crack under this loadcase. However, contractor is to perform own check if ram force exceed the assumed values in the calculation.
This check is done just to confirm segment is able to withstand certain amount of uneven support during erection. However, it is essential that dimensional tolerance of segment be ensure and contractor to take all precautions to avoid such load cases from happening. Use of packers in the circumferential joints will help further to reduce occurance of such loadcases.

File No.: Longitudinal_Settlement_Analysis_OAP-TJK.xls

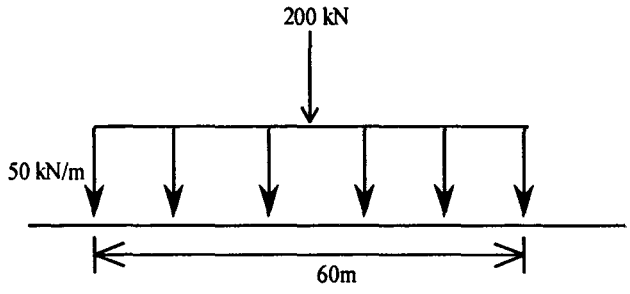
Calculated by: John Poh Date:_____

Drawing No.: _____

Checked by:Wen Dazhi Date:_____

LONGITUDINAL SETTLEMENT ANALYSIS
OLD AIRPORT ROAD - TANJONG KATONG

The longitudinal settlement analysis of the lining is checked in accordance with Clause 7.3.4.1 of the Design Criteria. The railway live load to be applied consists of single 200kN point load and a uniform loading of 50kN/m over the train length of 60m.
The train loading is based on BS 5400: Part 2: 1978: Specifications for loads, and is given in Figure below:



TUNNEL GEOMETRY AND PROPERTIES

Nominal Diameter of Tunnel	D_n	=	5.60	m	
Construction Allowance	ΔD	=	100.00	mm	
Thickness of Lining	t	=	275.00	mm	
Excavated Diameter of Tunnel	D	=	6.35	m	
Internal radius of tunnel	r_i	=	2.90	m	
Radius to extrados of lining	r_e	=	3.175	m	
Radius of lining centroid	r_o	=	3.04	m	
Cross sectional area of tunnel	A	=	5.25	m^2	
Second moment of inertia	I	=	24.21	m^4	
Effective second moment of inertia	I_e	=	12.11	m^4	(Since lining is segmented)
Length of bored tunnel	L	=	520.00	m	

TUNNEL MATERIAL PROPERTIES

Grade of concrete	f_{cu}	=	60	N/mm^2
Density of concrete	ρ	=	24	kN/m^3
Young's modulus of concrete	E	=	32000	N/mm^2
Poisson's ratio of concrete	μ	=	0.15	

SOIL PROPERTIES

Type of Soil		=	Marine Clay/OA		
Young's modulus of soil over 3B	E_s	=	12000	kN/m^2	
Width of beam	B	=	6.35	m	(Taken as diameter of tunnel)
Poisson's ratio of soil	μ_s	=	0.3		
Modulus of subgrade reaction (3m apart)	k_s	=	$\{E_s / B (1-\mu_s^2)\}*0.5*\pi*B*3$		
		=	62141.393	kN/m	

File No.: Longitudinal_Settlement_Analysis_OAP-TJK.xls

Calculated by: John Poh Date:_____

Drawing No.: _____

Checked by:Wen Dazhi Date:_____

MODELLING OF TUNNELS UNDER RAILWAY LOAD

The railway load, the section and material properties of the tunnel are entered into STAAD III for analysis.
The tunnel will be supported on elastic springs having stiffness, k_s obtained as above.

RESULTS

Maximum deflection	=	1.00	mm	Deflection is < 3mm, OK
Maximum angular rotation	=	0.0000		Angular rotation is < 0.0005 radian, OK

The calculated deflection is very conservative.
It is expected that the marine clay in this region will have a much higher Young's modulus, E.
Primary gouting from the TBM will cause the marine clay to have a much higher value of E.