THE GIBRALTAR TUNNEL

Project presentation and challenges

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1. LECTURE ORGANISATION

The project considered, from its history to the actual aspects of the present design stage, is very complex.

As it will be illustrated in the following, the preliminary design of the Gibraltar tunnel (actually the underground solution for the fix link through the Strait) was completed in 1996 but its revision became necessary because of new insights on the geological conditions.

This lecture will summarise some aspects of its long history (the last few decades out of a very long history, known since the ancient Greeks) as well as of one of its most relevant technical issues, geomechanics.

2. THE CHALLENGE

The Gibraltar tunnel represents a challenge in several respects.

First of all, the idea itself of connecting two continents. But also from the technical point of view, several other aspects of the project go well beyond the existing experience.

Many have the tendency to compare the Gibraltar tunnel with the Eurotunnel. Although the importance of the Eurotunnel (source of inspiration for the Gibraltar tunnel), the technical difficulties faced and solved as well as the knowledge acquired during its design and construction are well recognized, the Gibraltar tunnel imposes new challenges to the engineers, now and for some more years:

- Deepest tunnel under the sea level designed at present (tunnel depth and seabed depth).
- Largely unknown geological conditions, nearly impossible to be investigated in more details with marine drillings (which actually constitute an important disturbance to the tracing).
- Very weak breccias, more similar to a hard soil than to a weak rock.
- Extremely severe stress conditions at the tunnel’s elevation, with the pore pressure higher than the effective stress.
- Extreme operating conditions for the TBM, beyond the characteristics of the machines currently available on the market.
3. THE PROJECT AREA

From time immemorial the Gibraltar strait represents the ideal occidental crossing point between the two continents, Europe and Africa.
The existing intense naval traffic (both of goods and persons) is an evidence of it.
Here the main connection axes through Europe and the North of Africa find a natural link through the Strait (Figure 1). The missing connection in Gibraltar is evident.

4. A BIT OF HISTORY

4.1 The Origin of the Name

The rock of Gibraltar (which gave its name to the Strait) owes its name to the Berber general Tariq ibn Ziyad, who led the first invasion of the Iberia (Spain), as vanguard of the main Moorish force, in 711 BC. Originally two names were used: Jabal Tariq (mountain of Tariq) or Gibel Tariq (rock of Tariq).
The dispute (not yet settled) between Spain and the United Kingdom because of the perpetual sovereignty on Gibraltar, ceded from Spain in 1713 under the Treaty of Utrecht, is well known.
Several battles took place here since the “Eighty Years’ War” (1568-1648) until 1782, as a result of a Spanish siege, regularly stating the superiority of the British troops.
This also testifies the strategic value of the Strait and the blocking Rock.

4.2 The Proposals by the Architects

The idea of a fixed structure for crossing the Strait hit the mind of some architects at the beginning of the 20th century.
Their solutions represent a mix of art and technique. The artistic touch is more striking than the engineering.

In 1930, the German architect Herman Sörgel developed a concept, in a certain way exotic, pharaonic and unattainable (Figures 2 and 3). Nothing less than a dam crossing the Strait not far from the narrowest point, including a channel for boats and ships and a power plant.
This conception was actually developed to a certain degree of accuracy, including considerations on the protection from the strong tides of the Atlantic Ocean. This would have provided furthermore a significant level difference between the ocean and the Mediterranean Sea, which could have been exploited by means of a very impressive power plant (68 millions PS), and the possibility of winning a significant extension (approx. 660’000 km² by a level lowering of 400 m) of land for further use (agricultural, residential, etc.).

As a lateral consideration rising to the mind of an engineer, no information is available about
– how the volume of the 400 m water depth could be expelled from the Mediterranean sea and
– how the inflow of all the rivers could be diverted
in order to avoid larger expenses in pumping the waters than the profit arising from energy production.

Although high, surely the evaporation is not sufficient to allow for a positive balance. Without mentioning aspects like:
– the evaporation balance by the smaller surface (approx. -30%),
– the impact on the existing coastal environments,
– the impact on the currently submerged biotopes,
– the fundamental impact on the navigation network through the Mediterranean sea.

As a further example, the American architect Eugene Tsui developed a revolutionary design (Figures 4 and 5), which does not resemble any existing bridge and features an original floating and submerged concept, based on the creation of a three mile wide floating island in the middle of the Mediterranean Sea. It features about 2 x 14.5 km floating bridges, contains 150 windmills and 80 underwater tidal turbines generating 12 billion kilowatt hours of electricity. Windmills and turbines can be easily added to the design as needed. The bridge can generate enough electricity to power the southern Spanish province of Cadiz and the entire nation of Morocco, making it the largest wind and waterpower farm in the world.

The bridge is designed to float gracefully on and under the water like a giant, elegant serpent anchored to the cities of Tarifa, on the southern coast of Spain, and to Point Cires, on the northern tip of Morocco.

As usual, we skeptical engineers would think (as for example) at:
– the extreme and regularly inversing current up to 6 knots,
– the lateral and slanted waves,
– the tides,
– the military submarine traffic,
– the watertight joints at the connection between land and submerged sections,
– the threats from sabotage or terroristic attacks and the necessary protections.

5. THE PRESENT TIME

In the very recent times, the governments of both facing countries, Morocco and Spain, decided to give new impulse to the study of a fixed link through the Strait. Two companies ("Les Sociétés") were founded according to the co-operation agreement between Morocco and Spain "Accord de Coopération Technique et Scientifique" of 1980.
– SNED (Société Nationale d’Etude du Détroit, Moroccan) et
– SECEG (Sociedad española de Estudios para la Comunicación fija a través del Estrecho de Gibraltar, Spanish).

These companies are since their foundation responsible for the study of a fixed link crossing the Strait.
The studies were focussed in two main aspects:
– The traffic across the Strait
– The design of this project in all its technical aspects.

6. THE ALTERNATIVES

6.1 The Bathymetric Features

The initial activity related to the design itself was the study of the bathymetric features of the Strait. Some of the findings concerning the seabed are clearly understandable from a hydraulic point of view:
– The narrower the Strait, the deeper the water
– The current is very strong at the centre but diminishes significantly (by one order of magnitude) near the shores.

The maximum depth in the Strait is approx. 900 m, between Gibraltar (and Algeciras) and Ceuta, where the Strait is nearly 14 km wide. At the narrowest point, the depth is more than 600 m.
The minimum depth is approx. 300 m between Tarifa and Tanger, along the underwater ridge visible on the western side, named "Le Seuil".
Such depths lead initially the Companies to studying the bridge solution.
6.2 The Bridge Solution

Two different routes were identified (Figure 6):
- The shortest one (with a depth of up to 900 m), named "Route du Détroit"
- The one with the shallowest seabed (approx. 300 m but more than twice in length), named "Route du Seuil".

After abandoning the idea to cross at the narrowest point ("Route du Détroit"), because of the excessive water depth, the most recent studies of the bridge solution focussed on the shallowest zone ("Route du Seuil") (Figure 7 and 8). The limits of its feasibility were reached:
- Extreme length, nearly 5'000 m, of the main bridge spans.
- Extreme height of the pillars, with 315 m depth and 485 m height, for a total height of the pillars of approx. 800 m, foundations excluded.

For comparison, we can mention that the highest skyscrapers are currently 508 m (Taipei) and 452 m (Kuala Lumpur) high and their service loads are fundamentally different.

The bridge was abandoned also because of other reasons, such as:
- wind loads
- risk of collision by a ship (the traffic through and along the Strait is impressive)
- extreme vulnerability in case of malicious hits.

6.3 The Tunnel Solution

So the "Sociétés" led to the only alternative: if not over the Strait, then under it.

6.3.1 The Tunnel

The zone where to place the tunnel becomes obvious by thinking that ramps are needed for descending under the seabed and regaining the surface and that the railway technology requires to limit the slope: every additional meter of tunnel depth requires an additional tunnel length of 33 to 50 m on every side (the ramps being 2 to 3%).

A narrow but very deep cross-section is not well suited. A balance between depth and width is required. These conditions are met along the "Route du Seuil" (Figure 9).
6.3.2 Exploitation Phases

As mentioned before, the design is inspired to the Eurotunnel. The cross section (Figure 10) includes two railway tunnel and a safety and service tunnel. Because of considerations on the traffic intensity and financial planning, the realisation is subdivided in phases, with different exploitation schemes. This scheduling is a binding condition for the tunnel design.

This leads to a particular organisation of the time schedule during the first phase, when only one of the railway tunnels is available together with a safety and service tunnel, with train groups moving back and forth between the two terminals. Large terminals are required, in order to allow the arranging and loading of the whole group of the trains at once, in order to allow for short departure intervals.

The final realisation phase, with the completion of the second railway tunnel, allows for unidirectional circulation and for simplifications of the stations’ layout, as the contemporary loading of a whole group of trains is not required anymore.

All these issues (train features, terminal layout, loading and unloading sequences, maximum allowable slopes) are included in the study but they will not be treated further in this paper. They nevertheless significantly contribute in approaching the Gibraltar tunnel to the limit of feasibility in several technical fields.

7. GEOLOGY

7.1 General Asset

Within the project area, the geology of the Betic (Spain) and the Rif (Morocco) chains is formed by a very complex system of flysch plates interrelated by folds and overthrusts, resulting in a continuous and irregular alternation of different facies of the regional flysch.

Figure 11 shows, as an example, the Beni Ider - Algeciras formation (Moroccan and Spanish definition of the same continuous facies, crossing the Strait).

A mostly sandstone flysch is the Tisirène-Nogales formation, while impressive sequences of folded banks of multi-coloured marls and rigid sandstones are visible at outcrops near Algeciras.
7.2 The Investigatory Openings

These formations were studied by means of several boreholes in the Spanish and the Moroccan mainland and 3 large-scale investigatory underground openings:

− A trial pit in Bolonia, on the Spanish hills, with 80 m depth and 4 m diameter.
− The Tarifa trial tunnel, with 500 m length and 3.5 m diameter, excavated by TBM.
− A shaft system in Malabata (Figure 12), with two subsequent shafts, each 150 m deep and 3 m diameter (locally 5 m).

The trial pit of Bolonia was fully abandoned after its exploitation.
There are currently no activities on the Tarifa tunnel.
The shaft system of Malabata was re-opened approx. one year ago, showing some astonishing features, related to the characteristics of the rock mass, not considered at the previous design stage.

8. THE PREVIOUS TUNNEL DESIGN

The Preliminary design (Avant Projet Primaire or APP) was completed in 1996. This will be called APP96 in this paper. The Preliminary design was based on the assumption of geological continuity of the formations on both sides of the Strait. At that time, a detailed geological survey of the centre of the Strait was not available and the information about the conditions of the seabed and the rock mass underneath was scarce. The flysch formations were supposed to exist along the whole tunnel length and were subdivided into 4 different lithotypes.

The cross section chosen in the APP96 is similar to the one adopted for the Eurotunnel (Figure 10). At that time this was the only example (excluding the older Seikan tunnel in Japan) of a modern long submerged tunnel with significant depth.
The concept is well known:
− 2 railways tunnels
− linked by safety by-passes
− to a central safety and service tunnel.
This cross section is representative of the geomechanical studies carried out at that time. Thickness and tunnel distances were optimised based on the data available.
9. THE REASON OF THE PRESENT JOB

9.1 The Knowledge

Some statements by several experts at that time are significant for understanding how limited was the information, therefore leading to wrong conclusions when designing the tunnel.

“In conclusion, from a technical standpoint, there is little doubt that a Gibraltar Strait tunnel can be built.”

“Taking into account the available information, the construction of a tunnel under the Strait of Gibraltar is feasible. … A lining thickness of the order of 40 cm for the service tunnel and 60 cm for the railway tunnel will provide sufficiently safety if reinforced concrete segments are used.”

“The technique of drilling long tunnels in hard rock with TBMs is highly developed. … All necessary construction elements exist and are proven. They only have to be assembled.”

This is not an indication of lesser understanding or missing knowledge, but the direct consequence of the lack of information on the real conditions.

9.2 The Investigations in the Strait Middle

Between 1997 and 2005, new geological investigations campaigns were carried out, with several long boreholes realised at the centre of the Strait, in the area with maximum water depth. New techniques for getting really long boreholes in the extreme conditions in the centre of the Strait were applied. These campaigns were mostly conceived in order to complete the geological database along the central part of the route, as a basis for the next design stage.

The new insight resulting from these campaigns showed the necessity for the clients (“Les Sociétés”) to update the preliminary design.

Along with the investigations, the technique for drilling boreholes at such a depth evolved.

In order to reach higher depth below the seabed, the “Piggy-back with re-drilling” technique was finally applied and achieved the best results (Figure 13):

– A concrete block is lowered on the seabed, with a directing funnel for the drilling equipment.
The drilling rods were protected with hydrodynamic profiles (Figure 14) in order to reduce the swinging by rotating in the strong current.

The difficulties were enormous. Just for mentioning some of them:

- The current reaches up to 6 knots while the self-positioning devices of the drilling ships perform satisfactorily up to 4 knots. Therefore the drilling had to be interrupted or suspended after short time (operational time windows 6 to 10 days).
- The configuration of the seabed did not always allow for a suitable positioning of the "piggy".
- The seabed is anyhow up to 300 m under the drilling equipment.

9.3 Realised Boreholes

The refinement of the drilling technique improved greatly the knowledge of the conditions of the seabed.

The three last campaigns applied the piggy-back technique (Figure 15). The "Piggy-back with position recovery" technique was applied for the two last ones, when the deepest boreholes were achieved. The green spots show the position of the deepest boreholes.

9.4 Results of the Investigation Campaigns

The drilling gave surprising results.

Bioclastic limestone was found on the seabed, near the submarine mount in the centre of the Strait. Possible origin: fossil coral reef.

Underneath, sand of bioclastic origin under the limestone. Possible origin: contemporary erosion of the coral reef during its formation or transportation of older eroded coral reef before the new growth by sea currents.

And the TRUE REASON for the need for a design revision: THE BRECCIAS.

A chaotic mix of blocks, stones and pebbles in a heterogeneous clayey matrix, which can reach up to 60% of the whole.

The pictures (Figures 16 and 17) show the conditions of this material from a few tens of metres depth under the seabed until very deep (100 m and 200 m) under the seabed. The structure is consistently chaotic.

Is not yet known at what level such material could stop, but certainly below 600 m under the sea level. The studies carried out by the specialists tend to discard the Gibralt...
The Gibraltar Strait as the location of the contact between the African and the European tectonic plates. The mutual movement of the two shores is not considered in this preliminary design. The available information is not sufficient in order to focus the aspect and to solve it. It will be a further challenge for the next design stage and subject to detailed investigation.

9.5 The Conditions in the Malabata Shaft

Another characteristic discovered after the re-opening of the shaft: the flysch swells significantly if not confined (Figure 18 and 19).

10. THE GEOLOGICAL PROFILE OF THE REVISED DESIGN

10.1 The Geological Profile

The revised geological profile as resulting from the most recent investigations is shown in Figure 20. Note the high number of boreholes along the axis, most of them reaching only a few meters depth and in the flysch. The new drilling techniques allowed for the deep boreholes visible in the profile.

10.2 Genetics of the Breccias

The more credited genetic model of these breccias refers to the filling of the Mediterranean Sea. Its level was significantly lower than the one of the Atlantic Ocean. As the water began to flow through the Strait and excavated a deep incision, which is locally divided into two separate channels (the "sillons"). Once the level was equalised, subma- rine landslides occurred from the lateral slopes of the "sillons", filling them. In a later phase, the coral reef grew and bioclastic sand was deposited in a possible secondary erosion, with further formation of coral reef over it. How you can easily imagine, excavating within these breccias is a challenge, which will be examined more in detail in the following part of the lecture.
11. THE NEED FOR A NEW ROUTE

Within the design activities, the revision of the tracing was necessary. The presence of the deep boreholes along the tunnel axis, normally a desired condition for geological investigation, is negative in the present case. Their proper injection after drilling in the given working conditions isn’t warranted neither tested nor fully documented.

In such material as the breccias, the direct interception of an open borehole and the tunnel during excavation as well as a too short distance between borehole and tunnel would result in a too elevated hydraulic gradient through the rock. The uncontrolled break out of water in the tunnel would be catastrophic.

On the other hand, the geological information from a borehole is useful only up to a certain distance. Therefore the deviations from the boreholes cannot be very large. In this project, a horizontal distance of 200 m was estimated to represent the acceptable compromise, resulting in the new route (Figure 21).

Furthermore, other conditions limiting the possible corridor need to be accounted for, like the lateral slopes, where the depth of the Strait increases considerably.

A further criterion was the minimisation of the water depth along the axis, in order to increase as much as possible the effective stress towards the pore pressure.

In Figure 22 the schematic longitudinal profile, based on the best information currently available, is shown, including the simplified geological interpretation. The longitudinal slope is limited to maximum value of 3% because of the needs of train circulation.

The flysch formations are quite heterogeneous but, considering the nature of the breccias, the key questions about the tunnel concentrate de facto in the central zones, stretching for a total of only 2.8 - 4.8 km. The extension of these zones is still unsure. This uncertainty is another consequence of the difficulties met while drilling boreholes under such extremely difficult conditions.

The definition of the maximum depth of the route is related as well to the minimisation of the hydraulic gradient. A safe distance from the highly permeable sands and limestone is 100 m. At this distance, the hydraulic gradient is approx. 5, considered as bearable during the tunnel excavation.

The crossing of the delicate sections of the "sillons", within the sands or the bioclastic limestone, was excluded because requiring intensive grouting in severe conditions:

- Unknown lithological structure in the scale of the soil volume to be grouted.
- Back-pressure exceeding 40 bar.
12. SAFETY ASPECTS

The safety aspect was originally developed based on the fact that, at that time, the Eurotunnel was the more modern solution in the matter of ventilation and user safety. By-passes between traffic tunnel and safety tunnel were planned at a distance of 340 m, which allow the people to escape in a safe zone. They are closed with doors and the safety and service tunnel is pressurised, in order to prevent smoke penetration.

Recent experiences, ongoing projects and new regulations show the need for reviewing and integrating this scheme.

It is now out of question to operate an approx. 40 km long tunnel without a safety station for evacuation, user protection and intervention in case of fire.

Therefore, such a structure (named ZAS in the present case) was included in the centre of the tunnel, in addition to the already foreseen pattern of by-passes (Figure 23).

The ZAS is located in the middle of the tunnel, where it is expected to find the flysch of the Mount Tartesos.

The idea at the base of this choice, in line with the solutions adopted for the new long tunnels through the Alps (Gotthard, Turin-Lion, etc.) is that a burning train should be able to reach this location (placed in the lowest point of the tunnel) by its own means. Here the smoke can be extracted with maximum power and efficiency while the passengers can leave the train and attain a safe zone in the easiest way as possible, through additional by-passes realised at a lower distance.

The black section in the figure is the regular one, 8 m internal diameter, by-passes every 340 m.

The green sections are enlarged in order to accommodate larger platforms, for a faster and safer train evacuation.

The yellow bypasses are realised at shorter distance than usual (113 m).

The red tunnel is the smoke-extraction duct, formed by the preliminarily excavated exploratory tunnel (discussed later in the lecture).

Generally, during normal operation, the tunnel is ventilated longitudinally.
13. REALISATION CHALLENGE

In addition to these aspects, determinant for the layout of the whole structure, the construction technique is fundamental for such a tunnel.

The following considerations, related to the realisation of the tunnel (technique and geomechanical analyses), refer to the safety and service tunnel and to the smoke-extraction duct. This tunnel (realised at a higher level than the transit tunnels) will serve during the realisation phase as an exploratory tunnel (Figure 24).

The geomechanical analyses show that the final loading conditions are not critical for this tunnel, but the transient conditions between the excavation face and the supported tunnel are determinant.

The hydraulic conditions around the excavation face, where the pore pressure will initially be drained, are fundamental, jointly with the poor quality of the rock mass. In the present case, the designers assume that the tunnel will be excavated with a TBM allowing the control of the pressure acting on the face, a hydro-shielded or an EPB (earth pressure balanced). The latter is illustrated in the Figure 25.

This machine should exhibit several particular features.

The first one is the resistance to an acting pressure between 10 and 15 bar during the excavation and 20 bar under static conditions (current realistic upper limits by the manufacturing of such machines).

Another condition is the possibility to realise guided perforations (Figure 26) in order to drain an annular zone around the tunnel far beyond the excavation face. This guiding is very effective because it allows to drain in a limited zone around the tunnel. Otherwise, the drainage will take place within a very large cone and will strongly reduce its efficiency because of the very low permeability of the breccias.

Further aspects have to be investigated and implemented in the final layout of the machine, such as:

- the grouting system of the annular gap between precast ring and excavation,
- appropriate means for transferring the pushing forces engendered by the 20 bar on the precast elements,
- appropriate means for facing possible infiltrations, localised but at high pressure.
14. GEOMECHANICS UNDER EXTREME CONDITIONS

14.1 Key Questions

A general overview of the performed computations is indicative of the approach followed in order to provide answers to some of the key questions. The analyses presented here consider the excavation of the safety and service tunnel as well as the excavation of the exploratory tunnel (both with internal diameter 6 m), which doesn’t interfere with the other underground structures. The need for an exploratory tunnel is discussed later.

Since the beginning of the computations, the need for getting the best possible answer to the following key questions became clear:

− Are the applied computation models suitable?
− Can be assured that no major problems will occur under the currently known conditions? In other words, is the tunnel actually feasible in all the predictable conditions?
− Are there limiting conditions to the achievement of the project?
− Is the available information sufficient for assuring the successful achievement of such a huge project?

14.2 Analyses and Assumptions

The analyses refer to the worst cases expected to be encountered, in order to allow for a clear answer to the key questions outlined above. The computations consider both the breccias as well as the clayey flysch, because it became evident that not only the strength of the material is determinant but also the permeability and the combination of both.

14.3 Determinant Conditions

The most critical conditions are met along the central section of the tunnel, with the highest total depth and where the seabed reaches its maximum depth (Figure 27).

The natural stress state is particular and never met in similar underground openings. The pore pressure $u$ (at 500 m depth) is more than double of the effective stress $\sigma'$ (200 m of residual soil laying over the tunnel):
- \( u = 5'000 \text{ kPa} \) (500 t/m\(^2\))
- \( \sigma' = 200 \cdot (2.2 - 1.0) = 2'400 \text{ kPa} \) (240 t/m\(^2\))
- \( \sigma_{\text{tot}} = u + \sigma' = 7'400 \text{ kPa} = 7.4 \text{ MPa} \).

In order to assess the real stress states, the coupled conditions "effective stress - water pressure" shall be considered in the computations. The percolation forces are determinant for several of the combinations analysed.

In the present case, it is not appropriate to perform analyses in total stress conditions.

### 14.4 Hydraulic Conditions

Because of the very complex hydraulic conditions around the tunnel and the very low permeability, the analyses were initially carried out considering the two extreme drainage conditions:

- fully active draining (at the excavation perimeter or inside the plastic zone) and
- fully non-draining conditions.

This doesn't mean CU (undrained) or CD (drained) conditions, which are related to the stress and pore pressure conditions inside the rock mass, but the imposed boundary condition during the excavation.

The hydraulic conditions around the tunnel are complex and the permeability is very low (order of magnitude: \( k = 10^{-8} - 10^{-10} \text{ m/s} \)).

The *draining mode* (Figure 28) is representative for an excavation in a middle to permeable ground or for a very slow advance in an impervious material. In this case, the position of the drainage (near to the excavation perimeter or at the limit of the plastic zone) is significant for the percolation forces, dissipated near the tunnel or within the rock mass.

The *non-draining mode* is generally representative for a fast advance within a low-permeability ground. In this case, the percolation forces are maximum because of the very high gradient. The drop of the pore pressure, visible in the figure near the tunnel profile, is given by the expansion and fracturing of the material in the plastic zone, creating available volume for the water in the rock fissures and pores, allowing for its expansion and de-stressing.

### 14.5 Results of 2D Analyses in the Flysch

The computations were carried out using the code FLAC. Figure 29 shows the results for the flysch. The lines correspond to:

- green non-draining mode
- blue draining mode at the excavation perimeter  
- red draining mode at the limit of the plastic zone.

The two couples of curves represent the same tunnel conditions, at the face and far from it.

The difference lies in the initial tensile state:
- total for non-draining conditions
- effective for the two draining modes.

Considerations about the conditions in the flysch:
- The drainage is very effective for reducing the deformations.
- The drainage near the tunnel is less effective than at the limit of the plastic zone.
- A minimum support pressure of approx. 1.2 MPa (12 bar) should be assured in order to avoid excessive deformation.

14.6 Results of 2D Analyses in the Breccias

In the breccias, the results show a different behaviour (Figure 30), much more sensitive to the drainage because of the lower permeability, which produce high initial percolation forces, and the lower strength.

In non-draining mode the excavation seems feasible considering a high stabilisation pressure acting on the lining (of the order of 3-4 MPa) but a similar pressure should be provided already at the excavation face, which is far out of the limits of present experience.

In draining mode, the excavation seems not to be feasible, with the curves going into vertical asymptotes as soon the rock mass is de-tensioned. The reason is related to the very high loads acting towards the tunnel because of the percolation forces.

14.7 Conclusion of the 1st Analyses Phase (2D)

14.7.1 Flysch CLASS IV

- draining the rock mass → the excavation becomes possible
- without drainage → the required pressure is 2'000 kPa
  → the squeeze of the shield cannot be excluded
- the stability of the front in draining or non-draining conditions is not very different.
14.7.2 BRECCIA

- draining the rock mass → the required pressure is 1’710 kPa
  → the squeeze of the shield cannot be excluded
- without drainage → the tunnel is unstable, with inadmissible deformations
  (vertical tangent of the ground reaction curves)
- the stability of the front in non-draining conditions seems to lead to unfeasibility.

Remarks:
- UNCERTAIN RESULTS - LIMITS OF THE COMPUTATION METHOD
  (e.g. transition face-tunnel, TBM stop)
- NEED FOR MORE DETAILED NUMERICAL ANALYSES, REPRESENTATIVES OF THE
  INTERACTION GROUND-STRUCTURE IN HYDRODYNAMIC CONDITIONS, CONSIDERING THE
  SITUATION AROUND THE TUNNEL FACE
- NEED FOR 3D ANALYSES.

14.8 3D Model

The computations were carried out by using the code FLAC. The model is based on the
following elements (Figure 31):
- Rotational symmetry (the axis corresponds to the tunnel axis)
- The constitution laws can be expressed in polar coordinates or in a plane defining the
  generating surface of the 3D model (the blue surface).
- The excavation progress can be defined through a central thin cylinder (the tunnel),
  de-compressed (support of the face and the first tunnel meter provided by the TBM
  head and by the shield) or de-confined (excavation) and on whose surface the water
  pressure is fixed at 0 (drainage). This can be done repeatedly, in smalls steps
  (advance with drainage).

The mesh has 1’700 elements (Figure 32), with a denser distribution in the central zone
in order to obtain more detailed information during the computation.

The excavation is simulated by the current system, annulling of the resistance of the
material inside the tunnel.

The lining (precast elements) is introduced at a certain distance from the excavation
face by reactivating previously annulled (excavated) elements, changing their charac-
teristics from rock to concrete.
The boundary conditions refer to the pore pressure, the effective stress and the support conditions.

- The hydrostatic pore pressure is 10 bar (= 1 MPa) on the excavation face and along the shield, simulating the EPB or hydroshield conditions.
- The pore pressure at the model boundary corresponds to -500 m (50 bar = 5 MPa).
- The effective stresses correspond to 200 m below the seabed.
- No dissipation is considered because of the drainage during the advance.

The support conditions of the whole model simulate the displacement of such a large cylinder of rock extracted from the surrounding rock mass. Therefore:

- fix joints in radial direction along the rotation axis
- free displacements towards the axis (radial direction) of the borders perpendicular to it
- limited deformations along the TBM shield, with a geometry corresponding to its possible shape, from the excavation face up to the full contrast on the lining.

For the project, a total of 125 cases were analysed. Some typical results are shown in the following:

- case 65, breccias at feasibility limit (LF), \( v = 3 \text{ m/j, } k = 10^{-10} \text{ m/s, without drainages.} \)
- case 64, breccias at feasibility limit (LF), \( v = 3 \text{ m/j, } k = 10^{-10} \text{ m/s, with drainages.} \)

"At feasibility limit" means that, at the given operational conditions, the conditions for the realisation of the tunnel are still in the field of the available technology or in an expectable near future.

The most delicate aspect of these computations is the balance between the low strength of the rock mass, the dynamic conditions determined by the advance (which needs several iterations to find the equilibrium) and the drainage of the pore pressure (which needs to be constantly equalised by every advance step). That means a fitting choice of the computation step and complex routines describing both the mechanical and hydraulic behaviour of the rock mass. The computations are performed in coupled stress conditions, also considering the effect of the saturation degree on the deformation modulus once the rock mass is de-stresses (Biot Model).
14.9 Case 65

14.9.1 “z”-Stresses (Tangential)

Figure 33.
The re-compression zone at the limit between plastic and elastic zone, higher than the natural stress state $\sigma_0$, is visible.
The distortion at the borders should be disregarded. Nevertheless, the recompression is clearly present around the excavation after the passage of the face, caused by the rigid behaviour of the concrete lining (pre-cast elements).
This is due to the incomplete relaxation of the 3D stress state around the face, leading to the consideration that the 2D models are not fitting the realistic conditions in such extremely poor materials.

14.9.2 Pore Pressure

Figure 34.
We notice the decompression at the location of the shield, where the pressure drops from the natural 5 MPa to the 1 MPa of the hydroshield working conditions.
The dark-blue zone far from the excavation represents an increase of the pore pressure over of the natural value (5 MPa) due to the recompression of the rock mass in the tangential direction.

14.9.3 Extent of the Plastic Zone

Figure 35.
Large extent, interesting to be compared with the result of case 64.

14.9.4 Radial Deformations along the Excavation Line

Figure 36.
The position 0 on the x-axis is the location of the tunnel face.
- On the left of it, the rock is still in place.
- On the right, the excavation releases the deformations, until the tunnel profile is blocked again by the segment lining (erection and injection) at approx 10 m from the excavation face.
The gradient is steep. The variation of any parameter can produce highly different results. For this reason, a parametric study was needed and is presented below.
In this case, the deformation from the face until the equilibrium is achieved on the lining is approx. 80 cm, not compatible with the use of a TBM.

14.10 “Case 64”

14.10.1 Water Pressure with Longitudinal Drains

Figure 37.
This diagram shows the water pressure in case of presence of active drains ahead of the tunnel face. The simulation considers a drained extension of 25 m ahead of the excavation face. It is shown that, with the given permeability, the core within the drains is not yet fully drained before the excavation approaches.

The effect of such drainage is the consolidation of a ring around the tunnel. Therefore the effective strength of the material is mobilised.

The more realistic effect of the presence of several drains around the excavation was also analysed separately by means of a planar model simulating a cross section beyond the excavation face, where only the drains are effective.

Considering a circular line at the drainage diameter, it results (Figure 38) that only after 30 days the drainage is effective for a 50% reduction of the pressure around the whole draining perimeter, by \( k = 10^{-10} \text{ m/s} \).

The very low permeability doesn't allow to directly accept the efficiency of such a system as granted. This aspect should always be considered carefully.

With higher permeability, the effect of the drainage would be consequently faster. Therefore, the assumption of a fully drained length of 25 m is actually representative for a real drainage length of approx. 100 m ahead of the excavation face.

14.10.2 Extent of the Plastic Zone

Figure 39.
The effect of the drainage is evident in comparison to the previous computation. The extent of this zone is reduced, particularly in the zone of the drainage drillings.

14.10.3 Radial Deformations along the Excavation Line

Figure 40.
The position 0 on the x-axis is the location of the tunnel face.
- On the left of it, the rock is still in place.
On the right, the excavation releases the deformations.

The deformations are less than in the previous case 65, with approx. 30 cm radial deformation from the tunnel face up to the equilibrium on the lining.

14.11 Parametric Study

Because of the high sensitivity of the model to some of the parameters, a parametric study was carried out.

Definitions applied in the following diagrams:
- \( C_r \): the radial convergence at the face (pre-convergence).
- \( C \): the total radial convergence between the face and the considered point.

As a suitable maximal radial deformation from the excavation face to the lining (precast elements), 40 cm were assumed (\( C_{\text{max.,admissible}} \)). These consider:
- the over-excavation between TBM head and shield,
- the conical shape of the shield and
- the tail gap between shield and the outline of the precast lining.

Same conditions are valid in the flysch as well as in the breccias.

The reference advance rate is assumed with 3 m/day.

14.11.1 Results in the Flysch

14.11.1.1 Effect of the Drainage by Varying the Permeability

Figure 41.

The drainage is very effective in order to stabilise the behaviour of the excavation. Such a condition becomes fundamental for the project.

14.11.1.2 Stop of the TBM

The stop of the TBM is considered as a real occurrence during the advance. This condition was analysed in case of regular excavation, and for 1 and 2 weeks stop, both with effective or ineffective drainage system. Such situations become important when analysing the real exploitation of the TBM. The realisation of the drainage and other works can lead to a stop of the TBM.
In drained conditions (Figure 42), it seems that no significant consequences should arise until 2 weeks from the stop.

In undrained conditions the radial convergences are significantly higher (Figure 43). The red line simulate the longitudinal profile along a modern TBM, with over-boring tools, conical shaped shield allowing the better escape in case of squeezing and the step between the shield and the precast elements.

The model (which needed a specific programmed routine) simulates the contact with the possible shield (the red line) and corrects the applied forces in order to prevent the rock to penetrate into the shield.

The rock, squeezing the shield, could block the TBM totally. The power, torque and thrust of the machine should be sufficiently high in order to unblock the TBM after a stop, since the computed radial pressure in case of such stops amounts to approx. 2.7 MPa after 1 week stop and 3.2 MPa after 2 weeks.

14.11.1.3 Discussion

Higher permeability influences negatively the possibility to excavate and support the tunnel. Drains are needed. By lower permeability, the convergences remain small even in case of a less effective drainage.

By draining, the effect of the permeability is less sensitive and the deformation smaller.

In case of TBM stop, the pressure within the front chamber should be increased up to the order of 20 bar. The activation of drainages is recommended in this case.

14.11.2 Results in the Breccias

14.11.2.1 Effect of the Advance Rate

The effect of the advance rate is significant but its variation of the radial convergence strongly depends from the hydraulic conditions (Figure 44).

It is interesting to note that the increase in the advance rate brings the excavation, in both draining and non-draining modes, to behave in a similar manner. The time for consolidation becomes insufficient, so that the draining mode is no longer effective, behaving as the non-draining mode.
The conditions similar to the non-draining mode become more evident with the increase of the advance rate because of the shortening of the time available for developing the percolation forces.

In other words: the faster the advance, the less time is available for the development of percolation forces (decreasing the relevant convergences) in the non-draining mode but also the less effective is the drainage in the draining mode (increasing the relevant convergences), allowing the relevant curves to converge.

14.11.2.2 Effect of the Working Pressure

The effect of the working pressure is significant (Figure 45).

If drained, a working pressure at the tunnel face and along the shield higher than 1.5 MPa (15 bar) allows the excavation with limited radial convergences.

If possible (at this time no experiences available yet), a working pressure of about 3 MPa (30 bar) would allow to excavate and support the tunnel without a significant drainage.

14.11.2.3 Discussion

With a working pressure of 1 MPa (10 bar), the conditions are very tight, if not beyond the limits, in order to allow the excavation through the breccias.

A stop of the advance in these conditions should be considered as unacceptable.

The maximal radial convergence of 40 cm is achieved in many of the computed cases. A basic condition allowing the excavation and support becomes the drainage for at least 30 to 60 days before the excavation and a sufficient low advance rate, in order to let develop the necessary consolidation.

Another option is the increase of the working pressure, but the assumptions for the design are already at the limit of the present experience. Some manufacturers are approaching such high working pressures but none of them is yet working on projects of an equivalent scale. It'll be in the next techniques development.

14.12 Variation of the Rock Mass Strength

With the aim of a general overview over all the other identified rock types, the radial convergence at the face and from the face until the final equilibrium is represented as a function of the rock mass strength (Figure 46).
The difficulties actually foreseen in the breccias are visible, as the convergences strongly increase below the rock class V. The presence of such poor materials is the substantial difference in respect to the previous project phase (1996). The rock mass properties at that time (only flysch formations foreseen) were not supposed to be worse than the herein applied parameters for the rock mass class IV-V.

The main conclusions of this last simulation are of primary importance for the next phases of the project:
− up to class V (flysch) the deformations are not critical, for a TBM respecting the conditions posed by the analyses.
− This is true in draining as well as in non-draining mode.
− The geotechnical conditions of the APP96 are verified with this new approach as being suitable for the construction of the Gibraltar tunnel.
− The excavation through the breccia brings the risk of squeezing, with higher deformations than the admissible ones, if shields complying with the currently highest standards are used.
− Better conditions than the worse ones (named breccia LF), nearer to the currently assumed superior limits of the characteristics of this formation, would allow the excavation with a minor risk. The available information is not sufficient in order to exclude major problems by lower rock mass strength and low permeability.
− Generally, the computations are really sensitive to the variation of the rock mass property, especially approaching the lower strength.

15. FINAL CONSIDERATIONS

15.1 Complex Analyses

I would like to point out that the geomechanical analyses carried out in this case are much more extensive than the ones generally carried out at a preliminary design stage, but are strictly necessary in order to assess the feasibility in such extreme conditions. The reason is the presence of the breccias, which imposed a much detailed analysis of both the conditions related to the rock properties and to the possible working conditions of the TBM. The analysis needs furthermore to be carried out in terms of effective and total stresses as well as under dynamic conditions (excavation and drainage).
In our opinion, this is the only effective way to get a picture of the possible behaviour of the excavation and support of a tunnel under such extreme conditions. The analyses and the related considerations are focused on the exploratory tunnel. The investigations carried out in this design phase focussed on the worst conditions expected, the crossing of the breccias at the maximum tunnel depth and with the maximum sea depth. In order to understand the realistic stress states, the coupled conditions "effective stress - water pressure" shall be considered for the computations. In the present case, it is not appropriate to perform analyses in total stress state.

15.2 Design Characteristics

– Long-term loads
  • Water pressure (5 MPa)
  • Minimum effective stress
  • Note: swelling is not yet considered
    → As a working assumption, the whole total load of 7.4 MPa is considered as acting in the long term on the final lining.
– The percolation forces are significant and essential for the stability of the plastic zone during excavation.
– Effects of the drainage beyond the excavation face around the tunnel:
  • Reduction of the percolation forces near the cavity
  • Stability increase
  • Reduction of the deformation after the excavation.
– Effect of a stop of the TBM:
  • Risk of squeezing during the time required for executing the drainages if the confining pressure is not sufficiently high.
– Two different advance concepts can be selected depending on the characteristics of the rock mass (strength as well as hydraulic properties):
  • a slow and drained excavation (which could require extreme drainage time)
  • a fast and un-drained excavation (which could on the other hand face the higher risk of squeezing, once an unforeseen stop occurs without the consolidation procured by the drainage).
15.3 Open Questions

– Very tight or even missing margins, force the engineer to be extremely careful while assessing the feasibility of such tunnels.

– An exploratory tunnel is extremely useful, because it provides the following advantages in comparison with other strategies:
  • Geological investigation along the axis without the risk of siphoning the water of the sea in case of boreholes too near to the tunnel axis.
  • Possibility to realise short boreholes along the tunnel and laterally with short investment as from the sea surface.
  • Possibility to realise plenty of investigations impossible from the sea (one for all: piezometer).
  • Benefits from a direct, 1:1 scale experience and possibility of optimisation for excavation of the main tunnels.
  • By means of a careful choice of the route, this tunnel will be independent of all other part of the Gibraltar tunnel.
  • During exploitation this tunnel will be used as smoke-extraction duct from the secured zone in the tunnel middle (the ZAS).

The Clients ("Les Sociétés") are considering initiate the final design of the exploratory tunnel and of the related geological and geotechnical investigations. The discussion on the technical, economical and strategic considerations will continue for some time.

16. MEMENTO

– Some statements by several experts at the time of the previous design stage are significant for understanding how limited was the information, therefore leading to wrong conclusions when designing the tunnel. This is not an indication of lesser understanding or missing knowledge, but the direct consequence of the lack of information on the real conditions.
  
  We must never give as known a complicate condition and search for possibility to improve the available information.

– The traditional analysis method in terms of total stresses is not sufficient. the analysis should be performed considering the effective stresses and the water pressure.
results obtained using 2d approaches are uncertain - limits of the computation method (E.G. TRANSITION FACE-TUNNEL, tbm STOP) need for more detailed 3d analyses, representative of the interaction ground-structure by hydrodynamic conditions, considering the situation around the tunnel face.

Exceptional conditions, as higher pore pressure than the effective stress, need new computational approaches.

– Very tight or even missing margins oblige the engineer to be extremely careful before to assess the feasibility of such a tunnel.

   The advantage of an exploratory tunnel should be carefully evaluated in terms of geological and geomechanical knowledge but also for the optimising of the excavation and support methods.

– Because of the need for large scale and expensive investigations, is it still interesting or appropriate to proceed further with such a design

   ... or a dream?

   we believe yes

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The Partners
– TYP SA (Spain)
– GEODATA (Italy)
– INGEMA (Morocco)
Figure 1
Figure 4

Figure 5
Figure 6

Les possibles routes de la Liaison Fixe.

Figure 7

Profils du fond marin de la Route du Seuil (AB) et de la Route du Cañon (CD)

Figure 8
Figure 9


Figure 10
Figure 11
Figure 13

Figure 14
DEEP BOREHOLES

Figure 15

Figure 16
Figure 17

Figure 18
Figure 19

Figure 20
FIGURE 23

Smoke aspiration duct

ZAS
"Zone d'Arrêt sécurisée"

Regular section
By-passes every 340 m

FIGURE 24
Figure 26
Figure 27

Figure 28
Figure 29

Figure 30
Figure 31

Figure 32
Figure 33

Figure 34
Figure 35

Figure 36
EFFECTIVENESS OF THE DRAINAGE SYSTEM BY $k = 10^{-10}$ m/s

Flysch IV - Evolution de la pression à 10 m du centre
The Gibraltar Tunnel - Project Presentation and Challenges

Figure 39

Figure 40
FLYSCH IV – VARIATION OF THE PERMEABILITY

Figure 41

FLYSCH IV – EFFECT OF A TBM STOP WITH ACTIVE DRAINS

Figure 42
FLYSCH IV – EFFECT OF A TBM STOP WITHOUT ACTIVE DRAINS

Effect of the TBM stop leaning on the shield

Figure 43

BRECCIAS – EFFECT OF THE ADVANCE RATE

Effect of the drainage

Figure 44
BRECCIAS – EFFECT OF THE WORKING PRESSURE

Figure 45

VARIATION OF THE ROCK MASS STRENGTH

Figure 46