DESIGN OF THE COATZACOALCOS IMMERSED TUNNEL

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INTRODUCTION

Coatzacoalcos is a port city in the southern part of the Mexican state of Veracruz, on the western bank of the Coatzacoalcos River. The city has a population of about 250.000, making it the third-largest city in the state of Veracruz. The largest community in the municipality, aside from Coatzacoalcos, is the town of Allende, with a population of about 25.000. The town of Allende is situated on the east bank of Coatzacoalcos River.

The city's industry is dominated by the petrochemical sector. The main petrochemical complexes are however located on the east bank of Coatzacoalcos River and can only be reached via an old and heavily congested bridge south of Coatzacoalcos.

Work is now under way on a tunnel to directly connect the towns of Coatzacoalcos and Allende (see figure 1). The immersed tunnel will be constructed out of 5 tunnel elements of 138 m each, resulting in a total length for the immersed section of 690 m. At both ends of the immersed tunnel, access ramps will be made by the cut&cover method using diaphragm walls. The water depth of the Coatzacoalcos river varies between 5 and 12 m. The maximum depth of the base of the tunnel will be about 30 m below water level.

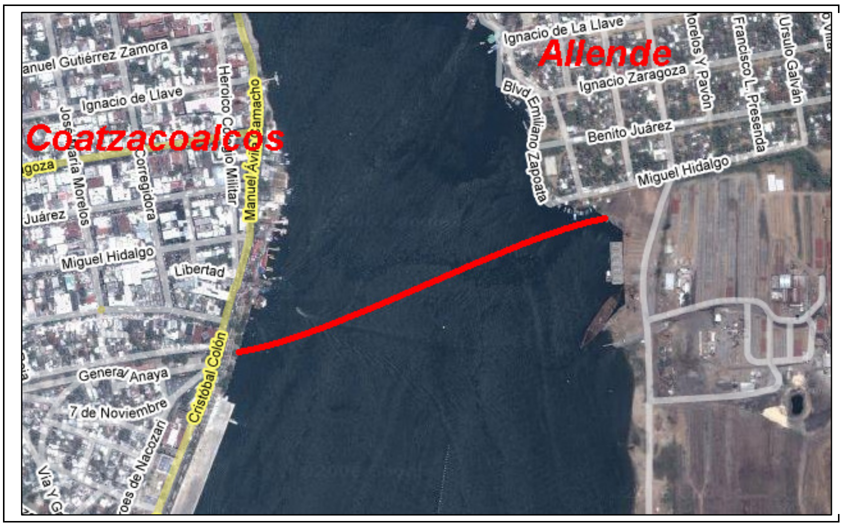


Figure 1: Tunnel alignment (picture source: Google Maps)

The contractor for the work is Constructora Tunel de Coatzacoalcos (CTC) a joint venture of FCC from Spain and the Mexican contractors Impulsa and OPC. The marine works (dredging, transport and immersion operation) will be carried out by a joint venture of Boskalis and Volker Stevin.
The main challenging design aspects are the rather poor soil conditions and the seismic hazard which are the subject of this paper.
Design is carried out at various locations: immersed tunnel elements in the Netherlands by TEC (Tunnel Engineering Consultants), a joint venture of Royal Haskoning, Witteveen+Bos and DHV, cut and cover sections in Spain and Mexico by engineering departments of the contractor and tunnel technical installations in the Netherlands and Mexico. This split of activities at different locations and at a great distance of the construction site has of course an impact on the design process.
Early 2007 construction works started for this first immersed tunnel in Mexico.

SITE CONDITIONS

The results from the site investigation show that silty sands and silty clays are present down to about 30 m depth. Figure 2 shows part of a CPT record from the east bank of the river (Allende). The spatial variability of the layers is high (heterogeneous channel deposits). Below 30 m, a more continuous layer of densely packed sand is present.

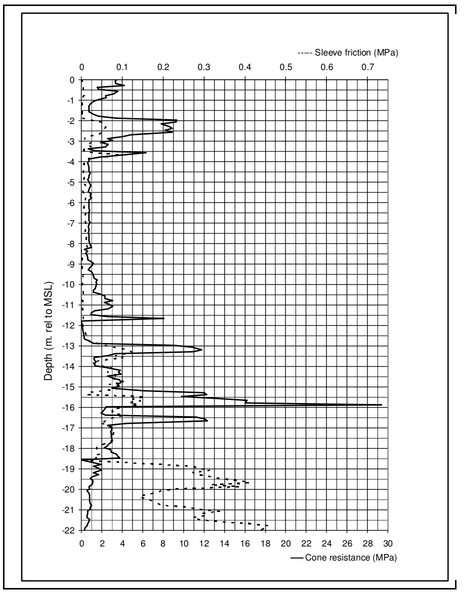


Figure 2: CPT record (Allende river bank)

The Coatzacoalcos area is seismically active. For the immersed tunnel ground accelerations between 2.0 and 3.5 m/s^2 are predicted (750 year return time).

SETTLEMENT ANALYSIS IMMERSED TUNNEL

The geotechnical design for the immersed tunnel focussed on the time-settlement behaviour of the tunnel elements during the following construction phases:

0. Dredging of immersion trench.
1. Installation of tunnel elements and construction work, ending with the final completion of the immersion joints.
2. Operation of the tunnel (Design Working Life of 100 year).

Initially, the dredging of the immersion trench will cause heave of the trench floor. The predicted heave varies between 0.1 and 0.3 m. After installation of the tunnel elements, settlements will develop. The final load of the tunnel (with fill material and roof protection, see figure 3) does not cause an increase of soil pressures. The installation of the tunnel therefore only causes re-loading, compensating for a large part the heave caused by the dredging of the immersion trench.

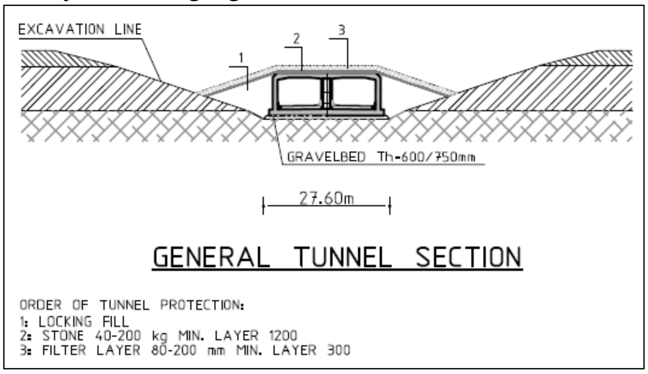


Figure 3: tunnel section

The heave rate after dredging of the immersion trench proved to be a difficult factor to include in the geotechnical calculations. In regular settlement calculation programmes (such as Delft Geosystems MSettle) only the vertical consolidation coefficient (c_v) for compression can be introduced. Heave takes place faster, but the exact vertical consolidation coefficient for heave could not be determined from available laboratory data. Based on few literature sources available, it is assumed that heave takes place about 10 times faster than settlement.

Due to the variable thickness of the cohesive layers under the tunnel floor, the load from the tunnel elements will cause differential settlements. For (Dutch) immersed tunnels, it is normally required that 90% of these settlements are achieved prior to finishing the immersion joints. This requirement is usually met during the construction time of the tunnel (normally about 3 months).

In the initial design, it was envisaged that the immersion trench would be allowed to fill up with sediment as a result of natural sedimentation. Preliminary calculations showed that such sedimentation would cause significant settlement after completion of the immersion joints. Thus, an additional ‘resting time’ was introduced to allow for more settlements to develop during construction. Also, a less severe settlement criterion of 50 mm after tunnel completion was adopted. With a resting time of 6 months, preloading of the tunnel elements directly after immersion will probably be required for 2 out of 5 elements to increase initial settlements.

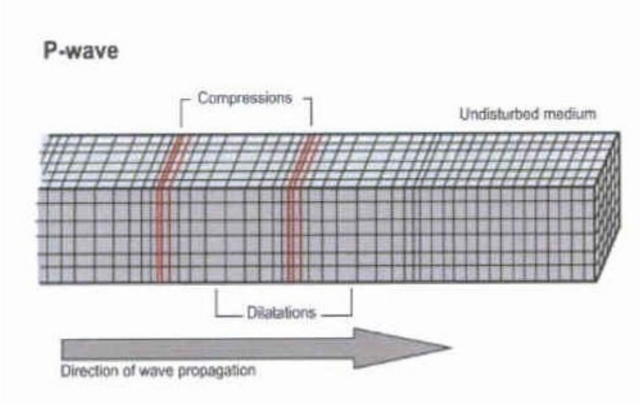
SEISMIC DESIGN

Underground structures are in general terms less sensitive for seismic loading than above ground structures. Response of an above ground structure is dominated by resonance with the ground motion at the foundation level. Resonance in free-standing structures will induce displacements (loads) much greater than those from the non-seismic combinations, potentially causing failure of the structure. The response of a buried structure is typically compliant with the ground motion itself. The loads on the structure are controlled by the ground strains and can usually be absorbed without the structure losing it’s capability to carry the static loads.

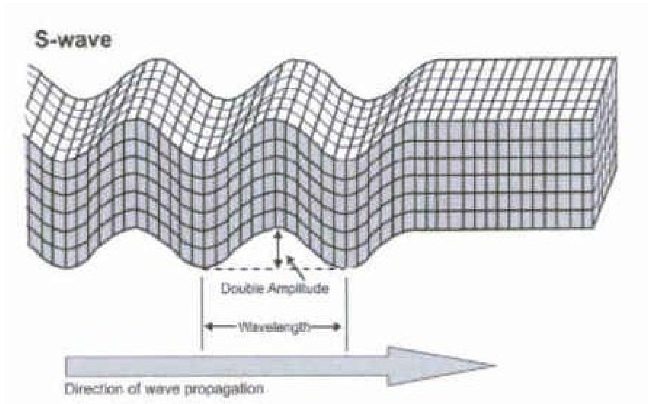
Tunnels are a special type of buried structures. Their linear nature contributes in part to their good seismic performance and their relative simple structural form allows for simplifying their analysis.

The seismic analysis of an immersed tunnel usually includes 2 main aspects: ground shaking (seismic motion) and geotechnical ground failures.

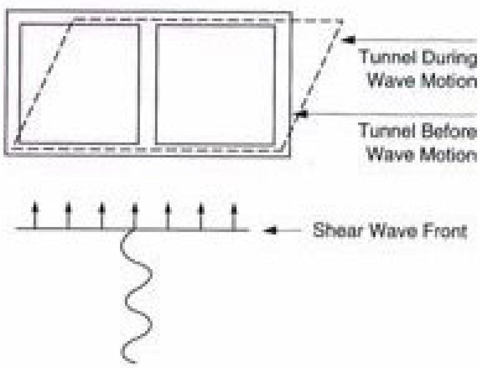
Ground Shaking
As shown in figure 4 there are three types of structural responses of an immersed tunnel during ground shaking (Owen and Scholl, 1981): (1) worming: axial compression and extension; (2) snaking: longitudinal bending; and (3) racking/ovaling. A flexible tunnel will in general conform to the ground motion and experience equivalent strains. A stiff tunnel will resist deformation and will experience lower strains but higher imposed loads.

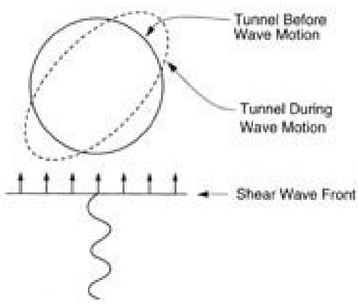


Worming: Axial compression and extension along extended lengths of the tunnel



Snaking: Longitudinal bending and curvature (in the vertical and horizontal planes)





Racking (rectangular structure) and Ovaling (circular structure): distortion of a transverse cross-sectional geometry due to local soil deformations

Figure 4: different types of structural responses (Owen and Scholl, 1981)

Geotechnical ground failures
Besides the structural response of the tunnel during seismic ground motion also the effects of possible ground failures and loss of foundation support should be investigated.

- Loss of shear capacity due to partial or complete liquefaction (and its consequence in terms of loss of foundation strength during and under post earthquake conditions).
- Discrete displacements along geological features like faults.
- Foundation failure and/or slope instability under the seismic induced load.

STRUCTURAL SEISMIC RESPONSES OF THE IMMERSED TUNNEL
Axial compression and extension (worming)
The tunnel is subjected to an imposed ground strain. The maximum frictional forces that can be developed between the tunnel and the surrounding soils limit the axial strains in the tunnel cross section. The numerical analysis for the detailed design has therefore focused on the interaction between soil and tunnel structure.

Modelling of the longitudinal behaviour:
Initially a semi-empiric analytical approach has been adopted (Hamada et , 1992). This approach has resulted in a first estimate of the seismic performance of the tunnel and the impact on the tunnel design (prestressing).
Based on the same concept of an axially embedded beam as the analytical model a numerical model is developed with compared to the analytical model a number of enhancements, which incorporated the project specific conditions.

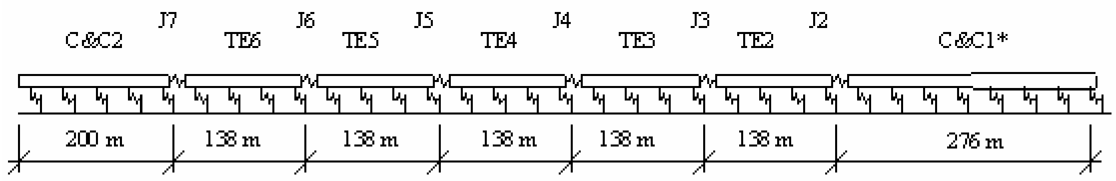


Figure 5: Schematic overview of numerical mode

Table 1 : Enhancements effectuated by numerical model

Analytical model	Numerical model
Articulated beam of infinite length	Articulated beam with length of the tunnel
No localised boundary conditions	Rigid fixation of tunnel at abutments
Linear joint characteristic	Non-linear joint characteristic based on GINA characteristic
Non-differentiated joint characteristic	Differentiated joint characteristic based on GINA characteristic
Non-differentiated axial embedding	Differentiated axial embedding
Non-differentiated imposed strain (based on friction force)	Differentiated imposed axial strain (based on individual friction forces on elements)

The numerical software used is DIANA (www.tnodiana.com), which is a state of the art FE program, dedicated to non-linear structural and geotechnical analysis.

The earthquake is modelled by an imposed ground strain resulting in lengthening of the soil around the tunnel leading to a relative displacement between concrete and soil. The relative displacement causes a (limited) introduction of shear stresses into the concrete elements. Because of the relatively high axial stiffness of the elements and the more flexible immersion joints, displacements will be concentrated in the joints. During an earthquake, the primary task of the joints is therefore to be able to follow the imposed displacements without fully opening, ensuring water tightness. Secondarily the joints should absorb a considerable part of the earthquake energy, so limiting high tensile forces into the concrete.

Longitudinal bending and curvature (snaking)
The bending strains are in general relative small compared to the axial strains. In the design an upper limit for the bending strains is used based on the maximum Peak Ground Displacement (PGD) determined by IIE in the Seismic Hazard Analysis (IIE, 2007), an idealised wave length and an equivalent stiffness for the flexible chain of tunnel elements and joints.
A more detailed analysis was not necessary as the proposed method gave conservative results which are acceptable (a more detailed analysis of the bending strains would have given no or only a small reduction of the overall maximum strains).

Distortion of transverse cross-sectional geometry (racking)
From the tender design analysis it was clear that based on a conservative and simple analysis the racking effects would require a lot of additional reinforcement. A more detailed numerical analysis to reduce uncertainties is therefore performed in the detailed design.
The racking analysis is carried out using 2 approaches, which are clarified below.

1. *Site response analysis using SHAKE 2000 (Ordóñez, 2004):*

This approach aims at the determination of the seismic induced displacements of the ground at tunnel depth for a selected number of design profiles. These design profiles are characterised by specific combinations of tunnel depth and soil profile. In this approach it is considered that the tunnel-element follows directly the motions of the ground; the soil-tunnel interaction as such is not included. The established displacements of the ground at tunnel depth are used as 'pre-scribed' displacements in the subsequent structural analysis.

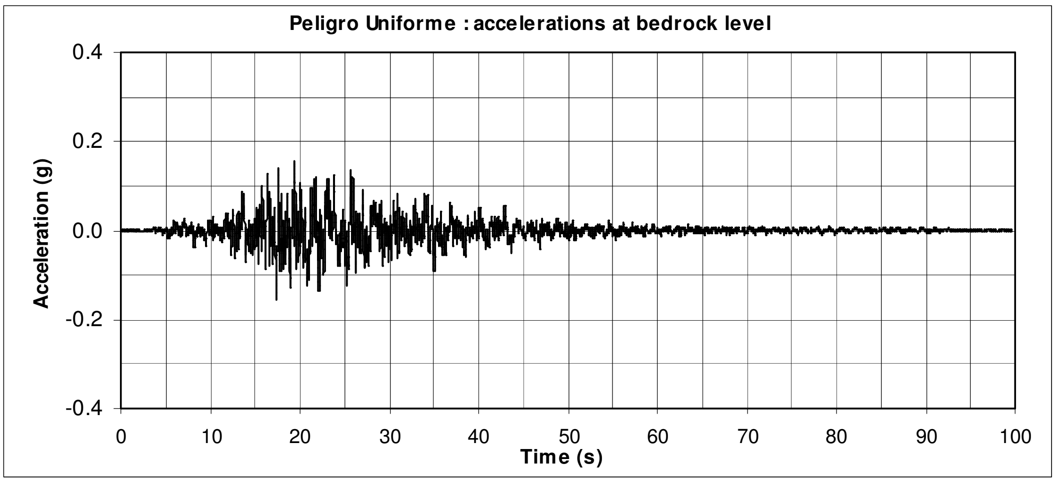


Figure 6: Representative input signal for SHAKE analysis

2. Dynamic soil structure analysis using PLAXIS (Plaxis, 2007):
The dynamic soil structure interaction analysis (between tunnel and surrounding soil) is assessed by a two-dimensional plane-strain analysis. The calculated displacements of the tunnel, considered in a transverse section, are used as input, e.g. as 'pre-scribed displacements', in the subsequent structural analysis of the tunnel-element.

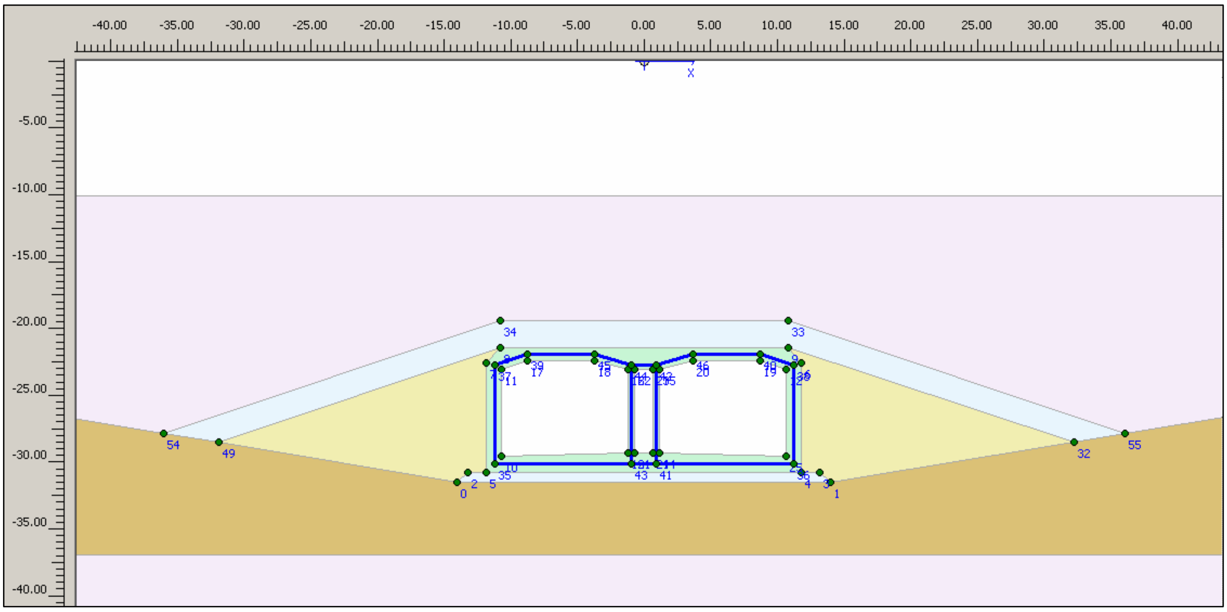


Figure 7: An example of the tunnel profile as modelled in PLAXIS

SEISMIC INDUCED GEOTECHNICAL GROUND FAILURES

Liquefaction
Liquefaction is linked to the changing effective stress in the soil under seismic induced (cyclic) strain. Liquefaction can occur only if the soil is fully saturated with pore fluid and the loading (or strain) is applied so quickly that the developed excess pore pressure, if any, cannot be drained effectively away. The loading is considered (largely) undrained.
A soil is considered to be in a 'liquefied' condition, if the seismic induced strain results in a 'close-to-zero-effective stress condition'. In this case most of its shear strength capacity is lost.
The liquefaction analysis is carried out using the procedures as described in NCEER (Youd et al, 2001). This simplified procedure elaborates further on the procedure known as the 'Seed and Idriss procedure', supported by liquefaction susceptibility criteria based on grain size, liquid limit and/or natural water content (Seed et al, 2001).
Seismic induced volumetric strain, or seismic induced settlement, are assessed from empirical correlation between safety factor against liquefaction and cyclic induced volumetric strain, for various relative densities (Ishihara, 1997).
The results of the liquefaction analysis showed, that the soil layers under the base of the immersed tunnel are sufficiently resistant to liquefaction. The (natural) soil layers above the tunnel roof are susceptible to liquefaction. This has however no consequences for the foundation of the tunnel.

Discrete displacements along geotechnical features like faults
According to the site specific Seismic Hazard Analysis (IIE, 2007) geological features like faults do not appear at the tunnel location.

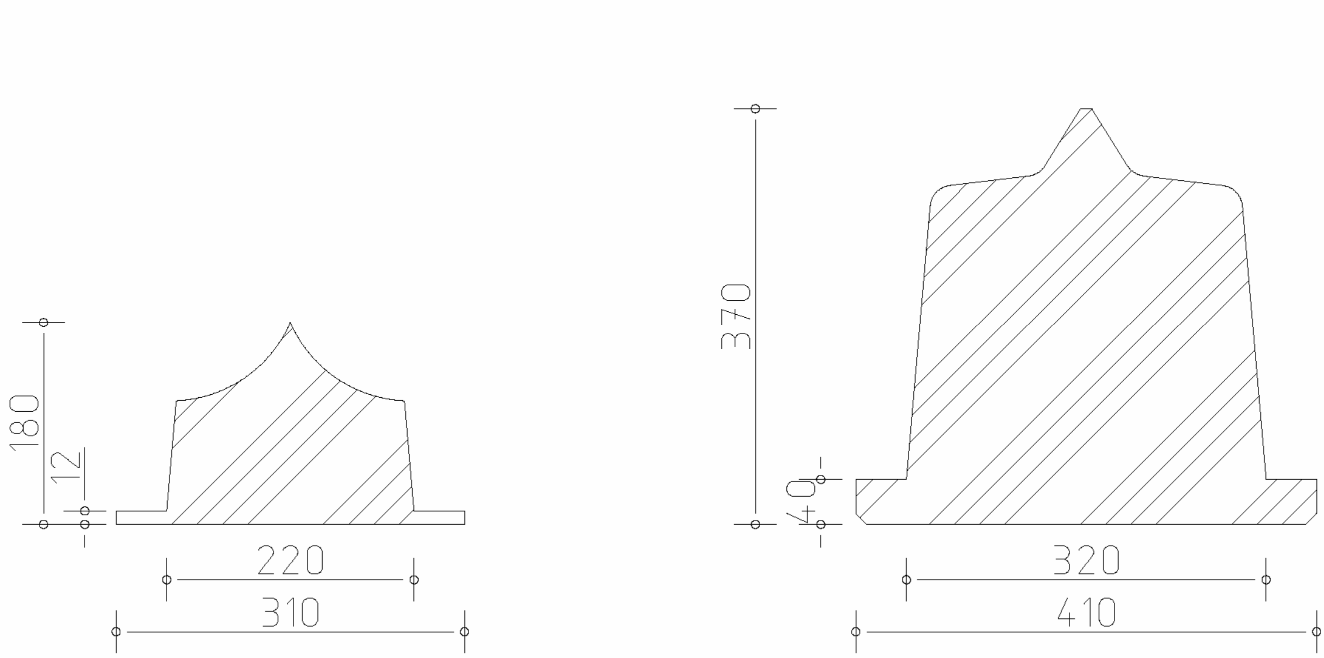
Foundation failure and/or slope instability under the seismic induced load.
To prevent liquefaction of the foundation layer a gravel bed has been chosen for the foundation of the immersed tunnel. For the construction period seismic effects are not considered, so the slope stability under seismic loading is not investigated for the immersion trench.

DESIGN CONSEQUENCES FOR THE IMMERSED TUNNEL

The poor soil conditions in combination with the seismic hazard had a large impact on the design of the immersed tunnel elements:

- special segment and immersion joint details shall limit the damage caused by a seismic event and allow for easy repair after a major seismic event
- permanent longitudinal prestress is applied
- heavy shear keys are required in the inner and outer walls at the immersion joints
- a gravel bed foundation is applied to prevent liquefaction of the foundation layer
- additional reinforcement for the racking effects is required.

Immersion joints
The element length of 138 meter is in fact a compromise: deformation and rotation at the immersion joints as well as the amount of prestress required can be limited by using short elements. Short elements do however require more immersion operations. The analysis resulted in the requirements for the flexible immersion joints (Gina profiles). It appeared that no existing profile met these requirements, so in cooperation with Trelleborg-Bakker new much larger rubber Gina gaskets are developed specifically for the Coatzacoalcos Immersed Tunnel (see figure 8).



Common Gina gasket in Dutch tunnels **New, Coatzacoalcos Gina gasket**

Figure 8: Gina gasket

The installation of these larger Gina gaskets made it necessary to increase the outer dimensions of the tunnel at the location of the immersion joints. At both ends of each tunnel element a 200 mm thick collar is applied all around (see figure 9).
The specific tunnel seals (Gina and Omega) guarantee water tightness of the immersion joints. To provide additional security against leakage of an immersion joint during a seismic event 10 to 16 passive tendons are installed over the immersion joints. The tendons shall first be activated as the

minimum compression of the Gina gaskets under normal conditions is reached. The activated force in the tendons shall compensate the loss of compression force in the Gina gasket due to further relaxation. In this way at least the minimum compression force acting in the Gina gaskets under normal conditions during lifetime will be kept in the immersion joint also during a major seismic event.
The longitudinal prestress in the tunnel elements is permanent and will not be cut at the segment joints. Cutting of the prestress at the segment joints, as is usually done in western Europe, is not possible as this would require very large deformation capacities for the seals at the segment joints to maintain water tightness during an earthquake.
The consequence is however that to prevent differential displacements of the individual 138 meter long continuous tunnel elements heavy shear keys are required in both outer and inner walls at the immersion joints.

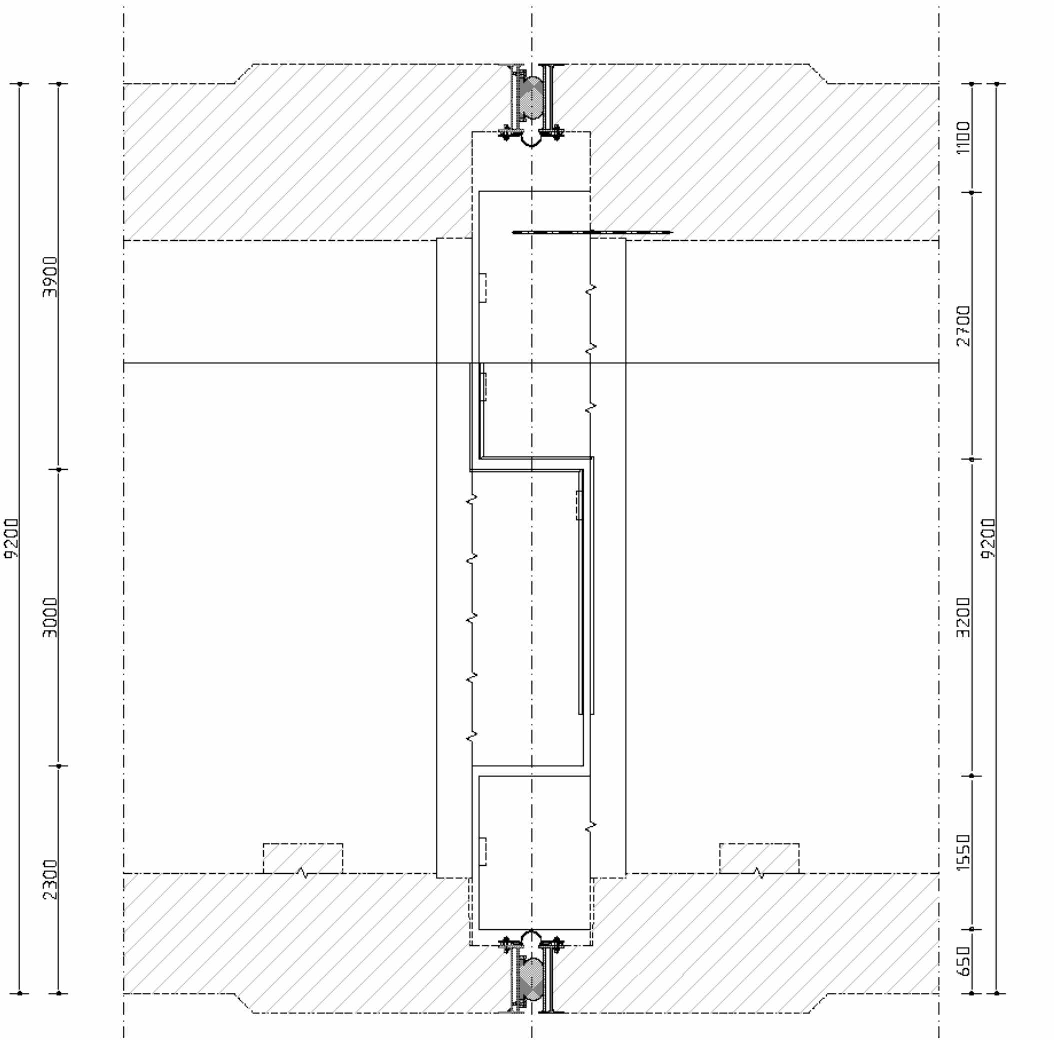


Figure 9: immersion joint with shear keys in the wall

Segment joints
Under the seismic conditions tensile stresses up to 2.0 N/mm^2 are accepted in the structure. In general the concrete can take these stresses. However, tensile cracks can not be completely excluded. Crack inducers at the location of the segment joints shall force such cracks to occur at the location of the segment joints. The installation of an injection hose at the location of the segments shall allow for repair of such cracks.

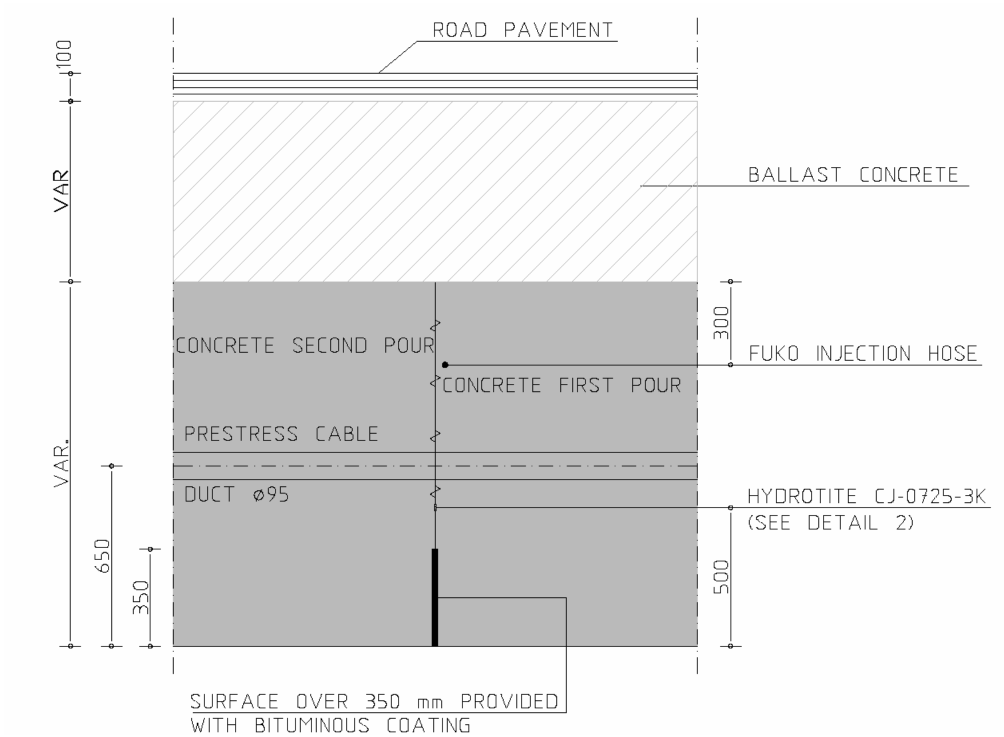


Figure 10: segment joint with crack inducer (bituminous coating) and injection hose

Permanent longitudinal prestress
The tensile forces in the concrete are defined by friction with the surrounding soil and by the immersion joints. With the chosen joints the latter one is of minor importance. Governing stresses are determined mainly by the backfill on top of the elements. The tensile forces resulted in required prestress levels between 0.5 and 2.2 N/mm^2 for the floor and roof slab of the elements, mainly depending on the maximum backfill on top of the roof.

Distortion of transverse cross-sectional geometry (racking)
The predicted maximum free field racking displacements from the SHAKE 2000 analyses vary between 40 and 80 mm.
Secondly, a racking analysis was made using PLAXIS finite element software, using stiffness parameters based on the outcome of the SHAKE 2000 analysis. The Finite Element calculation includes the strain / stiffness behaviour of the tunnel element.
The finite-element analysis show that the racking displacements are in the order of 15 to 20 mm, which is about 55% of the result of the analytical approach used for the tender design (the analytical approach resulted in a racking deformation of 27 to 35 mm).
In the detailed structural analysis finally a differential displacement (roof versus floor) of 20 mm is used, which resulted in a considerable reduction of the required reinforcement (up to almost 50% at some locations) compared to tender design analysis.

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