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Submerged Floating Tunnels: A review and study of their use for strait crossing.

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Abstract

Submerged floating tunnels (SFTs) are innovative structural solutions to waterway crossings, such as sea-straits. As the width and depth of straits increase, the conventional structures such as cable-supported bridges, underground tunnels or immersed tunnels become uneconomical alternatives.

In this thesis, we will understand how Submerged Floating Tunnels work, how they are designed and constructed, find advantages and disadvantages and, finally, when all that is clear, we will evaluate the possible application of this type of crossing on the Chacao Channel. All this in comparison with a much more common structure such as a long span bridge, in this case a suspension bridge.

In order to assess the viability of the SFT in the Chacao Channel, a multi-criteria analysis has been chosen to evaluate both construction processes in an objective and systematical manner. This analysis has evaluated and compared, both construction processes integrating diverse aspects such as costs, environmental impact, safety, social impact and functionality.

The results obtained for both structures have been considered very similar, being 0.78 for the Suspension Bridge and 0.80 for the Submerged Flotation Tunnel. In conclusion, both the construction of an SFT and an SB in the Chacao Canal is feasible.

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1. Introduction and Objectives

1.1. Introduction

The water crossings structures often give an optimal solution for achieving the minimum travel time and infrastructure length between two areas separated by water. Normally, in these cases, the terrestrial travel time (if it's possible) or boat crossing tends to be very long. Therefore, when the connections want to be more effective, analysing this type of structures, is of special interest.

Nowadays, the Submerged Floating Tunnel (SFT) is a new crossing solution under development and which concerns many countries. This infrastructure is considered to be a suitable solution for wide and deep water crossing and is thought that it will be able to offer access to places until now unthinkable. Apart from that, this solution has many advantages against conventional structures. SFTs are environmentally friendlier than conventional structures and cheaper after a certain distance.

Chiloe island is located in the southern of Chile and the Chacao Channel separates it from Chile mainland.
The connection between the island and mainland is by means of ferry. The ferry is known for being a transport mean which does not require any infrastructure, a part from docks for satisfying their services and so there is no need for big inversions. However, the ferries also have some disadvantages. On the one hand ferries depend on the weather and their services can't be always operative. On the other hand, the ferries velocities are slow compared to other transport means. As a result, the construction of a new suspension bridge has been approved.

The main objective of this thesis is to study the feasibility of an SFT in the Chacao Channel in order to see if it would have been a more viable solution for the new crossing. Firstly, in the second chapter the concept of a Submerged Floating Tunnel has been introduced followed by the explanation of the different design and construction methods for a SFT and a suspension bridge. Different similarities can be appreciated.
Chapter three evaluates and identifies advantages and disadvantages with the construction of both in different locations.
Finally, in chapter four the site conditions of the Chacao Channel are analyzed in order to choose a viable location for situating the proposed SFT. Once the location is chosen, a comparison between sites with similar conditions to proof its viability has been done. With the viability of the site, a suitable SFT design has been proposed and finally an objective analysis between the SFT and the SB has been carried out to evaluate their global impact and conclude which alternative is more feasible. It is interesting to notice how all the chapters of the thesis project are linked at the four chapter of the work.

1.2. Objectives

The objective of this thesis is to understand how Submerged Floating Tunnels work, how they are designed and constructed, find advantages and disadvantages and, when all that is clear, to see a possible application of this type of crossing on the Chacao Channel. All this in comparison with a much more common structure such as long span bridge, in this case a suspension bridge.

2.State of the Art

2.1. Submerged Floating Tunnel

The submerged floating tunnel (SFT), also named as Archimedes Bridge, is a novel way of crossing water beneath its surface. Unlike conventional immersed tube tunnels, a SFT is not an embedded structure, but instead is suspended above the sea floor, anchored by a support system such as pontoons on the surface or by anchoring to the seabed. It consists of one or more prefabricated hollow tunnel elements constructed in the dry at a location, which is not their final location. Their final location is somewhere between the surface and the bed level, surrounded by water as mentioned before.
SFT can provide a more economical way of crossing a body of water in comparison with an undersea tunnel or a suspension bridge, depending obviously on the local sea characteristics, (depth, traffic etc..) and hydrographic conditions.
Nevertheless, even though the concept of SFT has existed for many years and that there are many site studies and proposals, none have been constructed yet, probably due to the total lack of experimental data on the actual behaviour of the SFT, both in traffic and in environmental actions.
Submerged floating tunnels have applications not only for road and rail traffic, but also for use as pedestrian tunnels and service tunnels as well.

Even though, a submerged floating tunnel has never been built yet, several proposals have been presented by different entities.

1. English Channel, United Kingdom.
2. Strait of Messina, Italy.
3. Høgsfjorden, Norway.
4. Transatlantic tunnel (between North America and Europe) Atlantic Ocean.
5. Funka Bay, Japan.
6. Lake Washington, Seattle, United States.
7. Vancouver Island, Canada.
8. Lugano Lake, Switzerland.

2.1.1. SFT Classification.

SFTs can be classified according to their anchorage system.
There are four types of SFT (CEP, Civil engineering portal).

a. SFT with pontoons.
b. SFT supported on columns.
c. SFT with tethers to the bottom.
d. Unanchored SFT.

a. SFT with pontoons.

This type of SFT is sensitive to the wind, waves current and possible ship collisions. On the other hand, this system is totally independent of the water depth, and so its use could be beneficial in places where depth makes impossible the foundation construction and long crossing distances prevent the construction of bridges without support points in the middle of the crossing. In addition, the construction of underground tunnels would be unthinkable.

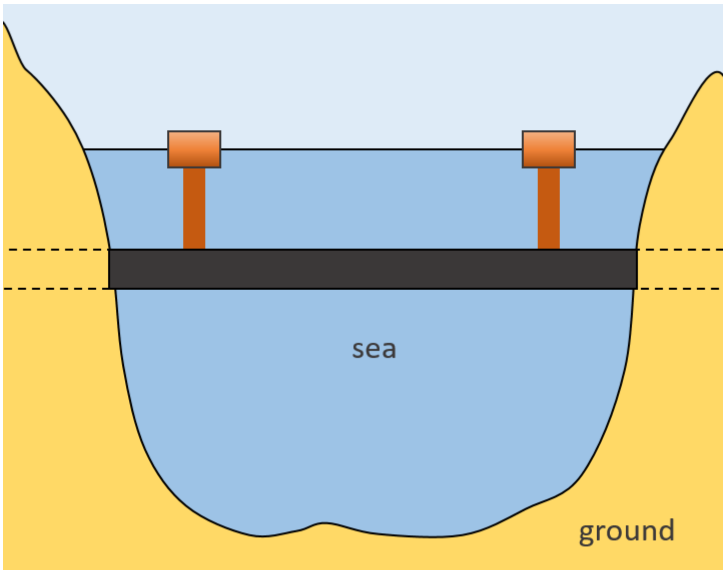


Figure 2.1 SFT supported by pontoons.

b. SFT supported on columns.

Some journals have referred to this type of SFT as an "underwater bridge", as columns attached to foundations, support this SFT. These columns can be either in compression or in tension depending on the buoyancy ratio (explained on the following chapters). This type of SFT will be very limited by the sea depth, at present a few hundred meters is considered a limit at present time. However, the construction of an underground bridge will have a less environmental impact.

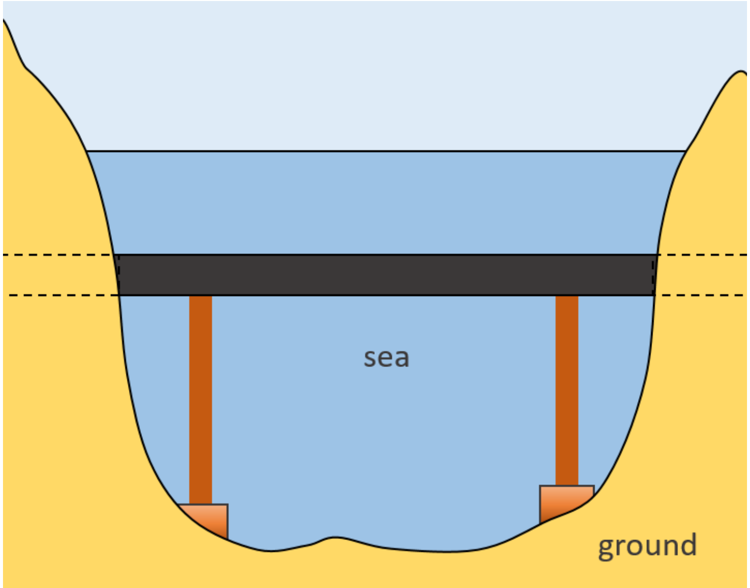


Figure 2.2 SFT supported by columns.

c. SFT with tethers to the bottom.

It is based on tethers being in tension in all future situations, no slack in these tethers may be accepted in any future load cases; consequently, the SFT must have sufficient net buoyancy under any load, wave, current, salinity or temperature condition in order to avoid any foul in the structure. The present practical depths for this type of crossing may be several hundred meters.

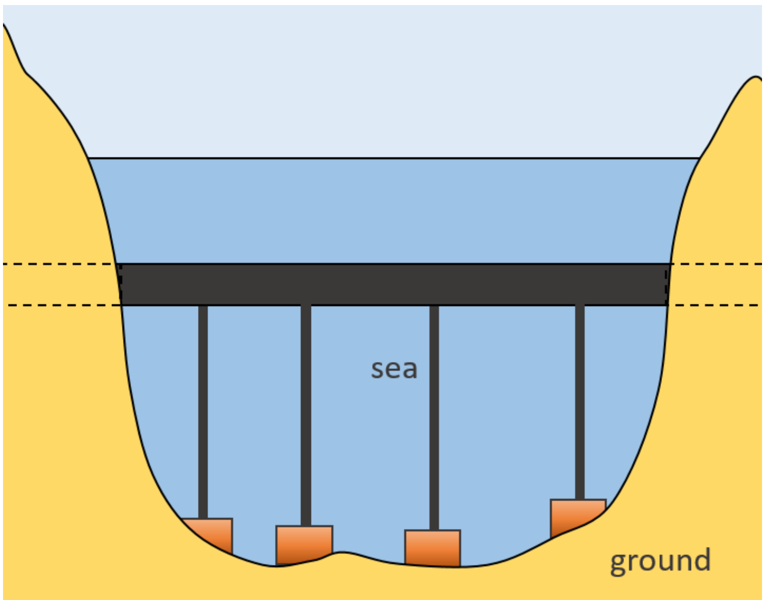


Figure 2.3 SFT supported by tethers anchored to the bottom.

d. Unanchored SFT.

This type of SFT has no anchoring at all except at landfalls, and so its independent of depth. However, its length is limited.

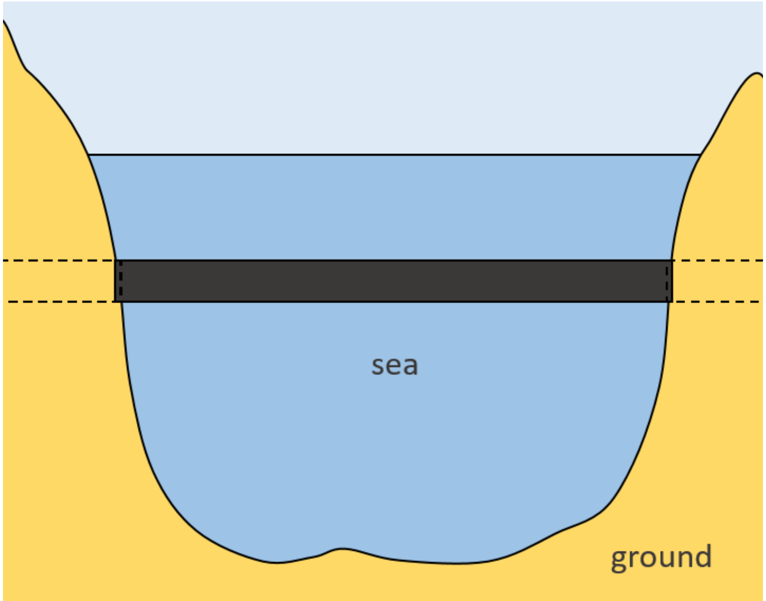


Figure 2.4 Unanchored SFT.

2.1.2. SFT Design Criteria

In this part we are going to analyse the most important concepts and issues that must be taken into account before focusing on the construction methods.
Even though, it is important to notice that the allowable design general criteria are mainly determined by the national codes; if not, the client/owner, together with the designer/contractor, must establish them.
In the design of a Submerged Floating Tunnel, the definition of the functional and geometrical arrangement of the cross section and the structural configuration of the tunnel is one of the main aspects to be faced.

2.1.2.1. Geometry

In terms of geometry, different alternative cross sections are being studied, circular, elliptical, polygonal and rectangular. Optimization of the cross section will mainly depend on hydrodynamic effects, vortex shedding, and structure strength, construction, economy and functionality.

Site conditions such as the length, water depth, water currents and densities, geology at the entrance etc.. have an impact on the dimension of the structure in terms of criteria for both the design and the construction methods.

The dimensions of the main body of a SFT are determined by the internal, external and structural requirements. The secondary components such as the supports are mainly determined by structural requirements and installation methods.
Obviously, the internal dimensions will depend on the purpose of the tunnel, since a SFT could be used for the crossing of both the railroad and road traffic, even for the crossing of pedestrians or wiring; and therefore, different dimensions will be required to fit the different purposes.

On the other hand, the external dimensions will be mainly determined by the construction methods, the influence of external loadings, the overall structural concept and the local structural requirements, it is also important to notice that they will depend on the required internal dimensions.
The behaviour of the tunnel is significantly affected by the external dimensions as the most important design loads are related to the tunnels volume. Examples of such loads are the buoyancy and the added mass, the circumferential compression load, and the forces of inertia associated with the wave motion and the seismic excitation.

The design of the tunnel structure and geometry of a SFT must be made according to some criteria.

1) The tunnels structure must be designed in order to meet the desired structural performances in terms of serviceability and safety by providing enough stiffness, strength and ductility. In addition, the waterproofing of the tunnel and the durability must be assured.

2) The internal dimensions of the cross section should be large enough to accommodate the infrastructures facilities and implants, necessary to guarantee the normal development of the operations inside.

3) The tunnel cross section must be designed so that the buoyancy ratio (the relationship between buoyancy and self-weight), is larger than the minimum value, if the buoyancy is limited inferiorly (upward force) and smaller than the maximum if the buoyancy ratio is limited superiorly (downward force).

4) Finally, issues related to the fabrication and transportation of the tunnel modules to site must be considered.

2.1.2.1.1. Circular Cross Section.

According to Brancaleoni et al. a circular cross section with respect to hydrostatic pressure features a very rational structural behaviour, as this induces only comprehensive stresses and no bending in the cross-section plane. In addition, Grantz W.(1997) says that thanks to the fact that generally the outer ring shell of a circular SFT is at leads partially made up of concrete, no longitudinal cracks are produced by hydrostatic pressure, and so the tunnel waterproofing is not compromised. Furthermore, regarding hydro-elastic stability issues, Solari (2010) says that a circular cross section features a good response, as, thanks to its polar symmetry, it should not be subjected to flutter or torsional divergence phenomena.

Figure 2.5 SFT with circular cross section.(a) Messina Strait crossing(Italy) proposed by ATI-SSST (Scolari et al., 1989); (b) Høgsfjord crossing (Norway) proposed by Aker Norwegian contractors (Skorpa and Østlid, 2001); (c) Sulafjord crossing (Norway) (Jakobsen et al., 2009).

In addition, another suitable geometrical configuration for the SFT cross section is to have one or more circular tubes holding the required traffic lanes and other related facilities connected to each other through a frame substructure and enclosed inside a streamline shell. This solution was envisaged in the first SFT proposal, developed by Alan Grant in 1969 for the Messina Strait crossing (**Figure 2.6. a**) and considered again later on, such as in the casa of the Northern Japan Exchange Axis (**Figure 2.6. b**).

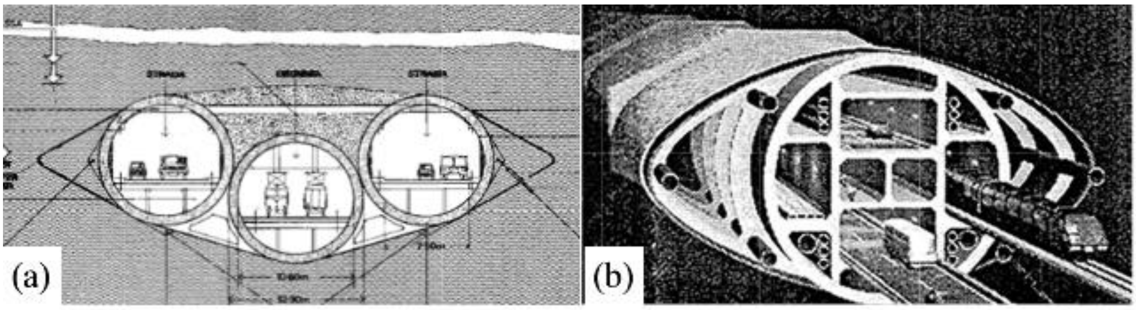


Figure 2.6 (a) Messina Strait (Japan) crossing proposal (Alan Grant, 1969); (b) Northern Japan Exchange Axis.

2.1.2.1.2. Elliptical and Polygonal

When the water conditions are stronger, instead of using circular cross section, elliptical and polygonal cross sections can be employed. Thanks to the elongation of the cross section in the horizontal direction, the impact of the hydrodynamic actions on the tube are decreased. In addition, these shapes provide larger values of stiffness and strength in the horizontal bending plan, which ensures the good hydrodynamic behaviour (Panduro J. Omar 2013).

In terms of distribution, it is easier to accommodate the traffic and facilities requirements in a polygonal than elliptical cross section, but the opposite happens in terms of structural requisites. Furthermore, the production process of the elements is easier for a polygonal cross section. Some proposals can be seen in **Figure 2.7.**

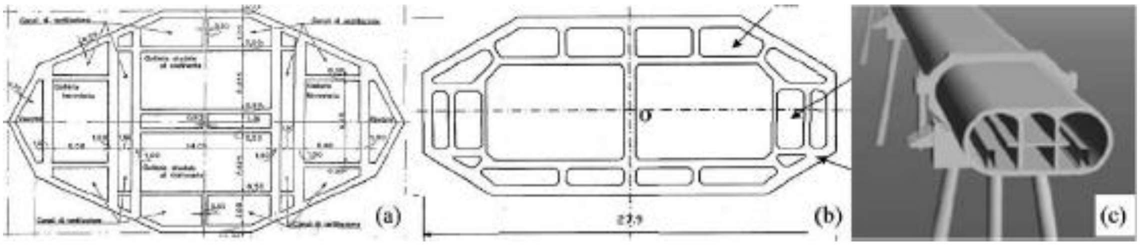


Figure 2.7 a) Messina Strait crossing, Italy (ponte di Archimedes S.p.A., 1984); (b) Jintang Strait crossing, People Republic of China (Faggiano, et al 2001a); (c) Washington Lake crossing, USA (Felch et al, 2001).

2.1.2.1.3. Rectangular

Rectangular cross section would represent the most rational solution, considering the easiness of the production procedures and the versatility in the organisation of the internal spaces and facilities. M. Kristofersen et al. says that in terms of functionality and distribution, the rectangular cross section works better than a circular cross section but with respect to blast loading, a circular cross section indicates a superior behaviour.

Under severe conditions, the hydrodynamic behaviour of a complete rectangular shape is not suitable, the water flow passing through a rectangular SFT would generate turbulence, thus increase the regime of dynamic pressures induced on the structure. In order to improve the hydrodynamic behaviour, one solution could be a rectangular cross section with rounded edges. This solution is recommended when the magnitude of currents and waves are not large. Another suitable solution in extreme natural conditions is the employ of hydrodynamic lateral keel, which can be fabricated of steel shells and trusses. This would improve the fluid dynamic behaviour of the SFT, preserving the advantage of the rectangular cross section.

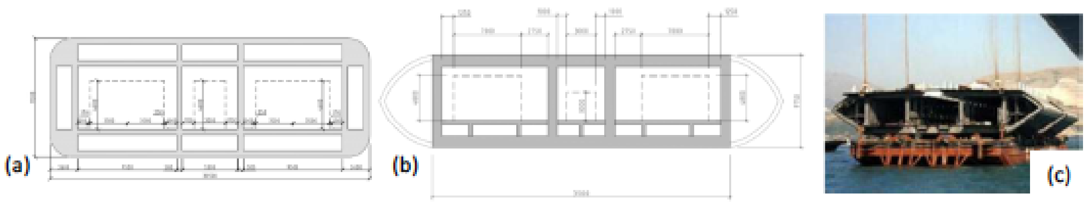


Figure 2.8 (a) Sognefjord crossing, Norway (Sweco Norge AS, 2012); (b) Sognefjord crossing, Norway (Cowi AS, Aas-Jakobsen AS, Johs Holt AS, NGI and Skanska AS, 2012); (c) Tsing Ma Bridge element, Hong kong.

2.1.2.2. Materials

When building a marine structure, the main factors when choosing the materials are; safety, (which would include ensuring structural and functional performances, show resistance to marine environment etc..) sustainability and economical points of view (constructional and maintenance cost).

According to Martire G. (2010) the most suitable and rational solutions are the ones who involve more materials, leading to a multi-layer/multi-material composite structure. In this way, each material has a particular function that exalts the material advantages and neutralizes its defects (Faggiano et al. 2001b). In this chapter, the most recommended materials for the design and construction of a SFT will be analysed.
These materials are:

a) Steel.

It is commonly employed in offshore structures. It has a very good behaviour against tensile and compressive efforts at the same time that has a lightweight. On the other hand, the corrosion is one of the main problems. Eiichi et al (2003) proposes a graph where in terms of the SFT vertical position, corrosion rate increases or decreases.

New types of steel have been introduced, featuring a lower content of carbon and resistant to corrosion, in order to improve its performance in maritime applications. But other types of problems affect them such as the difficulty to produce them in large scale (Ramasco et al., 1991).

The following table (**Table 2.2**) shows the different advantages and disadvantages of steel in marine conditions.

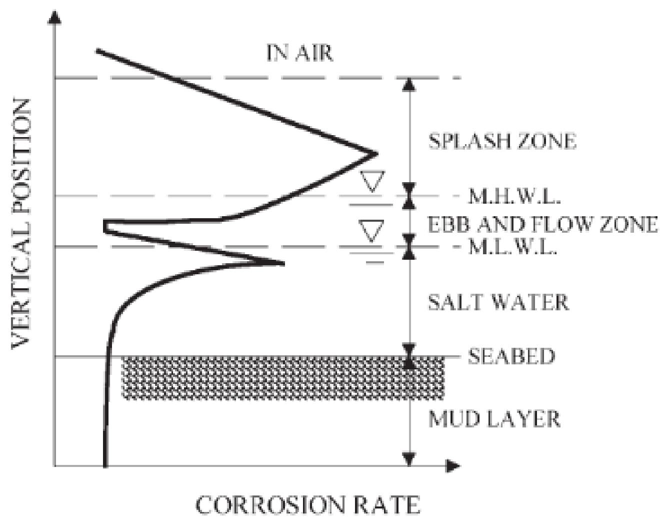


Table 2.1 Distribution of corrosion rate of steel (Ramasco et al., 1991).

Advantages	Disadvantages
Good mechanical properties Good resistance to fatigue Good resistance to abrasion Good workability Good weldability Large strength-to-weight ratio	Low resistance to corrosion Low performance of welded connections with respect to fatigue due to the cyclic loads imposed by environmental actions Low resistence to fire

Table 2.2 Advantages and disadvantages for steel. (Martire, 2010).

b) Concrete (reinforced and/ or prestressed).

Concrete is recommended when a large structural weight is required in order to stabilize the structure. Concrete in a SFT can contribute to structural strength and stiffness and provides the weight needed in order to counteract the tunnel buoyancy (Martire, 2010).
In addition, implementing prestressed concrete can lead to better mechanical performances and to a larger degree of waterproofing.
Table 2.3. shows the main advantages and disadvantages when employing concrete in a marine environment.

Advantages	Disadvantages
good resistance to the corrosion in marine environment Good resistance to abrasion Good resistance to fire and high temperatures Possibility to be cast to realize complex shapes Low cost	Negligible resistance to tensile stresses

Table 2.3 Advantages and disadvantages for concrete (Martire, 2010).

c) Aluminium Alloys.

Their main application in offshore structure is in the emerged part of the offshore platform, its used as to protect the internal structure from corrosion, external impacts and acts as a waterproofing surface. The following table summarizes its mechanical properties in a marine environment.

Advantages	Disadvantages
Wide range of strength, comparable to the one of steel grades	Poor resistance to fire
Good workability	Stiffness lower than steel
High resistance to marine corrosion	Specific weight relatively low, it being equal to 1/3 of the one of steel
High resistance to resilience	High cost

Table 2.4 Advantages and disadvantages for aluminium alloys. (Martire, 2010).

d) Rubber Foam.

This material has been considered for the design of a SFT as an external layer as it's an impermeable material which would act protecting the inner structure from corrosion, also is an extremely light material which is able to dissipate the energy transmitted by external impacts (Grantz,2003). In addition, it is a porous rubber made up of expanded polyurethane used in the Naval Engineering to increase the buoyancy of vessels and so it could help stabilize the buoyancy ratio too.

f) High-Performance Fibre Concrete (HPFC)

These materials have appeared in the last decade and are known as high performance fibre concrete materials. Reinforced concrete is being replaced in water crossing by HPFC's due to the fact that HPFC's have higher tensile strength and crack resistance and also, they are widely being used because of their good behaviour against compression, their extent tension strength, their anti-impact resistance, their waterproofing and their durability.

The following table shows the HPFC's current properties, also it can be seen how mechanical parameters increase by using different types of fibre (SINTEF (2009))..

Concrete material	Characteristics	Main mechanical parameters	Note
RC (C25)	—	f_{cd}=11.5Mpa, f_{td}=1.23Mpa, E_c=28.0Gpa,v=0.16	—
SRC	Advantage: higher compressive strength and tensile strength, good flexural and impact toughness properties, strong anti-explosion and anti-permeability Disadvantage: difficult to mix uniformly	f_{cd}=120Mpa, f_{td}=19.4Mpa, E_c=52.2Gpa,v=0.22	high cost, need to ensure that steel fiber bond strength with concrete construction technology requirement: steam curing can increase significantly quality and performance of SRC
GRC	tensile, bending, shear, impact resistance, fatigue resistance and fracture toughness are significantly improved compared with RC	f_{cd}=23.3Mpa, f_{td}=2.16Mpa, E_c=29.9Gpa,v=0.24	poor resistance to alkali, easy to embrittlement
PPFC	tensile strength, freeze-thaw resistance, anti-carbonation and fatigue resistance are all greatly improved, suitable for hydraulic structures	f_{cd}=20.5Mpa, f_{td}=2.20Mpa, E_c=31.5Gpa,v=0.20	poor effect to late shrinkage cracks and temperature cracks
S-PPFC	mechanical properties (strength and toughness) are determined according to mixed quantity of different fibers	f_{cd}=55.0Mpa, f_{td}=7.2Mpa, E_c=32Gpa,v=0.20	avoid the negative effects of fiber mixing; mixing should be sufficiently and uniformly

Table 2.5 Properties of high-performance fibre concrete with different fibre.

The table shows some abbreviations which mean:
SRC: Steel fibre reinforced concrete.
GRC: glass fibre reinforced concrete.
PPFC: polypropylene fibre reinforced concrete.
S-PPFC: multi-layered materials, also called Sandwich.

Archimedes Bridge Prototype in Qiandao Lake

Features an internal layer made of steel shells, so that its high mechanical performances are exploited; an intermediate concrete layer protecting steel shells from corrosion and cooperating with them with the axial, bending and shear resistance of the tunnel. In addition, this concrete layer has the function of assuring the ballast weight and as mentioned before, provides the weight needed in order to counteract the tunnel buoyancy. The steel and concrete parts act as a composite structure by means of shear connectors, **Figure 2.9.** shows a possible way of how steel shells and concrete layer could be disposed together with shear connectors.

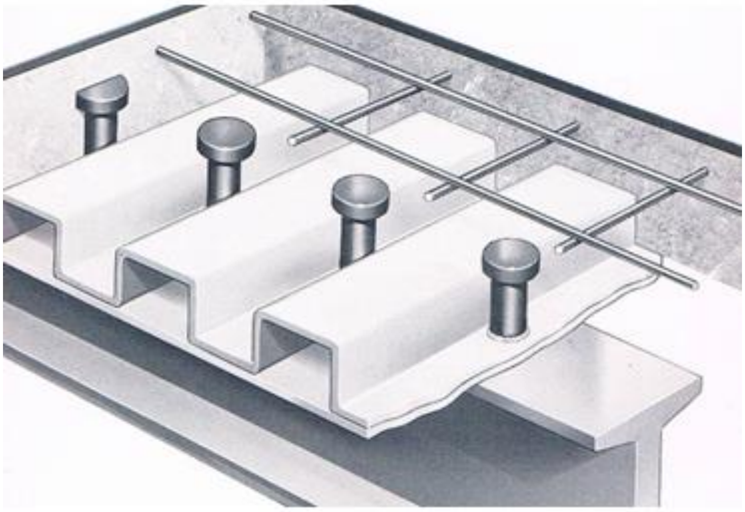


Figure 2.9 Steel shells and concrete layer act as a composite by means of shear connectors.

Finally, an external aluminium layer, protecting the inner layers from corrosion, external impacts and water penetration.
Mazzolani et al. (2007) says that this structural configuration also known as 'sandwich' structure, presents good characteristics of flexural strength in both elastic and post-elastic fields and that under exceptional events such as internal impacts or explosions is able to face up without great damages.

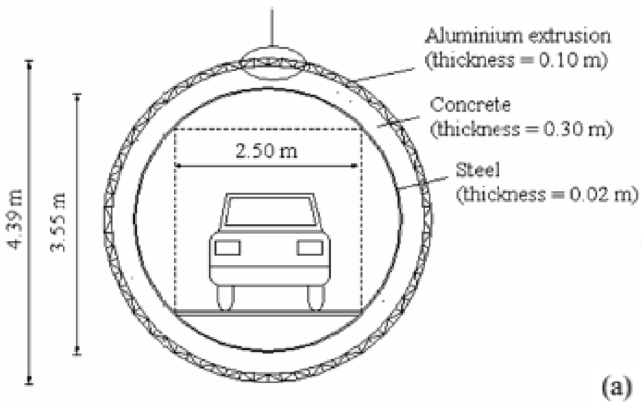


Figure 2.10 Sandwich tunnel structure of AB prototype in Qiandao Lake.

The Baltimore Harbour Tunnel

An example of a double steel shell Immersed Tunnel, is similar to the one of the prototypes, with the difference that the outer shell is made out of steel. This external shell has the same function as the aluminium outer layer from the prototype, to protect the inner structure from corrosion, impacts and to reduce the hydrodynamic actions due to incident wave flows (Grantz, 1997). Grantz also in 2003 made a similar proposal, a double steel shell hull reinforced with reinforced concrete rings on correspondence with the connections of the tethers and covered with an external layer of rubber foam, protecting and waterproofing the inner structure.

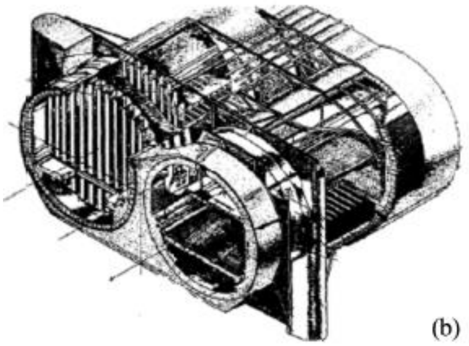


Figure 2.12 The Baltimore Harbour Tunnel, was constructed on 1957 in Baltimore, Maryland, United States .

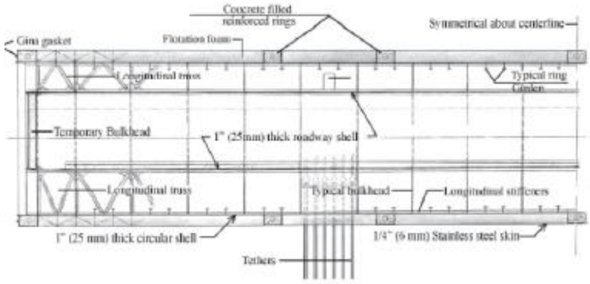


Figure 2.11 The proposed SFT structure by Grantz in 2003 SFT proposed with an outer foam rubber layer.

2.1.2.3. Stability of the Tunnel

The vertical stability of a SFT is a very important consideration. In order to understand better this concept, Ahrens D. (1997) compares the SFT with other types of tunnels. For a bored tunnel, the vertical stability is sufficiently ensured by the soil weight on top of the tunnel. In the case of an immersed tunnel, the vertical stability after the element is placed on the trench; additional ballast concrete is added in order to ensure the vertical stability.

Also, for a SFT, the vertical stability has to be ensured by the structure and its support system as it will never be covered by soil.
In addition, as the SFT in placed between the surface and the seabed, the appearance of the relationship between buoyancy and self-weight, also known as the buoyancy ratio, is very important and is crucial to analyse it. In fact, it is a critical structure parameter tremendously affects the dynamic behaviour of both the tunnel tube and the cable system (Ahrens D. 1997).

Two ways of ensuring the vertical stability of a SFT could be:

- Tether system: A situation where there is more buoyancy than weight and so the structure delivers a resulting upward and positive force, which is taken by a downward force created by the support system.

- Pontoon system: A situation where there is less buoyancy than weight and so the structure delivers a resulting downward and negative force, which is then taken by the upward force created by the support system.

Depending on the site conditions, this relationship will vary, as it will be of a much more interest to use one type of support system or another. It could exist the possibility that there is too much traffic in the surface and so a tether system would be more beneficial and safe (buoyancy more than weight), on the other hand deep sites can lead to difficulties and so much more expensive projects, as a consequence a pontoon system would be more viable (weight more than buoyancy).

The resulting load known as residual buoyancy in the tether or the pontoon, will be the difference between the upward and downward force, or in much more detail, the algebraic sum of the permanent loads, live loads and the buoyancy of the tunnel. This load is determined by the internal arrangement and the external dimensions of the cross section, by the material used and the destination of use.

The third design requisite, where the tunnel cross section must be designed so that the buoyancy-weight-ratio (BWR) is larger than the minimum value, is of particular importance in terms of stability and deserves a more detailed discussion.
BWR determines the tension force in the tether section and influences the dynamic behaviour of SFT structure (Hong Y. et al. 2010).
Firstly, it is important to highlight that the aim of this design condition is to conveniently limit inferiorly the residual buoyancy.
When a upward (positive) residual buoyancy is assumed, in order to avoid anchorage slackening due to the induction of environmental actions, it is necessary to ensure a minimum value of the residual buoyancy in operational conditions. By contrast, for a SFT where its weight is bigger that the buoyancy force, a different criterion has to be considered.

The design value of the positive BWR can be evaluated considering the load combination under vertical dead and live loads valid for the Ultimate Limit State, defined according to the Eurocode 0 provisions [CEN, EN 1990, 2002] (Panduro J. 2013).

Fd= γg.(Gk + Bk) + γq.Qk (γg = 1.35; γq = 1.5) (eq2.1)

Where:

- γg, γq are the partial safety factor for dead and live loads, respectively
- Gk is the characteristic value of the dead loads;
- Bk is the characteristic value of the buoyancy;
- Qk is the characteristic value of the live loads.

Very large values of the lower limit of the BWR were considered in the first studies and preliminary designs, up to 1,70. Numerical studies confirmed that larger values of the BWR can improve noticeably the structural performance of the SFTs, when they are subjected to severe environmental loading scenarios (Martire 2010). In particular, Brancaleoni et al. (1989) found that increasing the BWR from 1,25 to 1,40 can lead to impressive improvements of the SFT response to extremely severe sea states.

Condition	Buoyancy Ratio *BR*	Proponent
Positive (upward force)	1.2 ≤ BR ≤ 1.3	Mazzolani et al.
Negative (downward force)	0.5 ≤ BR ≤ 0.8	Zhang et al.

Table 2.6 Typical design values for buoyancy ratio proposed by Mazzolani et al. and Zhang et al. on their works.

As see in **Table 2.5.** Mazzolani et al. and Zhang et al. proposed optimal intervals in which the BWR should be stablished depending in the direction of the residual buoyancy.

The following table shows how the amplitude of tunnel vibration varies with the increase of the BWR having a fixed section diameter and for a SFT. As seen, the dynamic response in the current direction increases with the increasing of BWR, and shows that when BWR is between 1.1 and 1.2, the vertical response decreases dramatically with the increase in BWR, and so it is seen how Mazzolani was right when stablishing the optimal BWR between 1.2 and 1.3.

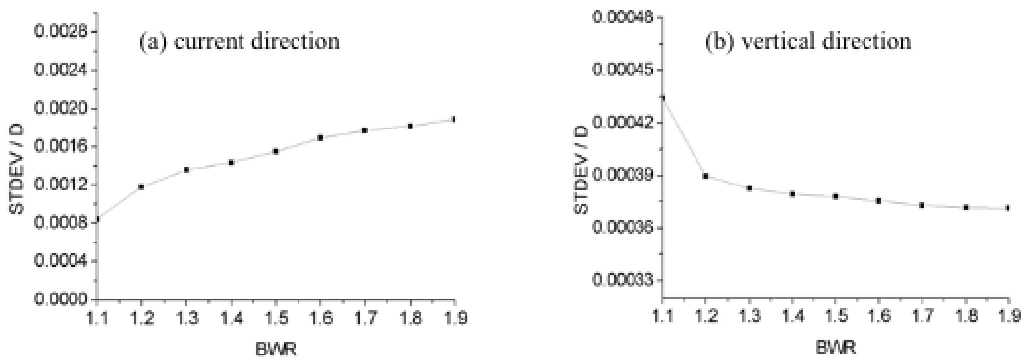


Table 2.7 STDEV at SFT mid-span under different BWRs. (a)Current direction. (b) Vertical direction. (Hong Y. et al. 2010)

Figure 2.13. provides a flow chart which describes the general procedures for the cross-section design of a SFT. It is being considered both cases, when the tunnels weight is larger than its buoyancy and opposite, when the resulting force is negative and when it is positive. In both cases is sought the design value of the buoyancy ratio. In addition, when the preliminary design does not meet this requirement, the calculation of ballast could be the solution for satisfying the point.

It also considers the structural analysis of the SFT once the other structural issues of the tube are designed. In addition, it is taking into account the anchorage of the support system, which will depend on the decision made after evaluating the natural conditions of the crossing.

Once the anchorage or support system is designed the static analysis can be made, both transversal and longitudinal analysis (Mazzolani et al 2008). Then it is designed the inter-modular joints and shore connection of the SFT for finally realize the dynamic analysis of the whole structure and meeting the limits of the design (Remseth S et al. 1999; Xiang Y. et al. 2019).

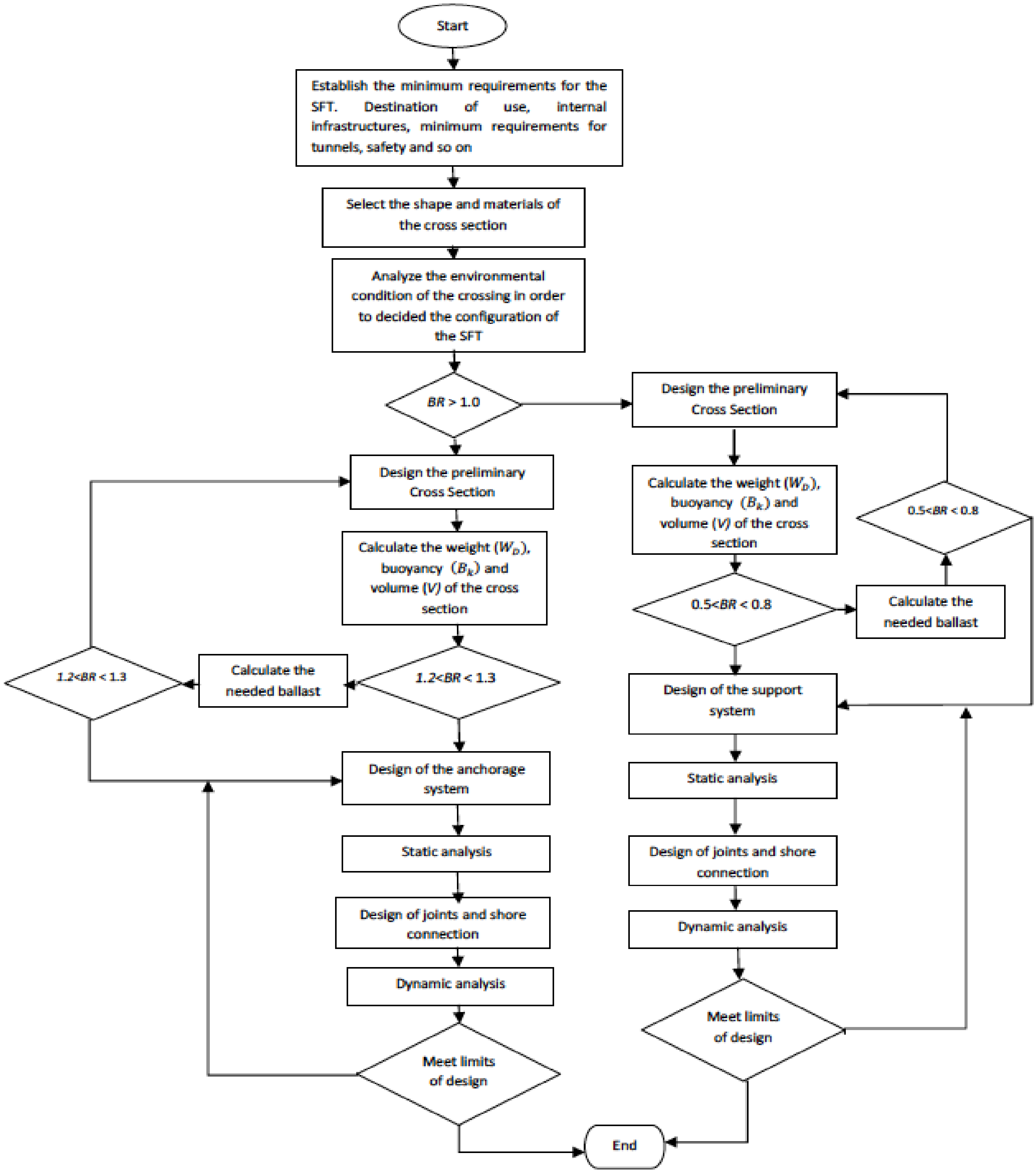


Figure 2.13 Flow chart of the design of a Submerged Floating Tunnel. (Panduro J.Omar 2013).

2.1.3. Supports

As mentioned in previous chapters, a SFT can be fixed at the seabed by foundation systems or on the surface by pontoons, it also exists the possibility of a self-supported tube where the presence of supports is not needed.

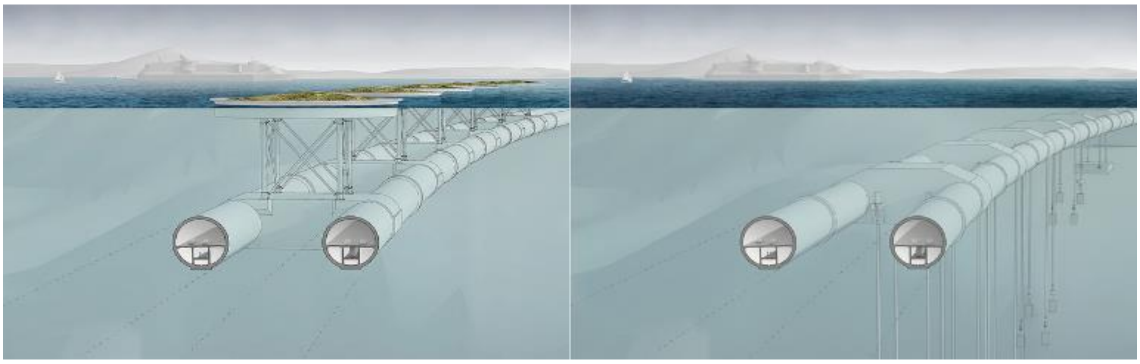


Figure 2.14 (a) SFT supported by pontoons. (b) SFT anchored to the seabed by tethers.

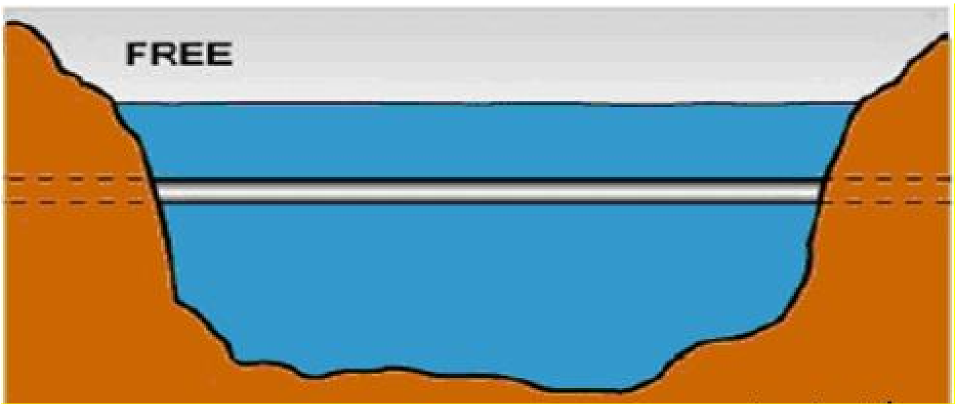


Figure 2.15 Unanchored SFT

In some cases, depending on the site and the environmental conditions, a combination of pontoons and tethers can be proposed.

2.1.3.1. *Pontoons*

When the residual buoyancy has a negative value, pontoons are employed. The weight of the structure is bigger than the buoyancy of it and so an extra force is needed in order to maintain the SFT stable and in position (vertical and horizontal). These elements are fixed on the surface of the water crossing.

Their design usually depends on the SFT cross section and the types of boats that will have to allow the passage. The first and most important issue is the fact of having to dimension the structure in order to assure the enough buoyancy for supporting its weight itself and the SFTs.
The separation between pontoons will depend on the issue mentioned before, and so as it is required to provide enough clearance to allow boats crossing, in some

cases space between pontoons would not be enough for the vessels in the region, thus, a navigation channel must be provided. In the following figure, we can see an example of a navigation channel, this figure represents a design proposed by NPRA for the Norwegian fjords.

Figure 2.16 A proposed SFT supported by pontoons for the crossing between the cities of Kristiansand and Tronheim.

Pontoons are only able to provide vertical support and lead to a system that is more flexible. They should be applied in less severe environmental conditions, as even though pontoons have the advantage of being independent of the water depth, they will have to cope with ships, waves, tides, currents and ice. Also, the applicability under severe conditions is reduced due to the fact that when fixing the pontoons to the SFT should be made through a ''weak link'' joint. Pontoons might be fabricated of steel or concrete, having several compartments of ensured buoyancy in the event of a ship collision (FEHRL, 1996).

Fixing the pontoons to the SFT through weak links, will isolate the tunnel from impact overload and so will limit the forces that can be transferred. Also a weak link will be beneficial with respect to damages to the ship (Fjeld et al. , 2013)

Here are three alternative weak link concepts which where nominated for further assessment for the current Bjørnafjord crossing (Engseth M et al. , 2016).

2.1.3.1.1. Truss Integrated WLA

Aas-Jakobsen et al. (2012) propose a tubular truss tower. This tower is designed to possess enough lateral drift capacity for the columns to fail in tension at the column ends prior to forming any plastic hinge action. The maximum load transferred to the bridge is limited by a bolted connection at the base of the columns designed to break at a certain load. The columns are placed on the inside of the tubes to exclude potential damage to the tunnel for a climbing ship scenario.

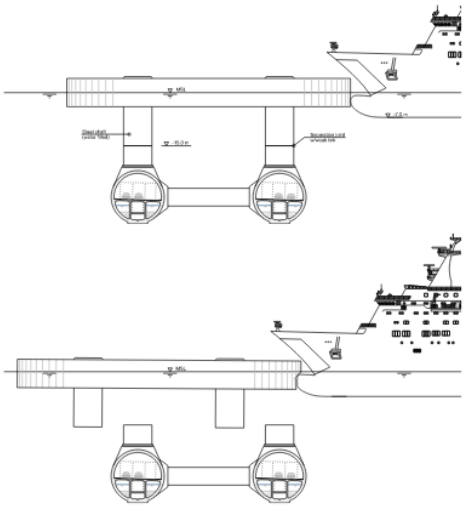


Figure 2.17 Tuss integrated weak link.

2.1.3.1.2. Shaft Integrated WL

Olav Olsen et al. (2012) developed a shear type weak link in the Sognefjord feasibility study. The load path is obtained by separating the horizontal shear from the other load effects in the shaft. The weak link is designed to break at a target impact energy level, while remaining elastic under normal operational conditions.

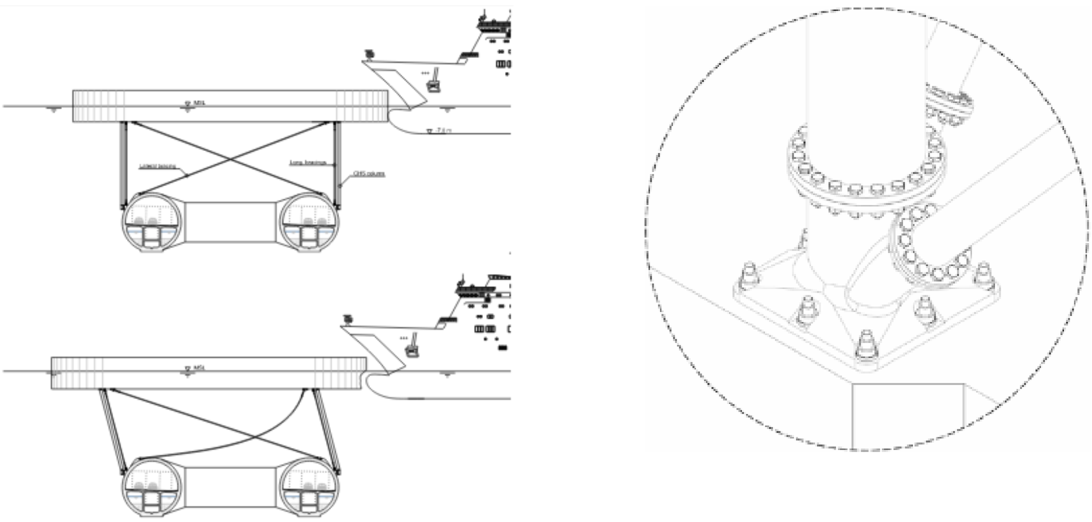


Figure 2.18 (a)Shaft integrated weak link. (b) Bolted connection at the base of the columns.

2.1.3.1.3. Bracing Integrated WL

The sacrificial connection to the pontoon shown in Figure 4.2-4 (Dr.techn. Olav Olsen, Reinertsen, Norconsult, 2015) is based on the same philosophy as the previous WL concept 1, except that the columns are placed outside the tubes. This will give a more stable platform for a long pontoon, but in return, the diagonal struts will be much longer which makes them more exposed to Vortex Induced Vibrations (VIV). To overcome this problem the bending stiffness of the diagonals must be increased.

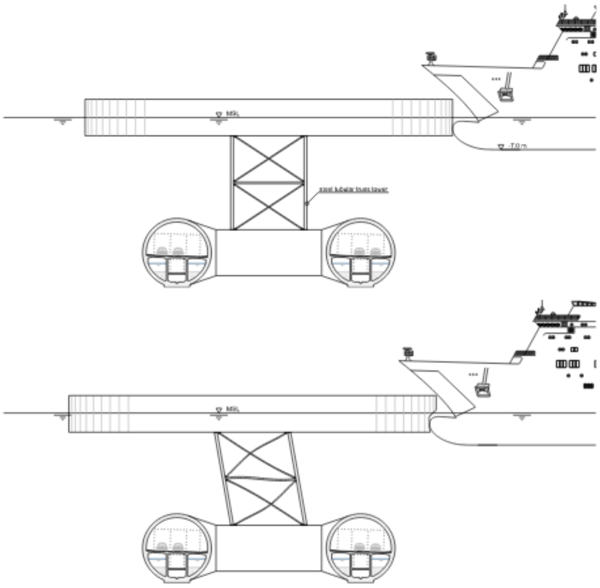


Figure 2.19 Bracing integrated weak link.

Magnus Engseth et al. (2016) explains how to design a pontoon and analyses how would a proposed SFT subjected by pontoons and weak links within them, would react to certain ship and submarine crushes, and from that, makes a capacity evaluation of the SFT. It is talked about the concept of designing sacrificial elements to dissipate energy while preserving the integrity of primary structure and presents calculations that show how a shaft designed to break at a certain load, can reduce the load transferred.

It is also mentioned that, after an event where severe deformations are caused to the pontoon shaft and to the pontoon itself, it is possible that the pontoon losses its ability to support the SFT and for that reason a SFT is designed to withstand loss of one pontoon in order to maintain its integrity when such an event happens. Finally, it is explained that to prevent a pontoon from sinking after being separated from the bridges, pontoons are designed with watertight compartments.

2.1.3.2. Tethers

In the case where the residual buoyancy is bigger than the SFT weight, the SFT must be stabilized by a foundation system together with a linking system. Obviously, first the foundation system would be constructed and finally the linking as in the pontoon system.

2.1.3.2.1. Foundation System

There are several foundation systems, which the use of them depend on various factor, mentioned as follows:

- The seabed condition (type of ground, seismicity and fouls).
- Depth (constructability point of view).
- Strength to support.
- Cost.

Different foundation systems proposed for SFT such as ;

- Gravity foundations.
- Suction caissons.
- Piers.
- Rock bolts.

Even though, depending on the factors mentioned before, it will be more viable the use of one system or another, Panduro J. Omar (2013) says that in general any type of structure that could help to hold the Submerged Floating Tunnel on position can be considered suitable for employing while stability is guaranteed. In this chapter, the gravity foundations and suction caissons will be analysed.

2.1.3.2.1.1. Gravity Foundations.

Massive blocks, which are designed to have enough, weigh in order to counterbalance the residual buoyancy of the SFT. Main problems related to gravity foundations are the need of a superficial layer with good mechanical properties and the low horizontal bearing capacity, which can lead to displacement and SFT stability problems (i.e. severe seismic events produce a combination of vertical and horizontal dynamic forces on the foundation which might produce permanent horizontal displacement on it leading to a modification on the geometrical configuration of the anchoring system) (Martire, 2010).

These elements generally consist on initially empty concrete boxes that are precast in a dry yard and that thanks to their own buoyancy capacity, are transported until their correspondence place. There, they are filled with concrete and then sinked until the seabed, where finally act as gravity foundations.

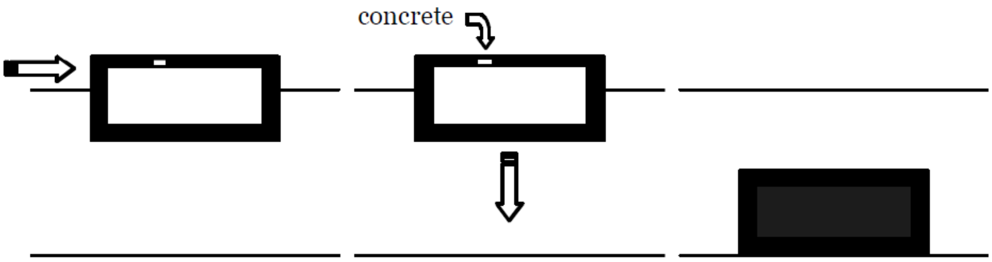


Figure 2.20 Sequence of transport and erection of foundation (Mazzolani et al. 2007).

2.1.3.2.1.2. Suction Caissons

Suction caissons are most new form of offshore foundation, which have greater advantages than the conventional ways, mainly being easier to install and to remove during decommissioning. Nowadays, suction caissons are widely used for anchoring large offshore installations at great depths.

This technology was developed in order to maintain the installations stability under sever conditions where the anchorage suffered from large tensioning due to waves and stormy weather. The suction caissons work very well in a seabed with soft clays or other low strength sediments. It is important to notice that the installation is much easier than piles, which must be hammered into the ground and so less expensive.

The installation of a suction caissons starts by its own weight penetration to the ground and then forced to the design penetration by pumping water out of the caisson in order to create under pressure/suction within the caisson. This difference in pressure results in a downward force on the exposed end of the caisson, which slowly pushed the caisson into the seafloor.

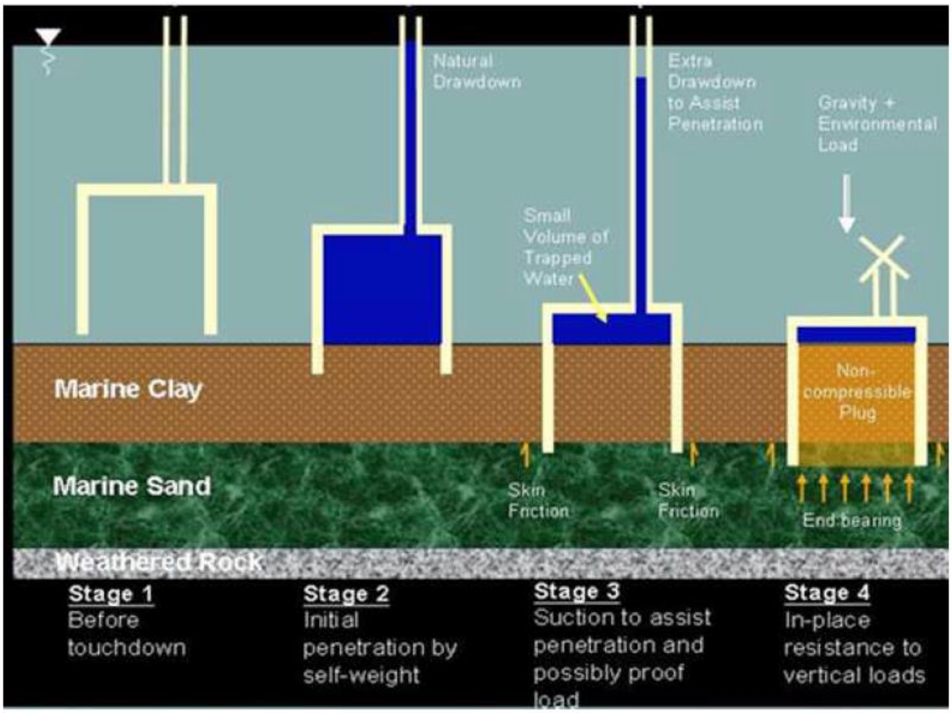


Figure 2.21 Suction Caisson installation (Arup).

2.1.3.3. Linking System

In order to attach the SFT to either the foundation or the pontoon, the most common way is by using steel tethers as cables, bars or tubes. This system is usually conceived as a series of cables groups, disposed in the tunnel cross-section plane and repeated along the tunnel axis with a fixed inter-axis.

There are several types of mooring cable arrangements in order to support a SFT, the following picture shows some of the most proposed arrangements.

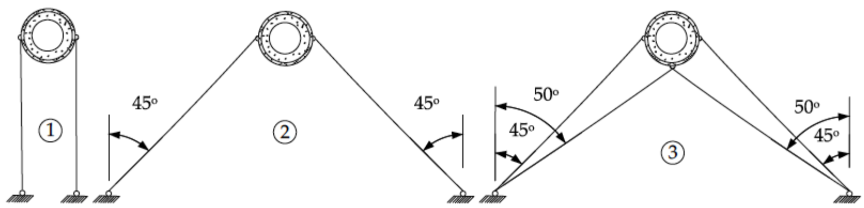


Figure 2.22 Tether configurations.

Naik Muhammad et al. (2017) evaluates the performance of a SFT supported by the three different cable configurations shown in **Figure 2.22.** and concludes after analysing the dynamic and the static response of the SFT, that groups made up of two vertical cables configuration are effective only in the vertical direction, thus being suitable only in a calm environment; groups made up of four inclined cables are the most effective.

Also, Hong Y. et al. (2010) investigates the slackening phenomena; their study focuses on the relationship between relative dynamic tension and tether angle, where it is concluded that at first the dynamic tension increases sharply and then decreases and that if the angle is larger than 30º, the response becomes stable. Finally, the dynamic tension reaches its peak value around the tether angle of 15º.

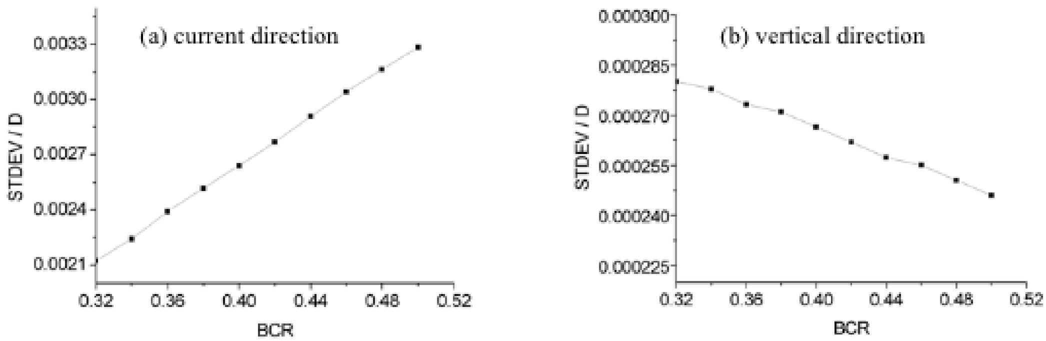


Table 2.8 Effect of tunnel outer diameter on SFT dynamic response (Hong Y et al. 2010).

Table 2.8 indicates that the dynamic response in the current direction increases as tunnel outer diameter increases (or BCR increases, stiffness coefficient of tether system) increases, whereas in the vertical direction, the dynamic response increases as the outer diameter of tunnel segment decreases (or BCR decreases).

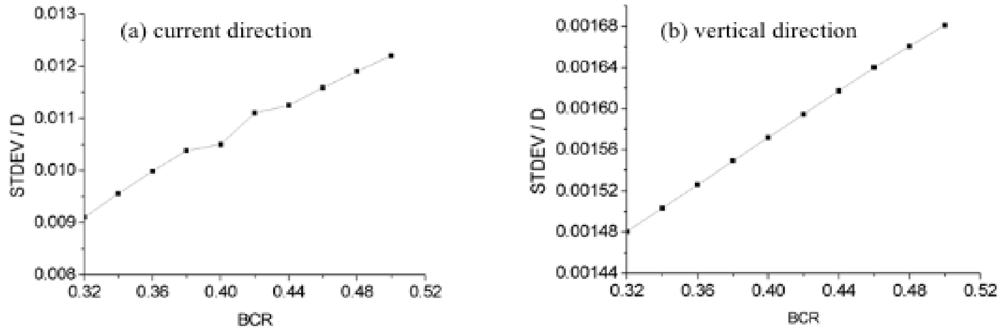


Table 2.9 Effect of tether system stiffness on SFT dynamic response (Hong Y et al. 2010).

Table 2.9 shows that the dynamic responses on both current and vertical direction increases as the tether system stiffness decreases or BCR increases (stiffness coefficient of tether system). Even though net buoyancy and tether stiffness are essential factors with regard to SFT stability. From **Table 2.8** and **Table 2.9** can be seen that BCR is not a characteristic factor with respect to SFT dynamics (Hong Y. et al., 2010).

Obviously, the cables must be able to resist the tensile force generated by the residual buoyancy. Therefore, given the permanent residual buoyancy and the cable length, it is chosen the diameter of the cables which determines it axial tensile strength, stiffness and other parameters such as the ratios between initial axial force and cables strength and weight. Martire (2010) considers that these two ratios are of great importance for the performance of each cable, as they define the axial force increment due to live or environmental loads that can be carried by the cable and the importance of non-linear effects in the cable response.

So, for the design of the anchoring system made out of steel cables, it's important to pay special attention to the geometrical configuration of the cable system, the diameter to be assigned to the cables and the restraint condition to provide at the ends of the cables.

2.1.4. Joints

The whole structure behaviour of a SFT has a great dependence in the joints between modules, as they must guarantee the linking between each of them as well as the whole tightness of the tube. They must assure the correct structural behaviour for which the SFT is designed. If the design allows displacement along the tube a flexible joint must be employed or if not a rigid joint has to be adopted. Finally, special joints must be located at the end and entrance of the SFT, as they must allow different displacements according to the design carried on.

There are two ways of tube joint based on stiffness and deformation: rigid joint and flexible joint. **Table 2.10** Enumerates differences of two types of joints.

Items	Rigid joint	Flexible joint
Structural composition	End steel shell, GINA water stop, junction steel plate and reinforced concrete	End steel shell, GINA water stop, OMEGA water stop, shear key, longitudinal displacement limiting device
Deformability	Poor deformability	Absorbing some temperature deformation, consuming some seismic energy for their good deformability
Construction	Long period and great difficulty	Convenient construction
Cost	Cheap	Expensive
Applicable position	End joint	Intermediate joint

Table 2.10 Comparison between rigid and flexible joints. (Zhang et al., 2010).

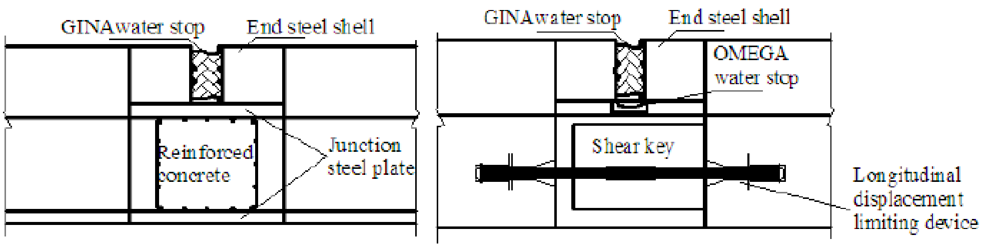


Figure 2.23 (a) Schematic drawing of rigid joint design (b) Schematic drawing of flexible joint design.

On the other hand, one of the most relevant issues in the design of a SFT is the configuration of the shore connections, as in these zones a transition between two different states of equilibrium happen (FEHRL, 1996):

- The tunnel in equilibrium with the ground pressures and, eventually with the seismic pressures induced by earthquakes.
- The tunnel in equilibrium with the water actions, with the retaining forces induced by the supports and if the SFT is anchored to the seabed, to the seismic actions.

The following figure shows two proposed shore connections for the Messina Strait crossing situated in Italy by the ENI Consortium.

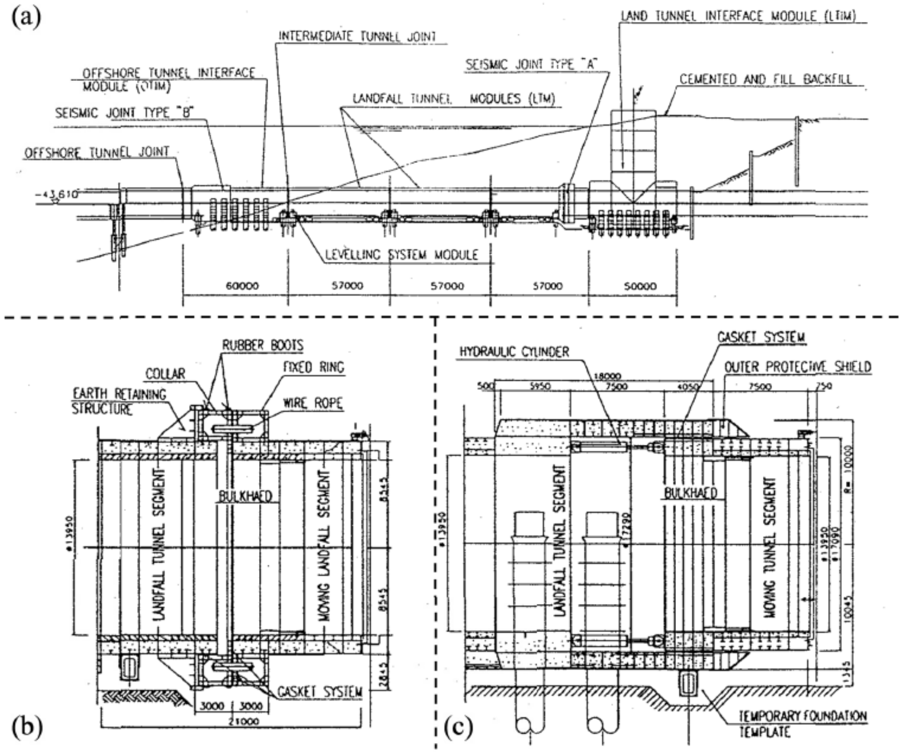


Figure 2.24 Shore connection of the SFT Messina Strait. (a) Global longitudinal view. (b) Joint A. (c) Joint B. (Nicolussi and Casola, 1994).

In order to prevent water leakage, both joints include a gasket system, also, natural rubber gaskets are squeezed between the surfaces of the joint elements in contact; and in order to reduce friction forces, Teflon sheets are stuck on the steel surfaces. In Joint A, a pretension system with wire ropes allows free relative movements in all six degrees of freedom with its release. Moreover, joint B includes a hydraulic system of 20 jacks distributed along the joint cross-section; these jacks absorb axial loads induced in quasi-static conditions. If the established threshold is exceeded due to big loads (earthquakes), the hydraulic system opens allowing for free axial movements; hydraulic accumulators connected to the jacks, meanwhile, develop a residual reaction to maintain the maximum relative moment within allowable design limits. Finally, an active back-up system, which consists on acceleration sensors, controls the good functionality of the hydraulic system (Nicolussi and Casola, 1994).

2.1.5. Loads

Like every other structure, a SFT must be designed based on expected and possible combinations of loading cases.

By using partial load factors, the individual loads are combined to give the design loads. These factors can either be specified by the client or established in the national design codes.

The following loads are the ones that usually must be considered during the design stage:

1. *PL (Permanent Loads):* Weight of the various structural and non-structural component (Total dead weight), the water buoyancy and the hydrostatic pressure. This load will be permanently present during the lifetime of the structure. The resulting force in the tether or pontoon system will be the difference between the buoyancy and total dead weight.

   The self-weight per unit of the cross section of the SFT can be calculated with the following equation (Panduro J.Omar 2013).

   $$W_D = \Sigma \gamma_m A_m + W_f \quad \text{(eq2.3)}$$

   Where:

   W_D : Weight of the structure per unit (kg/m).
   γ_m : Specific weight of each material involved in the cross section (kg/m2).
   A_m : Area of each material involved in the cross section (m2).
   W_f : Facilities weight (kg/m).

   The water buoyancy also known as Archimedes buoyancy is obtained with the following equation.

   $$B_k = \gamma_w A_{tot} \quad \text{(eq2.4)}$$

   Where:

   γ_w : Specific weight of water. This value is related to the salinity and temperature of the water. The salinity is generally measured as part per million (ppm).
   A_{tot} : Total area of the SFT cross section.

   Finally, the algebraic sum of the self-weight of the structure and the water buoyancy gives the residual buoyancy of the tunnel. This value is a fundamental factor for the stability of the structure. The calculation can be made with the following equation.

   $$RB_k = B_k - A_{tot} \quad \text{(eq2.5)}$$

   The accuracy when determining this force is very important, and in order to ensure that, the following uncertainties must be taken into account (Ahrens D. 1997).

   - Tolerances in geometry and dimensions: During the design stage, the acceptable tolerances in geometry and dimensions, depending on the choice of the construction method, must be established. During construction, an experienced contractor keeps the tolerances under control. After construction, this parameter is no longer variable. In case of significant variations from the weight design value, the ballast quantity can be modified at the end of the construction.

- The specific weight of concrete: Although the specific weight of the concrete will vary during the construction, it can be easily measured. The acceptable range has to be established beforehand. After the construction, this parameter is known exactly. The weight of the structure may change slightly over the time, as the concrete absorbs water.(Alteration minimal in comparison to the capacity of the support system).

- The specific gravity of water: The range in the specific gravity of the water will be particular of the site, and should be obtained at an early stage in the design process. The variation in buoyancy resulting from the change in the specific gravity of the water is permanent variable, and may have special importance in coastal areas, where the amount of river runoff or melting ice or snow can change the value rapidly.

- The amount and stability of marine growth: marine growth is known to concentrate at the sea floor and at the surface. If the SFT is not located in the critical surface layer, the effects of marine growth will be minor. However, where such growth does occur, it will increase the weight and the current resistance. Therefore, accurate predictions of the amount of marine growth are required.

The aforementioned uncertainties lead to time variations of the residual buoyancy; in order to keep these variations under control, avoiding negative effects on the structural stability, it is possible to use water as ballasting material (or part of it) and counteract the weight changes by varying the amount of ballast water. This operation can be easily made through hydraulic pumps (Martire, 2010).

2. *FL (Functional Loads) or variable loads:* Are those caused by the usage of the structure, in a SFT, which are due to traffic, changes in ballast conditions and the variable loads during construction.

Depending on the destination of use of the SFT, different functional loads will be produced. As it is known a SFT is proposed for pedestrian, roadway, railway or combination of these.

In a single day, the amount of traffic in the tunnel suffers great variations and so defining an associated load is quite difficult. Codes such as Eurocode 1 , part 3, UNI ENV 1991-2, 1991 or Norma española. UNE-EN 1991-2. CAPÍTULO 4. ACCIONES DE TRÁFICO RODADO Y OTRAS ACCIONES ESPECÍFICAS PARA PUENTES DE CARRETERA define conventional loading conditions determined on the basis of statistical data gathering and analysis; these codes are intended to reproduce most critical stress conditions produced by traffic loads on the structure.

The following table shows some load values proposed by Martire (2010) according to the SFTs destination of use.

Destination of use	Variable Load Value (kN/m)
Pedestrian	12
Roadway	720
Railway	840

Table 2.11 Variable load values for SFT depending on its destination of use.

3. *DL (Deformation Loads):* These are cause by geometric changes in the structure itself; these loads are usually associated to the material properties involved. Some of the causes of deformation loads are due to post and pre-tensioning, temperature variations, differential settlements, shrinkage, as mentioned before ship or submarine collision etc…

4. *Hydrodynamic loads:* Hydrodynamic actions due to the interaction between the water and the structure in presence of waves and currents, it often represents the most important environmental load for a SFT.

   The effects of the hydrodynamic actions for a circular cross section are evaluated, through both static and dynamic analyses, by means of the Morison. (Mazzolani F.M. et al. 2008)

$$F = F_D(t)+F_I(t) = \frac{1}{2}\rho_W D C_D u(t)^2 + \rho_W \frac{\pi D^2}{4} C_I a(t) \qquad \text{(eq2.6)}$$

with the following meaning of the symbols:

- F = Hydrodynamic force
- F_D= Drag force, depending on the velocity vector u(t);
- F_I= Inertia force, depending on the acceleration vector a(t);

- ρ_w= Water mass density;
- D = Tunnel external diameter;
- C_D = Drag coefficient
- C_a= Added mass coefficient where for a cylinder = 1.0.
- C_I= Inertia coefficient =1 + C_a =2.0, taking into account the added mass of the water surrounding the tunnel, which participates to the tunnel motions (Efstathios Konstantinidis 2017);
- u(t) = Velocity vector, equal to the relative velocity between the fluid particles and the tunnel;
- a(t) = Acceleration vector, equal to the relative acceleration between the fluid particles and the tunnel.

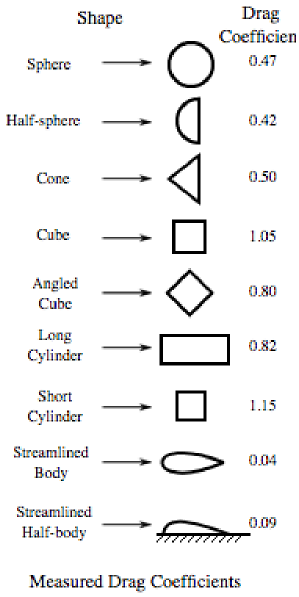


Figure 2.25 Drag coefficients.

4.1. *Currents:* There can be different types of currents in waterways, those generated by wind and those generated by tide variation. Wind blowing over the water surface transfers energy to it, creating water motion. By contrast, the rise and fall of the water level due to tides, leads to horizontal water motion. Obviously, it will depend on the site whether if current will create a larger or smaller load.

Usually water motion due to currents take place in the horizontal plane and can be assumed constant, as small variations in its velocity occur in a sufficiently long-time period. However short and long-term fluctuations around the velocity mean value occur, therefore when the former ones are significant they should be considered. Generally, for design purposes, water current is modelled as a horizontal velocity distribution along the water depth; this distribution can be roughly assumed to be constant or, more generally, can be represented by as a polyline, thus requiring observed data relative to the current velocity at the depths of the polyline vertices (Martire, 2010).

An analytical distribution often adopted for the current velocity can be shown as:

$$V_C(z) = V_T \left(\frac{z+d}{d}\right)^{\frac{1}{7}} + V_W \left(\frac{z+d}{d}\right) \qquad \text{(eq2.7)}$$

Where:

$V_C(z)$: Current velocity at a depth equal to z (z axis with the origin in the free surface and directed upwards).

V_T : Surface current velocity generated by tides.

V_W : Surface current velocity generated by wind.

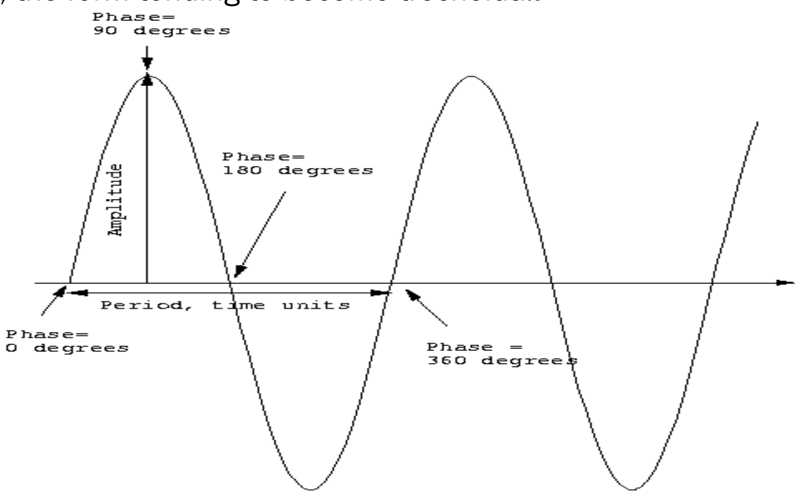
4.2. *Waves:* Waves differ from currents because an oscillating motion of the water particles characterizes them. Can be of two types, they can be either generated by wind due to the wind blowing over the surface, or they can be created by the force of gravity acting on small differences in density which keep water particles in motion, known as internal waves.

- Wind generated waves: When directly being generated by the local winds, a wind wave system is called a wind sea. On the contrary, wind generated waves that are not affected by the local wind at that time and have been generated elsewhere, or some time ago, are called swells.

- Internal waves: Different density can exist between two fluids or between different parts of the same fluid because of a difference in temperature, salinity, or concentration of suspended sediment.

  Although internal waves can be important and induce significant loads on marine structures, in most cases wave water motion is made of wind generated waves (Martire, 2010).
  The size of the wind waves depends on three variables: the wind speed, the wind duration and the fetch the exposure of the site to wind action.

Small amplitude waves in deep water acquire a sinusoidal form, by contrast in shallow water the wave crests become steeper and the troughs (minimum) flatter, the form tending to become trochoidal.



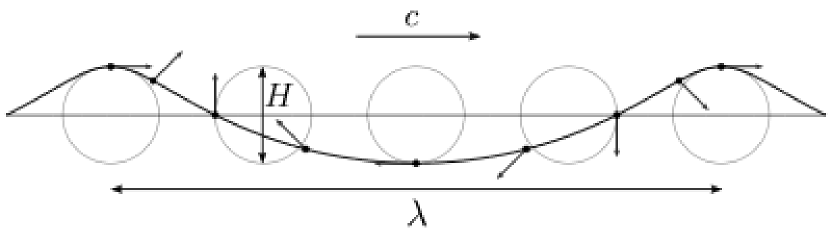


Figure 2.26 Sinusoidal wave.
Figure 2.27 Trochoidal wave.

In deep water a single sinusoidal wave can be described by three parameters (CIRIA Underwater Group, 1978).

- Wave period(T_W): The temporal distance between two crests or two troughs.
- Wave height: (H_W) : The vertical distance from trough to crest.
- Wave length(λ_W): The geometric distance between two crests or two troughs, measured in the direction of propagation of the wave. Wave length is closely associated to wave period and also to wave celerity, being λ_W = c · T_W their relationship. Moreover, there is a first order relationship between λ_W and T_W.

$$\lambda_W = 1.56 \cdot T_{W2} \qquad \text{(eq2.8)}$$

(λ_W in metres and T_W in seconds)

In order to describe the motion of water particles due to waves, several theories have been developed, generally focusing on the determination of the velocity potential, satisfying the Laplace Equation and assuming an irrotational and incompressible fluid. The following table explains the most common theories used to carry out the hydrodynamic analysis due to wave loads.

Theory	Features	Application
Airy Linear Wave Theory also called Sinusoidal	The wave height Hw is small (Sarpkaya and Isaacson, 1981).	When the wave height Hw is considerably smaller tan both the wave length Lw and the seabed depth d. (Sarpkaya and Isaacson, 1981).
Stokes theory	The considered wave is not too steep and water depth is not too small (Sarpkaya and Isaacson, 1981)	When the wave height-to-length ratio is largely lower than one (Hw/Lw<<1; (Peregrine, 1972)) and when the wave length Lw is less than 8 times the water depth d (Laitone, 1962).
Cnoidal Wave Theory	Waves with very steep and sharp crests and flat roughs.	In shallow waters. When d/Lw < 0.05.
Solitary Wave Theory	large wave length compare to the height	When d/Lw < 0.05
Linearized Long Wave or the Trochoidal Theory	Circular particle orbits, rotational fluid and trochoidal wave surface profile (Gerstner, 1802; Rankine, 1863).	It serves as a link from linear theory to the finite amplitude oscillation wave theory (Wilson, 2003) .

Table 2.12 Linear and non-linear wave theories features (Panduro J.Omar 2013).

Choosing the right and adequate wave theory, will depend on the different wave parameters. In fact, each wave theory can be considered reliable in different fields of application in terms of wave period, height and seabed depth. In **Figure 2.28.** can be seen the different behaviours of each theory.

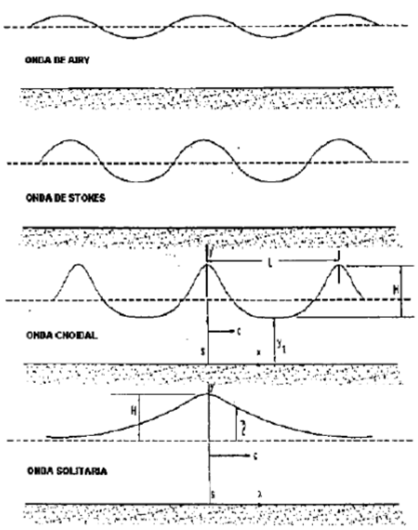


Figure 2.28 Graphical representation of the different wave theories. (Medina, 2009).

There are many diagrams defining the field application of the various wave theories. In **Figure 2.29.** the range of validity of the different theories of wave.

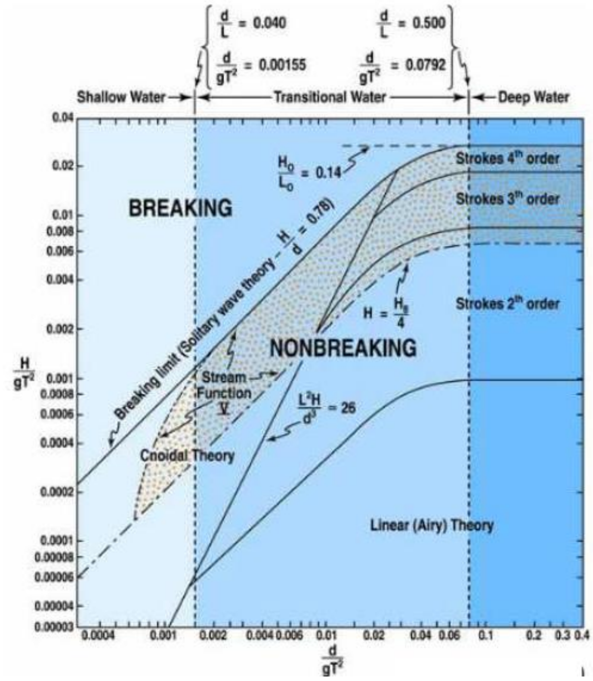


Figure 2.29 Range of validity for the wave theories. (Le Méhauté, 2008).

Bernt Jakobsen (2010) says that it is not so much the difference in wave heights that causes higher wave loads on the structure from the sea waves, but more the difference in wave periods. **Figure 2.30.** illustrates the effect of different wave periods on the wave force as a function of the depth at which the tube is located.

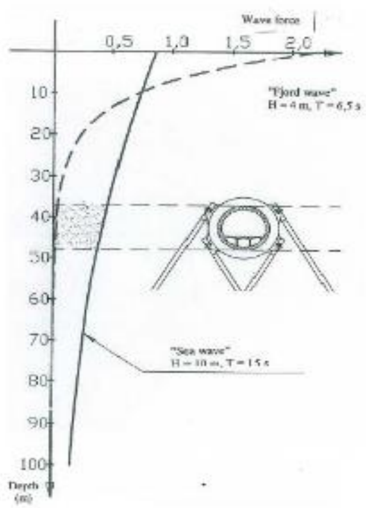


Figure 2.30 Wave force as a function of wave period and depth (Bernt Jakobsen ,2010).

5. *Earthquake or seismic loads:* Strong ground motion due to release of energy from the accumulated earth's crust apart from volcanic and tectonic activities, which originate mainly at the edges of the plate. Special attention should be paid to the tunnel shore connections and also to the anchoring system as the seismic loads will propagate through (downwards residual buoyancy). A SFT supported from bed level by means of any anchoring method would suffer the seismic loads much like a bridge, but supported in any other way could be almost independent of seismic loads. This does not mean that seismic events can be ignored. So, it is necessary to assure that every structural component supports safely extreme seismic events. (Ahrens D. 1997)

   Xiao J. et al (2010) made a study of the seismic response of the Submerged Floating Tunnel with different types of connections between the ends of SFT and the shore, including the rigid connection, hinged connection, elastic bearing connection, bi-linear elastic bearing connection, and passive isolation connection. Hi concluded firstly, that the hinged connection system can greatly reduce the seismic response with respect to the rigid connection, and finally, that the elastic bearing and passive isolation connection in comparison with the hinged connection by choosing proper parameters of the bearings can be much more effective.

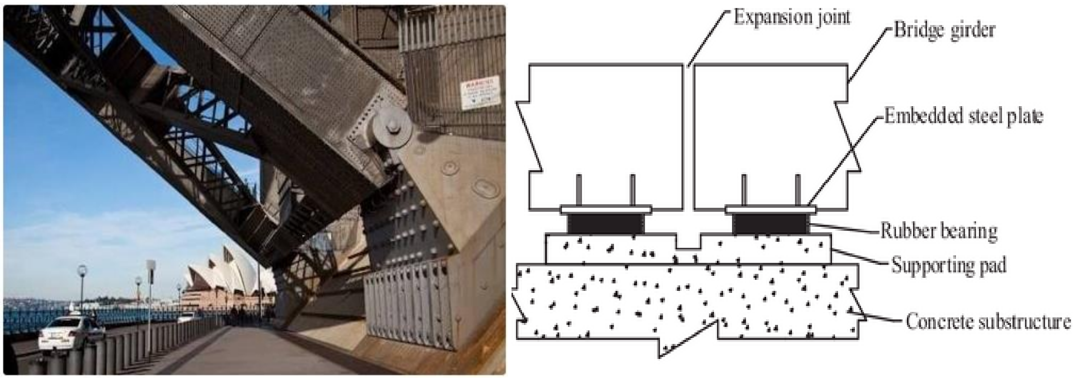


Figure 2.31 (a) Hinged connection. (b) Elastic bearing connection.

Figure 2.32 Passive isolated connection.

6. *AL (Accidental Loads) :* Accidental loads, are not meant to happen, but they do happen from time to time, so they must be considered, and specified, a rational way of dealing with these loads must be studied. Explosions inside or outside the tube, loss of buoyancy, fire from burning cars or fluids, failure within the support system are some of the accidents that could cause this type of loads. On the next table can be seen several studies related to accidental events and how may they affect a SFT.

Topic	Author
Elastic-plastic capacity of a submerged steel bridge in case of impacts with sinking ships	Rambech et al. (1994)
Fire safety systems available for underground tunnels and applicable to SFTs	Fiorentino (2009).
Impact analysis of Submerged Floating Tunnel for conceptual design	Lee et al. (2013)
Global response of submerged floating tunnel against underwater explosion	Seo et al. (2013)
Simplified collision analysis method for submerged floating railway using theory of beam with elastic foundation	Seo et al. (2013)

Table 2.13 Studies related to accidental event which could occur on a SFT. (Panduro J.Omar 2013).

2.1.6. Limit States

The SFT design method can be based the semi-probabilistic limit state approach, using partial safety coefficients on both loads and strength of materials.

The semi-probabilistic approach divides the design into the following design limits states:

SLS (Serviceability Limit State): These conditions are set to ensure that the whole structure meets the necessary criteria with regard to deflections, crack widths, factors of safety, accelerations etc.

ULS(Ultimate Limit State): This condition is set to confirm that the structure has the necessary margin of strength to survive factored loads and load combinations, with the factors being set to provide acceptable risk of failure. The factors must also be sufficient to ensure after an unfeatured event, that the structure is capable of continuing to operate satisfactorily.

FLS(Fatigue Limit State): Some materials lose strength due to repeated loading. The FLS is required to account this fact. By computing the accumulated damage in the material and checking the computed life of the structure against the operational life the sensitivity of certain components of the structure can be established.

2.1.7. Construction Methods.

2.1.7.1. Incremental Construction and Launching.

In this method, the construction takes place at one of the abutments which are modified and increased in size to accommodate the construction site and the associated plant, equipment and materials.

The tube is constructed in consecutive sections on an inclined skid way in the abutment. After each section is completed, the tube is moved forward into the water, over the length of one section. For this reason, the segment is constructed in saddles and pushed forward by hydraulic jacks. Before it is pushed through the gate, the tube is coupled to the previous segments with pretension cables. The part of the tube pushed out into the water must be kept under control. A temporary cable system and/or a pontoon support system is a likely method for this purpose.

If the pontoons are used for vertical support during pushout, a shore-based cable system may be needed to keep the SFT under control in the horizontal direction. The stiffness of the cable system in relation to the stiffness at the pushout gate will require careful consideration. New pontoons are connected to the tube as it moves forward. The pontoons, which may be temporary or permanent, follow the structure across the waterway.

An alternative would be to use final tethers, installed in advance and supported by temporary pontoons. To allow the tube to move, a special guidance and support system is needed at the top end of the tethers. It may be possible to install rollers, guidance plates or hydro jet bearings in a saddle, which may be temporary or which may become part of the final construction. As the tube is pushed forward, it meets the guidance structure. The tether tension is then provided by the buoyancy of the tube (Ahrens D. 1997).

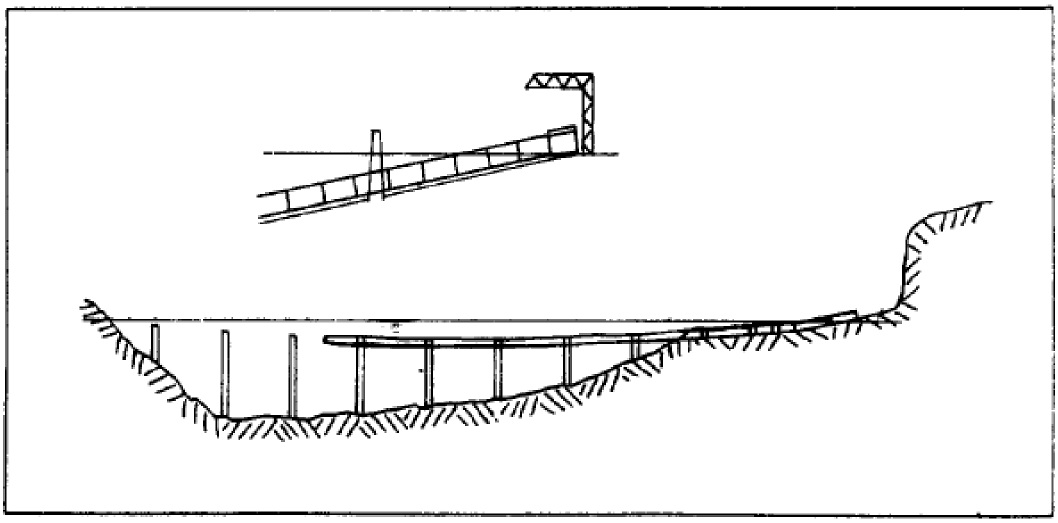


Figure 2.33 Incremental construction and launching of SFT elements (Ahrens D. 1997).

2.1.7.2. Element Construction and Launching

The SFT elements are constructed in a dock. After all, or a number of elements have been completed, the dock is flooded and the elements are towed to the site. Bulkheads used to seal the ends and to maintain positive buoyancy during construction, are joined at the site. The length of each element is determined partly by the structural capabilities of the SFT (the designed distance between the supports) and partly by the available length of existing ship docks, slipways or construction docks.

After reaching the site, the installation barge supports the element during assembly and lowering to the intended depth: At each joint location, a set of tethers is pre-installed and coupled in a horseshoe-shaped support. The element is lowered under the support while temporarily pulling the support aside.

After the element has been fitted into the predetermined tether support system, it is de-ballasted, causing the load to transfer from the installation barge to the tether system. During this process, the length of the tethers is adjusted at the support shoe to prevent unacceptable deflections of both tethers, the position of the new element and of previous elements.
An initial watertight seal is provided by a rubber gasket. After de-watering the area between bulkheads, the permanents join between the two elements is made and post-tensioned to obtain full structural strength. The rubber gasket has only a temporary function during construction.

Several solutions may be used for the final connection between the tunnel and the tethers. These connections depend on the location of the connection i.e. whether they are external or within special chambers added to the tunnel. It also depends on whether the preference is to connect the tethers first to the anchor points on the bed or to the elements before it is lowered (Ahrens D. 1997).

Figure 2.34 SFT elements being towed to the site.

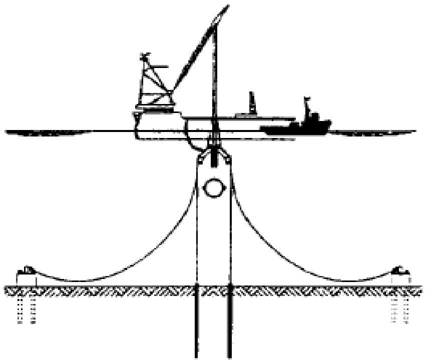


Figure 2.35 Type of installation on site (Ahrens D. 1997).

2.2. Suspension Bridges

A suspension bridge is a type of bridge, which has suspended cables between towers, in addition to vertical suspender, cables that carry the weight of the deck below, destined for the crossing of traffic.

The basic structural components of a suspension bridge system include girders/trusses, the main suspension cables, main towers, and the anchorages for the cables at each end of the bridge.

Figure 2.36 Akashi-Kaikyō Bridge, Japan.

Figure 3.2. briefly shows the different components of a suspension bridge. In these type of bridges, the main cable made out of high-strength steel wires, together with hanger ropes, support traffic-carrying stiffening girder. Any load applied to the bridge turns into a direct tension that is transmitted through the cables to the towers and anchorages, due to this, suspension cables must be anchored at each end of the bridge or sometimes to the bridge itself. The intermediate vertical structure, also known as the main tower, support main cables at a level of optimal sag ratio and required clearance. These towers transfer the external loads to the bridge foundations.
An anchorage is generally a massive concrete block, which anchor main cables and act as end supports of a bridge against horizontal movement (Lin W. et al. 2017).

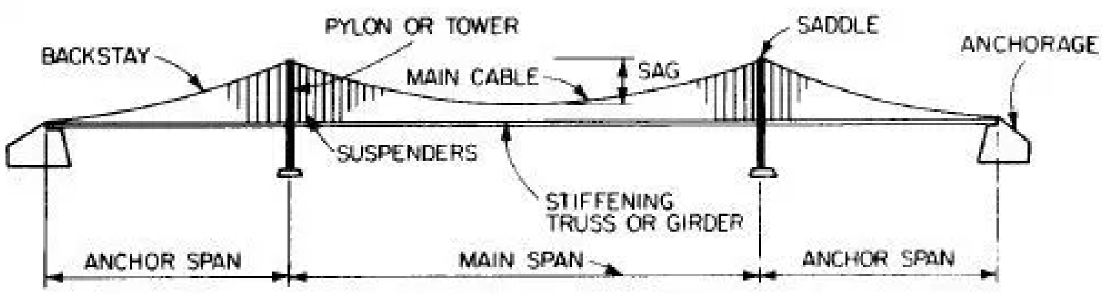


Figure 2.37 Main suspension bridge components (Lin W. et al. 2017).

Suspension bridges are widely used as engineering structures to cross long distances and enables the use of domain under de bridge. A SFT is able to provide the crossing of long stretches, but is more limited in terms of crossing traffic below or above it, since it will depend on whether it is subject by pontoons or tethers, and thus will harden sea traffic passing through. However, a SFT has less environmental impact than a suspension bridge.

2.2.1. History of the Suspension Bridge

It is hard to say who really had the first idea of suspension bridge. In 1433, Thangtong Gyalpo build several bridges in eastern Bhutan, the difference of these bridges from modern suspension ones is that Gyalpos iron chain bridges did not include a suspended deck bridge, instead, both the railing and the walking layer used wires and the stress points that carried the screed were reinforced by the iron chains. This is the reason why Gyalpo is not considered the first designer of a suspension bridge (Gazzola F., 2015).

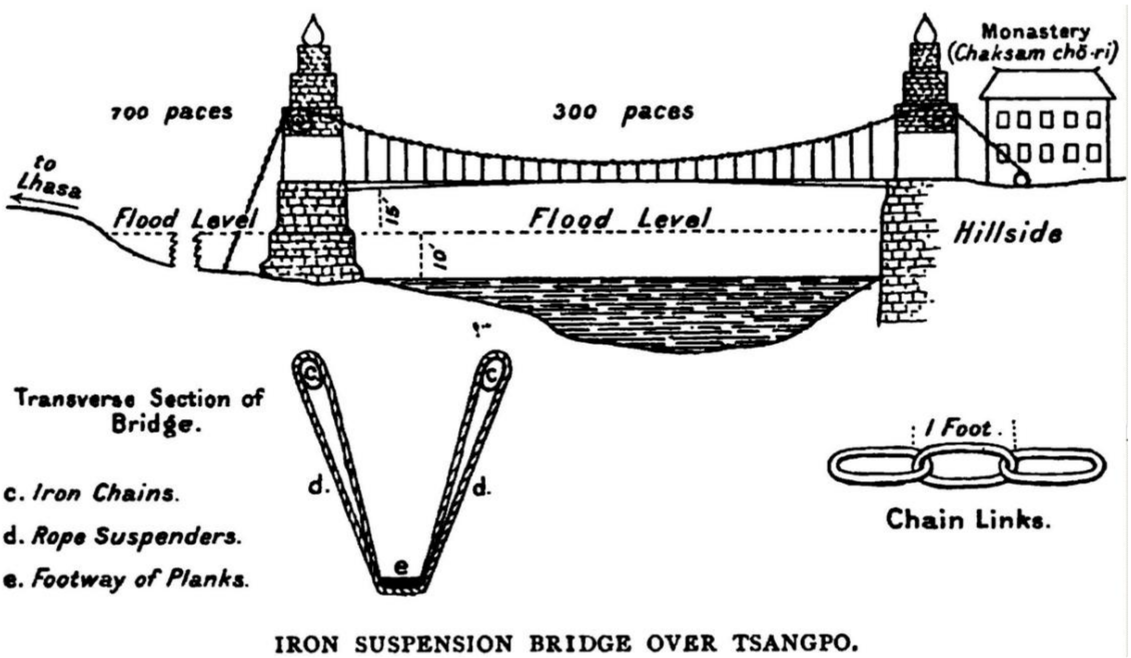


Figure 2.38 Picture of the Chushul Chakzam suspension bridge (Gazzola F., 2015).

The Italian engineer Fausto Veranzio is known to be the first one to design a modern suspension bridge in 1595. Veranzio proposed an iron bridge (Pons Ferreus) which is somehow a combination between a suspension bridge and a cable-stayed bridge. Even though, this bridge was never build.

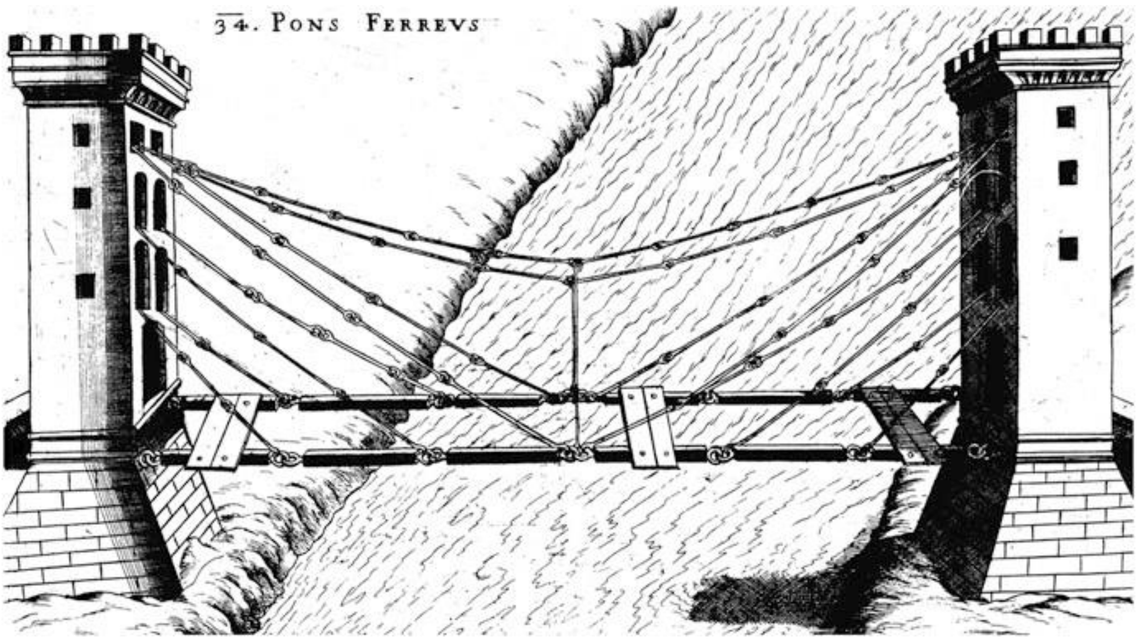


Figure 2.39 Picture of the Pons Ferreus by Fausto Veranzio (1595) (Gazzola F., 2015).

However, it was not until two centuries later in 1801, when it is thought to have been erected the first suspension bridge, using wrought iron chains with a level deck, known as Jacob Creek Bridge situated in Pennsylvania. This bridge, was built by the Irish judge and engineer James Finley.

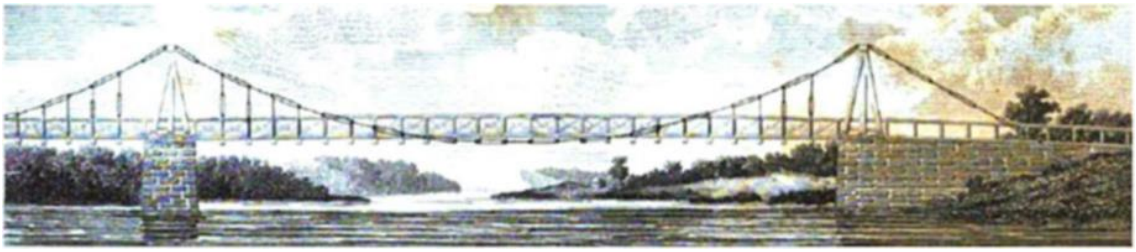


Figure 2.40 View of the Jacob Creek Bridge (1801). (Gazzola F., 2015).

2.2.2. Suspension Bridge Classification.

Suspension bridges can be classified according to:

- Span Number
- Stiffening Grinders
- Suspenders
- Anchoring Conditions

2.2.2.1. Span Number

Nowadays, there can be several types of suspension bridges based on the number of spans and towers. Single-span, two-span, three-span or multi span suspension bridges. Among them, the most commonly used in engineering practice is the three-span suspension bridges with two main towers.

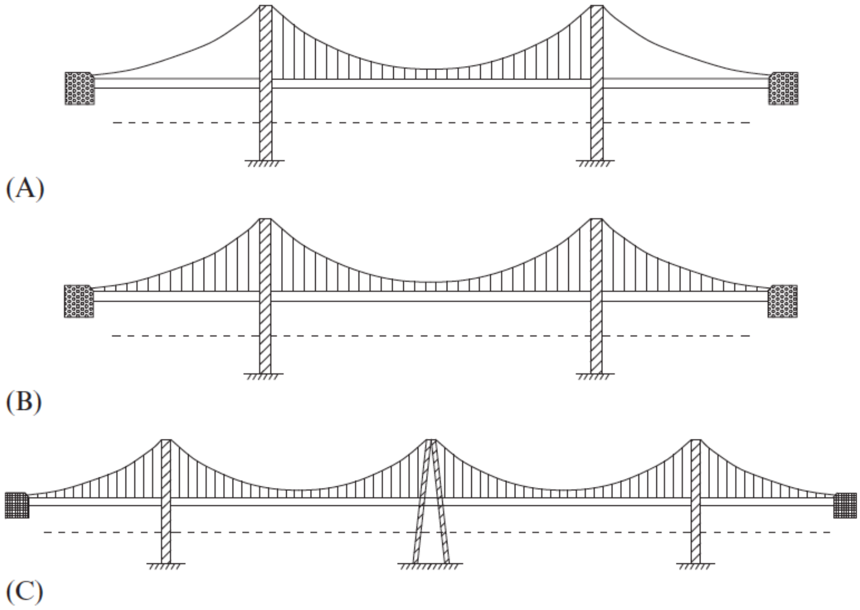


Figure 2.41 Suspension bridge classification according to span number. (a) Single-span. (b) Three-span. (c) Multi-span *(M. De Miranda, 2016).*

Typical single-span suspension bridges could be the Tsing Ma Bridge in Hong Kong and the Pingsheng Bridge in Guangdong.

Figure 2.42 Tsing Ma Bridge in Hong Kong

Figure 2.43 Pingsheng Bridge in Guangdong.

An example of a multi-span suspension bridge could be the Tamate Bridge built in 1928 in Japan. For these type of bridges, where more than two towers are present the horizontal displacement of the tower tops due to live loads can be a concern and measures for controlling such displacement becomes necessary.

Figure 2.44 Typical multi-span suspension bridge, Tamate Bridge, Japan.

2.2.2.2. Stiffening Girders

According to Alampalli et al. 2015 there are two types of stiffening girders, based on the continuity. The two-hinge or the continuous types. They can be appreciated on **Figure 2.44** The use of one and other varies. For highway bridges the two hinge stiffening girders are commonly used, and the continuous stiffening girder is often used for combined highway-railway bridges, in order to ensure continuity between spans and secure the smooth operation of the trains. The longest suspension bridge in the world, Akashi Kaikyo bridge in Japan (see **Figure 2.36**), was designed with a two hinge stiffening girder system.

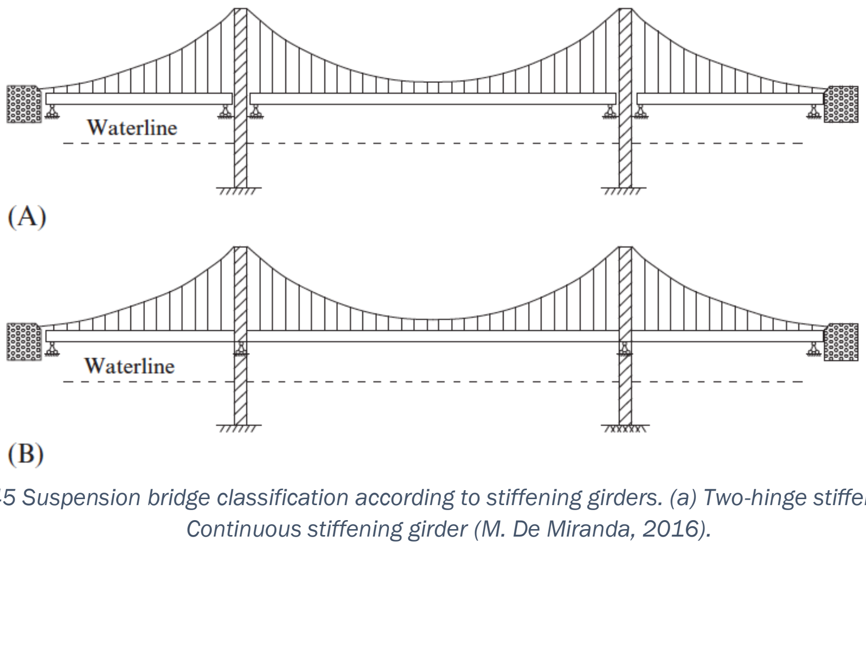


Figure 2.45 Suspension bridge classification according to stiffening girders. (a) Two-hinge stiffening girder (b) Continuous stiffening girder (M. De Miranda, 2016).

2.2.2.3. Suspenders

Suspenders or hangers in suspension bridges can be designed either vertical or diagonal. Vertical suspenders are more often used in suspension bridges. As for the diagonal hangers, the purpose of its use, is to increase the damping and improve the seismic performance of the bridge. In order to have a higher stiffness of a cable supported bridge, a combined suspension and cable-stayed cables system can also be used. A typical example for a suspension bridge with diagonal hangers would be the Humber bridge situated in England.

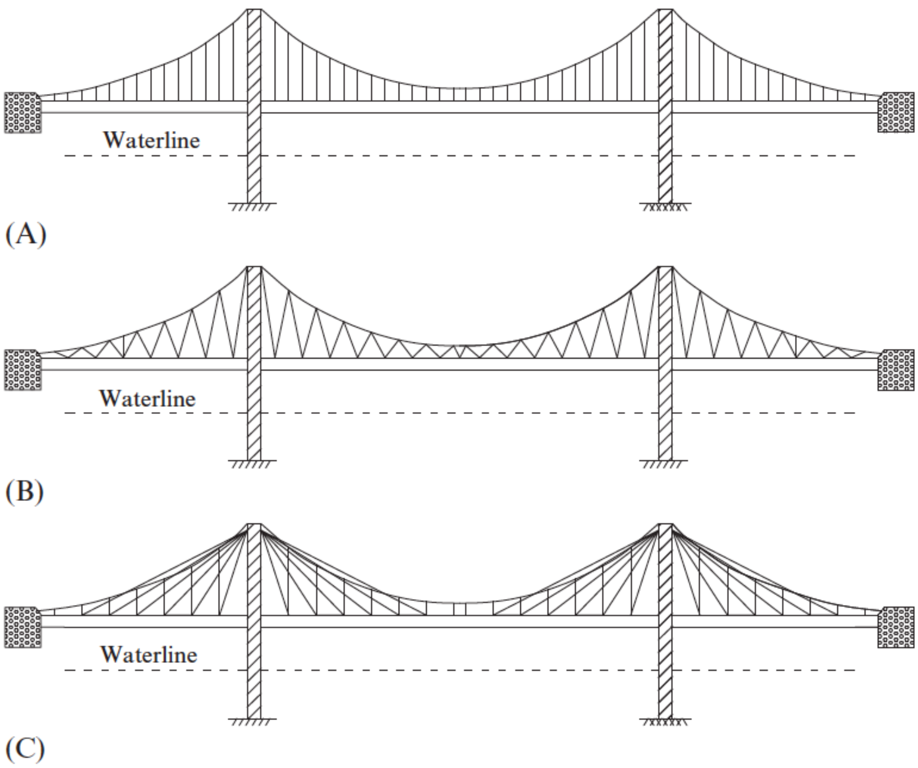


Figure 2.46 Suspension bridge classification according to suspenders. (a) Vertical suspenders. (b) Diagonal suspenders. (c) Combination of suspenders and cable-stayed system. (M. De Miranda, 2016).

Figure 2.47 Humber bridge, Kingston upon Hull, England, UK.

2.2.2.4. Anchoring Conditions

Suspension bridges can be classified according to the anchoring conditions; they can be either externally anchored or self-anchored, as shown in **Figure 2.48** the most common type of suspension bridges are externally anchored.
In order to do so, anchorages need to be built on both ends of the bridges to sustain the tensile forces from the main cable. The disadvantages of Earth anchoring occur in the difficulty of anchoring very large horizontal forces, and transmitting them to the soil, as usually these forces are applied meters above the strong layers. The self-anchored suspension bridge removes this last difficulty, as there is no need of anchorages, and only vertical loads are transmitted to the soil. Inversely, construction is more difficult, at least for long-span bridges, since the deck must be present when the cables are installed. Therefore, the deck must be erected on temporary supports (M. De Miranda, 2016). Usually for long-span bridges, external anchorages are more convenient (Lin W. et al. 2017). The San Francisco Oakland Bay Bridge is a typical example of a self-anchored suspension bridge.

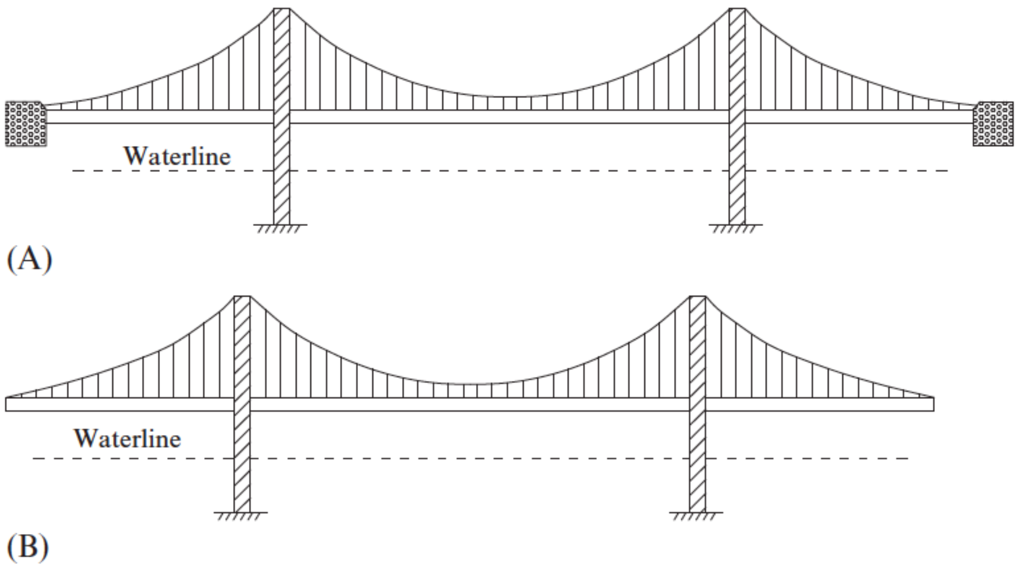


Figure 2.48 Suspension bridge classification according to anchoring conditions. (a) Externally anchored suspension bridge. (b) Self-anchored suspension bridge. (M. De Miranda, 2016).

2.2.3. Design Criteria

2.2.3.1. Geometry

A suspension bridges geometry can vary with respect to main towers and stiffening girders, the use of one type or another, will depend on the site conditions and the load applied.

2.2.3.1.1. Main Towers

Main towers of suspension bridges can be made out of steel or concrete structures. Concrete structures are typically more economical, at least in areas of low seismicity and with good soil conditions (M. De Miranda, 2016). At the top of these pylons, cable saddles are accommodated; here practically all the load of the half bridge is concentrated.

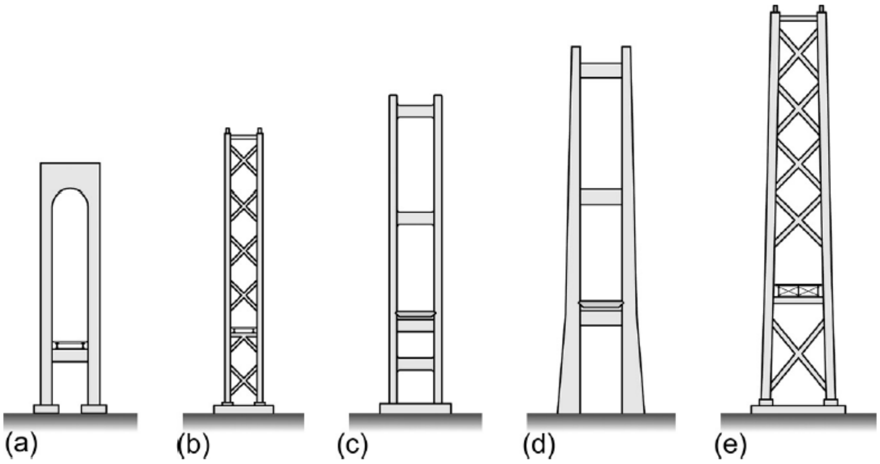


Figure 2.49 Typical layouts of suspension bridge towers, in size ascending order. (a) Simple frame. (b) Multiple frame. (c) Trussed. (d) Stain lined frame. (e) Stain lined truss (M. De Miranda, 2016).

In the longitudinal direction, towers can be classified into three types; rigid, flexible, and locking types. Flexible towers are commonly used in long-span suspension bridges, rigid towers are used for multi span suspension bridges, to provide enough stiffness to the bridge, and locking towers occasionally for relatively short span suspension bridges (Okukawa et al., 1999).

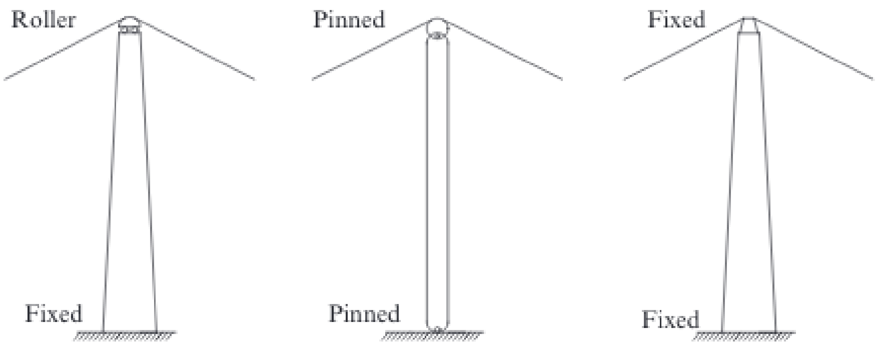


Figure 2.50 Towers in suspension bridges (Okukawa et al., 1999).

For towers in suspension bridges, the buckling due to the large compressive forces deserves special attention. As for a SFT, if subjected with columns, buckling due to tensile forces may happen and if subjected by tethers, foundations due to the same forces could be displaced.

In addition, the horizontal seismic performances in the tower for bridges and in shore connections and foundations for SFTs should be analysed.

2.2.3.1.2. Stiffening Girders or decks

The stiffening girder or deck is the surface of the bridge, it may be constructed of concrete for small spans, but they are usually made out of steel, or a combination of both. Usually, a railroad bed and track, asphalt concrete, or other form of pavement for vehicle crossing covers the deck.
In suspension, deck grinders usually come in three types;

- Truss structure
- Plate girder or I-girders
- Box girder

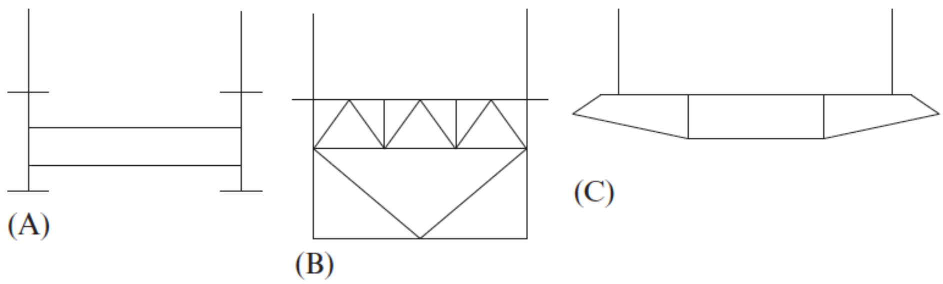


Figure 2.51 Typical suspension bridge cross-sections. (a) Plate girder. (b) Truss girder. (c) Box girder. (Okukawa et al., 1999).

In comparison to other decks, plate girders are very light and simple with respect to their design, but are disadvantageous with regard to their aerodynamic behaviour as they have a relatively low flexural stiffness and, mainly, very low torsional stiffness. However, the use of aerodynamic fairings can improve their aerodynamic performance and bottom bracing can improve torsional stiffness.

For modern long-span suspension bridges, trusses and box girders are more often used.
The use of truss-stiffened girders, gives great stiffness to the whole structure while a relatively low aerodynamic drag is maintained. However, for long spans, the weight of

steel is higher than for other systems. These leads to an increase of the deadweight and with it an increase of the bending moment when temperature variations occur. For long-span bridges, deadweight takes up a big percentage of the load carrying capacity, and so it should be taken into account during the bridge design.
The world's longest suspension bridge, Akashi Kaikyo (see **Figure 2.52**) has a truss-stiffened grinder (M. De Miranda; Lin W. et al. 2017).

Figure 2.52 Akashi Kaikyo Bridge, Japan.

With regard to box girders, even though, this kind of decks have a relatively high fabrication cost, they present many advantages, mentioned as follows;

- Streamlined-aerodynamical profile
- Aerodynamic drag is low
- Good flutter stability
- High flexural and torsional stiffness

However, the aerodynamical stability for a classical box girder suspension bridge depends on its main span length. Therefore, for long spans (>1600m) or very high design winds, the system would not satisfy the required stability. Richardson (1984) says that by splitting the deck into two streamlined box girders, flutter stability is greatly increased. Still, Farquharson (1950-1958) states that the higher stability in decks is satisfied with central openings. **Figure 2.53** shows the construction of a box girder suspension bridge situated in Guangzhou, China.

Figure 2.53 Second Humen Bridge, Guangzhou, China.

As in SFT, there are several cross sections for suspension bridges, but some act better than others depending on the site conditions and the purpose of each structure. Comparing both types of cross sections, it can be appreciated, that suspension cross sections are much simple, in terms of materials, and SFT cross sections are much simpler, in terms of geometry.
Both cross sections have to act against loads, aerodynamic and hydrodynamic loads, which lead to stability problems. In SFT hydrostatic loads must be considered too.
The main problem for a SFT cross-section that does not concern a suspension bridge is impermeability. A poor arrangement in terms of materials or a crack in the structure due to specific events (ice, ships, submarine collision etc..) could eventually lead to collapse due to pressurized water access.
In terms of security, for a SFT the construction of a parallel tunnel or exit towers could be proposed, and so, in case of an accident, traffic can be redirected and emergency exits are available. In suspension bridges, traffic access must be cut.

2.2.3.1.3. Stability of the Bridge

In a suspension bridge, the length and the stability of it depends on the sag ratio.
The sag ratio of the main cables is known as;

$$n = f/L \qquad \text{eq (2.9)}$$

Where,

f = vertical interval of the main cable in the main span known as the sag.
L = length of the span.

The sag ratio is generally taken between 1/8 and 1/12 (a generally accepted optimum ratio is a 1/10). A bridge with a low sag ratio or said in another way, if the main cables have a flat curve, the bridge has more vertical stability but the cable stress is high and strong anchorages are required. On the contrary, a higher ratio or a deeper sag, reduces cable force, but increases the height of the main towers and makes them more susceptible to large forces and consequently the bulking increases.

The geometrical shape of a main cable is shown in **Figure 2.54.**

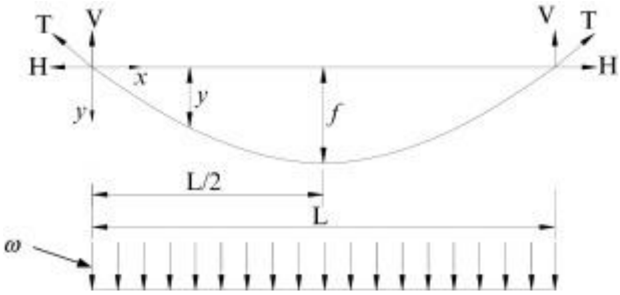


Figure 2.54 Shape of the main cable (Lin W. et al. 2017).

Lin W. et al. (2017) demonstrates that generally the main cables deform as a parabolic curve if it is considered that the cables and the dead weight (w) of the stiffening grinder are uniformly distributed.

As explained before, the stability of a SFT will depend on a similar ratio, the buoyancy ratio, where instead of relating the vertical interval with the length of the span to stabilize the deck is vertical displacement, it relates the buoyancy of the tube with its self-weight in order to stabilise the tube in a precise depth.

2.2.3.2. Cables

Suspension bridge cables are always made out of strong steel in parallel wires and are galvanized, with diameters of 5,2–5,7mm.
Hangers are made with wire ropes, spiral cables, locked coil or parallel wire ropes. As previously mentioned, always in galvanized steel and protected by sheaths and often in high-density polyethylene. The clamps that support and anchor the hangers to the main cables are typically made of cast steel, with bolt anchors.
The hangers are attached to the deck by hinge type anchorages next to the mooring block, as there is maximum longitudinal rotation on the hangers, and rigid type anchorages for the intermediate hangers.
De Miranda 2017 analyses vertical and horizontal loads produced on suspension bridges, in order to make a good sizing of the cables. For vertical loads, she analyses permanent and moving loads and for horizontal, she focuses on the actions of the wind.

Figure 2.55 Cross section of the main cable in Akashi Kaikyo Bridge (De Miranda M., 2017).

2.2.3.3. Anchorages

The anchorage structure usually includes; the foundation, anchor block, bent block, cable anchor frames, and protective housing.

These members receive through the main cables, most of the self-weight and other loads of the bridge. In order to prevent cable force concentration inside the anchorages, the cables are spread over a large area so that the load is distributed evenly.

Bad dimensioned anchorages can suffer from rotary movement and slippage, and so before anything, safety check should be confirmed.

The most often anchorage used nowadays in construction are the gravity type anchorage systems as shown in **Figure 2.56** (A) and the tunnel type anchorage as shown in **Figure 2.56** (B).

Gravity anchorages usually consist on the construction of huge concrete blocks, which relies on the mass of the anchorage itself to resist the tension of the main cables, that is why, before their construction a precise knowledge of the total systems load should be given. On the other hand, the tunnel anchorage takes the tension of the main cables into the ground. On this type of anchorage, adequate geotechnical conditions are required.

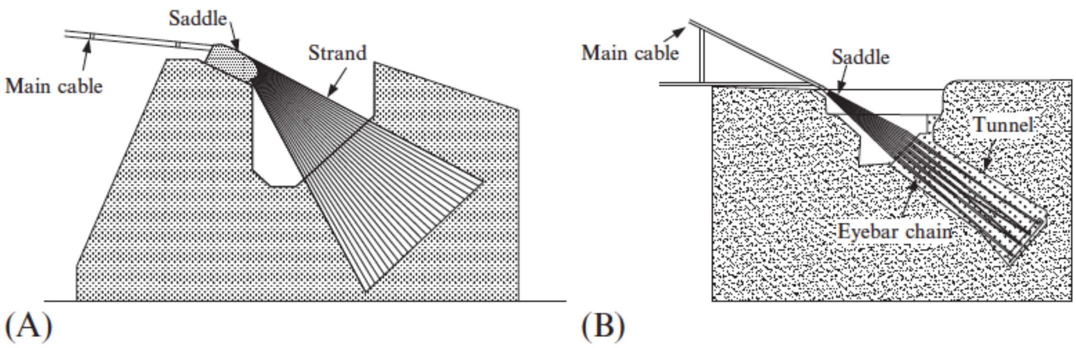


Figure 2.56 Anchorage systems. (A) Gravity type. (B) Tunnel type. (De Miranda M., 2017).

2.2.3.4. Loads

The following loads presented as follows are based on the AASHTO specifications. In general, this loads can be divided into two big groups: permanent loads and transient loads (functional, fluid, seism, deformation and accidental loads). Additionally, depending on the structure type, other forces due to creep, shrinkage, or movement of the structure supports can take place.

1. *PL (Permanent Loads):*

Permanent loads include;

- Dead weight of structural elements and non-structural elements (DC).
- Dead weight of coating surface and accessories (DW).

The structural elements are those forming part of the resistance system. Inside the non-structural elements include barriers, parapets, signals etc. In the case where no information of the technical specifications of the precise weight is given, unitary weights from the **Table 2.14** Extracted from AASHTO specifications can be used. Dead weight of coating surface can be estimated taking the unitary weight for a surface thickness.

Materials	Unitary weight(Kg/m3)
Aluminium	2800
Bituminous surfaces	2250
Sand, clay or compacted silt	1925
Lightweight concrete (including reinforcement)	1775
Lightweight concrete with sand (reinforcement included)	1925
Normal concrete	2400
Sand, silt or loose gravel	1600
Soft clay	1600
Ballast	2450
Steel	7850
Stone masonry	2725
Hardwood	960
Soft wood	800
Rail for road traffic	300 Kg/ml

Table 2.14 Unitary weights.

2. *Fluids*

Structural loads due to fluid flow (water or air) are stablished as seen in the SFT chapter by the Bernoulli equation in combination with correction coefficients.

2.1. *Wind loads*

Wind speed varies with the altitude and roughness of the terrain it travels. The wind speed increases with altitude as seen in **Figure 2.57** V_g represents the limit speed, regardless of any surface, δ is the layer thinkness and V_{10} is the velocity at 10m.
For bridges at heights less than 10m the V_{10} speed will be used, and for bigger heights the AASHTO equation will be used;

$V_{DZ}=2{,}5V_0(V_{10}/V_B)\ln(Z/Z_0)$ eq(2.10)

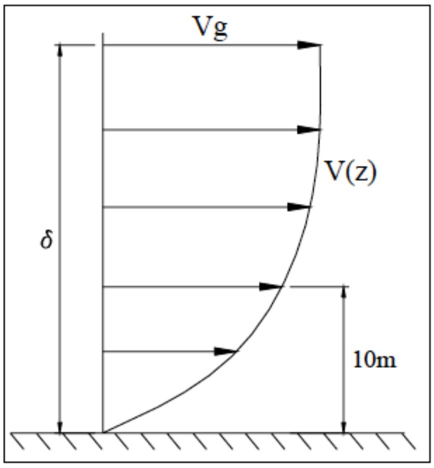


Figure 2.57 Speed profile

V_{DZ} represents the expected design speed in Km/h at an altitude Z. V_B is the base wind speed at 160 Km/h. V_0 is the friction speed and Z_0 the friction length, both are obtained from the following table.

	Open field	Semi-urban	City
V0(km/h)	13,2	15,2	25,3
Z0(mm)	70	300	800

Table 2.15 V_0 and Z_0 values.

The wind pressure acting at the structure is related with the base wind speed V_B = 160 Km/h.

$$P_D = P_B(V_D/V_B)^2 = P_B V_D^2/160^2 \qquad eq(2.11)$$

Pressures for the base wind speed are given in **Table 2.16**

Structural element	Windward load(MPa)	Leeward load (MPa)
Columns, arches, armour etc...	0,0024	0,0012
Beams	0,0024	N/A
Long flat surfaces	0,0019	N/A

Table 2.16 P_B values corresponding to V_B.

A 1,6 N/mm wind load applied at 1,8m above the road surface must also be considered for each vehicle.

2.2. Hydraulic loads

The water that surrounds the substructure of the bridge creates lateral forces that act directly on the structure, due to that sediment can be accumulated on the bridge towers foundations and so increase lateral forces. The following equation represents this lateral forces or drag forces.

$$F_D = 1/2\ \rho_w C_D u(t)^2 \qquad eq(2.12)$$

Where C_D as mentioned before is the drag coefficient given in **Table 2.17**and u(t) is the water velocity in m/s. If the sub-structure is at an angle to the current, corrections must be made in accordance with the specifications of AASHTO, the same will happen for sediment deposited in the substructures of the bridge, the area must also be corrected. This could also apply to a SFT and should be considered in its design.

Type	Cd
Semi-circular pillar	0,7
Square pillar	1,4
Pillar with diverted flow	1,4

Table 2.17 Drag coefficient values.

3. *Seismic loads*

Depending on the bridge location, seismic effects can be ignored or might influence on the bridge design. J.D. Yau et al. (2007) analyses the response of a suspended bridge due to moving loads and seismic loads. E.S. Manrique (2004) explain how to determine these charges by following a few steps.

a) Preliminary design (bridge type, span number, tower height, road section, etc..).
b) Acceleration coefficient, which depend on the bridge location, this coefficient is obtained from the seismic zoning map.

c) Importance of the bridge category. **Table 2.18** Summarizes the characteristics of the three categories of importance.

Importance category	Description
Critical bridges	After the design earthquake (475 years return period) they must be open to all traffic and after a long earthquake (2500 year return period) must be open to emergency vehicles.
Essential bridges	After the design earthquake must be open to emergency vehicles.
Other bridges	Can be closed to be repaired after a long earthquake.

Table 2.18 Importance category.

d) From the acceleration coefficient determinate, the seismic operation of the zone.

e) Determinate the place coefficient S. This coefficient depends on the soil conditions.

f) Finally determine the response modification factor, which reduces seismic forces based on the elastic analysis of the bridge system.

4. Deformation loads.

4.1. Temperature

Two types of temperatures must be considered in the structural analysis. The first type is the uniform temperature change. This type of change in temperature affects long and short bridges, and if the supports are restricted it will cause forces in the stirrups of the bridge. This type of deformation is seen in **Figure 2.58** a). The second type the temperature change is a non-uniform heating or cooling gradient, this deformation can be seen in **Figure 2.58** (b).

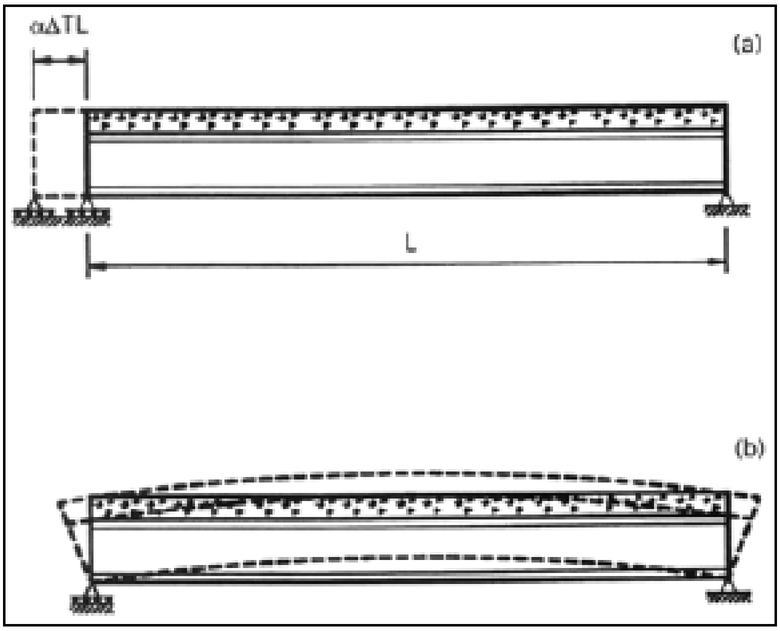


Figure 2.58 (a) Temperature which causes dilatation. (b) Temperature which causes curvature.

Exposure to sun rays, the bridge platform will get hotter than the beams from below, and so beams try to bend upwards. If this movement is prevented by internal supports, internal forces will be induced. If the movement is completely free internal stress will occur, just like the simply supported beams.

4.2. Creep and Shrinkage

The creep and shrinkage effects in the concrete produce structural stress and fatigue. These effects are of special importance in prestressed concrete and in superstructures with large volumes of concrete.

4.3. Settlement

Support movements can occur due to elastic deformations or inelastic deformations of the foundations. Elastic deformations include movements that affect the response of the bridge to other loads but permanent deformations will not be seen.

5. Accidental loads

The impact forces on a bridge during collision depend on parameters such as the vehicle type, size, mass, and speed as well as the circumstances of the accident (eccentricity of impact, size and shape of structures etc…). Normally the duration of action of the maximum force in vehicle/bridge collision is very brief. Z. Kamaitis (1997) proposes a formula to calculate the average impact force caused by vehicle. He also proposes a formula to determinate the same force for collision of floating debris, ice and marine traffic.

2.2.3.5. Limit States

In a suspension bridge, the same limit states are considered as in a SFT (Serviceability, ultimate and fatigue), the difference is that possible combinations of loads for each design limit state can be precisely known due to that fact that previous bridges have been constructed, these values can be found on Eurocode 1 – Actions on structures.

2.2.4. Construction methods

Classic suspension bridges always have to follow de same operation sequence (de Miranda M., 2008);

- Construction of towers and mooring blocks.
- Formation of the supporting cables and installation of hangers.
- Installation of the girder deck.

The deck installation is done by lifting the structural elements of the deck (or panels of the truss segments or whole segments) from the sea (or from the river or ground depending on the environment) by a crane positioned on the cables or on the already-erected deck.
The installation of box-girder decks in bridges starts from the centre line, where the segments are assembled initially, and proceeds symmetrically to the towers to reduce the risk of flutter during construction (see Figure 2.59). For bridges with trussed decks, truss segment installation normally starts from the towers, proceed symmetrically toward the middle of the central span, and toward the end of moorings (see Figure 2.60).

During the erection of adjacent segments, previous lifted segments are temporarily connected with devices designed to allow mutual rotation and to guarantee the necessary stability due to the wind dynamic effects (de Miranda and Petrequin, 1998).

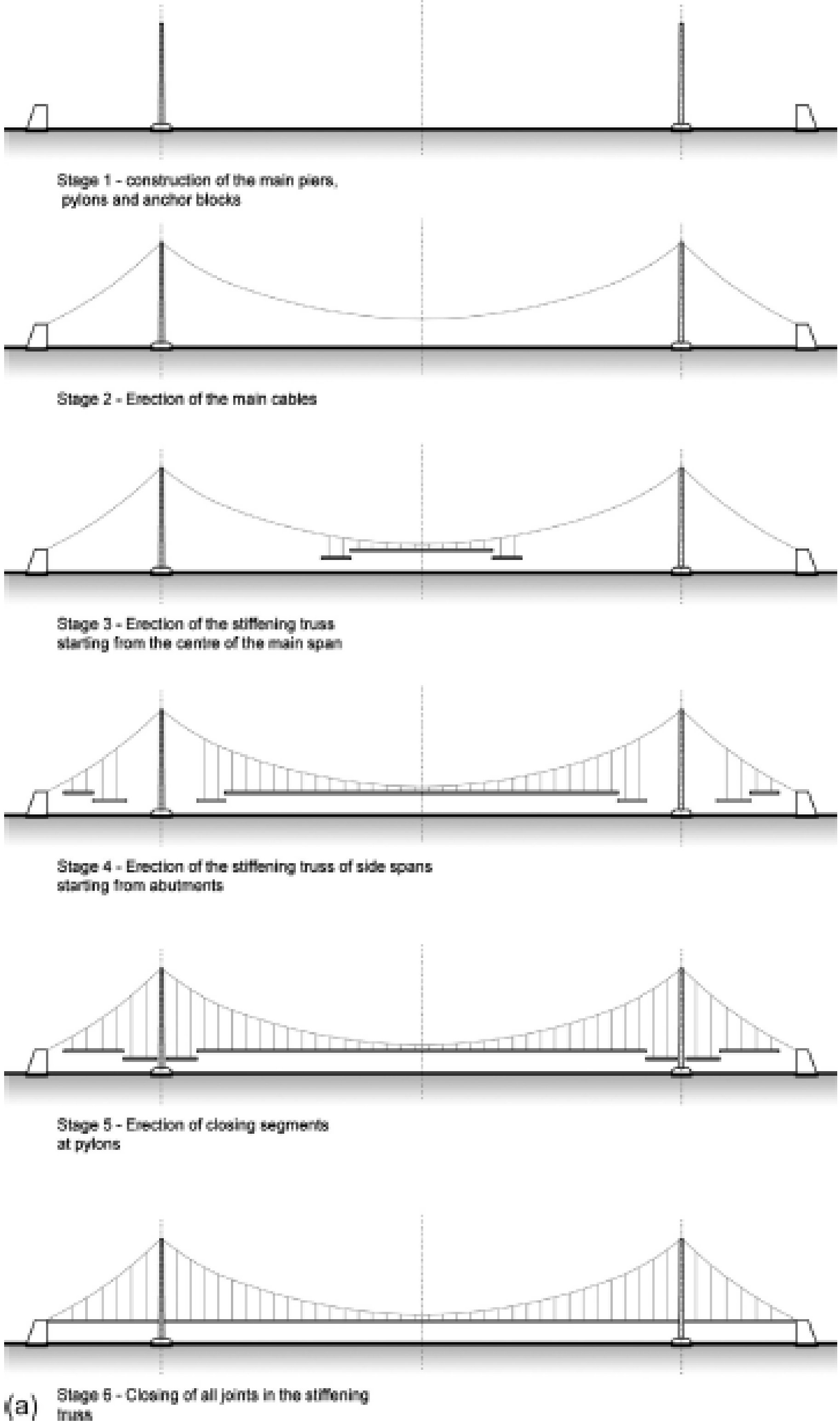


Figure 2.59 (a) Erection sequence of typical box-girder suspension bridge, by starting from mid-span in order to minimize the risk of flutter during construction (Miranda M., 2008).

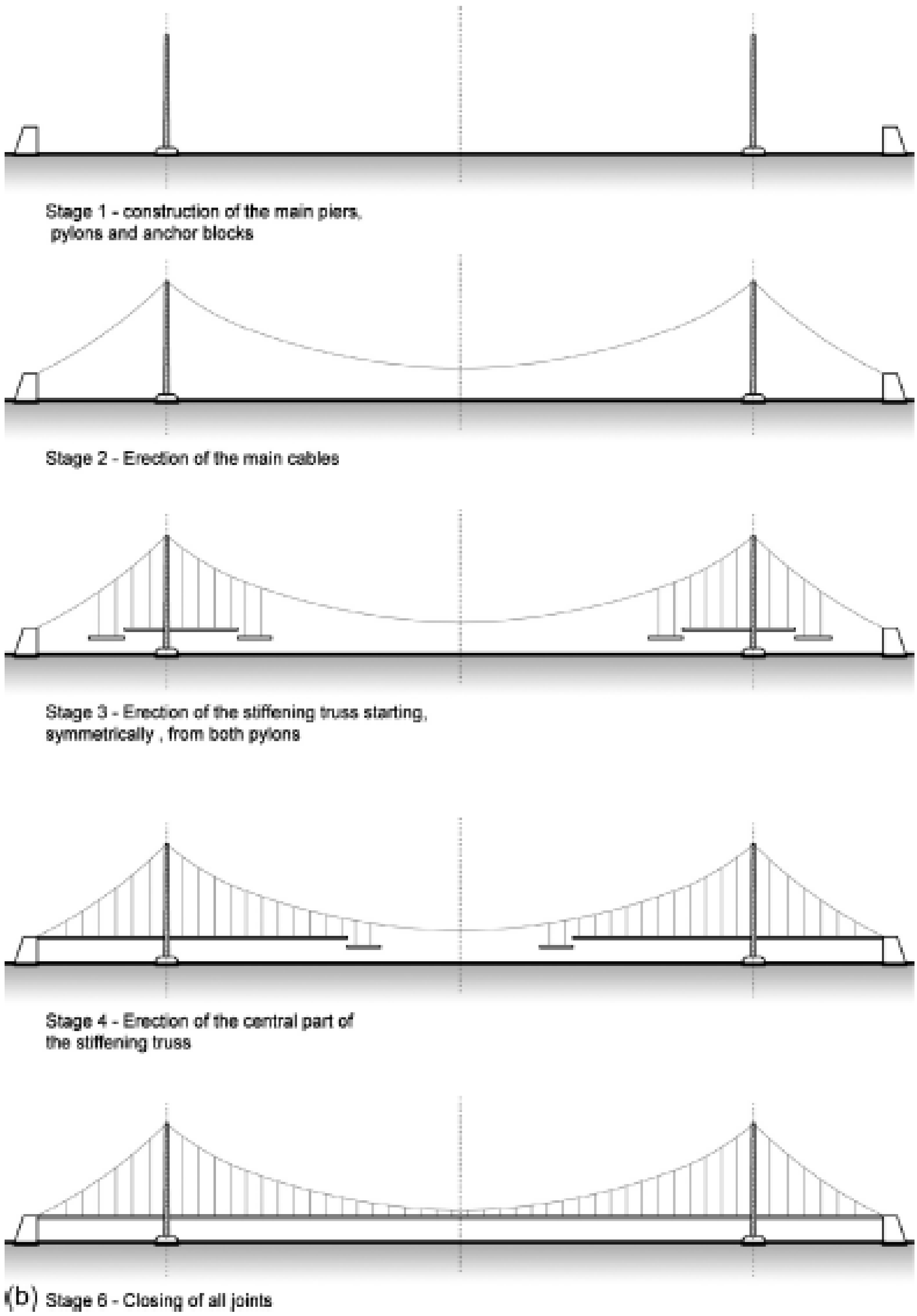


Figure 2.60 Continued. (b) Erection sequence of deck starting from pylons *(Miranda M., 2008)*.

3. Comparison between a Suspension bridge and a SFT in different locations.

In general, there can be several similarities noticed between the SFT and a suspension bridge. For example, the loads are analogous; the bridge deck is supported through vertical or inclined cables and the tunnel tube too; the gravity loads on the cable-supported bridges are replaced by the upward residual buoyancy in the SFTs and the role of wind being played by currents and waves actions. Of all these similarities, the most important which has not been mentioned is that, both infrastructures offer the possibility of crossing long distances in the fastest way. That is why nowadays in many cases where a suspension bridge is proposed to be build, the possibility of constructing a SFT may also appear. In this chapter, it will be seen examples of different locations where a comparison is made between both ways of crossing in order to obtain the most viable way.

3.1. Qiongzhou Strait, China.

Due to the demand traffic and development of civil engineering in China, several proposals of waterway crossing have been carried out. At present Qiongzhou Strait is frequently discussed. Qiongzhou Strait is located in the south of China, between Leinzhou Peninsula and Hainan Island.

The Qiongzhou Strait is one of the larges three straits in China. The length is about 80 kilometres from east to west, and the widths ranges from 18 to 35,5 kilometres from north to south. The water depth ranges from 80 to 120 m along the central axis. On the north and south shores, scraps with a maximum height of 70 metres and a maximum slope angle of 22º-24º are located.
The major geological structure is made out of fractures. The submarine strata are mainly sedimentary shaped in the Tertiary and Quaternary Periods. The upper layer is filled with sludge, sandy clay or silt, and thick-bedded clay and silty sand are distributed on bottom layer.

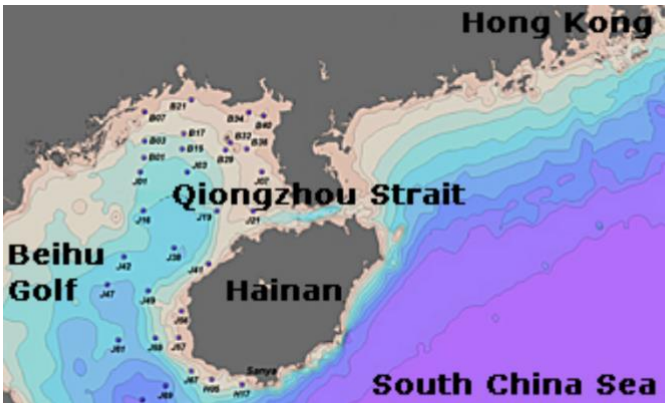


Figure 3.1 Situation map.

About ten earthquakes that caused lots of damage have occurred in this area, nine of which are over six in magnitude. The average annual temperature is 24ºC. Being from May to October the rainy season and from May to November the typhoon season, especially in September.

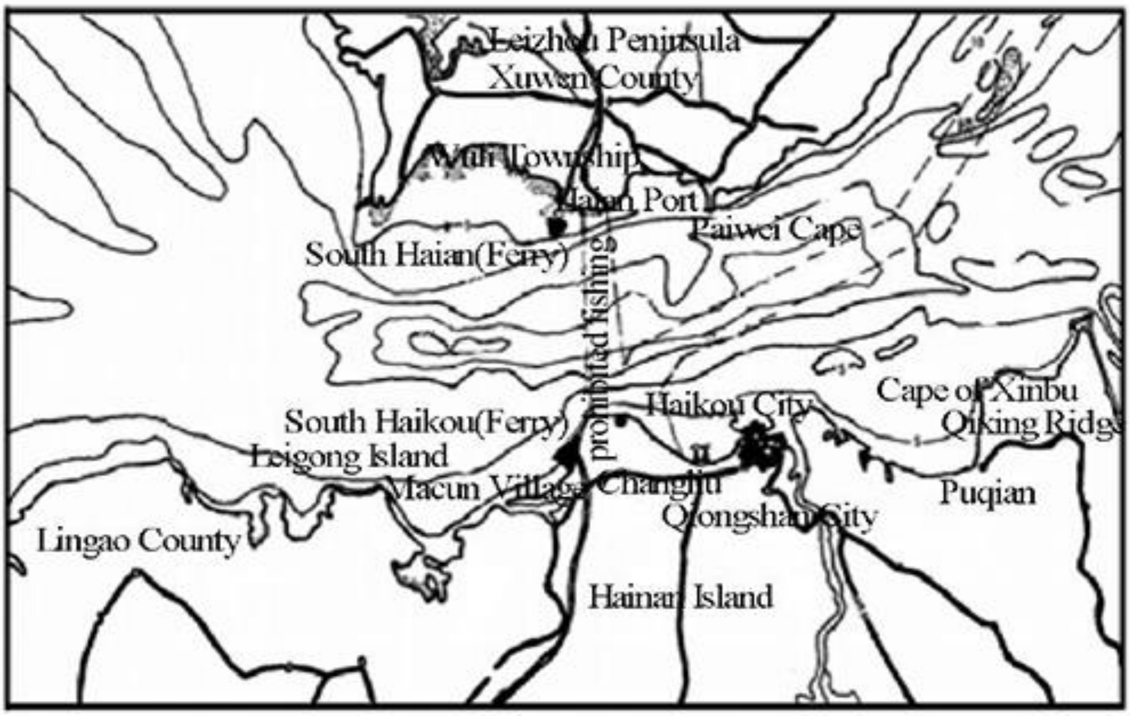


Figure 3.2 Seabed topography (Yan et al. 2015).

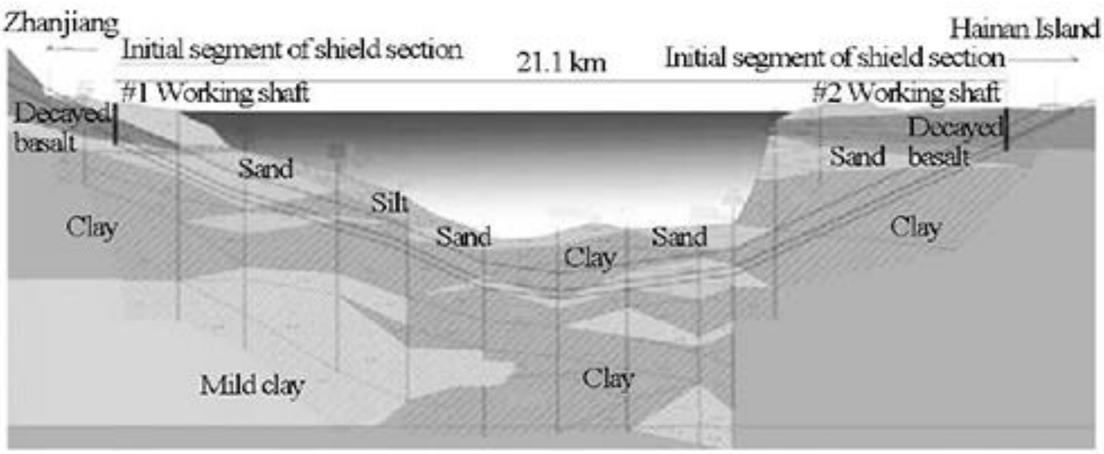


Figure 3.3 Geological section and bored tunnel proposition (Yan et al. 2015).

According to careful investigations and evaluations, in general there are three mostly proposed ways of crossing; the east line, the central line and the west line.

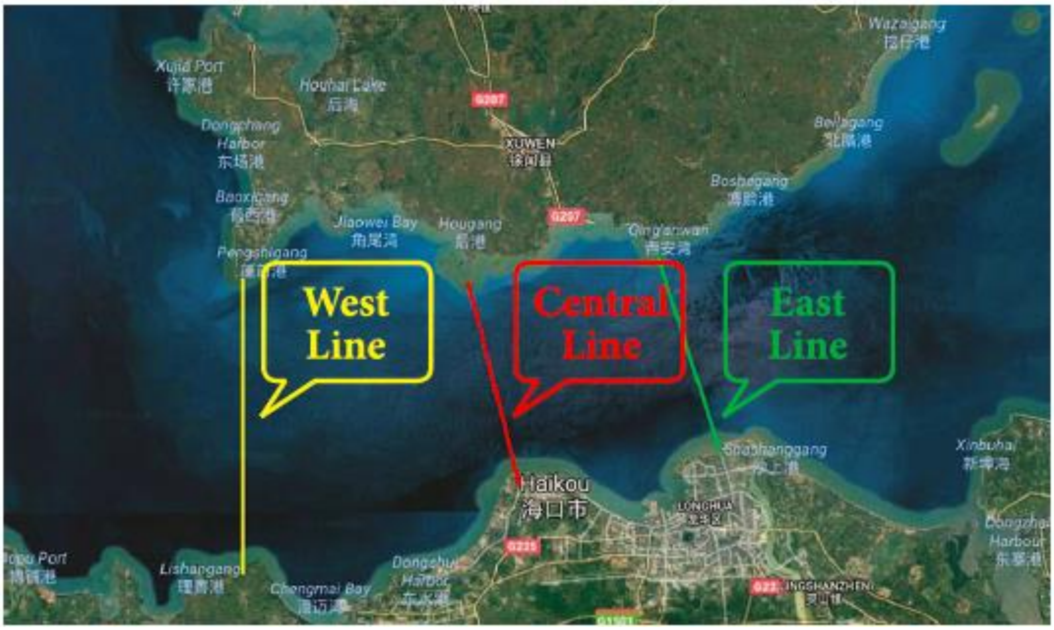


Figure 3.4 Proposed ways of crossing for Qiongzhou Strait (Yan et al. 2015).

Jiang B. et al. (2018) make a feasibility study, he proposes three ways of crossing through the central line location, which would be resumed on **Figure 4.5.**

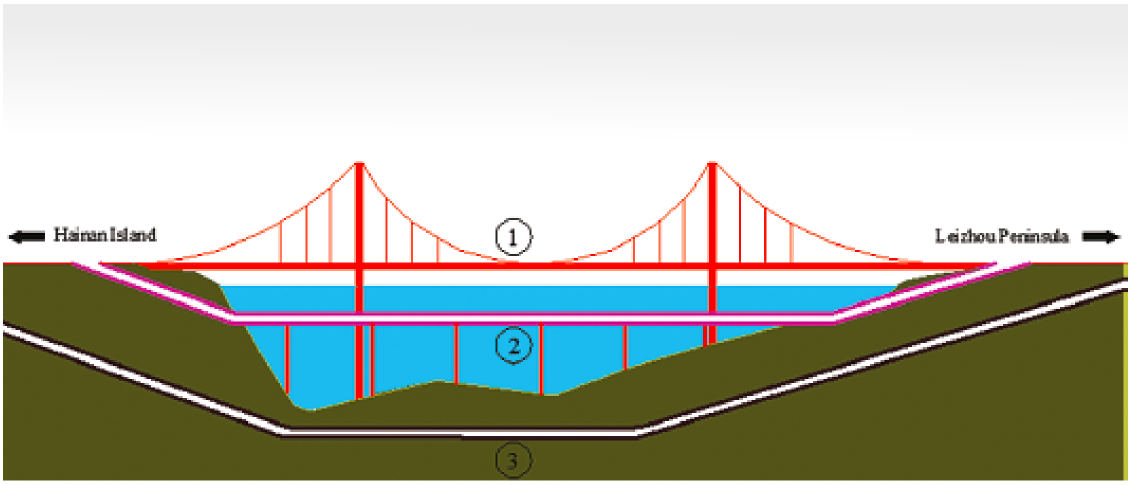


Figure 4.5. Alternatives of solution: 1) Suspension bridge. 2) SFT. 3) Submarine tunnel.

It is proposed a suspension bridge, a SFT and an underground tunnel, but we will focus only in the comparison between the SFT and the suspension bridge made.

It is evaluated which is most viable, by focusing on previous studies done, like for example the study made by Yan et al. (2015) which make several proposals for SFT schemes in Qiongzhou Strait. Advantages and disadvantages with the implementation of each way of crossing are pointed out in order to have a better view on which is more viable.

Suspension Bridge

Advantages

- *Advanced technology of design and construction*

Despite the hydrological, meteorological, geological and navigation conditions in the strait, China has rich experience in the construction of long-span bridges, such as the Zhoushan Xihoumen Bridge and the Jiangyin Yangtze River Bridge.
In the central line location, it is feasible to construct a multiple spans bridge with a single span of over 1000 metres. Nevertheless, the construction of the piles is very difficult due to the conditions aforementioned; this problem can be solved by means of updated construction technology.

- *Better traffic conditions*

Bridges offer better lighting, ventilation, vision, comfort and so on when the vehicles are running on the bridge compared to tunnels.

- *Easier and more economical maintenance*

The bridge requires less expense on lighting and ventilation compare with tunnels. The bridge is easier for maintenance service.

Disadvantages

- *Higher expense of construction*

The crossing line is around 20km. Jiang et al. says that if the bridge is designed with single spans of around 1000m, about 20 piles and foundations will be needed. Therefore, as the average water depth is around 50m, piers and foundations will be difficult to construct, and the cost will be extremely high. In order to decrease the number of piles, the span length could be increased, but this would affect directly the bridge main towers height. He analyses the relationship between the critical length of span (Lc) and the height of the main tower (h) taking into account only the self-weight of the cables and different materials.

HSS: High strength steel.
LSS: Low strength steel.
FRP: Fibre reinforced polymer.

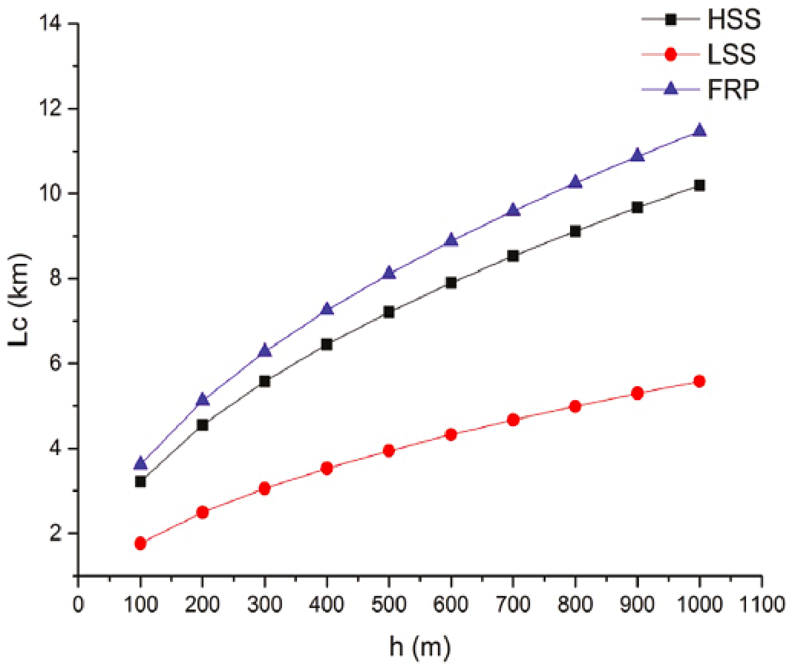


Table 3.1 Relationship between height main tower and critical length of span.

From **Table 3.1** we can see that if HSS is selected as cable material, when the single span measures 5000m, the height of the main tower increases up until 1000m. That is another reason why the design and construction of foundations becomes very complicated. In addition, the current technology and economy can hardly support such height.

- *Weather*

The Strait is located in a tropical area where it is subjected to tropical monsoon climate. In this area, every year, from May to November is the typhoon season, and so an average of 5 days in September, that the wind force is stronger than Level 8 (62-74km/h). Moreover, there are more than 24 days with dense fog in average each year. A bridge is directly exposed to the natural environment, and so the traffic condition is largely affected by weather conditions.

- *Impact on air and water traffic.*

The Qiongzhou Strait is a very busy strait. The construction of a suspension bridge will have impact on the navigation of ships, due to the bridge piers. Moreover, the height from the deck to the water surface limit big ships from crossing the strait, and an increase of the decks height will difficult the bridges access to certain vehicles due to steep road access. In addition, the main tower height as seen before us quite tall, thus will affect flights.

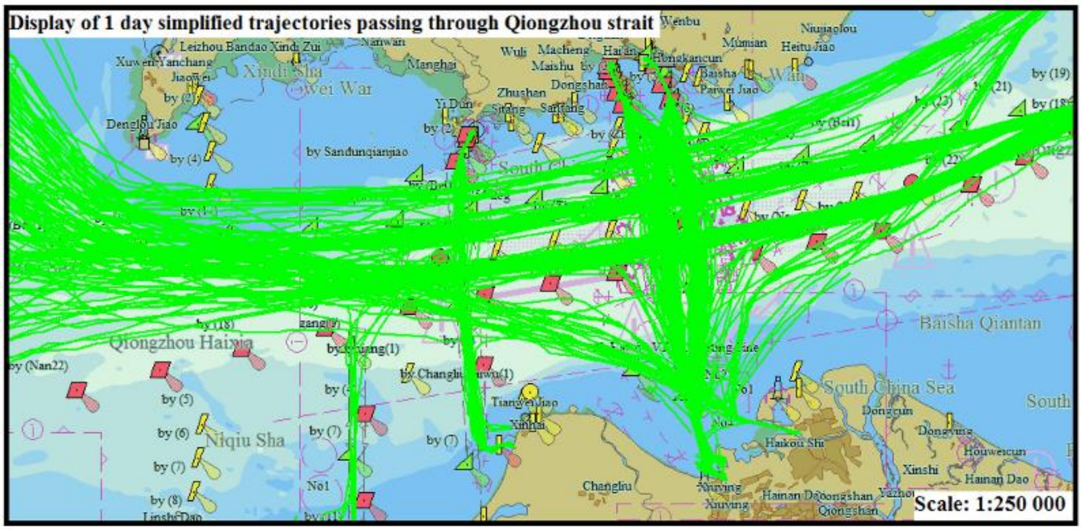


Figure 3.5 Ship traffic of one day in the Qiongzhou Strait.

- *Environmental impact*

The construction of bridges piles and foundations will have a direct impact to the seabed and as a consequence to the sea flora and fauna. In addition, no matter how elegant and high-tech a suspension bridge can be, its construction cannot avoid interfering in some way with the surface environment.

- *Pollution*

While the main purpose of structural systems is to enable people to travel from one point to another, several of them litter the route. As a result, areas around bridges, overhang, passes often contain mounds of littler. This is why areas in and around these bridges are often affected, influencing local life, both flora and fauna. Furthermore, increased traffic over the bridges often results in chemical and air pollution, thereby making it harder for those residing close by.

- *Noise and vibrations*

In pre-construction stage, mobilization of construction equipment will generate noise but it will have a low impact on the environment. By contrast, a high negative impact will be seen at the time of construction of bridge substructure especially during pile driving, which generates high underwater and air noise levels that affect the aquatic life. Also, during the operation and maintenance stage, there will still be a negative noise impact to the environment.

Submerged Floating Tunnel (SFT)

Based on the geological condition of the central line and the 30m of clearance depth between the water surface and the submerged floating tunnel proposed by Xiang et al. 2017 in order satisfy the requirement of navigation space for big vessels and reduce the impact from the water surface wave, **Figure 3.6** represents the longitudinal profile for the proposed SFT.

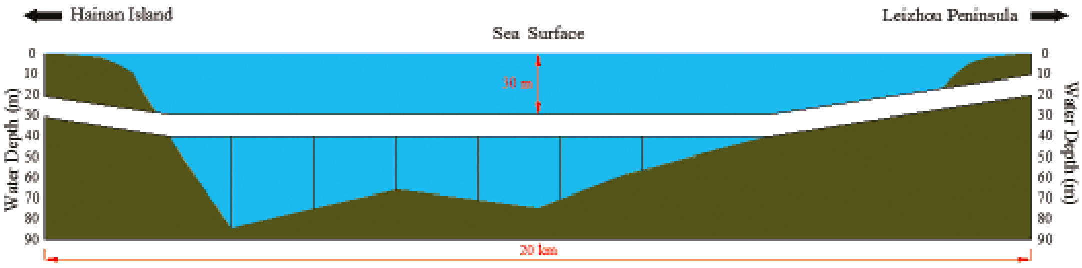


Figure 3.6 Longitudinal profile of proposed SFT.

Advantages

- *Shorter distance*

From **Figure 3.6** we can clearly see that the distance of the submerge floating tunnel is the shortest, for it does not need to consider the slope.

- *Influence on air and water traffic*

Firstly, it is obvious to see that the SFT will not have any impact to air traffic. Besides, the gap between the SFT and the water surface where it is intended to be build, can guarantee the security of navigation. Big ships will still be able to cross.

- *Environmental impact*

The SFT will not interfere with the natural beauty of the surroundings. However, its construction will still affect the seabed flora and fauna due to the tunnels foundations.

- *Sea current*

In summer, southwest monsoon prevails and currents flow from west to east with a high speed. In other seasons, currents flow from east to west with low speed. This is a disadvantage to bridge, but such regular current flowing is an advantage to submerged floating tunnel, as currents vertically act on SFT, no matter flow from east to west or otherwise.

- *Lower cost*

The construction costs of a SFT, per meter length, do not increase significantly with the length of the crossing or with the depth of the waterway, whereas the cost of the suspension bridge will rapidly increase as the span increases.

- *Noise and vibrations*

As a suspension bridge, pre-construction stage mobilization of construction equipment will generate noise and construction of foundation plus the tunnel colocation will have an important impact to the sea environment. But, during the operation and maintenance stage, the noise and vibration impact will be less than in a suspension bridge (World Road Association Rodiale de la Route). In conclusion, allows reduction of acoustic noise pollution.

Disadvantages

- *Foundations*

The cables will need a foundation, and the requirements of such a foundation is strict and important.

- *Pollution*

Outside a tunnel, the concentrations of pollutants can be harmful to people and to the environment.

- *Experience on construction*

Until now, no SFT project has been constructed, and so the lack of relevant construction experience is a big challenge.

Finally, he develops a numerical model by means of the FEM software ANSYS/Fluent considering appropriate cross section, support system, materials, joints and connection schemes for a SFT and analysed the way this type of crossing responds to different water depths, different wave loads and different current velocities.

From this model, it is obtained that the proposed SFT structure is safe even under extreme weather conditions. By considering previous cost studies made which say that the construction of a SFT would suppose a lower cost than a suspension bridge and the viability analysis, he finally concludes, that the SFT is a better solution.

3.2. The Sognefjord, Norway

Until now, travelling from the southern port of Kristiansand in the south to Trondheim in the north of Norways (1000Km, E39) takes around 21h by car, at an average speed of 48km/h and thanks to the seven ferry crossings needed.
In the past years, different studies have been made in order to upgrade this route. About half of Norway's traditional export is generated by industries and companies from the six counties route E39 passes through. Studies show how crucial this route is to regional development in western Norway, as many synergy effects depend on how effectively the corridor interconnects areas with large populations and substantial trade and industry. The upgrade of this route will also enable direct access to hospitals by car, as until now, neighbours had to wait for helicopters. The new infrastructure replacing the ferry crossing could cut the travel time up to 10,5h by car.

The Sognefjord which is about 3,7 km wide and 1300m deep, is of particular interest as it has been used to develop and design new concepts of crossing, hitherto unthinkable. This fjord is considered the most difficult and challenging to cross.
While the depth of the Sognefjord is extreme, the other fjords along the route are more typically some 500-600 m deep.

The Norwegian Public Roads Administration which is responsible for the planning, construction and operation of the national and county road networks, has initiated several development studies in order to determine which way of crossing would be more viable. There are three ways proposed; through a suspension bridge, a floating bridge or a submerged floating tunnel, also a combination of these crossings have been considered as a solution. Even though there are several ways of crossing proposed, as on the past example, we are only going to pay attention to the suspension bridge and the submerged floating tunnel.

Figure 3.7 Situation map of the proposed crossings.

The studies of the suspension bridge have been undertaken by the Bridge section of the NPRA Directorate of Roads in Oslo, and have considered a suspension bridge with a main span of 3700 metres, as the very steep slopes into the fjord, require that the main span have the same length as the width of the fjord. There are also considered two main towers with a total width of 33 metres and 455-metre-high.

Figure 3.8 Proposed suspension bridge for crossing the Sognefjord.

For the case of the Submerged floating tunnel, studies have been undertaken by the consulting groups Reinertsen/Olav Olsen, Snøhetta Architects, Rambøll, Faltinsen, Johansson, Arup and Berger ABAM. After studying several possibilities, they have found that the most feasible way of crossing through a SFT is by means of two separated and interconnected curved circular concrete tubes that are anchored to floating pontoons. The tunnel will have a length of 4083 metres, and will enter traditional rock tunnels at both sides of the fjord for connection to the main corridor.

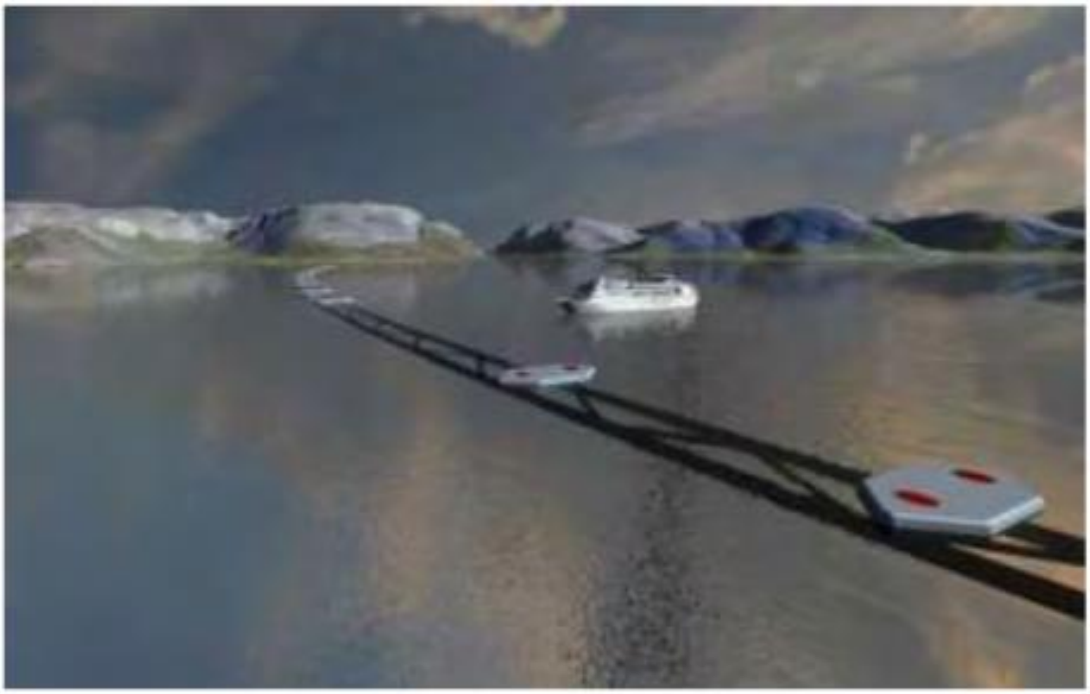


Figure 3.9 Proposed SFT for crossing the Sognefjord.

The overall design requirements in this area are said to be satisfied, as both ways of crossing have fulfilled width, depth and height sailing clearances of 400 metres and 70 metres, respectively. In addition, both are designed to absorb collision energies without risk of fatal accidents for road users as well as for passengers or crew on board the ship. Until now, the conclusions obtained are that crossing the Sognefjord is feasible by means of either a single-span suspension bridge and submerged floating tunnel. On the other hand, O.M.K. Iversen (2014) on his master thesis, makes a early phase life cycle assessment considering material production, construction, operation and maintenance over 100 years for both ways of crossing. He focuses mainly on their environmental impact and finally concludes that the Submerged Floating Tunnel (SFT) has the highest total emissions, with about 605 900 tonnes of $CO2$-equivalents and the Suspension Bridge (SB) had emissions of 493 200 tonnes. Yet, the Norwegian Public Roads Administration does not seem to have completed the considerations of construction cost or technical optimisation of the designs on the technical feasibility study.

4. Feasibility study of a SFT in the Chacao Channel.

The Chacao Channel is located in Los Lagos Region, and separates Chiloé Island from mainland Chile. This Channel connects the Pacific Ocean and Gulf of Ancud. The new connection will replace the current car ferry service and will reduce travel time from 45 minutes to under three minutes.

This channel has prevented the development of the Chiloe Island. Building a new connection between mainland and Chiloe island, apart from reducing the time travel as said before, a better social development in terms of health, services, education and employment will be possible. In addition, this new connection will have a positive impact on the economic development of the island. Studies show that tourism will increase, there will be an improvement on the industrial sector and will influence on the need of more residential infrastructure. That is why, finally it has been approved the construction of a new suspension bridge.

The aim of this chapter is to analyse the feasibility of a SFT in this specific location and see whether it is more viable either to construct a suspension bridge or a Submerged Floating Tunnel.

4.1. Introduction to the Site

This channel is approximately 26 Km long from NW-SE between Chocoi and Coronel, and varies between more than 2 Km wide in the Roca Remolinos and up to 5 Km wide between Lenqui and Pugueñun.

As mentioned earlier, currently, the car ferry service is responsible for the crossing of cars, offering a time of between an hour and a half and twenty minutes, depending on the weather conditions. However, the construction of a new bridge is being developed, this new connection will provide a less than three minutes cross by car. **Figure 4.1.** shows the actual cross and **Figure 4.2.** shows the future cross.

Figure 4.1 Current car ferry service that crosses from Chile mainland to Chiloé.

Figure 4.2 Crossing of the new suspension bridge.

Site Conditions

The Chacao Channel is known for its extreme conditions. The presence of strong currents, earthquakes, strong winds etc.. makes construction of infrastructure difficult. For this reason, *FHECOR Ingenieros Consultores* together with *COWI* and *Ingeniería Cuatro*, have carried out several studies in order to precisely know the site

conditions. Also, Nils Erik Miguel Lindeen de la Fuente (2012) and Herrera (2010) study the current behaviour of the channel for carrying out a preliminary energy resource assessment.
The results of those studies have been summarized below.

The studies focus on four principal areas which are of particular interest:

a) Seismic conditions
b) Marine conditions
c) Geotechnical conditions
d) Wind conditions

a) Seismic conditions

This channel is situated in the famous ring of fire, which is a major area in the basin of the Pacific Ocean where many earthquakes and volcanic eruptions occur.

Figure 4.3 The Ring of Fire.

The biggest earthquake known, had its epicentre nearby the studied area and where the suspension bridge is being constructed. For this reason, it is of extreme importance to consider this in the design. From the seismic analysis is was found that there were two possible origins for seismic movements: the continental fault (subductive seism) and an hypothetic fault parallel to the axis of the channel, but yet it has not been determined if it's an active fault or not ; the fault of the Ancud Gulf. The following figure shows the fault of the Gulf of Ancud.

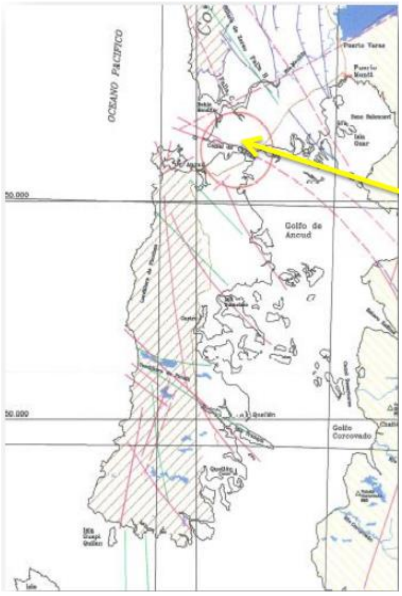


Figure 4.4 The fault of the Gulf of Ancud.

b) Marine conditions

Sea tides

Different measurements were made in specific points of the channel in order to have more knowledge of the Channels behaviour. The specific points analysed where; Tique, Manao ,Carelmapu, Pihuio and finally the axis of the Chacao suspension bridge which will be introduced lately. The following picture shows those specific points placed in a map to understand better the results obtained.

Figure 4.5 Map situating the different specific points of interest.

Name	Tidal race(m)
Tique	6,44
Manao	6,24
Bridge axis	5,74
Carelmapu	3,59
Pihuio	3,47

Table 4.1 Difference between the maximum tide and the minimum tide in the different points.

Finally, the following picture would represent the Channels situation where it is clearly seen how the real-time sea level at the Ancud Gulf is at a higher level than at the open ocean this situation is traduced on the high flow velocities analysed as follows.

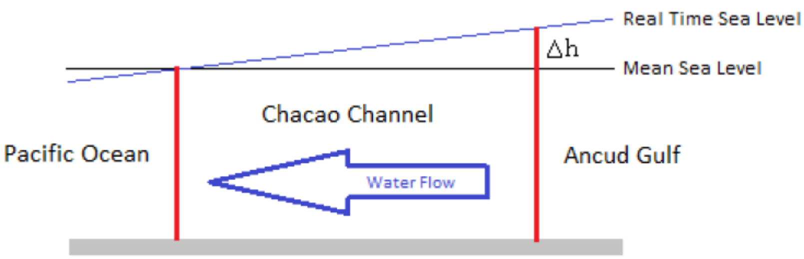


Figure 4.6 Diagram of tide level differences between both sides of Chacao Channel.

Sea Currents

This channel present large currents, which makes its study of great importance. These strong currents are linked to the filling and emptying of the channel by the rise and fall of tides.

Due to the fact of the decrease in mean channel width and depth from one end to another, causes the same volume of water go through a smaller section, causing an increase on velocity and on power. As well, both mean and maximum velocity and power are larger along the Channel at ebb (exit the Channel towards open ocean) tide than at flood (enter the Channel from open ocean) tide, meaning flows from the ocean to the Gulf are lighter than those flowing the opposite direction. This can be clearly seen from **Figures 4.7. 4.8. 4.9** and **4.10.**

The following figures show the current speeds and the flow patterns for the instants corresponding to the maximum Syzygy (full moon and new moon) and the minimum Quadrature (sun, earth and moon aligned in a right angle) , both filling and emptying of the Channel, which constitute extreme situations. These figures where extracted from the work done by Herrera (2010).

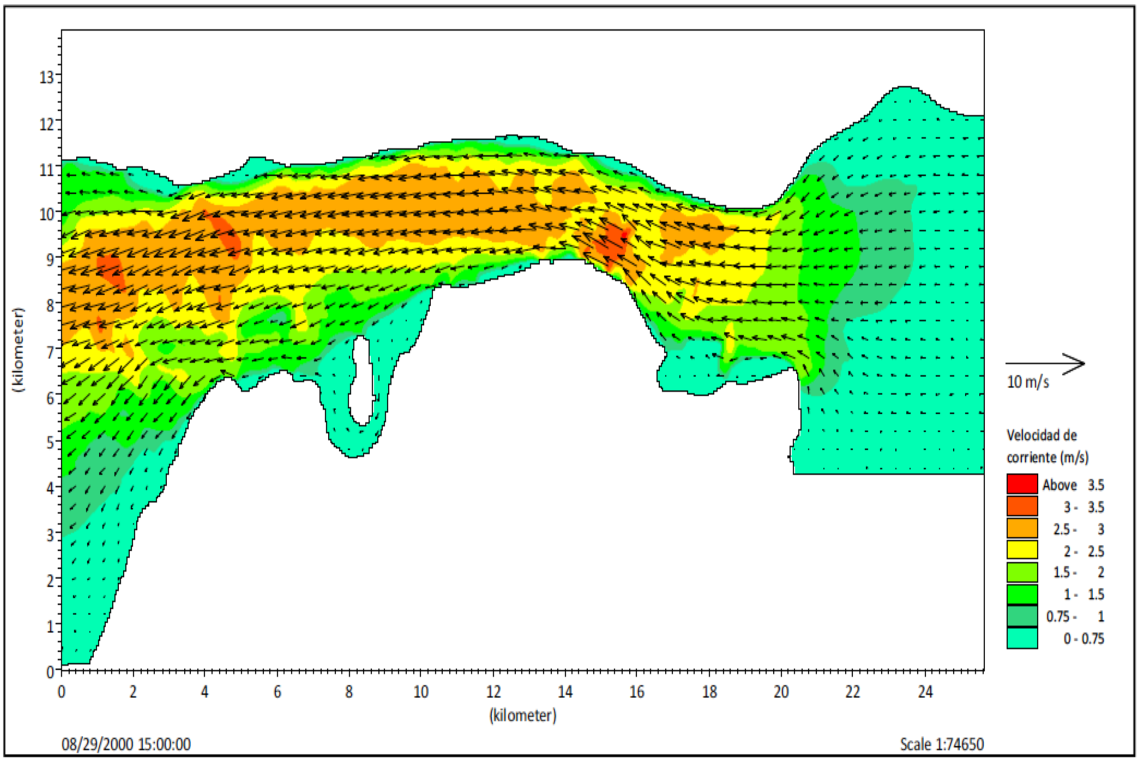


Figure 4.7 Flow pattern emptying of the Channel (Sygyzy).

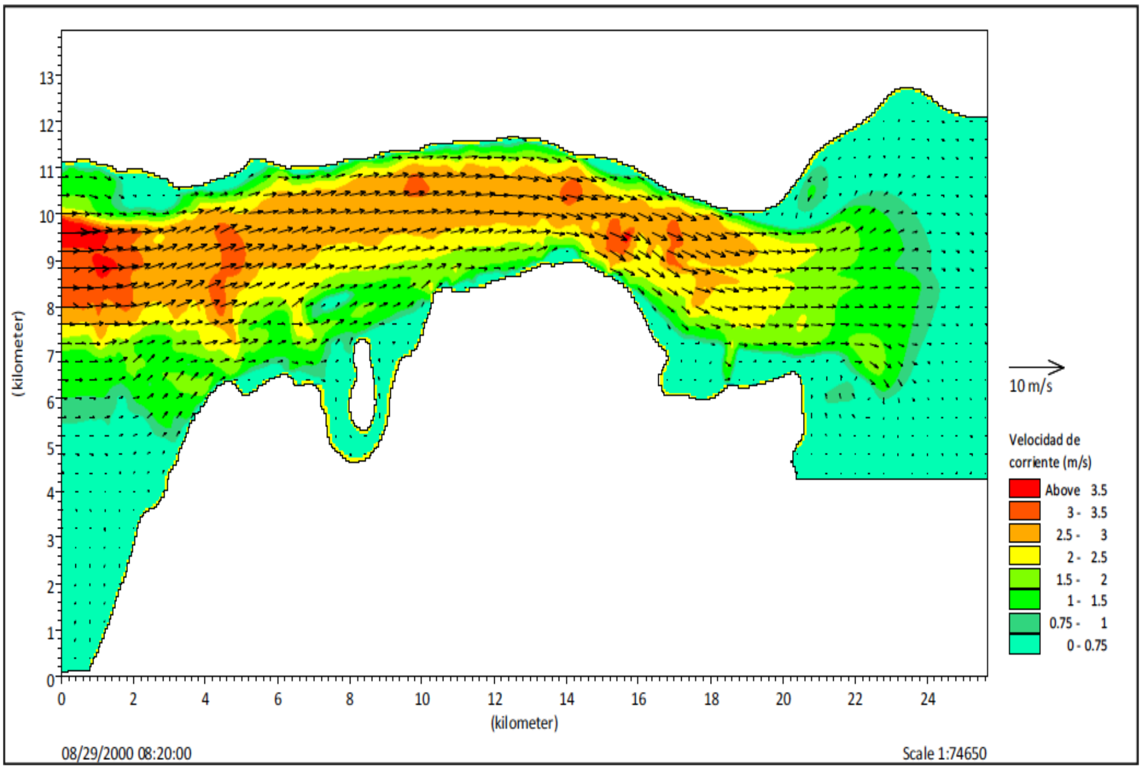


Figure 4.8 Flow pattern filling of the Channel (Sygyzy).

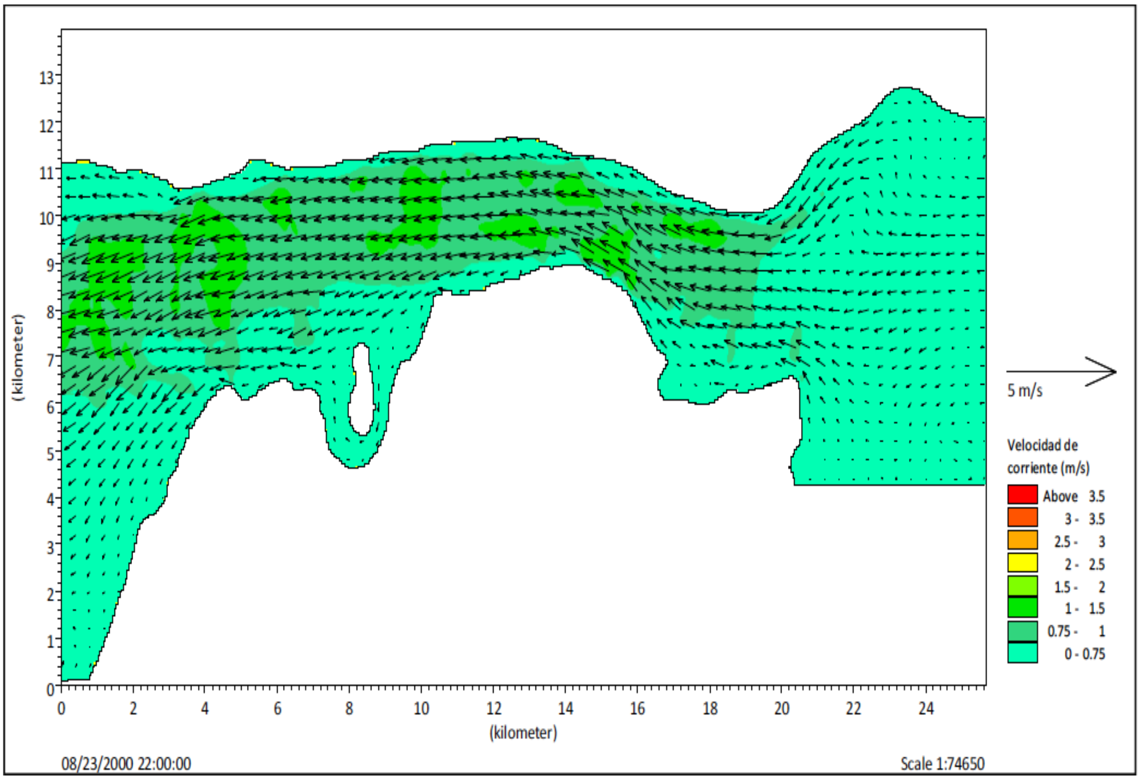


Figure 4.9 Flow pattern emptying of the Channel (Quadrature).

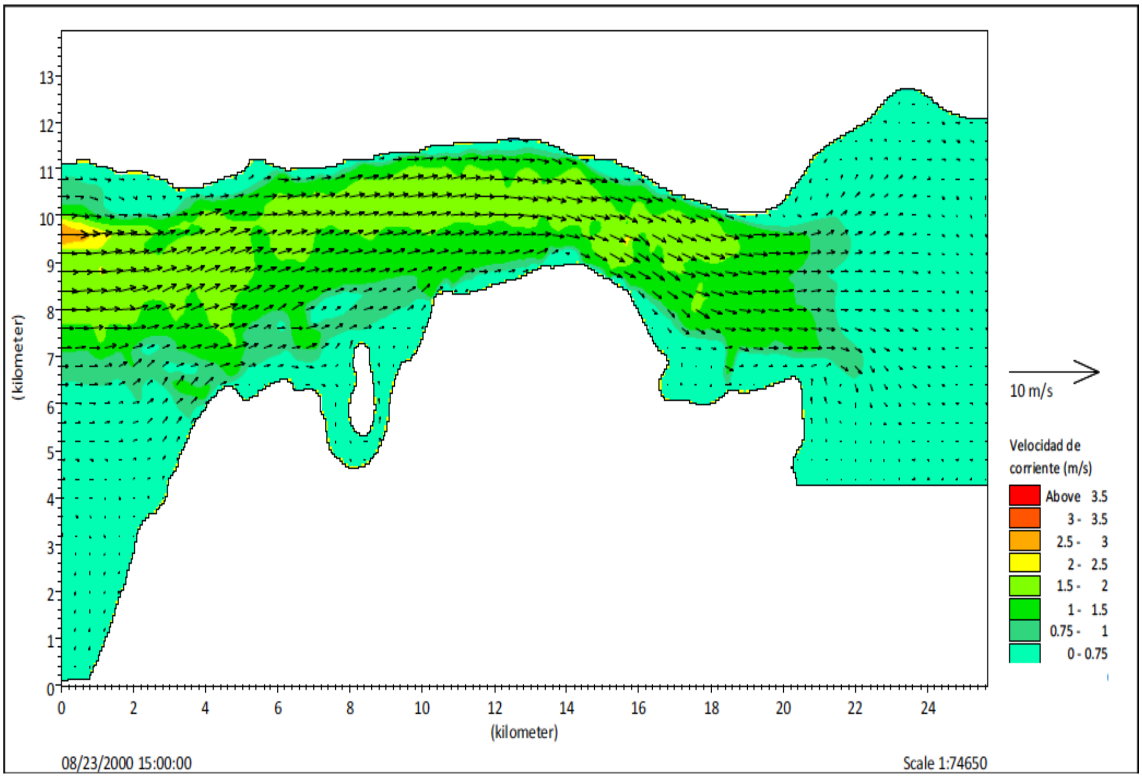


Figure 4.10 Flow pattern filling of the Channel (Quadrature).

These figures clearly show the main flow in the Chacao Channel, in where the high speed that the currents reach stands out. In condition of Syzygy, speeds close to 4 to 5m/s are reached in the area near Roca Remolinos.

Nils Erik Miguel Lindeen de la Fuente (2012) in his work pays special attention to how water behaves in Roca Remolinos and concludes that, there, mean flow is flood dominated at all depths in shallow areas, and ebb dominated in the deeper areas of the same cross section. This way, recirculation reflects strong divergences and lateral shears that translate into a relevant contribution of nonlinear terms (advection, horizontal and vertical friction) to the momentum balance (Cáceres, Valle-Levinson, & Atkinson, 2003).

Conclusions seen before can be better understood from **Figure 4.11** , as the arrows show in and out flow in different areas of the same cross section.

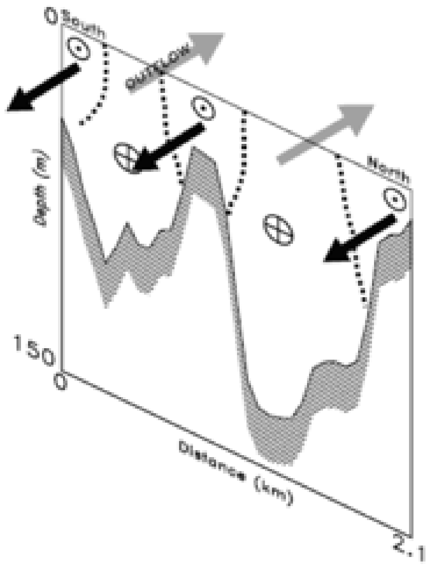


Figure 4.11 Roca Remolinos section with the inflow and the out flow from the rise and fall of tides.

Sea Waves

Waves where analysed in three parts of the channel (entrance, bridge axis and exit) to understand more their behaviour inside and before entering or after leaving the channel.

Waves at the bridge axis resulted having low height, being between 0-0,25m and having periods between 3 to 4 seconds. Waves in Bahia Guapacho mostly where about 0,5 to 2 meters with larger periods of about 15 to 16 seconds. Finally, was of particular interest to analyse what was the extreme case that can be generated in

the Gulf of Ancud as our area of study is more exposed. So, waves in the Gulf of Ancud resulted having heights below 1,7 meters and periods between 4 to 6 seconds. These values have been extracted from the UK Meteorological Office Global Wave Model.
Until now, even though the Chacao Channel is situated in a high seismic zone, the largest wave recorded which was caused by the Valdivia earthquake (22th of May, 1960) is of about 7m. This is due to the fact the area of study is relatively protected. However, this wave generated currents up to 8 m/s.

c) Geotechnical conditions

Based on the sample visual inspection and test results, four different layers were identified in Roca Remolinos (W. Romberg, 18 January 2006, **Figure 4.12**). All elevations given herein are below the Mean Sea Level, in meters (K. Syngros et al. 2008).

1. A Tuffite (Caprock) layer exists (grey colour), about 40-meter-thick, comprising of fine to medium sands, well to highly cemented, with low to medium silt content.

2. Below the Tuffite layer is a layer comprising of Pleistocene Sands about 35-meter-thick, comprising of medium sand, well to low cemented, with a few layers of gravel. Within the same layer are sublayers with concentrated gravel and low silt content. The layer colours vary from medium to dark grey and grey-brown.

3. Below the Pleistocene Sand layer, there is a layer of Pliocene/Pleistocene Silt about 30-meter-thick, comprising of hard or cemented Silt, with light brown to yellow colour.

4. Below the Pliocene/Pleistocene Silt there is a layer of Pliocene/Pleistocene Sands, comprising of very densely compacted, partly cemented medium sands. Both layers form the seabed of the channel.

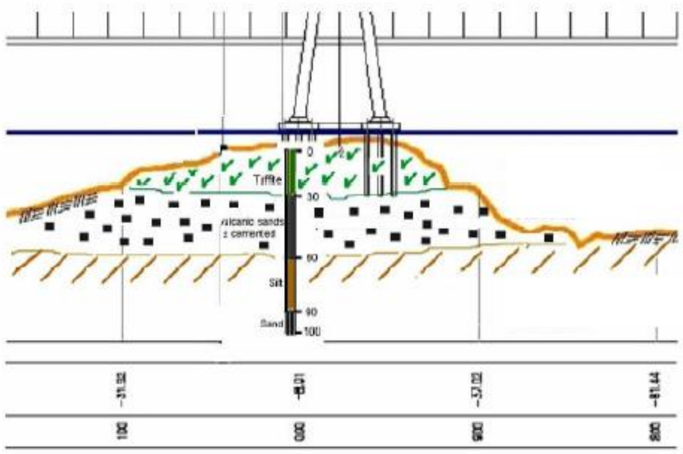


Figure 4.12 Roca Remolinos soil layers (W. Romberg, 2006).

d) Wind Conditions

After more than a year doing measurements of the site, the *Ministerio de Obras Públicas de Chile*, said that by using the Peak Over Threshold method, base wind speeds of up to 23,4m/s where recorded.

Also previously, FHECOR studied how wind affected perpendicular to the bridge deck and obtained the following results.

Height	Sustained wind (10 minutes average)		Gust	
z	V10(z)	P10(z)	Vg(z)	Pg(z)
250	45,1	1,27	60,7	2,3
200	44,1	1,22	59,7	2,23
150	42,8	1,15	58,4	2,13
100	41	1,05	56,6	2
75	39,7	0,99	55,3	1,91
50	37,9	0,9	53,5	1,79
25	34,8	0,76	50,4	1,59
10	30,8	0,59	46,4	1,34
(m)	(m/s)	(kN/m2)	(m/s)	(kN/m2)

Table 4.2 Different winds and preasures affecting the bridge deck depending on its position

4.2. The Chacao Suspension Bridge (SB)

The project consists on the construction of a long double suspension bridge with 2,754km of distance between anchor blocks. It will have three reinforced concrete pylons, two exterior shoulders and a central (A-shape) tower on Roca Remolinos, a point where the rock reaches the surface offering a point of support for the central pillar; a suspended north span of 324m, two main spans (1,155m and 1,055m) and a south approach viaduct of 140m. Least but not last, an orthotropic box girder deck of 23.8m wide will carry four lanes of traffic.

From **Figure 4.13** we can see again represented in a landscape view the SB crossing. Also from **Figure 4.14** and **Figure 4.15** we can appreciate the SB proposed design and cross section respectively.

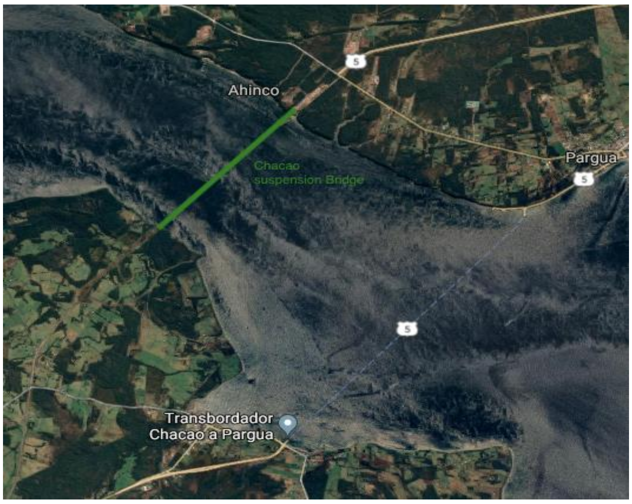


Figure 4.13 Current suspension bridge placement.

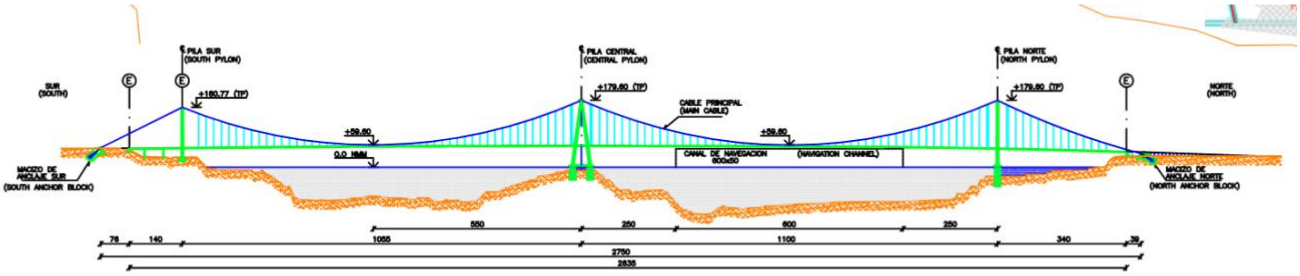


Figure 4.14 Design of the Chacao Bridge.

Figure 4.15 Bridge cross section.

4.3. The proposed Submerged Floating Tunnel

4.3.1. Viability Study

From the distribution of the site, it is thought to be possible the design of a submerged floating tunnel, the proposed way of crossing is seen in **Figure 4.18** resulting to be a crossing of 4 Km approximately, while the crossing of a suspension bridge is 2,754Km. A SFT will not be able to cross through the same path as the suspension bridge, firstly because of the depth in Roca Remolinos and secondly because it is a location where as seen before, extreme and recirculating currents are created. In order to choose the new way of crossing it has been considered the following:

- Obviously, the depth of the Channel (**Figure 4.16**).
- Try to use the already constructed roads in order to avoid an increase on cost and on environmental impact.
- Areas of Benthonic resources extraction seen in **Figure 4.17.**
- Areas of aqua farming activities seen in **Figure 4.18.** These would only be affected during the construction of the tunnel.

From **Figure 4.16** can be appreciated the depth of Roca Remolinos, which prevents the construction of a SFT but facilitates the construction of the suspension bridge, by offering a third support point in the middle of the channel.

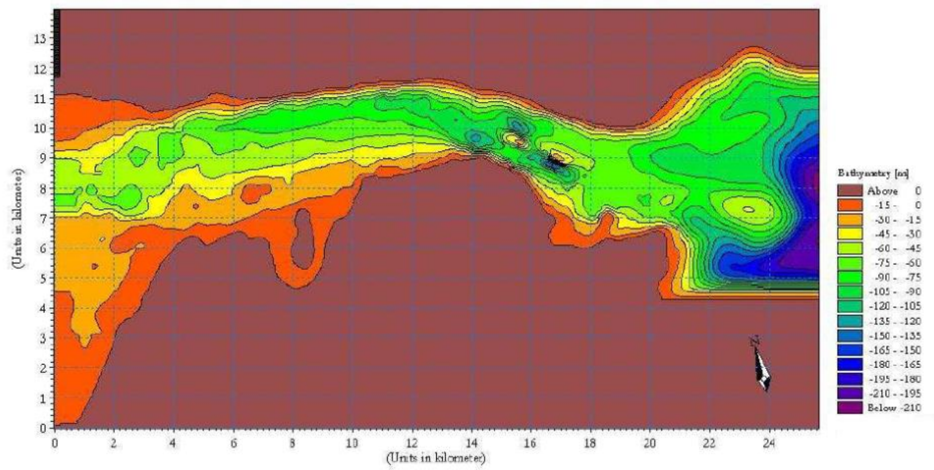


Figure 4.16 Bathymetry of the Chacao Channel. (Herrera, 2010).

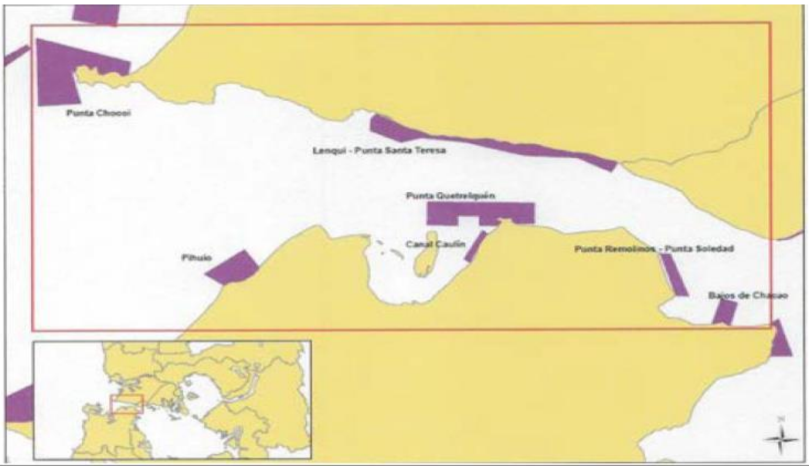


Figure 4.17 Areas of aqua farming activities.

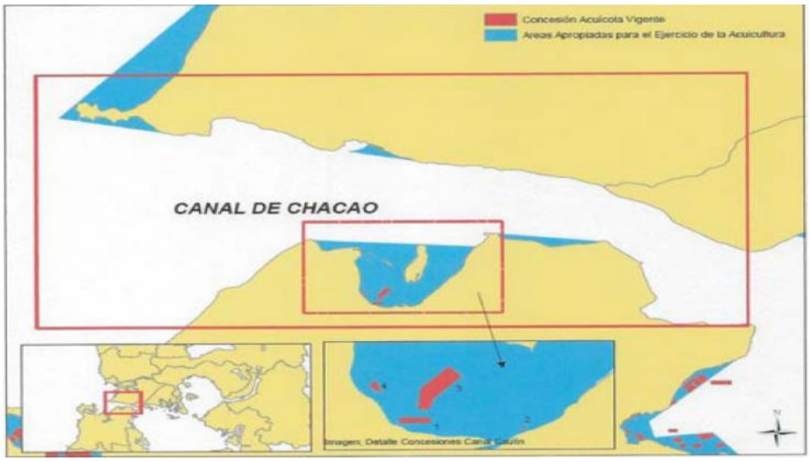


Figure 4.18 Areas of Benthonic resources extraction.

Figure 4.19 Current suspension bridge placement and the proposed SFT placement.

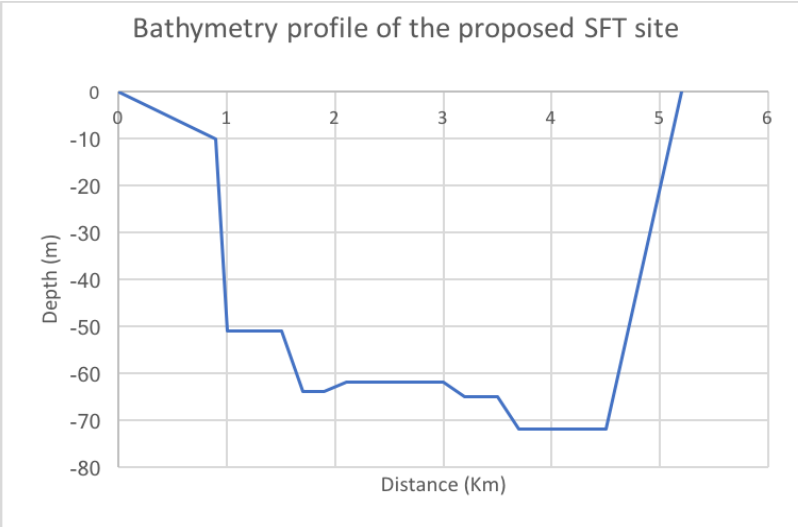


Figure 4.20 Bathymetry of the proposed site (Points from the graph have been extracted from the Instituto Geografico Militar de Chile).

Approximately the new proposed crossing will be of 4km (Can be appreciated on **Figure 4.31**, proposed design, end of chapter). For the suspension bridge the distance will be less, 2,754km. The fact that the suspension bridge offers a shorter crossing does not mean that its construction will be more economical. Martire (2010) on his paper made a cost comparison between a SFT and suspension bridge with similar characteristic to the same crossings situated on Messina strait where the conditions where more severe than in the Chacao Channel. He concluded that for a residual buoyancy bigger than 1 (positive force) an so anchored to the sea bed, the cost of a SFT varied linearly for both increase on depth and length as for a suspension bridge constantly and exponentially respectively. He considered two types of SFT and suspension bridge, one with the most expensive arrangement and the other with the most economical, for the site.

From the following figure extracted from the work done by Martire, we can clearly see how costs increase linearly with the increase of depth and length in a SFT and how a SB costs increases exponentially with length and are constant with depth

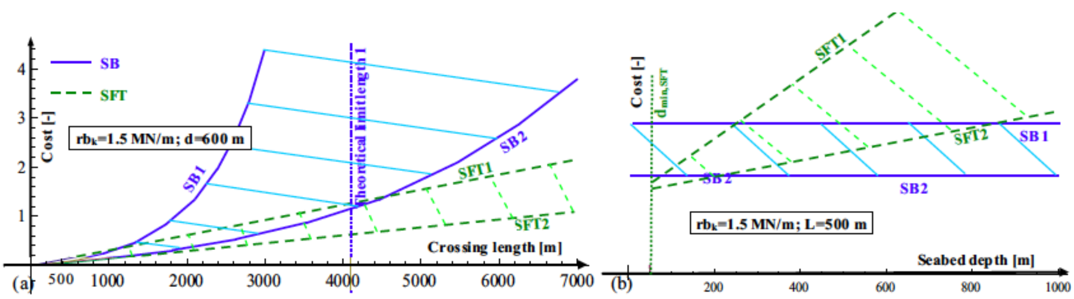


Figure 4.21 SFT and suspension bridge cost curves as a function of the (a) Crossing length (b) Crossing depth. (Martire, 2010).

To see whether or not the construction of a SFT is viable, a comparison has been made between our weather conditions and those of sites where a SFT design has been proposed, to have an idea of how should be our design to be able to support all loads and operate in the most optimal way.
Martire (2007,2010) analysed four different environmental conditions in his work. The waves and current characteristics of the Qiandao Lake, Jintang Strait, Messina Strait and Akashi strait, which represent mild, intermediate and severe conditions, and Panduro J. Omar., (2013) analysed the wave and current characteristics of the Gulf of California, which represent intermediate – severe characteristics. It is important to notice that these parameters are analysed as the wave and current parameters are the most important issues which define the severity of the hydrodynamic actions. The Table **4.3** represents the different characteristics for each site as well as the Chacao Channel in order to make the comparison between them.

Location site	Condition	Depth	Clearance	Wave height	Wave length	Period wave	Current velocity
Qiandao Lake	Mild	30m	2.0m	1.0m	8.25m	2.3s	0.1m/s
Chacao Channel	Mild - Intermediate Severe(tsunami)	70m 70m	25m 25m	1,4m 7m	20m --	4s --	2.5m/s 8m/s
Akashi Strait	Severe	80m	30m	9.4m	--	--	4m/s
Jintang Strait	Intermediate	100m	25m	5.8m	76.5m	7.0s	4.1m/s
Quiongzhou strait	Intermediate	150m	30m	5m	--	--	3.5m/s
Messina Strait	Severe	200m	30m	13,5m	200m	11.5s	3.45m/s
Gulf of California	Intermediate-Severe	213m	25m	9.5m	99.8m	8.0s	2.9m/s

Table 4.3 Comparison between locations for SFT proposals.

In the table, for the Chacao channel it has been considered 25m of clearance as Christin Ingerslev (2010) said, currently, few if any vessels have a draught exceeding 25m. Even though, the traffic is mostly consisted on recreational traffic and ferries destined for the strait crossing. This type of boats has small drafts and so it has been oversized. It also has been considered the fact that there are approximately 5m meters of difference between high and low tides. This has influenced on the SFT buoyancy and so on its anchorage, as for a SFT subjected by pontoons it will be very difficult to have waterproof shore connections which enables rotation due to tide variation. So, the SFT must be anchored to the ground buoyancy by means of tethers and gravity foundations, having a positive buoyancy. It is important to notice that by having the SFT anchored to the ground, when high tides, the SFT clearance will be of 25m and when low tides the clearance will be reduced to approximately 20m.

So, from **Table 4.3** we can see that the characteristics of the natural conditions of the crossing are among the mild (Qiandao Lake) and the intermediate (Jintang Strait). The wave conditions are similar to the Qindao Lake presenting a little variation. Moreover, the current velocity in the Channel even under extreme conditions is minor to the intermediate condition crossing but higher than the mild condition crossing. Hence, the natural conditions in the Chacao Channel at the chosen section are acceptable for the placement of a Submerge Floating Tunnel.

On the other hand, even though for natural conditions it is acceptable, it is of extreme importance to be aware of the high seismicity and of the probability of a tsunami to take place. As seen in the table, it is not so the height of the wave produced, but the currents produced that may concern the placement of the SFT, as the site is relatively protected. The values shown in the table are from a wave created by the Valdivia earthquake (22th of May, 1960), this wave caused huge damage in the city of Maullín, situated at the entrance of the channel and so far, it is the largest we have recorded in this zone.

4.3.2. SFT Proposed Design

The design chosen is based on a comparison of our case with other similar locations where different designs have been proposed and see which one fits better. If more precise data could be available it would be possible to optimize the design.

4.3.2.1. Cross Section

Locations such as the Akashi strait (severe conditions), the Gulf of California (intermediate – severe conditions) and the Quiongzhou Strait (intermediate) have been considered and compared to our case. Our case normally consists of mild conditions but there is the possibility of tsunamis occurring and causing severe conditions that is why the project must be overestimate and locations with severe conditions have been considered. In these locations Martire (2010), Panduro J. Omar., (2013) and B. Jiang et al. (2018) proposed different cross sections respectively.

a)

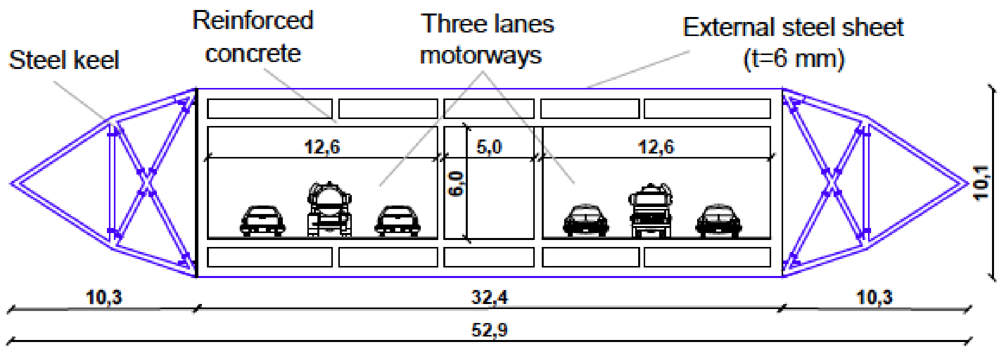


Figure 4.22 Martire 2010 cross section proposal for Akashi Strait.

b)

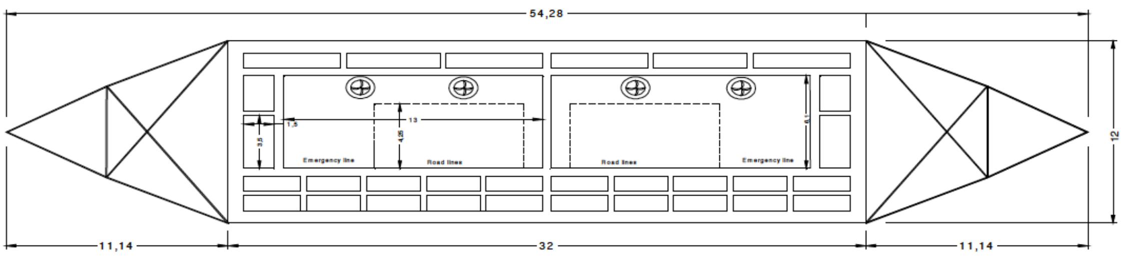


Figure 4.23 Panduro J. Omar., (2013) cross section proposal for the Gulf of California.

c)

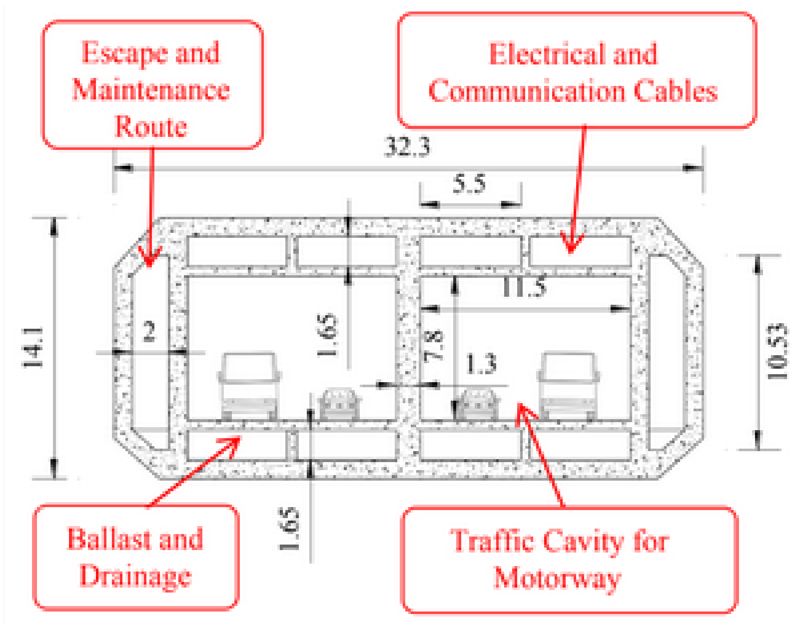


Figure 4.24 Jiang et al. (2018) cross section proposal for Quiongzhou Strait.

The idea is to offer the same operability as the suspension bridge, carrying four lanes, two for each direction. So, the internal dimensions of the cross section should be large enough to accommodate the infrastructures facilities and implants, necessary to guarantee the normal development of the operations inside. Also, as mentioned before, the tunnel cross section must be designed so that the buoyancy ratio is larger than the minimum value.

It is easily seen that the only difference between (a) and (b) is that (a) is carrying six lanes and (b) four. On the other hand, the difference between (a), (b) and (c) is that (c) has a polygonal cross section while (a) and (b) a rectangular combined with hydrodynamic lateral keels.
In terms of distribution, the three cross-sections offer an easy way to accommodate the traffic and facilities requirements, but for the production process of the elements (a) and (b) are easier. Moreover, M. Kristofersen et al. said that this types (a,b) of cross sections are suitable for extreme natural conditions and adding the fact that Martire analysed their behaviour under severe conditions considering seismic motion and obtained positive results and that in the Chacao Channel there is a large possibility of severe conditions to take place. It has finally concluded that the cross section proposed for the Gulf of California (b) and for Akashi Strait (a) are the ones that fit better to our case. However, dimensions have been adjusted to our scenario taking into consideration the Spanish regulations for road sizing, established in *El Boletin Oficial del Estado (BOE)*.
The final cross-section proposed can be appreciated on the following figure.

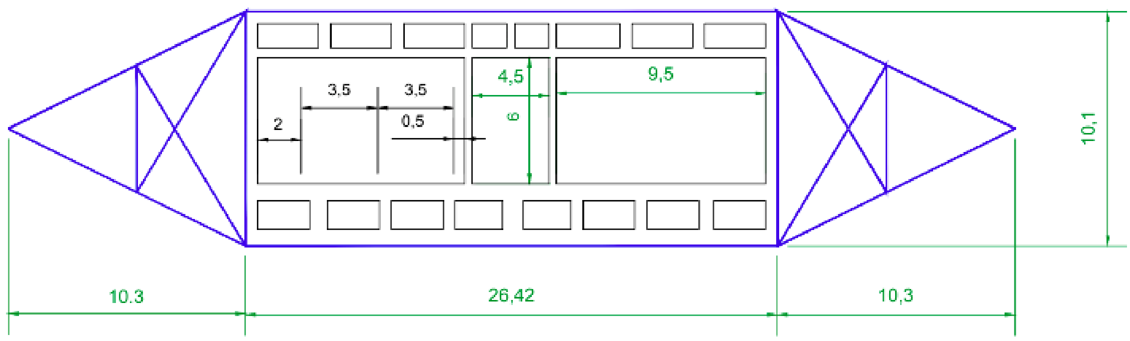


Figure 4.25 Proposed cross section for the SFT (all in meters).

The cross section combines a rectangular shape, which facilitates its production and the versatility in the organisation of the internal spaces and facilities, with a hydrodynamic lateral keel, that improves the fluid dynamic behaviour of the SFT, preserving the advantage of the rectangular cross section. In addition, the length of each module is established of 200 m to assure the anchorage system with a distribution of symmetric spans.

4.3.2.2. Joints

Inter-modular Connections

The connection between the elements is fundamental for the SFT behaviour. Therefore, there will be flexible connections between the elements allowing a minimum flexion and extension between modules. In our case, it is proposed the normal practice for immersed tunnels, which consist on attaching and sealing each element by means of the Gina gasket and Omega seal. The combination of both not only allows for sealing but also for transfer of hydrostatic loads and movements between elements. The normal practice adopted is described below, following the Trelleborg Baker instructions, as they are expert suppliers on this type of joints.

To ensure that the tunnel elements are watertight and capable floating, elements are temporary provided with bulkheads at both ends. On one end of each tunnel element, an endless Gina gasket is mounted. Once on site, the tunnel element is pulled firmly up against the preceding floating element with hydraulic jacks.

When the gasket has full contact around the total section of the adjacent element, the water between bulkheads is pumped, normally inside the tunnel. Due to the different pressure between bulkheads and the hydrostatic pressure on the outside of the tunnel, the GINA profile compresses and seals the joint.
The Omega seal is then clamped across the joint on the inside faces of the two elements. In general, after the approval of the pressure test between Gina and Omega, bulkheads are removed. The following figure shows an outline of the steps previously described.

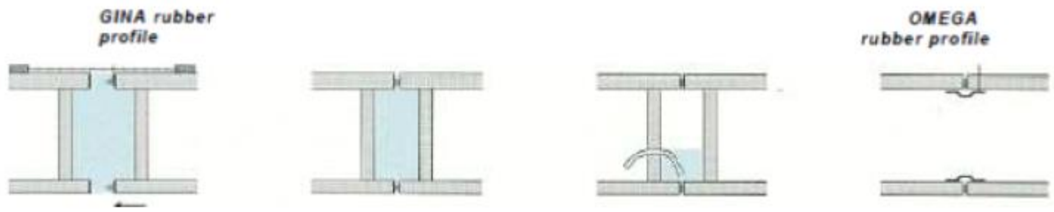


Figure 4.26 J Construction steps of the inter-modular joints (Trelleborg Baker).

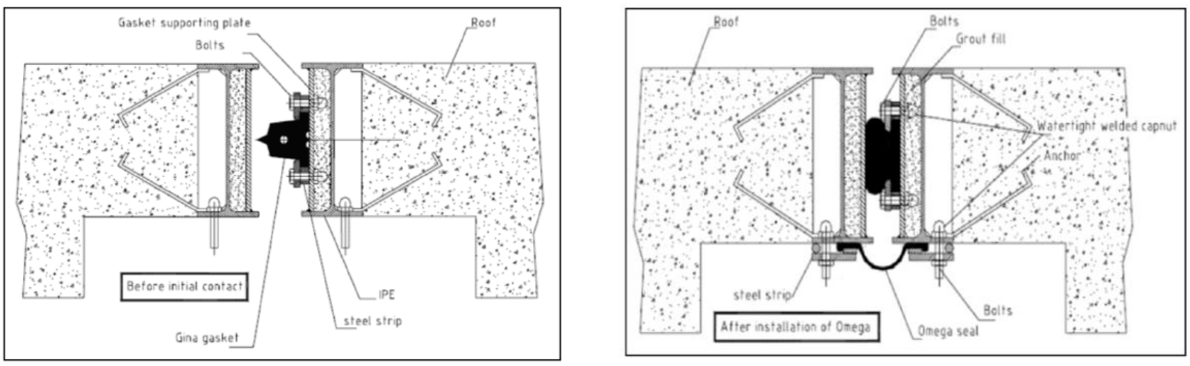


Figure 4.27 Details of the inter-modular joints. (a) Before installing contact (b) After installation of Omega. (Bistoon Baspar LTD)

Shore Connections

Due to the high seismicity of the site and considering the great length of the SFT proposed, the shore connections must allow free rotation and axial displacement as the displacement and the affectations produced by natural events must be considered as local issues. The proposed joint adopted in our design was the one proposed in the Qiandao Lake, China (Mazzolani et al. 2007). This shore connection allows the structure to expand in presence of thermal variations and acts well against water tightening. The general concept is based on separating the waterproof and mechanical functions of the device (Panduro J. Omar., 2013).

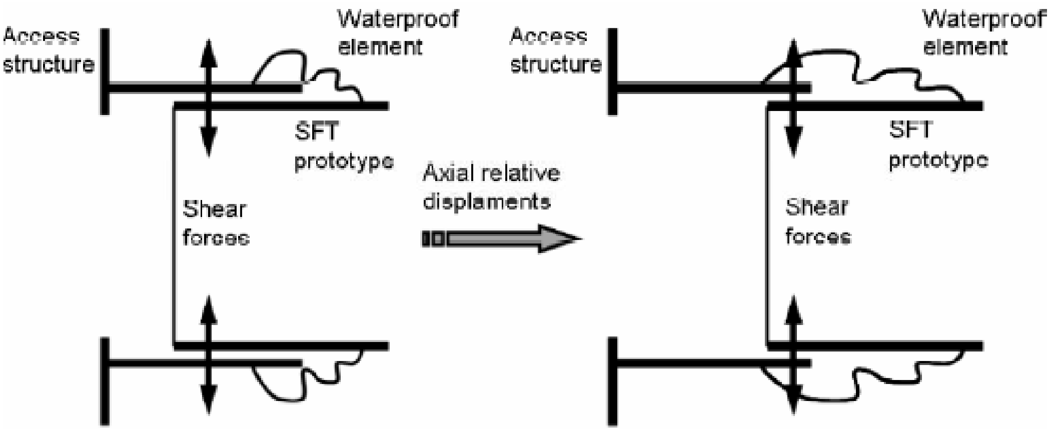


Figure 4.28 ptual design for the shore connection (Mazzolani et al., 2007).

4.3.2.3. Anchorage System

The anchorage system will consist of steel cables subjected to gravity foundations as seen in **Figure 4.29** It is important to see that the distance between the foundations and the SFT modules will vary depending on the depth of the crossing, being the distance shorter in shallow depths and so having stiffer cables and bigger distances in deeper parts of the crossing. Martire's results show that, anchoring systems gets considerably more involved in carrying the external loads as the crossing length increases due to the reduction of tunnel bending stiffness, so that , the critical point in larger crossings is generally ensuring enough strength to the anchorages and/or reducing their maximum axial forces induced by hydrodynamic loads. In our case, it is considered a long cross and so, this has been considered.

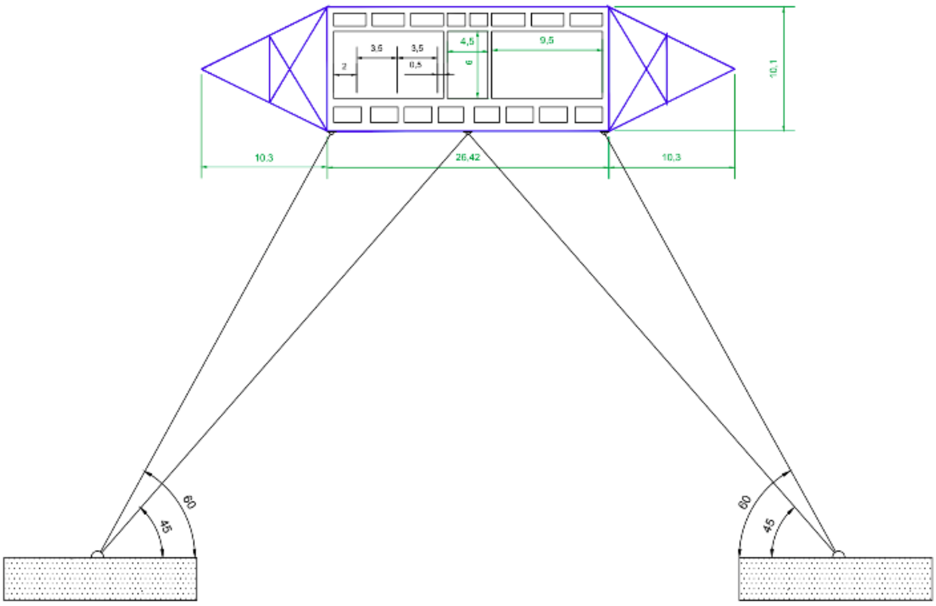


Figure 4.29 Transversal anchorage configurations.

Having this type of cable configuration brings relevant advantages to our design in terms of maximum displacement and reduce maximum moments, if two cable systems are provided per module as seen in **Figure 2.26.** It is also seen that increasing the number of cables more than two per tunnel module produces no relevant advantages in terms of maximum displacements (Martire, 2010). As Hong Y. et al. (2010) concluded the tether angle must be larger than 30º. As in similar cases such as in the Qiongzhou strait or in California Gulf, the tether angle employed has been of 60º and 45º, the same angles have been considered.

Finally, four anchorage systems have been considered along the longitudinal axis as seen in **Figure 4.30** this means that for each 200m module there will be a total of sixteen cables and between each anchorage system a distance of 50m.

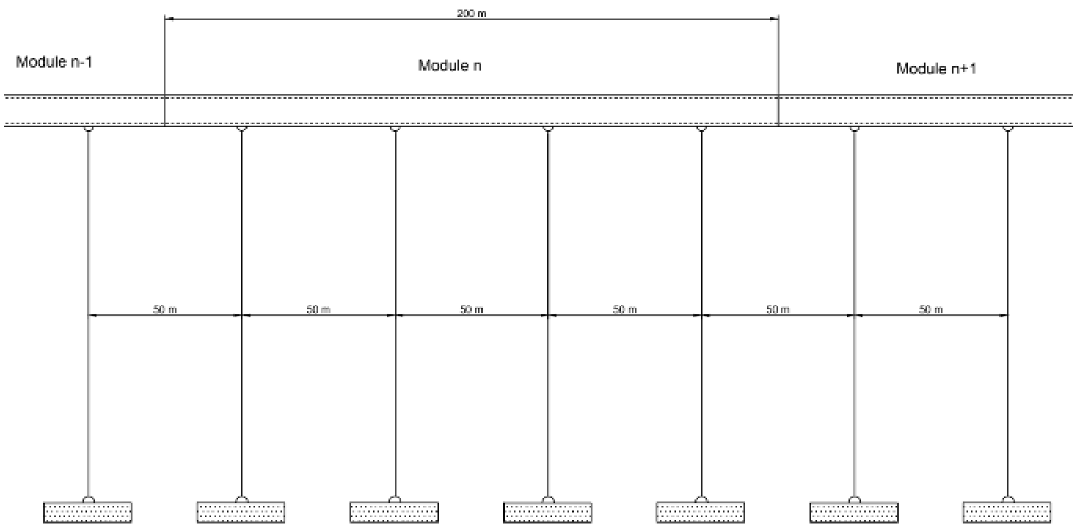


Figure 4.30 Longitudinal anchorage configuration.

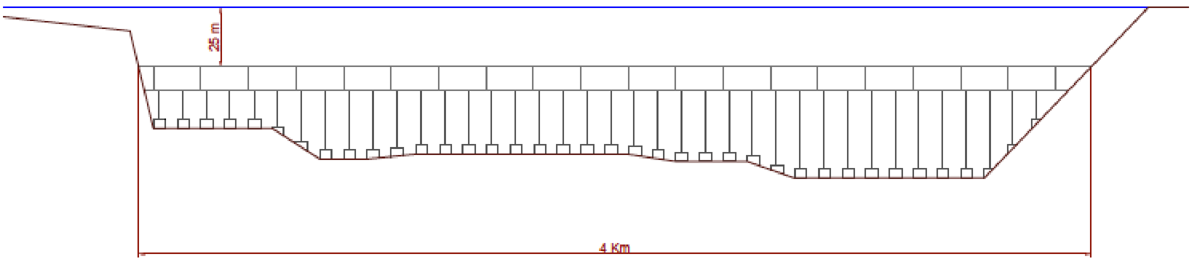


Figure 4.31 Proposed design set on site.

In order to deal with seismicity, Martire proposes a simple and economic solution, as the anchorage groups located close to the shore have reduced lengths (less length, greater stiffness), they will be subjected to extremely larger axial forces. The idea to build the shorter anchorages by means of structural elements able to yield and dissipate energy through a stable post-elastic behaviour, or alternatively to introduce specific mechanical devices providing special dissipative connection between the tunnel and the anchorages.

4.4. Multi-criteria Analysis

The main objective of this chapter is to evaluate whether it is more feasible or not the construction of a SFT in the Chacao Channel. To do so a multi-criteria analysis has been carried out, based on the doctoral thesis by Casanova, M (2014). This methodology allows to the administration, or any another entity, in the bidding phase of a project, to quantify and compare the value of the different alternatives evaluated, prioritize them and choose the best one.

This multi-criteria analysis will evaluate and compare, objectively, both construction processes integrating diverse aspects such as costs, environmental impact, safety, social impact and functionality. Aspects will be referred as categories and categories will be divided into different criteria as seen on **Table 4.4.** Both terminologies are an adaptation of the work done by Casanova (2014).

For each category and criteria, different weights will be assigned depending on their importance within the project. Casanova (2014) carried out a series of interviews with different experts in the sector in order to stablish the different weights for each category and criteria. To have more precise results, similar ones have been considered.

Due to the framework of this project, indicators for each infrastructure have been wisely estimated considering different scientific publications and the authors criteria himself. Indicators are the way to measure or value the alternatives with respect to the categories and criteria. As the diverse indicators will measure in different units, in order to evaluate each alternative, it is necessary to stablish a value function. This function will transform the different measuring units of the indicators into satisfactory or value units.

For each construction process (SB, SFT) an indicator between 1-0 will be assigned. The value of 1 will be assigned to the structure which the least value is obtained during its lifecycle (more satisfactory) and 0 to the one that obtains the highest value (less satisfactory).

Casanova 2014 says that the value functions can adopt different trends and forms depending on how the satisfaction varies with the indicator response variation. In our case to simplify our analysis, a linear variation has been considered.

The following figure shows this relationship between indicators. On the y axis the value for each indicator, while on the x axis, the indicators response.

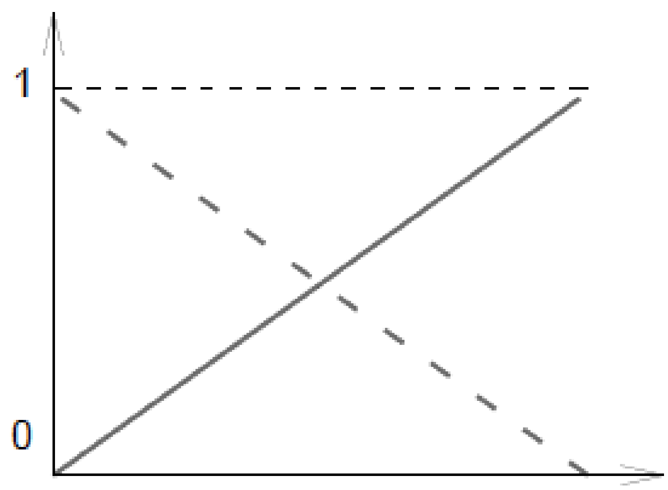


Figure 4.32 Lineal relationship between indicators. (Casanovas, M. 2014)

From **Figure 4.32** the following equation (**eq 4.1**) can be deducted. This equation represents the relationship between both indicators and which would be used to calculate the less satisfactory indicators.

Ratio = 1*lowest value/highest value eq 4.1

Global weights (%)	Category	Criteria weights (%)	Criteria
21.3	Economic Impact	29.5	Construction Costs
		33.2	Maintenance Costs
		37.3	Durability
20.1	Environmental Impact	19.1	Landscape Impact
		19.5	Fauna and Flora Impact
		20.3	Sea water Impact
		20	Air Impact
		21.1	Noise Impact
19.1	Social	31.8	Execution Time
		36.5	Travel Affectation
		31,7	Inconvenience to neighbours during construction
20.5	Safety	40.2	Against Earthquake
		29.4	Against Tsunami
		30.4	Against Collision
19	Functionality	40.2	Cross section
		13.5	Extension Possibility
		46.3	Connectivity to existing road network

Table 4.4 Impact table for the construction process evaluation.

4.4.1. Indicator Estimation

Construction cost

Given that there are no experiences in the construction of tunnels in the marine environment and that there are no records of costs for already constructed immersed tunnels, the price will be estimated from a land-based tunnel and applying a factor to take into account that it is an infrastructure to be built in the sea. In order to obtain this factor, it is necessary to study the relationship between a terrestrial structure and its counterpart built in the sea. One case that is currently common is the construction of wind turbines on land and at sea and, therefore, it has been considered that the relationship between their costs gives us an approximate idea of the relationship between the cost of a conventional tunnel and one at sea.

Before calculating the sea factor, we must see if the sections of both tunnels coincide, since the price will not only vary with the length, but also with the section. If they don't coincide, a correction factor between the different areas will have to be stablished.

The cost of the conventional tunnel constructed in Barcelona for the new AVE line, using an EPB type TBM, with 11 m external diameter, was about 200M euros with a length of 5Km and a section of 85m2 with approximately 40.000 euros per m.

At first glance, it can be appreciated that our section is larger, so it will be necessary to calculate a corrector.

After calculating our section obtaining a value of 200m2, the correction factor will be 200/85=2,35.

The construction of an offshore wind tower costs approximately 11M euros and the construction of a inshore wind tower cost approximately 3,5M of euros; this prices have been extracted from the work done by Moraleda M. (2013) and Hoyo L. (2018) respectively. Finally, the sea factor obtained is 3.14.

Multiplying the approximated cost per m of the construction of the AVE by our total distance (4000m) and by both factors, we obtain that the price of the SFT is about 1125M of euros.

Knowing that the Chacao bridge cost will be approximately 900M of euros we can now calculate the indicator. For the SB = 900/900 = 1 and for the SFT = 900/1125 = 0.8

Construction	SB	SFT
Indicator	1	0.8

Table 4.5 Construction cost indicator.

Maintenance cost

The usual maintenance costs related to tunnels in general, such as lighting, ventilation, traffic control, drainage, interior cleaning etc... which are expected to be low, usually in tunnels is estimated to be an annual 1,5% of the total cost (Egil Lundebrekke, 2014). On the other hand, as bridge is directly exposed to the natural environment there are additional costs. Maintenance task such as dehumidification, protection against scour, improvement on paintings are extra costs compared to a tunnels maintenance and adding the fact that the Chacao Channel is situated in a place with a lot of humidity, strong winds and where it rains a lot, maintenance costs increase. Approximately maintenance costs for a suspension bridge is around an annual 2% of the total cost (Abed, M. 2009). Then, the indicators are, for SFR = 1.5/1.5=1 and for SB = 1.5/2=0.75

Maintenance	SB	SFT
Indicator	0.75	1

Table 4.6 Maintenance cost indicator.

Durability

It is very difficult to estimate the durability of both structures without a deep and precise study. However, it is considered to be an important criteria inside the economic impact, and so it has been considered in this final project, but it cannot be assessed within the framework of this project. Then, it is assumed than both solutions will be designed for the same life span and so they have both the same indicator of 1.

Durability	SB	SFT
Indicator	1	1

Table 4.7 Durability indicator.

Environmental impact

a) Landscape impact

No matter how elegant and high-tech a suspension bridge can be, its construction cannot avoid interfering in any way with the surface environment; its construction has a visual impact on the surroundings. It is inevitable not to witness such a large infrastructure. Its presence changes the way people looks to the environment, takes away the natural essence of the landscape. This does not mean that it has a total negative impact on the environment; some people believe that it can bring charm to the site. By contrast, a SFT, as the name says, will go under the sea surface and so it is invisible. That is why the following indicators have been considered:

Landscape	SB	SFT
Indicator	0.1	1

Table 4.8 Landscape impact indicator.

b) Flora and fauna impact

The Chacao Bridge will have three main towers, one supported in seabed (North Tower), another in Roca Remolinos and the last one in land (South Tower). The SB anchorages plus the North Tower and the road extension constructed to provide access to the infrastructure, will have direct impact on flora and fauna. Trees have been transplanted and grassland has been destroyed, also, the new road will make animal cross much more difficult due to the high traffic and speeds of vehicles. In addition, the North Tower and the Roca Remolinos tower will have an impact on the sea flora and fauna. However, even though the SFT construction will only have an impact on the sea flora and fauna, 4km will be affected by approximately 160 gravity foundations laid on the seabed. Tethers attached to the foundations and subjecting the tunnel will difficult sea animal cross and sediments accumulations on each gravity foundation will cause variations on the ecosystem and as a consequence nearby flora and fauna will be affected. In addition, the tunnel pipe could obstruct the water flow, which could cause a variation of the mixing process among the different water layers, unbalancing the habitat for some biological species.

Flora and Fauna	SB	SFT
Indicator	1	0.8

Table 4.9 Flora and fauna impact indicator.

c) Sea water impact

During construction, in a suspension bridge, main towers will be constructed on site, and so the possibility of dropping harmful substances or materials such as concrete will increase.

Even though the construction of a SFT is underwater, no workforce is needed outside the tube, thus reduces the possibility of dropping materials. Elements will be precast on land and then transported to site, minimizing the possibility of pouring concrete onto water or other materials. Boats responsible for transporting elements and decks will have a direct impact on water contamination. As the SFT is longer, more boat traffic will be needed to place the elements on site and so it is thought that the construction of a SFT will have a greater impact onto the sea.

Sea water	SB	SFT
Indicator	1	0.7

Table 4.10 Sea water impact indicator.

d) Air impact

A study made by Ole Magnus Kålås Iversen (2014) showed how during production phase most CO_2 emissions were produced and that during construction operation and maintenance still were produced, but much less. The production of the emissions depended on the length of the crossing due to the need of more segments and so, materials. In this case, as the SFT will have 4Km approximately and the SB 2,754Km, the SFT will have a higher impact. Approximated values are obtained by considering the results from Iversen study and correlating them to our case. Then, the indicators are, for SB= 367,092/367,092=1 and for SFT = 367,092/605,895=0.6

	SB	SFT
Production	348,671	571,361
Construction	10,789	18,311
Operation and Maintenance	7,632	16,223
Total	367,092	589,672

Table 4.11 Tonnes of CO_2-equivalents

Air	SB	SFT
Indicator	1	0.6

Table 4.12 Air impact indicator

e) Noise impact

As mentioned earlier in Chapter 3, in a SFT as in a SB, the mobilization of construction equipment in the pre-construction stage will generate noise and the construction of the foundations, but in addition to the placement of the tunnel, it will have a major impact on the marine environment. By contrast, during operation and maintenance stage, the impact of noise and vibration will be less than on a suspension bridge (World Road Association Rodiale de la Route, 2018) and considering that actually there is a huge traffic crossing the strait. The following indicators have been considered:

Noise	SB	SFT
Indicator	0.3	1

Table 4.13 Noise impact indicator

Social impact

a) Execution time

The bridge was intended to be built in 6 years and the SFT is estimated to be built in 8 years, A comparison between the Messina Strait (3300m) has been made which was estimated to be 7 years, so approximately 2 years/km. Then, the indicators are, for SB = 6/6=1 and for SFT =6/8=0.75

Time	SB	SFT
Indicator	1	0.75

Table 4.14 Execution time indicator.

b) Travel affectation

To calculate an approximate indicator, it has been considered two random points in order to evaluate the cross time. It is clearly seen, that an increase of distance will suppose an indicator variation as the difference between crossings is of about 5,5 minutes. The velocity considered for the Chacao Bridge crossing was 100Km/h (*Dirección General de Concesiones del Ministerio de Obras Públicas de Chile*) established and for the SFT 80km/h, due to the long and claustrophobic cross which cause ease to get distracted. Having higher speeds would increase probability of accidents and as a consequence its impact, as a reference Spanish tunnels have been considered such as the Viella tunnel in Lerida (Barios, J., 2005). Also, it has been considered that the SFT alternative will cross through Pargua and so velocity will have to be reduced to 50Km/h before accessing the tunnel. Then, the indicators are, for SB = 9/9=1 and for SFT =9/14.5=0.62

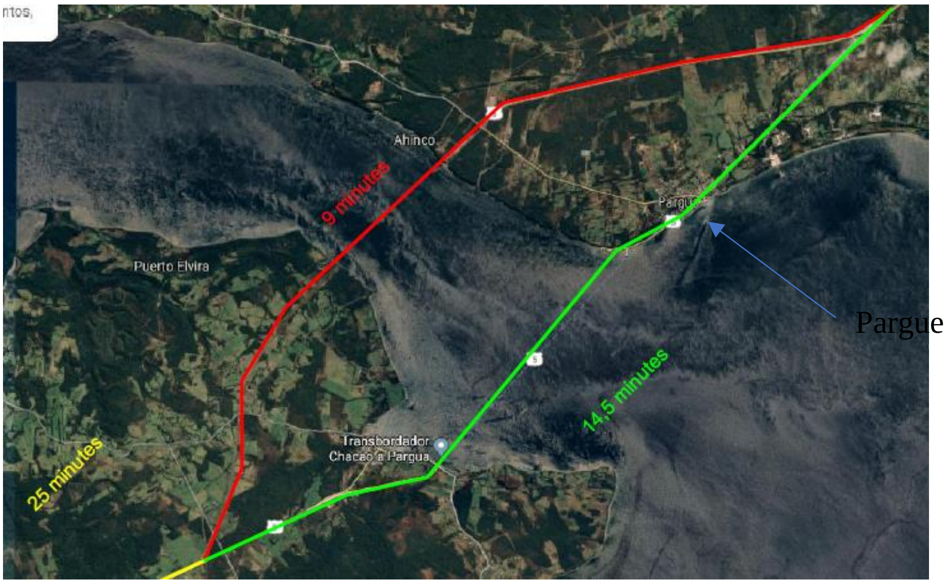


Figure 4.33 Different cross times for the different infrastructures.

Travel	SB	SFT
Indicator	1	0.62

Table 4.15 Travel affectation indicator.

c) Inconvenience to neighbours during construction

During the tunnel construction, all the material will have to be transported by truck through Pargua, which will cause great inconveniences, such as noise, traffic, close of streets and so more withholdings. By contrast, the bridge construction is far apart from cities.

Inconvenience	SB	SFT
Indicator	1	0.3

Table 4.16 Neighbours inconvenience indicator.

Safety

a) Against earthquake

The structure of the SFT is much more flexible than the bridge structure. The idea to build the shorter anchorages by means of structural elements able to yield and dissipate energy through a stable post-elastic behaviour and the combination of Gina gasket and Omega seal between elements will allow a minimum bending and extension between modules. Even though, a suspension bridge is considered one of the most flexible bridges, it has still considered that a SFT has a better behaviour.

Inconveniences	SB	SFT
Indicator	0.3	1

Table 4.17 Earthquake indicator.

b) Against tsunami

The design of this SFT is considering currents up to 4,5m/s, the fact that the largest tsunami recorded until now created currents up to 8m/s could create loads, which SFT anchors would not be able to support and would eventually collapse. Waves of 7m though, do not suppose any inconvenience, as similar designs are able to support constant bigger waves. For FHECOR *Ingenieros Consultores (2002)*, this tsunami does not represent to be a big problem to the security of the bridge structure.

Tsunami	SB	SFT
Indicator	1	0.6

Table 4.18 Tsunami indicator.

c) Against collision

It is thought to be more probable that the proposed SFT collapses by collision than the Chacao Bridge. There are more possibilities of things that can collide the SFT than the bridge, and so probability increases. Firstly, the collision of an anchor dragged by a ship it is thought to be the most probable thing to collide. Secondly, the presence of whales in these zones are very common and so the collision of a disoriented whale it is believed to be possible. Finally, but least probable, the collision of a sinking ship. On the other hand, a ship collision with the main towers suspended on the seabed's channel, it is thought to be very improbable as the tower which is thought to be more likely to be collided, is only accessible during high tides. During small tides, the main tower is protected by what stands out from Roca Remolinos.

Collision	SB	SFT
Indicator	1	0.8

Table 4.19 Collision indicator.

Functionality

a) Cross section

Even though a rectangular cross section is used for the proposed SFT, which would be one of the most effective in terms of functionality within tunnel cross sections; still, height would be a restrictive characteristic of it (In the European Union, vehicles with a gauge greater than 4 to 4,3m are not allowed in tunnels). By contrast, a bridge would obviously not be affected as vehicles circulate in open air, more space will be offered for facility distribution and will be able to guarantee access to every vehicle (weight restriction). However, *el Ministerio de Transportes y Telecomunicaciones de Chile* in his last resolution of the articles 56º and 57º of the law Nº 18.696 (http://bcn.cl/1vg3n) establishes 4,2m as the maximum height permitted and for special vehicles 4,3m, such as trucks destined to the transport of vehicles. Steer Davies Gleave consultancy in 2002 studied the increase of traffic over time and estimated the future demand of the new suspension bridge. Considering his estimation for 2018, a total of 6762 vehicles crossed the channel, being 1130 heavy vehicles, a 16,7% of the total traffic. If we consider that five percent of this traffic exceed the tunnels maximum height stablished in 4,3m, 56,5 vehicles will have not been able to cross through the SFT, 0,83% of the total traffic and would have to use ferries to cross. (In this case, the less has considered being the worst). Then, the indicators are, for SB = 6762/6762=1 and for SFT =6756 /6762=0.99.

Cross section	SB	SFT
Indicator	1	0.99

Table 4.20 Cross section indicator.

b) Extension possibility

It is obvious that the extension of a SFT is not possible; the solution would be building a parallel tunnel. On the other hand, even though is complicated, the extension of a suspension bridge, is possible. A recent example of a bridge extension would be the Rande Bridge in Vigo (Bernardo, H. et al. 2019).

Extension	SB	SFT
Indicator	1	0.1

Table 4.21 Extension indicator.

c) Connectivity to the existing network

As seen in previous situation maps, the SFT will end up connecting to the existing roads, and so, an extension of the existing road will not be needed. On the other hand, for the correct connection of the new suspension bridges, the construction of an extra 13,27 Km approximately of road will be needed, and so in comparison the new bridge offers a bad connectivity.

Connectivity	SB	SFT
Indicator	0.1	1

Table 4.22 Connectivity indicator.

4.4.2. Value Calculation

We will now proceed to calculate the different values considering de different weights established for each criteria.

			Indicator		W*I	
Category	Criteria Weight(%)	Criteria	SB	SFT	SB	SFT
Economic Impact	29.5	Construction Costs	1	0.8	0.21	0.30
	33.2	Maintenance Costs	0.75	1	0.25	0.33
	37.3	Durability	1	1	0.37	0.37
Value					**0.92**	**0.94**

Table 4.23 Economic value.

From the calculated results, we can appreciate that the SB alternative is more unfavourable with respect to the economic impact. In addition, it is also seen how the SFT cost does not vary exponentially with length. Durability should be more deeply investigated in order to have more precise conclusions.

			Indicator		W*I	
Category	Criteria Weight (%)	Criteria	SB	SFT	SB	SFT
Environmental Impact	19.1	Landscape Impact	0,1	1	0.02	0.19
	19.5	Fauna and Flora Impact	1	0.8	0.20	0.16
	20.3	Sea water Impact	1	0.6	0.20	0.12
	20	Air Impact	1	0.6	0.20	0.12
	21.1	Noise Impact	0.3	1	0.06	0.21
Value					**0.68**	**0.80**

Table 4.24 Environmental value.

From the results obtained it can be seen that the SFT will have a less environmental impact, considering the fact that a noise impact has more weight than a sea water or fauna and flora impact. Noise will be a part of the whole lifecycle for both infrastructures. On the other hand, it is thought that seawater, flora, and fauna will only be affected during the construction of both, as once infrastructures are on its operation phase, the impact will be already caused. In addition, landscape impact it is thought to have the least weight, as a view impact will only affect the natural concept of the site.

			Indicator		W*I	
Category	Criteria Weight (%)	Criteria	SB	SFT	SB	SFT
Social Impact	31.8	Execution Time	1	0.75	0.32	0.24
	36.5	Travel Affectation	1	0.62	0.37	0.23
	31.7	Inconvenience to neighbours during construction	1	0.3	0.32	0.10
Value					**1.00**	**0.53**

Table 4.25 Social value.

From the table can be appreciated how the construction of the Chacao bridge will more satisfying value. Less time for its construction will be needed as the crossing is shorter and there is more experience on the construction of bridges. Its less lower time of construction means its access will be guaranteed before. In addition, disturbances on the surroundings will last less. As seen in the table and analysed previously, the bridge will provide a quicker crossing and so vehicles will be able to get to their destinations before, even though there is only a difference of 5,5minutes which is not much depending on the drivers destination. Finally, fewer inconveniences to neighbours will be caused, as its construction will take place far apart from the nearest village.

			Indicator		W*I	
Category	Criteria Weight (%)	Criteria	SB	SFT	SB	SFT
Safety	40.2	Against Earthquake	0.3	1	0.16	0.40
	29.4	Against Tsunami	1	0.6	0.29	0.18
	30.4	Against Collision	1	0.8	0.30	0.24
Value					**0.72**	**0.82**

Table 4.26 Safety.

From the calculated results, we can appreciate that the SB alternative is more unfavourable with respect to the safety of the infrastructures. Weights have been stablished considering the probability of occurrence and affectation. It can be appreciated, it has been considered a much bigger weight for safety against earthquakes as it is more probable for an infrastructure to fail due to the high seismicity of the zone, and SFT appears to have a much better response against them. It has also been considered the probability of failing due to collision and tsunamis, but its occurrence and affectations are considered minor. It is thought that the SB will be less affected by both as explained before.

			Indicator		W*I	
Category	Criteria Weight (%)	Criteria	SB	SFT	SB	SFT
Functionality	40.2	Cross section	1	0,95	0.40	0.38
	13.5	Extension Possibility	1	0,1	0.14	0.01
	46.3	Connectivity to existing road network	0.1	1	0.05	0.46
Value					**0.58**	**0.86**

Table 4.27 Functionality.

Finally, it can be appreciated from **Table 4.27** that the proposed SFT will be a much more favourable alternative in comparison with the SB in terms of functionality.

Connectivity to existing roads it is thought to be of great importance inside the functionality of these infrastructures. The need of new roads will have a direct impact on the environment, on the construction cost and on the execution time of the project. Therefore, taking advantage of the existing roads reduces these impacts.

The infrastructure cross section has also thought to be of great importance in terms of functionality. Depending on the section considered, the arrangement of facilities can be much more difficult and could determine if using a bigger or smaller section, this will have a great influence on the production cost variation. In our case, the most important fact that has been considered between infrastructures has been the height restriction, as both cross sections are considered to easily accommodate facilities and offer a width cable to support all types of traffic. As analysed before, the SFT due to the height restriction it is thought that is not able to allow the crossing of a small percentage of special vehicles. Therefore, it is believed that its section with respect to that of the SB, has a slightly worse functionality. It has not been considered of great importance the extension of both infrastructures, as considering the future and actual traffic crossing the channel (Steer Davies Gleave consultancy, 2002), the design of both are capable of offering a good service.

Once evaluated each impact for both infrastructures and having a better understanding of the results obtained, we can proceed to calculate the global value in order to obtain a final conclusion.

Global weight (%)	Category	Impact		Global value	
		SB	SFT	SB	SFT
21.3	Economic Impact	0.92	0.94	0.20	0.20
20,.1	Environmental Impact	0.68	0.80	0.14	0.16
19.1	Social Impact	1.00	0.53	0.19	0.10
20.5	Safety	0.72	0.82	0.15	0.17
19	Functionality	0.58	0.87	0.11	0.17
	Total value			**0.78**	**0.80**

Table 4.28 Final Global value.

The result obtained by the product of each category with its corresponding global weight and the final sum of all, shows a very small variation between infrastructures global values. It appears that the SFT has a slightly higher global value, but the difference is so small that it could be concluded that both infrastructures are feasible for the site. In this case, it should be more deeply analysed to come up with more precise indicators in order to have a better perspective of their value and on whether it is more viable the construction of the Chacao bridge or the proposed SFT design.

5. Conclusions.

The objective of this thesis is to understand how Submerged Floating Tunnels work, how they are designed and constructed, find advantages and disadvantages and, when all that is clear, to see a possible application of this type of crossing on the Chacao Channel. All this in comparison with a much more common structure such as a long span bridge, in this case a suspension bridge.

During the project, several similarities have been noticed between the SFT and a suspension bridge. Nevertheless, those that have really helped us to understand why in many cases where a suspension bridge is proposed to be build, the possibility of constructing a SFT may also appear, have been summarized below.

The bridge deck is supported and the Submerged Tunnel is anchored through vertical or inclined cables, being the gravity loads on the cable-supported bridge replaced by the upward residual buoyancy in the SFTs. The most important parameters responsible for the stability of a Floating Tunnel are waves and currents, by contrast, the stability of the bridge will be determined by wind. We could then say that the loads to which both infrastructures will be subjected, are analogous. Finally, to all this, can be added the fact that both structures are capable of offering long crossing distances.

Many advantages with the construction of a SFT were identified, most of them solving problems that conventional infrastructures cannot deal with. The clearest advantages that were present in many SFT cases are the following;

- The cost of a SFT will not vary exponentially with an increase in depth or length which makes such infrastructure very competitive in the construction world. Until now, both parameters were the main restrictions in many water crossings.
- In many cases it presents to offer a shorter and quicker way of crossing, because unlike a suspension bridge, end slopes are not considered part of the crossing.
- Air and sea traffic would not be affected by its construction and operability.
- The SFT will not interfere with the natural beauty of the surroundings.

To sum up, SFT will be able to offer an invisible, quick and economical way of crossing between sites hitherto unthinkable. However, disadvantages are also present, the most interesting found are listed below;

- In many studies, it has been found that the SFT has the highest total CO_2 emissions and have concluded that was due to the high material consumption.
- A seabed anchored SFT can provoke a very high sea water impact, this impact can increase with the distance and the type of foundations used. However, a

SFT can also be anchored by means of pontoons on the sea surface so less impact will be caused.

So, watching the great advantages and the possible solution or minimization of the main disadvantages that such a structure could offer, the reason why until now no SFT has been constructed was seek.

The answer was that, in addition to the many possible positive impacts that a SFT can cause, since no SFT prototype has been built so far, it has not yet been possible to prove the possible problems and difficulties that we could encounter with the structure interaction with water, the mooring to the tunnel, the anchoring to the water bed, the installation, the connection to the shore etc. and therefore, it has not been possible to establish safety coefficients yet.

In the past years, an attempt was made to build a 100m long prototype on Lake Qiandao in China, however due to some extra technical problems it was not possible to make it a reality. Moreover, many countries have proposed a SFT as an aquatic crossing solution in their territories.

In conclusion, studies are being developed (Recently in Madrid a conceptual design congress took place to discuss future directions for research inside the field of the floating tunnels. This congress was organized by ACHE (*Asociación Española de Ingeniería Estructural*) and fib) and pre-projects have been developed, but there is still no established body of doctrine for the construction of SFT.

In the final part of the project, the feasibility of a SFT at a given location has been evaluated in comparison to a Suspension Bridge.

To do so, an objective and systematic method has been chosen, such as the multi-criteria analysis. This analysis method has evaluated and compared, both construction processes integrating diverse aspects such as costs, environmental impact, safety, social impact and functionality.

From the analysis, we can see the benefits a SFT can contribute to these aspects. Even though it is seen that in comparison with the SB it has a unfavorable social value. We must say though, that this effect was due to the proposed designs location; its proximity to a village increased its social value (possible disturbances being caused during material transportation). A different location on a rural site would had varied the value obtained and a more satisfactory result would had been achieved. (in this case it is not possible as shown in chapter 4, due to bathymetry, benthonic resources extraction areas and aquafarming activities).

Finally, the results obtained for both structures have been considered to be very similar, being 0.78 for the Suspension Bridge and 0.80 for the Submerged Floating

Tunnel. It can be concluded so, that the construction of either a SFT or a SB in the Chacao Channel is feasible.

Nonetheless, in order to obtain a more precise result, a more in-depth study should be carried out, since the quality of the analysis has been limited by the availability of data. Consequently, the indicators had to be estimated, which were the way to measure the alternatives with respect to the different aspects. Therefore, the results reached are approximate.

However, with these approximated results, it is already demonstrated how competitive a SFT can be. In conclusion, with this thesis it has been proved, by watching the successful results, the possibility of having obtained better ones in a different location, and considering all the benefits a SFT could contribute to the water crossing concept, how viable could be the construction of such an infrastructure in a nearby future, once a body of doctrine is stablished and its construction is possible.

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