Finite element modelling of load shed and non-linear buckling solutions of confined steel tunnel liners.

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Keywords: Steel tunnel liner, finite element, deterioration, confined buckling, Jacobsen

ABSTRACT

This paper compares Jacobsen's closed form solution for the buckling of steel pipes (1974) with the results of finite element analyses. The study is based on the geotechnical and structural analysis of a 4m diameter concrete tunnel with a fully welded internal steel liner, back grouted against the existing lining.

A staged geotechnical finite element analysis of the existing segmental lining was carried out using PHASE2. This analysis simulated the deformation of the surrounding ground with time, and the accompanying load shed onto the steel lining.

The resistance of the liner to bucking was analysed using STRAND7. This analysis was extended to examine the effect of varying load and stiffness parameters on the buckling load, and the results of the analyses are compared with Jacobsen's solution.

1 INTRODUCTION

The Port Hedland Under Harbour tunnel was excavated in sedimentary sandstone and conglomerate formations underlying alluvial deposits. The tunnel was lined with 250mm thick, gasketed, precast concrete segments which are corroding due to infiltrating saline water. Due to the ongoing deterioration it was proposed to reline the tunnel with a prefabricated, internal circumferential steel liner, site welded to form a continuous structure, with the annulus backgrouted against the concrete segments to allow load transfer.

Thin steel tunnel liners subject to external hydrostatic pressures are subject to failure from buckling. Many closed form solutions for single lobe buckling of confined steel tunnel liners have been proposed however these solutions consider a short term, uniform external water pressure applied to a confined liner, and do not consider any asymmetric ground loading that may be present. Such solutions are common for the design of steel lined penstocks for hydropower projects, the most accepted of these being Jacobsen's method.

The deterioration of the concrete segments will result in part of the ground loading being transferred onto the steel liner. The Jacobsen solution does not account for ground load transfer or the interaction between the deteriorating concrete liner and back grouted steel. Thus to capture the combined state of stress and resistance to buckling of the steel liner, detailed FEA was undertaken using a staged construction sequence and composite liner interaction using PHASE 2. The response of the lining under constrained buckling loads was then analysed under a range of different conditions and compared with the predictions of Jacobsen's closed form solution.

2. CLOSED FORM SOLUTION FOR BUCKLING OF CONFINED TUNNEL LINER

2.1. Confined Liner Buckling

The equation for the buckling of an unrestrained pipe subject to external pressure is given by a simple equation, similar to the Euler equation for column buckling. For a pipe restrained by a relatively rigid material (rock, grout or concrete), the initial buckling will be restrained, but a gap between the pipe and surrounding material will allow multi-lobe buckling to occur. In most cases, buckling manifests itself by formation of a single lobe parallel to the axis of the tunnel (refer Figure 1). Buckling occurs at a critical pressure, which depends not only on the thickness of the steel liner but also on the gap between the steel liner and concrete backfill. Realistically, the gap can vary

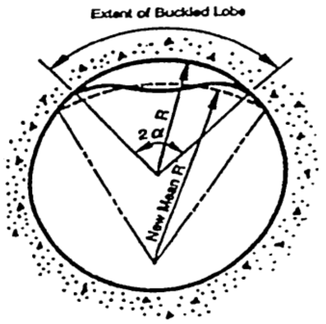


Figure 1: -Single Lobe buckling (Amstutz 1997)

from 0 to 0.001 times the tunnel radius depending on a number of factors, including the effectiveness and properties of contact grouting of voids behind the steel liner. Several alternative theories have been developed to predict the final buckling shape and pressure, including rotary symmetric buckling and single lobe buckling theories.

Berti (1998) compared single lobe buckling theories by Amstutz and Jacobsen, finding that the Amstutz approach was the simpler of the two, but included assumed constant values that may be unconservative. With the advent of computerized analysis the more conservative Jacobsen method has come into general use (Eskilsson 1997). Berti also found that the rotary symmetric equations are unconservative compared with the Jacobsen equations.

2.2. Jacobsen Closed Form Buckling Solution

Jacobsen derived three equations, relating the critical buckling pressure (P_{cr}) and parameters α and ß, related to the dimensions of the buckling lobe:

$$R_{rt} = \sqrt{\frac{\left\{\left[\left(\frac{9\pi^2}{4\beta^2}\right)-1\right]\left[\pi-\alpha+\beta\left(\frac{\sin(\alpha)}{\sin(\beta)}\right)^2\right]\right\}}{\left\{12\left(\frac{\sin(\alpha)}{\sin(\beta)}\right)^3\left[\alpha-(\pi\kappa)-\beta\frac{\sin(\alpha)}{\sin(\beta)}\left[1+\frac{(\tan(\alpha-\beta)\tan(\alpha-\beta))}{4}\right]\right]\right\}}} \quad \textbf{(1)}$$

$$\frac{\sigma_y}{E_m} = \frac{\left[1-\frac{\sin(\beta)}{\sin\alpha}\right]}{2R_{rt}} + \left[\left(\frac{p.R_{rt}\sin(\alpha)}{E_m\sin(\beta)}\right)\left(\frac{4p.R_{rt}^2\beta\sin(\alpha)\sin(\alpha)\tan(\alpha-\beta)}{\pi E_m\sin(\beta)\sin(\beta)}\right)\right] \quad \textbf{(2)}$$

$$\frac{12p.R_{rt}^3}{E_m} = \left(\frac{9\pi^2}{4\beta^2}-1\right)\Big/\left(\frac{\sin(\alpha)}{\sin(\beta)}\right)^3 \quad \textbf{(3)}$$

Where R_{rt} = R/t; R = radius of pipe to the neutral axis, t = pipe thickness, E_m = effective elastic modulus, κ = Radial gap / R; σ_y = pipe yield stress; α = half of the angle subtended to the centre of the pipe by the buckled lobe; ß = half of the angle subtended by the new mean radius through the half waves of the buckled lobe. For the purposes of this study these equations were solved by iteration using MathCad or an Excel spreadsheet, using an estimated value of α as a starting point.

Run No	Variable	Pressure	Pipe Deformation,mm				
1-3	Pipe deform.	Uniform	0, 10, 20				
4-6	Pipe deform.	Hydro.	0, 10, 20				
Run No	Variable	Pressure	Gap	Contact Friction	Contact Stiffness	Rock E	Surcharge Pressure
			mm	Factor	MN/m	GPa	Ratio
7-10	Pipe/restraint gap	Uniform	0, 1, 2, 5	0.5	10		
11-14	Pipe/restraint gap	Hydro.	0, 1, 2, 5	0.5	10		
15-17	Contact friction	Hydro.	2	0.7, 0.5, 0.3	10		
18-20	Contact stiffness	Hydro.	2	0.5	1, 5, 100		
21-25	Rock stiffness	Hydro.	2	0.5	100	10,1,0.25,0.1,0.05	
26-29	Surcharge press.	Hydro.	2	0.5	100	1	0, 0.3, 0.6, 1.2

Table 1: Summary of Finite Element model Runs

3. LINEAR BUCKLING FEA

The Jacobsen buckling solution has been derived from elastic theory and from the presented equations, it is obvious that the solution is independent of geotechnical variables such as confining rock modulus and shear interaction. The Jacobsen method also assumes a uniform pressure distribution to the steel liner, however for such a large diameter as the Port Hedland tunnel a hydrostatic distribution was considered to have an adverse effect on buckling pressure. Therefore, in order to assess the validity of the Jacobsen solution in deriving the capacity of the internal steel liner, confined by the deteriorating concrete segments and grout, a parametric FEA study was undertaken. The study involved a series of model runs where geotechnical attributes envisaged to have an impact on the critical buckling pressure were checked for sensitivity (Table 1).

All the analyses were carried out under plane strain conditions, modelling a 1 metre length of pipe. Runs 1 to 6 were carried out using linear elastic properties for the pipe, for comparison with the "Euler" buckling solution, including the effect of pre-deformation, to a maximum of 20 mm. The remainder of the runs used an elastic-plastic stress-strain curve for mild steel. Plate-shell elements were used for the pipe, restrained to enforce plane bending. For the restrained analyses the pipe was connected to the restraint points or the surrounding rock using frictional contact elements, which could be assigned a variable gap or zero gap. For Runs 21 to 29 the material surrounding the pipe was modelled using four node plane strain elements with a linear elastic stress/strain curve, with varying elastic modulus.

3.1. Modelling Results

The unconfined pipe under uniform pressure with zero pre-deformation had a clearly defined buckling pressure, close to that predicted by the theoretical solution (89 kPa). Introduction of a pre-deformation of up to 20 mm resulted in much larger deflections as the buckling pressure was approached, with no clearly defined buckling point (Figure 2). Under hydrostatic pressure the deflections were greatly increased, with no clearly defined buckling point.

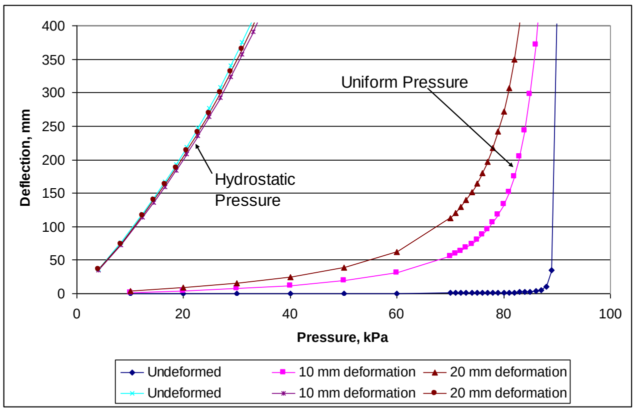


Figure 2: Unrestrained Buckling - deflection

Confining the pipe with fixed restraints greatly increased the buckling pressure. A gap between the pipe and the restraints significantly reduced the buckling pressure, and increased deformations (Figure 3). The hydrostatic pressure distribution increased deflections slightly, but had very little effect on the total buckling pressure. The buckling pressures found in these analyses matched those found from Jacobsens' method quite closely (Figure 4).

Varying the friction coefficient between the pipe and the restraints had very little effect on the results, but the contact element stiffness had a large effect, with the critical pressure varying between 550 kPa and 1160 kPa. Replacing the fixed restraints with plate elements representing the surrounding material gave similar results with the critical pressure varying between 840 kPa and 1170 kPa (Figure 5). Figure 6 shows the effect of applying a surcharge pressure to the top rock surface. A pressure ratio (PR, Surcharge pressure/ Pipe pressure) of 1.2 increased the buckling pressure by a factor of over 2. When confined with these high surcharge pressures the pipe failed in compression, rather than bending which was the failure mode with lower surcharge pressures.

4. APPLICATION TO THE PROJECT

It was determined that over the required design life of the tunnel, a loss of concrete thickness up to 50mm could be expected due to corrosion and spalling. To represent this load transfer with time, a staged construction sequence was modelled using the 2d geotechnical software Phase2: the stress condition prior to tunnel construction; the current stress state of the existing segmental concrete liner; and the load transfer condition that would result from the proposed future installation of the steel liner.

A staged geotechnical FEA modelling was carried out to determine whether the deterioration of the existing concrete lining would induce significant deflection and bending moment in the steel liner which would amplify the eccentricity of load, thus reducing the buckling load. Such a case could make the use of the Jacobsen solution inappropriate.

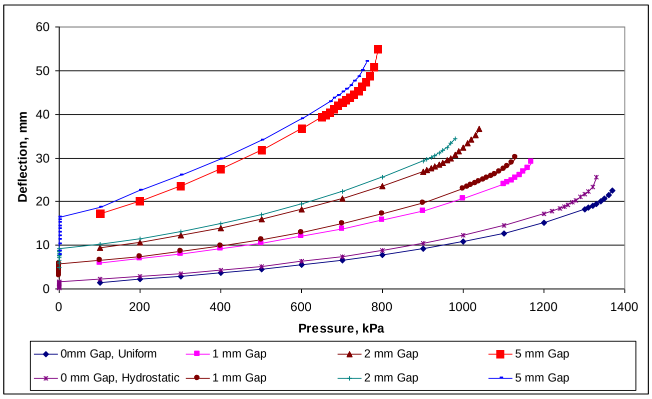


Figure 3: Restrained Buckling - deflection

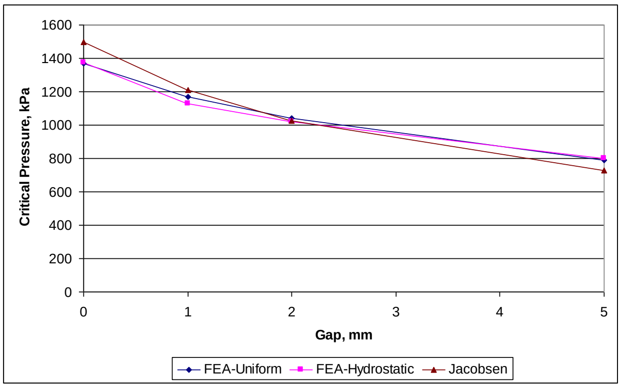


Figure 4: Restrained Buckling- gap

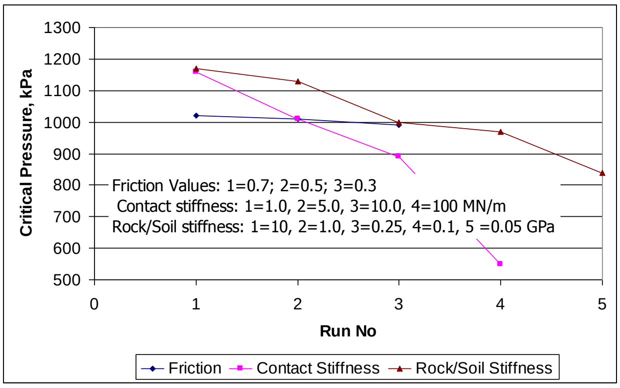


Figure 5: Effect of contact friction and stiffness and Rock/Soil stiffness

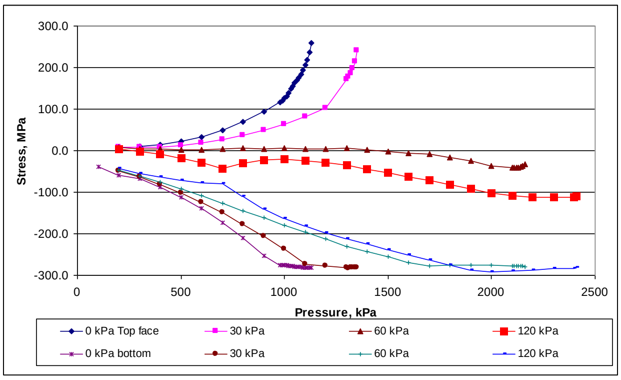


Figure 6: Effect of surcharge pressure

On review of the available geotechnical data and tunnel geometry, the 2D plane strain modelling was undertaken at the critical cross section which would have the most adverse load effect on the steel liner. The critical section was deemed to have a rock overburden of approximately 24m and a 40m head of groundwater, including tidal surge conditions. Due to the unavailability of existing design data, the actual stress state of the rock mass was unknown, and as such a sensitivity analysis was undertaken for a range of k_0 ranging from 0.3 to 3.0 to assess its influence on the load transfer condition; especially induced bending moments. The ground properties adopted for the model are given in Table 2

The geotechnical FEA was undertaken at this critical location as well as calculation of the analytical Jacobsen buckling pressure. It was determined that using the two part modelling approach described, if it was found the bending moments and deflections in the 25mm steel liner, from the identified load shed mechanism, were limited then the Jacobsen buckling solution could be applied to undertake the structural design of the steel liner.

4.1. Current Stress State in Tunnel Liner

To determine the stress condition in the existing concrete liner, excavation of the tunnel was modelled as the next stage. A 20% reduction in rock modulus was conservatively modelled to represent pseudo-3D displacement of the rock that may have occurred between excavation by TBM and installation of the segmental lining.

The next stage modelled represents installation of the tunnel liner directly after construction. The effect of concrete creep with age, to the present state, was modelled by reducing the modulus of the lining to an appropriate long term value, i.e. 50% of the (as new) modulus calculated in accordance with AS 3600. The present stress condition is presented in Figure 7, which shows the development of plasticity around the liner as well as the deflection induced from ground loading.

Material	γ (kN/m3)	E (MPa)	Cohesion (kPa)	ϕ	ν
Fill	18	25	30	25	0.45
Marine Mud	18	5	30	25	0.45
Red Beds	20	23	55	32	0.3
Upper Conglomerate	22	1000	250	36	0.3
Sandstone	22	100	130	34	0.25
Lower Conglomerate	22	1000	55	32	0.3

Table 2: Material Properties

4.2. Future Installation of Steel Liner

This final stage represents installation and back-grouting of the proposed 25mm steel liner, followed by deterioration of the existing segmental lining through the effects of steel corrosion and consequential concrete spalling. This deterioration mechanism was modelled by further reducing the concrete modulus by 50%, i.e. reduction of lining thickness from approximately 250mm to 200mm. A sensitivity study on the effects of in-situ stress, elastic modulus of the ground and the Mohr-Coulomb failure parameters was undertaken to determine the effect on distribution of axial force and bending moment in the steel liner.

The hoop thrust resulting from the critical buckling pressure calculated by the closed form buckling solution would have to be greater than the combined hoop thrust induced in the steel liner as a result of the hydrostatic water head plus the axial force calculated as a result of ground loading, from load shed. Appropriate geotechnical and structural factors of safety need obviously be applied in this calculation.

4.3. Geotechnical FEA Results

The Phase2 study showed that deterioration of the existing segmental lining did not induce significant bending moments in the steel liner (Figure 8). Variation in the elastic modulus or in situ stress condition of the surrounding ground also had little effect on the bending moment distribution (Figure 9). However, cohesion of the layer in which the concrete lining is confined can affect the bending moment distribution on the steel liner. This stratum must possess sufficient cohesion to limit significant development of plasticity as the lining is deteriorates.

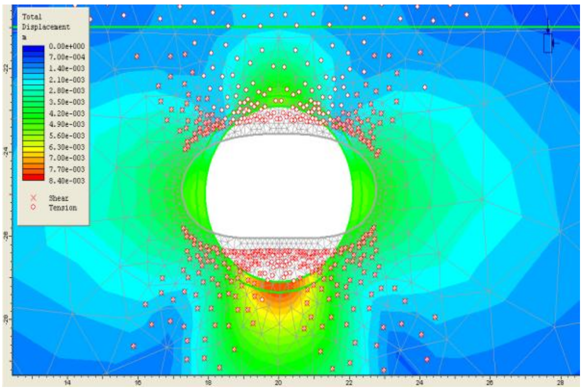


Figure 7: Stage 4 - Current State of Stress

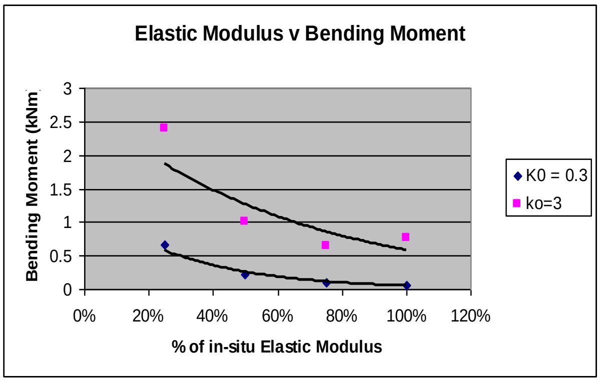
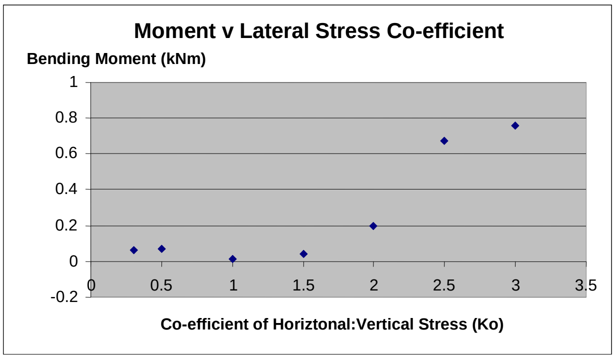


Figure 8: Effect of Elastic Modulus on Load Shed

The staged FEA showed that plasticity and arching of the strata developed during initial excavation and lining of the concrete tunnel. As the lining deteriorates additional displacements of the ground are small due to the developed arching. Increased areas of plasticity are also not evident. However this is only valid if the additional stress in the segmental lining due to deterioration does not result in plasticity of the tunnel lining, as shown in Figure 7.

If the segmental liner remains elastic, the bending moments induced due to the load transfer mechanism are small and do not induce a significant deflection of the steel liner.



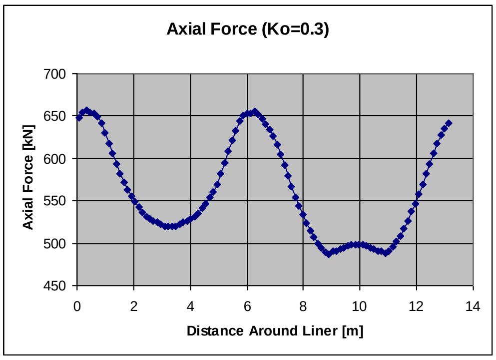


Figure 9: Bending Moment transfer to Steel liner **Figure 10: Axial Load Distribution in steel liner**

5. CONCLUSIONS OF STUDY

The finite element analysis of the load shed mechanism showed the effect of various parameters on the distribution of bending moment and axial force transfer from the segmental to the steel liner with deterioration. The study concluded that the co-efficient of in-situ stress (Ko) and elastic modulus of the ground both had an effect on the magnitude and distribution of axial load in the steel liner; however as plasticity had developed around the segmental lining prior to the installation of the steel liner, further deterioration of the concrete segments resulted in only small further strains of the ground. The arching action of the ground and the small increase in strain resulted in increased axial loads in the concrete segments which were transferred to the steel lining, but negligible bending moments were transferred to the steel liner.

The finite element buckling analysis results showed good agreement with the equivalent analytical predictions for both the unrestrained and restrained solutions for uniform load conditions. Under hydrostatic loads the critical pressure for the unrestrained liner was greatly reduced, but there was very little change to the restrained critical pressure. The finite element results were also in good agreement with the Jacobsen predictions when a gap of up to 20 mm was introduced between the liner and the restraints. Varying the stiffness of the restraints or the surrounding rock had a significant effect on the critical pressure, with reduced confinement stiffness reducing the critical pressure. Application of a vertical surcharge pressure to the surrounding soil or rock greatly increased the critical pressure, with the pipe failing in compression, rather than bending. Variation of the pipe/rock interface friction coefficient had very little effect on the critical pressure.

The study showed that the Jacobsen theory was suitable for the design of the steel liner for the case studied in this paper, since it gave a good estimate of the critical pressure under hydrostatic loading, and deterioration of the concrete liner was found not to significantly increase the bending moments in the steel liner. In situations with different constraint stiffness however the Jacobsen results may be unconservative over-conservative, and further investigation of the critical pressure by means of a finite element analysis may be justified.

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