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Design of tunnel liners: How important are bending moments?

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ABSTRACT: The design of the single shell liner of the 10.6m interior diameter Grauholz tunnel was performed with different design models. Besides ground loads, also transportation and handling loads have to be considered. These loads are actually more severe than most loads from ground and water. Design of the alternate proposal was made by the contractor based on a ring partially embedded with springs and applied external loads. Verifications were performed by the engineer using closed form solutions developed by Einstein-Schwartz (1977). Handling loads and loosened rock slabs induce bending moments with small thrust forces and require reinforcement. For loads from ground the couples of thrust and bending moment plot within an area for unreinforced concrete. The reinforcement in the liner will distribute the fine cracks and limit opening width, and is also necessary for force concentrations at the joints. Bending moments from ground loads are not important, but the liner has to resist the thrust forces.

1 INTRODUCTION

The Grauholz tunnel in Switzerland has been constructed with a mechanised system. In soil and mixed-face conditions the mix-shield operated as a slurry shield. In rock with sufficient cover it was operated as open face shield tunnel boring machine. (Steiner and Becker, 1991; Steiner, 1993; Scheidegger et al. 1993, Jancsecz and Steiner, 1994). The lining of the tunnel with 10.6 m interior diameter was constructed from segments of 0.4 m thickness and 1.8m width. Each ring consists of 6 segments and a keystone (Fig. 1).

The segments were manufactured in a purpose-built factory on site. Usually segments for 400 meters of tunnel were stored on site, in order to mature for 28 days. The tunnel was constructed to an alternate proposal submitted by a contractors joint venture, therefore the design was performed by the contractor and was verified by the consulting engineers. Both parties involved used rather simple design tools. However, the "loading" parameters were varied to encompass a range of loading conditions. The contractor used a ring with hinges which was supported by springs and loaded by external loads in the crown. For the verification the consulting engineer used a closed-form solution (Einstein & Schwartz, 1977). The different models yielded different loads in the lining, both normal forces and bending moments. The resulting pairs of normal force and bending moments are compared on Moment-Thrust interaction diagrams.

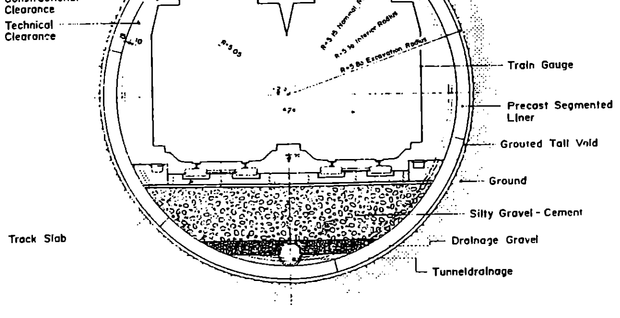

Fig. 1 Cross section of Grauholz tunnel

2 GROUND CONDITIONS AND DESIGN

A longitudinal geologic section along the Grauholz tunnel is shown on Fig. 2, together with the location of six design sections. These sections are also shown in Figs. 3 to 5. The Grauholz tunnel crosses a wide range of soils both above and below the ground water table. In the western soil section, first ice-marginal deposits (glacial till, silts and sands to clean gravel) with ground water were crossed (Fig. 3). The overburden reaches 25 meters with

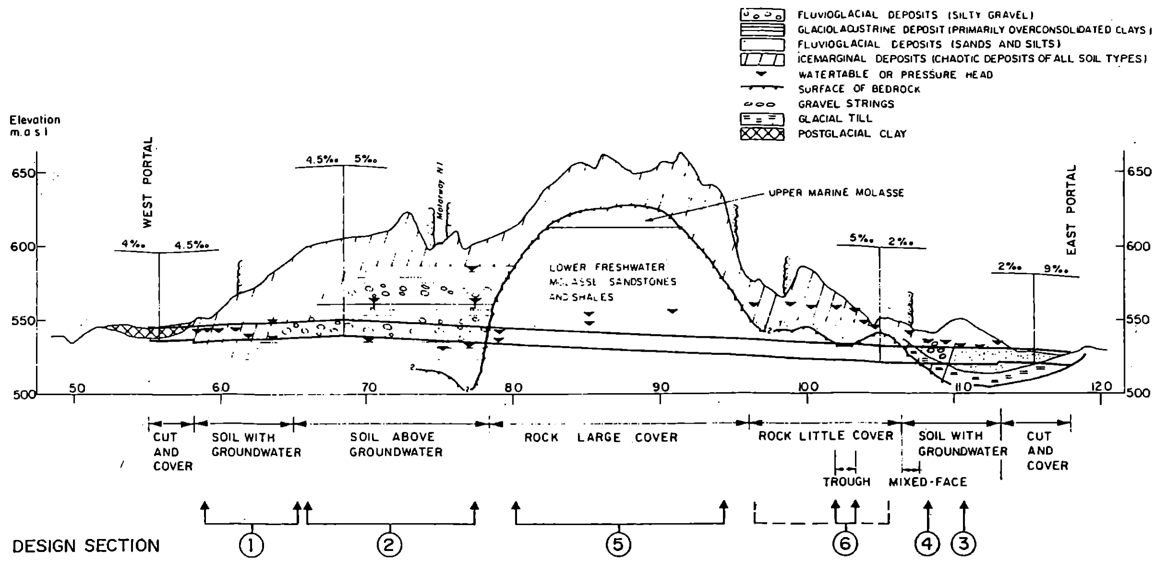

Fig. 2 Longitudinal geologic section

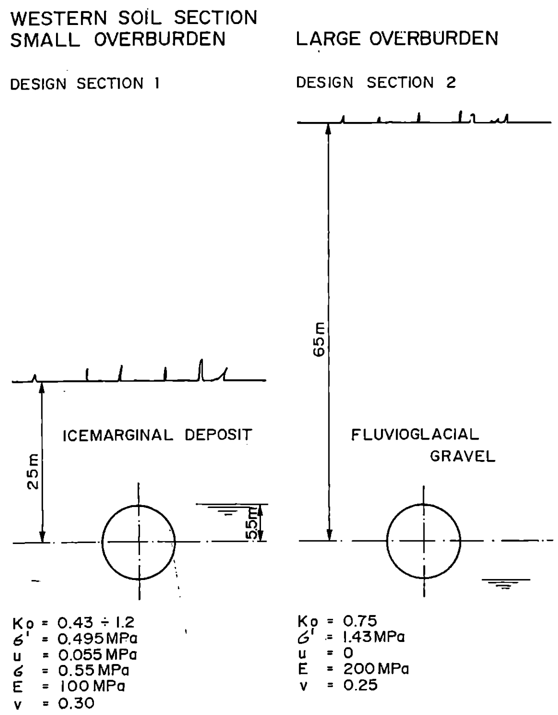

Fig. 3 Western soil section design conditions

ground water in the crown of the tunnel. The horizontal stress ratio was estimated $K_0 = 0.43$ -1.2. In the second section, the overburden is up to 65 meters above the tunnel axis and the tunnel is located in gravel without ground water.

In the following section (Fig. 4) bedrock, soft marls and sandstone of the Molasse formation, the overburden reaching 110 meters, was crossed without ground water. The horizontal stress was estimated as $K_0 = 0.7$-1.0. Then the rock cover reduces to little or none above the tunnel, soil overburden is 30 meters with ground water. Also some mixed face conditions with 2 meters of soil in the tunnel crown were encountered. Total overburden is around 40 meters, and water pressure 34 meters above the axis. The horizontal stress was estimated as $K_0 = 0.8$-1.8.

In the western part the eastern soil section the tunnel crosses glacial till and gravel (Fig. 5) with 15 meter overburden above the axis and 10 meter water pressure. The horizontal stress was estimated as $K_0 = 0.5$-1.7, with a likely value around $K_0 = 1.0$. The final section crosses dense glacial silts and sands. The overburden above axis reaches 24 meters and the water is 5 meters above the crown. The horizontal stress was estimated as $K_0 = 0.5$-1.2.

3 LOADING CONDITIONS OF SEGMENTS

The segments had to be designed to withstand the ground and water loads during the entire design life of the tunnel. This has been theoretically defined as 100 years. The ground and water loads act almost instantly on the lining once this has left the tail of the slurry shield and the tail void is grouted. Long-term changes are not expected

3.1 *Loads from ground and water*

Loads on the lining were determined for the six design cases presented earlier. The input parameters were varied. In particular the initial horizontal stress ratio was varied under consideration of overconsolidation effects due to glaciation. The computations were performed with total stresses that resulted from a superposition of initial effective stresses and water pressures. The contractor used a design model, based on German recommendations for the design of tunnel liners. An embedded ring with joints is simulated with spring support, which is loaded in the crown for ground loads from loosening and water pressure where it applies. Swelling pressure were applied in the invert.

The verification computation by the engineer used the closed-form analytical solution developed by Einstein-Schwartz (1977). The support delay effects were studied using the recommendations by Schwartz and Einstein (1980). The shield is more than 11 meters long, thus the theoretical recommendations would lead to no load in the lining. The three-dimensional factor is therefore based on engineering judgement and varies from 0.3 to 0.8.

3.2 *Handling Loads*

Once the segment is lifted from the steel mould in the prefabrication plant, the reinforcing steel is loaded due to gravity loading (Fig. 6). The segments may also be placed on the edges on the ground. This induces bending in the other direction. The segment

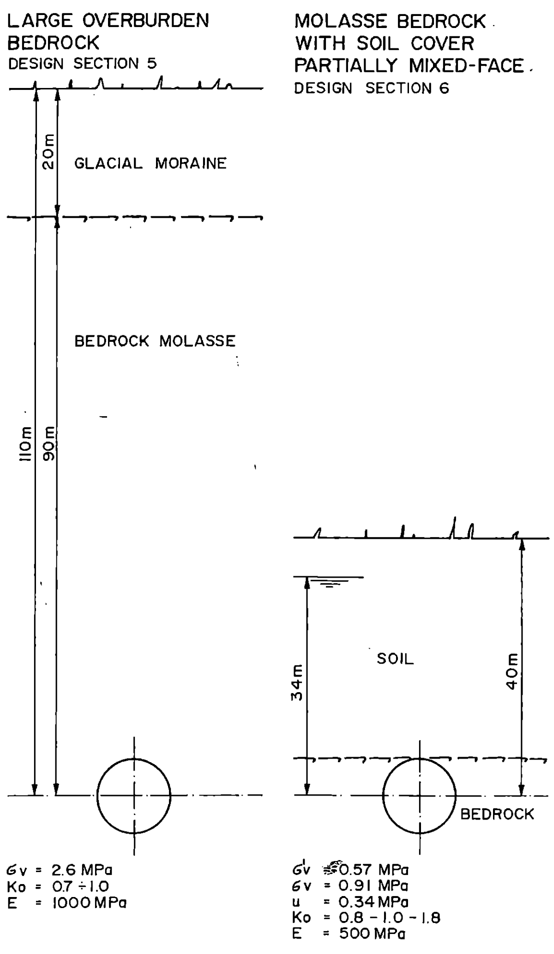

Fig. 4 Design section in bedrock

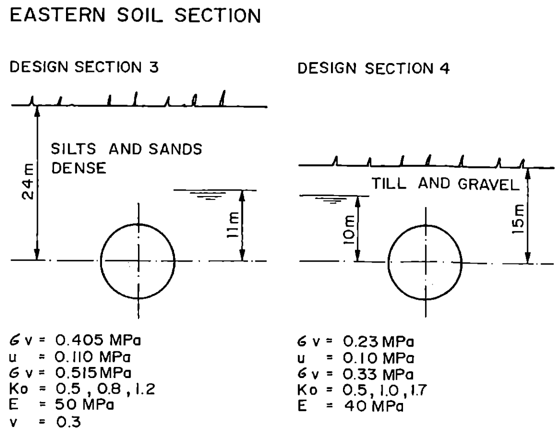

Fig. 5 Design eastern soil section

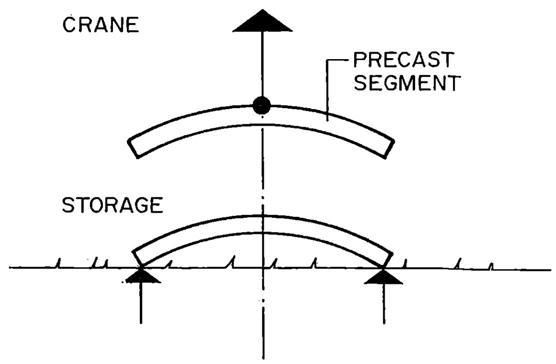

Fig. 6 Handling Loads on segments

has to be designed such that the reinforcement together with young age concrete, after 6 hours steam curing, can take this loading.

3.3 *Loosening Loads in the crown of the tunnel*

It is possible that in the sedimentary bedrock slabs may separate in the crown and load the segments in the crown. This loading condition was modelled with a three-hinged arch (Fig. 7). The segments forming a three-hinged arch may carry a slab of 1.3 meters thickness without large cracks. Governing the stability is the sliding in the joints that are assumed not yet to be in contact with the grout. The resulting forces in the segments are primarily bending moments with small normal forces.

3.4 *Shoving Loads*

During advancement of the shield the large shoving loads from the hydraulic jacks (Total =100 MN) are taken by the segments in the longitudinal direction. From computations it becomes evident that no economic reinforcing system can be designed to

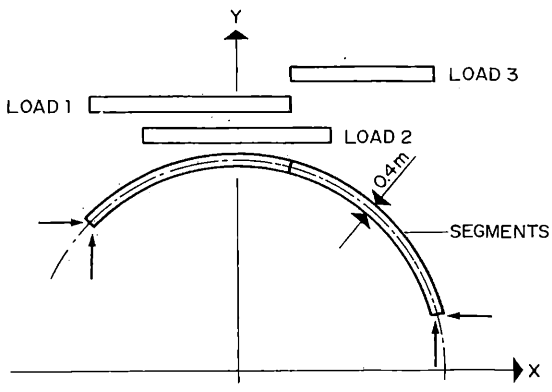

Fig. 7 Loosening Loads on Segments

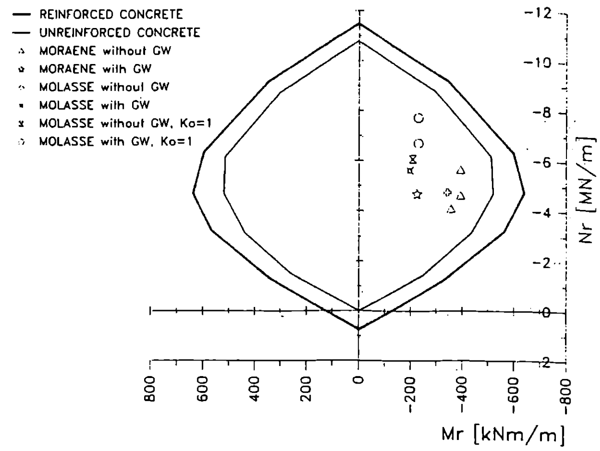

Fig. 8 Interaction diagram with contractor's results

equalise uneven support. The segments have to be manufactured to very small tolerances.

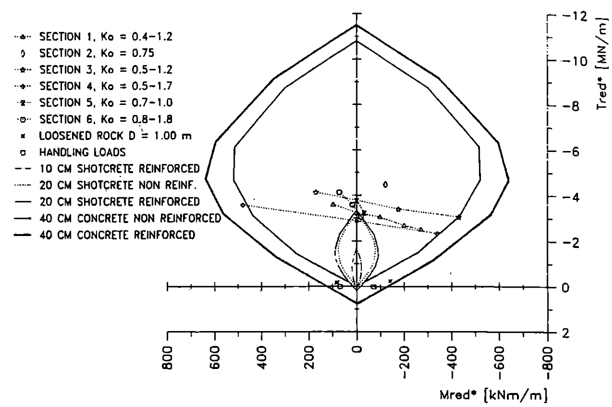

Fig. 9 Forces obtained from verification

4 COMPUTED FORCES IN THE LINING

The forces (bending moments and thrusts) are presented on a moment-thrust interaction diagram. The concrete is a BS 50/40 according to Swiss Standard SIA 162, corresponding to a f_{cu}= 24 MPa according to BSI. The steel has a yield strength f_y= 460 MPa. The reinforcing content is μ= 0.21%. The diagrams show the moment-thrust line for an unreinforced rectangular concrete section and the one with the above reinforcement.

In addition in one of the diagrams (Fig. 9) the moment-thrust interaction diagrams for shotcrete B35/25 according to Swiss Standard SIA 162, corresponding to f_{cu}= 16 MPa and thicknesses of shotcrete layers of 100 mm and 200 mm are shown. These curves show significantly smaller capacity.

4.1 *Design model of the contractor*

The results of the maximum couples (normal force and bending moments) are summarised on Fig. 8. The load in the crown is the overburden up to a maximum depth of two diameters of the tunnel. These computed loads are primarily obtained for the segment in the crown. Note that all couples fall in the right hand side of the diagram. This means that traction would always be on the same side. However, all the data points fall within the zone for non-reinforced concrete.

4.2 *Verification computation by the engineer*

The resulting forces obtained by analysing the problem with the closed form solution by Einstein-Schwartz (1977) including three-dimensional effects are presented in Fig. 9. The effect of the initial horizontal stress ratio on the couples is easily visible. For $K_o > 1$ the couples plot on the left side of the diagram. The normal force shows only little dependence on the horizontal stress ratio K_o. Practically all points, with the exception for extreme horizontal stress ratios, plot within the range of the unreinforced zone.

Also the couples for handling loads and from loosened rock are plotted on Fig. 9. The handling loads are on the line without thrust forces, whereas the loads from loosening are slightly above. All these data plot outside the non-reinforced but inside the reinforced concrete section. For these loading cases reinforcement is necessary.

Note also that most of the forces computed fall outside the range of the M-T diagrams for 200 mm and 100 mm shotcrete. In particular also the normal force exceeds the resistance capacity. As noted earlier the full ground and water load had to be expected behind the shield. A shotcrete support, even with reinforcement and lattice girders, would have been mostly insufficient, and numerous collapses of the tunnel would have been very likely.

4.3 *Conclusions from analysis*

The computations with different models show different couples of bending moment and thrust force. With other models, probably different force couples would have been obtained. There would be reasons and arguments to believe any of the computations, none of which can be disproved. The major conclusion is, however, that the resulting force couples fall within the range of the M-T diagram for unreinforced concrete. Thus one might conclude that reinforcement may not be necessary for the lining. However, reinforcement is necessary for the handling loads. In addition the reinforcement is a means to distribute fine cracks in the concrete. This is important in order to achieve a watertight concrete lining. Furthermore reinforcement is necessary near the joints in order to permit the load transfer across the zone of contact without cracking the concrete. The reinforcement is primarily necessary during the handling, but it helps to guarantee the serviceability of the concrete segments.

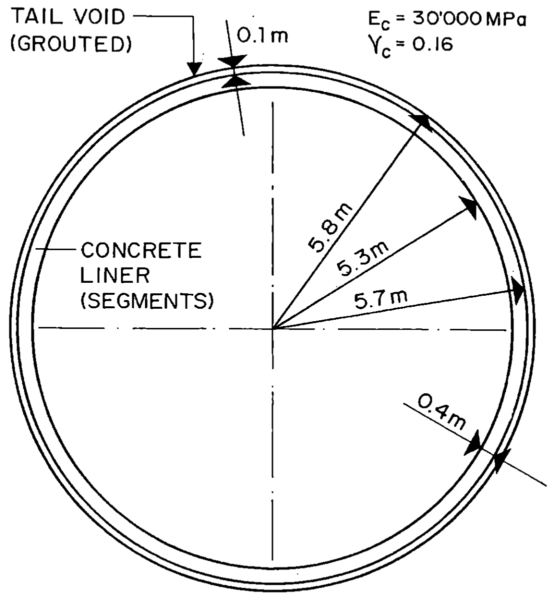

Fig. 10 Boundary conditions for design

5 EFFECT OF REDUCED BENDING STIFFNESS

The design verification assumed a continuos concrete beam with a rectangular cross section. The compressibility ratio C* and the flexibility ratio F*, according to Einstein-Schwartz (1977) were computed using properties of bending and compressive stiffness of uncracked concrete (Fig. 10). The effect of joints (Muir Wood, 1975) and of cracking due to bending may be simulated in closed form solution by decreasing the bending stiffness, resulting in an increased flexibility ratio F*. The effect on displacements of the lining can be demonstrated by an example only. The soil section with large overburden was chosen (Fig. 3, right side). The computations were performed for Ko = 0.41 and 0.6. Only plane strain effects without three-dimensional corrections were considered. The computed bending moments circumferential thrusts and displacements in crown and at the springlines are shown in Table 1, for full bending stiffness and bending stiffness reduced to one tenth. Displacements for an unlined tunnel in elastic ground are also shown. The bending moment drops to one tenth, however, the total displacements

Table 1: Influence of Bending Stiffness on Moments and Displacements of Liner

	Soil with large Overburden H= 65m γH= 1.43 MPa E = 200 MPa, ν = 0.3			Soil with large Overburden H= 65m γH= 1.43 MPa E = 200 MPa, ν = 0.3		
Ko	0.6			0.41		
Case	Stiff in Bending	Flexible in Bending	Unlined Opening	Stiff in Bending	Flexible in Bending	Unlined Opening
Compressibility Ratio C*	0.097		Large	0.097		Large
Flexibility Ratio F*	217	2170	Large	217	2170	Large
Moment (MNm/m)	±0.278	±0.029	0	±0.418	±0.0442	0
Thrust (MN/m)	5.66 to 5.77	5.71 to 5.72	0	5.07 to 5.23	5.17 to 5.26	0
Radial Displacement in crown positive = into opening	19 mm	20.3 mm	59 mm	27.5 mm	26.9 mm	63 mm
Radial Displacement at springline negative = outward	-14.3 mm	-15.1 mm	22.3 mm	-22.9 mm	-24.4 mm	9 mm

are in the order of ΔR= 15 to 20 mm.

Assuming that first full bending stiffness would be active and then the bending stiffness would be reduced, an incremental change of radius of only $\Delta\Delta R$= 1 to 1.5 millimeter would be necessary. The couples of normal force to bending moment are shown in Fig. 11. The point representing the reduced bending stiffness has moved closer to the central zone of the diagram. The incremental change in radius is one millimeter only, the relative change $\Delta\Delta R/R$ = 0.02%. This induces very small incremental strains in the tunnel lining that may come from creep, shrinkage or cracking of the concrete. By comparing the displacements of an unlined opening one notes that differential radial displacements are in the same order for all cases studied. A fully flexible lining in bending, but stiff in compression results in displacements that are only little larger. The displacements are controlled by the ground stiffness. Bending moments from ground loads are of lesser importance. However the liner must have sufficient capacity to carry the thrust loads.

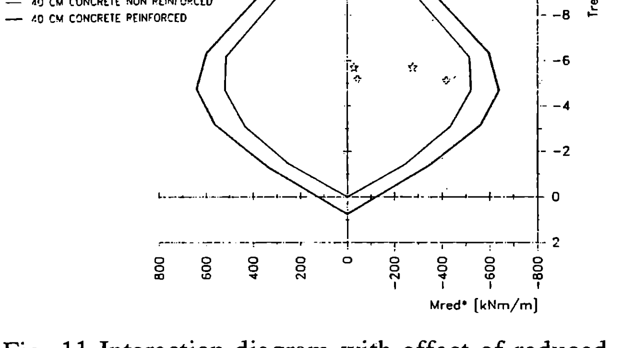

Fig. 11 Interaction diagram with effect of reduced bending stiffness

6 ACCURACY OF DESIGN COMPUTATIONS

The above computed example shows that deformations of the liner from ground behaviour are in the order of a few centimeters. Many of these changes occur along the shield which has a length corresponding to the tunnel diameter. Also important changes may occur with a few millimeters deformation. The tail void has a width of 100 mm, the deformations necessary to induce a significant change in the loading of the liner are a tenth to one hundredth of the thickness of the tail void. Tail void grouting has the most significant influence on the loading of the tunnel liner.

In the case of the Grauholz tunnel the grouting proved particularly delicate in case of the Molasse bedrock section. The initial grouting behind the shield did not lead to a sufficient prestressing of the neoprene gaskets, thus the tunnel was leaking. This had to be remedied with supplemental grouting that led to a prestress of the gaskets in the joints.

7 CONCLUSIONS

For loads from the ground different design models yield different thrust and normal forces. The couples plot, however, within the zone of resistance of a non-reinforced concrete cross-section. Segments undergo the most severe loading conditions during handling, transport and placement and bending moments are important. Reinforcement of concrete has to be designed for this load. The loading conditions from ground loads are much more favourable. Bending moments may be reduced due to creep and are not important for ground loads. However, the liner has to have sufficient capacity to carry the thrust loads.

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