Abstract: For centuries, a permanent 3km crossing over the Strait of Messina between Calabria in south Italy and Sicily has been considered, including fixed bridges, floating bridges and tunnels. In March 2009, previously shelved plans for the world’s largest single span suspension bridge were resurrected, arguably having overcome countless engineering, political, financial and cultural hurdles. This paper will critically examine these aspects and other important aspects of the bridge, and will attempt to judge the engineering feasibility of the current design in context of the present and future Italy.

Keywords: Suspension bridge, triple-box deck, aerodynamics, controversial, vented deck

1 Introduction

1.1 Background

The Strait of Messina is a 3km section of water between Sicily and southern mainland Italy (Calabria). The idea for a bridge has been around since the Roman times, yet engineering challenges caused by the strait’s deep water, fast flows, high seismic activity and high winds have hindered most designs so far [1].

Today, southern Italy has a weaker, less stable economy than its northern counterpart. Road and rail ferries across the strait take up to two hours, and are congested during peak times. It is believed that a permanent crossing will facilitate economic growth and social regeneration; the construction phase alone is predicted to provide an economic gain of €6bn, similar to the cost of the project itself [2].

1.2 Consideration of alternative schemes

In 1981, the concessionaire company Stretto di Messina was created by the Italian government to oversee the design, construction, operation and management of the crossing. Numerous solutions were proposed.

Both bored and floating tunnels were considered, but were rejected. Given a tunnel depth of -280m mean sea level (MSL) [3] and link height of about +50m MSL, an impractical 47km of autostrada tunnel links would be required [2]. Numerous active seismic faults run along the Strait [4] with potential for a repeat of the 1908 Messina
earthquake of about magnitude 7.2. Tunnels are commonly perceived to be invulnerable to earthquakes. However, the 1999 collapse of the Turkish Twin Bolu tunnels challenges this view, demonstrating vulnerability to comparable earthquakes [5]. Bored and floating tunnel behaviour under seismic action continues to be an ongoing research topic, such as Ref. [3].

A multi-span bridge involves construction of sea floor founded piers of an unprecedented depth of about 150m, making them hugely difficult to construct. Such piers must be constructed to withstand the Strait’s high design sea flow of 5.1m/s, wave height of 16m, a 200 year design life, and interference during construction and operation with 140,000 vessels through the Strait each year [1]. Furthermore, towers on the sea floor will have an increased total height, increasing their susceptibility to earthquakes.

1.3 Current Plans

A single span suspension bridge is able to either avoid or mitigate the aforementioned issues. In addition, it is by far the most tried and tested method, with a vast accumulated knowledge on their design, construction and maintenance, with a substantial, developed industry. By minimising geotechnical and subsea work, cost and construction time are more predictable. Key statistics are given in Table 1 and are compared to best precedents [2].

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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<tbody>
<tr>
<td>Largest span</td>
<td>3300m (Akashi-Kaikyo 1991m)</td>
</tr>
<tr>
<td>Tower height</td>
<td>382.6m (Millau Viaduct 343m)</td>
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<tr>
<td>Deck width</td>
<td>60.4 (Akashi-Kaikyo 35.5m)</td>
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<tr>
<td>Cable diameter</td>
<td>4x1.24m (Akashi-Kaikyo 2x1.12m)</td>
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<td>Design wind speed</td>
<td>75m/s (Akashi-Kaikyo 80m/s)</td>
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<td>Design earthquake</td>
<td>7.1 magnitude (Akashi-Kaikyo 8.5)</td>
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<td>Design lifetime</td>
<td>200 years (typical 120 years)</td>
</tr>
<tr>
<td>Approx. cost</td>
<td>€6.1bn (Akashi-Kaikyo €3.8bn)</td>
</tr>
</tbody>
</table>

Table 1: Key statistics of the proposed Messina Bridge

The bridge carries a service lane, a hard shoulder, two road lanes, two vented lanes (for deck aerodynamics) and a rail line in each direction.

![Figure 3: Deck section, units in metres, from Ref. [2]](image)

Towers are founded on the dry shores of Sicily and Calabria (Figure 1). Two pairs of main cables support groups of hanger cables every 30m, which support cross beams at deck level. These in turn support the deck sections. The deck uses a triple box aerodynamic deck, thus forming two longitudinal slots through the deck along the bridge length, designed to aid aerodynamic stability. Related design issues will be explored later on.

The choice of location of the bridge is restricted, since the funnel-shaped Strait opens up quite rapidly. The location is therefore set at the narrowest point of the Strait.

1.4 Controversy

Through years of development, the bridge has struggled to maintain political support. It was eagerly backed by Prime Minister Berlusconi from 1999 to 2006, but was dropped by Prodi, in power from 2006 to 2008. Cited reasons included a potential governmental need to fund the bridge, despite Berlusconi’s claims that no central funding would be required. Berlusconi planned to fund the project with 40% by Stretto di Messina’s shareholders, and 60% through loans to be repaid by the bridge’s operations. After re-election in 2008, Berlusconi resurrected the plans for the bridge.

Corruption in Italian politics and organised crime are both well linked in Italian history. Until the 1990s, the Christian Democracy party maintained a near-stronghold on the Italian government. This led to widespread corruption and mafia collusion, exposed in the Mani Pulite investigations from 1992. Since then, an effective two party system has developed, which to some extent ensures that a particular party now has to maintain integrity in order to retain power.

Organised crime is extraordinary problematic in Italy since the mafia is well accustomed to infiltrating the local economy and politics, certainly aided by the aforementioned corruption. Traditionally, the mafia would force local businesses to pay the ‘pizzo’, a protection tax, paid by 70% of Sicilian businesses and 50% of Calabrian businesses [6]. The pizzo is also paid for any construction work taking place on a mafia group’s territory. This is typically 3% of the contract value. Since the 1950s, the mafia has also been able to set up its own construction firms, gain sub-contracts, and infiltrate management and politics behind other such major public project tenders [7].

Clearly, a project substantially operated by the mafia, motivated by short term financial gain, will not be constructed to the necessary quality. Mafia infiltration of the Messina Bridge may have dire consequences, especially given the highly technical and precise construction required. A factor against the mafia is that a significant region of money will flow between large international consultants and contractors, with whom the mafia have no previous involvement. This makes infiltration more difficult, especially at the more lucrative management levels. Control of small and mid size contractors is a real problem however, and seems to have been disregarded by Berlusconi. Whether this is due to incompetence or mafia collusion is unclear, but Berlusconi’s questionable history, including alleged corruption and mafia collusion, has meant that the mafia’s potential involvement remains an issue of contention with the Italian public [7].

It has been questioned whether the bridge is required at all. The ferry service can be expanded for a much smaller sum, although problems of ferry pollution and socioeconomic isolation would remain. The road network leading to the bridge is claimed to be of lower quality than UK motorway standards, and the capacity of these roads is not great enough to provide a profitable traffic throughput on the bridge. However, given that there are
only two lanes in each direction on the bridge, this is not a
convincing argument.

There is substantial empirical evidence to suggest that
major infrastructure projects almost always run over
budget (34% greater than predicted on average, equivalent
to over £2bn for the Messina Bridge) and that traffic is
difficult to forecast. For instance, the £4.7bn Channel
Tunnel was 80% (£2.1bn) over budget, yet as of 2003 it
carries just 18% of forecast traffic [8]. Thus, history has
shown that the potential of the bridge being unable to fund
itself and relying on government support is very real,
despite Stretto di Messina’s claim that the bridge will be
economically viable under all foreseeable scenarios.

Environmental aspects of the bridge, including effects
on local wildlife and the effect of such an immense
structure on a poor, tradition-bound region have provoked
local resentment and protest. Measures to reduce
environmental impact, such as using waste material from
the tunnel links to refill the quarries used to mine
foundation materials, help to some degree, yet the size of
the structure and concerns of the other aforementioned
shortfalls are more difficult to overcome.

The final controversies concern the engineering
feasibility of the bridge itself, including its ability to
overcome wind and seismic effects. This will be
investigated in later sections.

1.5 Present status

Berlusconi recently committed €1.2bn to the bridge
as an investment against the global 2009 recession. It is
unknown if the heavily indebted government can afford
this. After the earthquake of L’Aquila in April 2009, he
has suggested that cash earmarked for infrastructure
projects could be diverted as relief funds. It is likely that
this will affect funding for the Messina Bridge. If plans
still go ahead, construction is expected to start in late
2009 and complete in 2016.

Due to the unstable background of the project,
numerous incarnations of the design exist. The most
developed and useful information comes from the plans
developed up until 2006. Hence, the bridge shall be
analysed based upon these plans, unless otherwise stated.

2 Aesthetics

When discussing the aesthetics of the bridge, it must
be borne in mind that all images are of architectural
impressions, models and imagination. Therefore,
discussion of the bridge’s appearance in real life is limited
somewhat, but the best attempt shall be made nonetheless.

The aesthetics of the Messina Bridge are hugely
dominated by cost and its engineering design, with
relatively little leeway for visual refinement. It is therefore
both fortunate and pleasing that the most efficient form
of the bridge is also highly elegant and attractive.

The design is based upon the classical European
design of suspension bridges, with simple horizontal
cross-beams and an aerodynamic deck. Aerodynamic
suspension bridges, by their very nature, are dominated by
the two towers and the main cables between them. This is
because the aerofoil basis of the deck gives it a very light,
slender appearance (Figure 1) compared to a truss system
(such as Akashi-Kaikyo). It could be argued that the deck,

at 60m wide but principally less than 4m deep, is so
shallow that it gives a feeling of instability and insecurity
to bridge users. However, it must be remembered that the
bridge will be uncommonly seen at an angle that
accentuates this slenderness, somewhat invalidating this
argument.

The shallowness of the deck helps to reinforce the
function of each component of the bridge in the eye of the
layman. If the deck is a truss type, it can be unclear why
the truss is there. It can look like it is helping the hanger
cables to carry the deck load, perhaps suggesting that the
cables are too weak to do this alone. Eliminating this truss
and making the deck look weaker actually helps to define
the function of the deck more clearly, in that it is only
carrying the loads directly imposed upon itself, and then
sending them to the nearest hanger.

Hereafter, the functions are obvious. The hangers
work in tension, sending forces to the main cable and into
the ground by compression via the towers. The
anchorages are necessary to contain the main cables’
horizontal component.

The height of the towers was dictated by the span in
order to give a depth to span ratio of 1:11. Any shallower,
and the cables would have been impractically big; any
deeper, and the towers would have grown, increasing
material use and earthquake susceptibility. It is again
fortunate that this ratio feels ‘just right’; the cables are
shallow enough to maintain a smooth flow over the water.

The towers are also well proportioned. Despite their
immense height, their large width ensures that they do not
feel too narrow. The cross-beams also appear to have
been spaced with the golden ratio in mind.

Perhaps one failure of the bridge aesthetically is that,
on closer inspection, its order breaks down. The triple-
box concept (Figure 4), incorporating grids between the
boxes, protruding service lanes, aerodynamic wind breaks
and supporting cross beams every 30m, come together
into a complex, seemingly disordered structure. Likewise,
the use of four cables (in two pairs) seems unnecessary. It
shall be seen later however that all these are important
tools in maintaining the aeroelastic stability of the whole
bridge.
A minor disadvantage of having the towers on the shore is that, once on land, the deck can be easily supported by regular piers on the dry land. This means that the hangers end prematurely on the approach to land, suddenly making the main cables seem isolated and vulnerable (Figure 5). On the contrary, this creates a highly symmetrical elevation of the whole bridge.

Refinements by consultants COWI in 2004 (Figure 6) included tapering of the crossbeams, making them more slender in their centres and reducing their visual mass. An octagonal leg section not only has desirable aeroelastic properties, but has excellent textural properties too. A shadow on one of the leg’s faces is visible from almost any position of the sun and viewer, thus lessening the visual impact of the huge 20m by 12m legs.

The sheer size of the bridge will play a huge part on how it is perceived visually. With potential to break countless world records, it will have the attention of the developed world during construction and operation. With this attention, aided by its size, character will become attached to the bridge. Whether this character will be positive or negative is currently hard to say. Internationally it may be favourable as a phenomenal engineering achievement, while locally the bridge’s scale currently attracts bad press, and may continue to do so.

Integration and sympathy with the environment is almost absent. Unfortunately, it is practically impossible to take such a cultured, traditional, relatively undeveloped region, and place such an immense symbol of modern technology and industrialism without expecting conflict. It could be argued that the bridge is a symbol of the development of southern Italy, and perhaps, if proponents of the bridge are correct, then this will be the case in the future. In any case, if a permanent link is required then a crossing of this nature is the only way to do it, as previously discussed. One can only hope that the locals learn to adopt the bridge as part of their landscape.

### 3 Development of the bridge

Given the unprecedented nature of the bridge, unusually extensive research has been performed to determine the load cases on the bridge, and to determine its instability modes. These have been used to precisely develop a bridge design that is stable, comfortable and safe under these cases.

#### 3.1 Environmental parameters

The first stage is to establish the worst case environment which the bridge must withstand. In the case of the Strait of Messina, the two critical scenarios are a high wind speed and a high seismic load.

##### 3.1.1 Design wind parameters

In order to accurately model the bridge behaviour in real wind patterns, it was insufficient to simply consider the maximum wind speed derived from standards or past records. For this reason, a weather station on the north east point of Sicily has been gathering wind records since 1985, thus allowing a very detailed wind profile of the site to be created.

It was found that the predominant wind direction was from the south west [9], likely to be an effect of the funnel-shaped Strait. As of 1994, the highest gust recorded was 32m/s and the statistical 2000-year return wind speed was 60m/s [1]. The eventual decision to use a design wind speed of 75m/s could be viewed as overly conservative. However, such pessimism is perhaps necessary to instil public and political confidence of the bridge’s stability in high winds in light of the bridge’s lack of precedence, the Tacoma Narrows disaster, and of potential wind speed increases from climate change.

##### 3.1.2 Design earthquake parameters

The Strait of Messina is geologically characterised by a system of active faults both within the Strait and on both shores, resulting in a normal expansion of the Strait, and in uplift of Sicily. Design parameters were derived from geophysical studies and statistical analysis have given a 1 in 2000-year design vertical ground acceleration of 0.58g, equivalent to magnitude 7.1 earthquake with an epicentre 20km away. The worst-case uplift has been established as 0.243mm per year, equivalent to 49mm over the life of the bridge [1]. This uplift should be within the movement tolerances of the structure.

A conflict to the analysis is that the infamous 1908 Messina earthquake, with an epicentre 15km from the bridge, was of magnitude 7.3 [10], and a similar earthquake is expected to repeat every 1000 years [11]. However, this is not a particular concern as a similar earthquake is not expected to reoccur during the life of the bridge.

The Strait’s expansion has been estimated to be 1.7 to 3mm per year, potentially 600mm over a 200 year life [4]. This should be borne in mind when designing expansion joints. Such an expansion will also raise the deck by about 2m through an increase in main cable tension; this will need to be accounted for in the main cable calculations.
3.2 Aeroelastic research

3.2.1 Aeroelastic principles

Decks of long span bridges have low natural frequencies which may allow wind to subject the deck to cyclic loads. Several requirements must be met for aerodynamic stability and structural efficiency of the deck, including high stiffness, low mass and low aerodynamic resistance. Traditionally, a truss has been used, exhibiting all of these qualities to some degree. However, as the bridge span increases, weight and aerodynamic resistance become increasingly important, meaning a truss system cannot be used. This necessitates the use of a lightweight aerodynamic section. However, this has a relatively reduced stiffness. To mitigate this problem, the shape of the aerofoil was fine-tuned to enhance its stability in high and turbulent winds. This was done by considering its interaction with the air moving around it aerodynamically [9].

In order to do this, three aeroelastic problems had to be solved:
1. Aerodynamic stability, including stability against flutter
2. Vortex shedding response
3. Buffeting response

Many strategies exist to solve these problems. Flutter instability occurs when a particular wind state brings the cables’ first vertical and deck's first torsional natural frequencies (Fig. 7) close to one another. This reduces the ability of the bridge to naturally dampen energy absorbed from the wind, potentially leading to collapse. To keep these natural frequencies apart, the deck’s stiffness should be as high as possible, thus raising the first torsional natural frequency well above the first vertical natural frequency of the cables. A truss is more suitable for this purpose, but its higher moment of inertia and associated stability issues mean that a suitably stiff aerofoil design is an acceptable compromise.

![Figure 7: Important modes of cables and deck](image)

Vortex shedding is the generation of alternate vortices as a fluid passes by a bluff body, which generates alternating forces on the body. If the frequency of the generation of vortices is close to a natural frequency of a given component, then ‘lock-in’ occurs, where the frequency of vortex generation locks in to the given natural frequency of the component. This is non-catastrophic, but must be limited to extend the fatigue life of the component. Prevention centres around improving the aerodynamic shape of the component, or breaking up these vortices at the point on the component where they form and break off, such as by using wind breaks or meshed surfaces.

Buffeting is the effects of turbulent wind on the structure. A stable structure should be able to dissipate absorbed energy; passive or active dampening can be installed otherwise. The effect on a structure is difficult to quantify, and so is typically tested using wind tunnels.

During the peak of research for the Messina Bridge, computing power was limited, necessitating the thorough use of wind tunnel testing, using 2D models for cross section design and 3D models for further verification of wind flow at oblique angles.

3.2.2 Cross-sectional deck design

Using aeroelastic theory, the static drag, lift and torsional forces on the deck respectively are as follows:

\[
F_D = \frac{1}{2} \rho U^2 B L C_D\]

(1)

\[
F_L = \frac{1}{2} \rho U^2 B L C_L\]

(2)

\[
F_M = \frac{1}{2} \rho U^2 B^2 L C_M\]

(3)

where \( \rho \) is air density, \( U \) is wind speed, \( B \) is the bridge chord, \( L \) is the bridge length under question, and \( C_D, C_L \) and \( C_M \) are coefficients of drag, lift and pitch respectively.

These equations assume a static structure, which in reality is far too simplistic for such a flexible structure. Modified expressions find the relevant forces by replacing \( C_D, C_L \) and \( C_M \) with functions of flutter derivatives and the reduced velocity (which is a dimensionless value that essentially provides similitude between model results and the real structure). These are as derived empirically by Diana et al [9]; full discussion of flutter derivatives is beyond the scope of this paper.

In order for the deck to remain stable as an aerofoil, the derivatives of the pitch and lift coefficients must remain small and positive with respect to the deck pitch angle, whilst satisfying all other criteria for stability. Various solutions were tested with emphasis on modifying the wind barriers and under-deck profile, which are the key areas of fluid-structure interaction. The optimum solution is shown in Figure 8, and aeroelastic properties are shown in Figure 9. The final design modified the originally proposed aerodynamic deck adding wind barriers around the rail lines, and adding aerofoils to the outer wind barriers [12]. These wind barriers are also to be made traffic height; this has the advantage that the presence or absence of traffic has minimal effect on the aerodynamic properties of the deck.

![Figure 8: Chosen deck design [12]](image)

![Figure 9: Deck aeroelastic properties, x-axis is pitch [9]](image)
Through numerical modelling, the first vertical and torsional natural frequencies are 0.061Hz and 0.081Hz respectively. (Numerical methods must be used for structures of this size and sensitivity in order to derive the natural frequencies for all modes of interest to a sufficient accuracy.) This is a satisfactory separation ratio of 1.33, providing stability against flutter at wind speeds in excess of 75m/s [9]. The vortex shedding response was also verified as being minimal [12].

3.2.3 Deck vent design

Perhaps the greatest innovation of the Messina Bridge is the development of the slotted box girder deck design, pioneered by Brown Beech & Associates during the early development (early 1990s) of the current design. Wind tunnel tests showed that such a deck with a slot running between the railway line and each of the road decks had a substantially higher wind flutter velocity than a comparable deck without slots. Although unexplained theoretically at the time, continued wind tunnel testing gave researchers sufficient confidence in the design, eventually leading to the present deck construction.

Conclusions empirically deduced by the Messina Bridge researchers related to the aeroelastic design of the deck, parapets and other devices were subsequently confirmed by Sato et al. [13]. In particular, the deck’s stability was explained.

If aerodynamic damping is \( \delta_a \), then flutter occurs when \( \delta_a \leq 0 \). This condition was applied to flutter derivatives and the Nakamura equations [14] to define the onset of flutter as given in Eq. 4:

\[
\frac{2\pi\alpha L_{R}}{M_{b}} + \beta B_{R} \geq 1
\]

where \( M_{XX} \) and \( L_{XX} \) are coefficients of unsteady dynamic forces, and \( \alpha \) and \( \beta \) are constants as defined by Nakamura. By performing a 3D flutter analysis of a bridge section, these coefficients can be derived. It was shown that adding a slot decreases the value of \( M_{XX}L_{R}/M_b \) while the use of aerodynamic parapets decreases the value of \( M_{b} \).

In real terms, this is probably the result of a disturbance of the vortices and air flows which generate flutter in the first place, whilst keeping the deck aerodynamic enough to mitigate other stability issues. On the Messina Bridge, the decision to use two vents rather than one ensures the bridge is wide enough to provide the required torsional stiffness, perhaps whilst maintaining the vents at an optimum width. It also ensures that the full rail system is kept on the same deck box.

It is worth noting that the longest aerofoil bridge in the world is currently the Xihoumen Bridge in China, with a main span of 1650m. Since it employs a single vent twin-box steel deck, this will likely be the origin of further research prior to Messina for gaining a greater knowledge on associated phenomena.

3.2.4 Tower design

At 382.6m high, the towers also have significant aeroelastic stability issues. Their relatively low weight and lack of structural damping reduces their ability to deal with any aeroelastic instabilities.

Numerical analysis showed the natural frequency of the towers in the weak axis to be 0.1Hz and in the strong axis to be 0.3Hz. Upon installation of the main cables the weak axis natural frequency increased fourfold. Since the wind speed at which vortex shedding excitation occurs is a function of the natural frequency of the structure by the dimensionless Strouhal number, the lower the natural frequency, the lower the wind speed is needed for this excitation to take place. The critical phase for the towers for vortex shedding excitation is thus during construction; initial designs would induce vortex lock-in at wind speeds as low as 10m/s [15].

The solution proposed was to line the edges of the tower with a mesh in order to break up these vortices. However, in recent images, this doesn’t seem to be the case. This is perhaps because the meshes are removed after construction to improve the tower aesthetics. Alternatively, this idea may have been subsequently discarded. The most recent tower designs show the tower leg to be 20m by 12m in plan, rather than the original 16m by 12m. This extra width may have increased the second moment of area, and thus the natural frequency, enough to make the excitation occur at a suitably higher wind speed.

3.2.5 Cable design

Loading calculations demonstrate that the bridge shall require approximately 4m\(^3\) of steel in the main cable. This would require two cables of 1.75m diameter, or two pairs of cable of 1.24m. The latter was chosen for aerodynamic reasons: not only is the area of cable open to the wind reduced, but there may be an aeroelastic advantage akin to the decision to use a vented deck.

3.3 Articulation and expansion requirements

The south of Italy may experience a worst case temperature range of between about -4°C and 46°C. Assuming the bridge is constructed at the mean temperature of 21°C and the thermal coefficient of expansion for the whole 3666m bridge is 12x10\(^{-6}\)°C\(^{-1}\), then the bridge will experience a longitudinal strain of ±1100mm. This translates to an average stress of 63MPa across the deck’s section if the deck is not permitted to expand, which itself is a significant percentage of the strength of the S460 steel used to make the deck.

It is possible that such a force will have already caused the deck to fail through buckling. There are many possible buckling modes that must be considered. In particular, the feasibility of a given vertical buckling mode depends on the ability of a hanger to deck connection to be displaced relative to a fixed reference point. This largely depends on the hanger elasticity and cable mass, thus the critical buckling mode is best established computationally.

A potential vertical mode crudely assuming \( A=1.4m^2 \), \( I=2.59m^3 \) and effective length \( L' \) of 1900m between fixed supports gives a buckling load \( P_{crit}=1.4MN \), equivalent to 1.0MPa. This is very low, but is unlikely to occur due to the restraining effect of the hangers. The mode assuming \( L'=30m \) gives \( P_{crit}=5681MN \) (4058MPa). In the horizontal plane, \( I=2112m^3 \), meaning for all applicable modes \( P_{crit}=383MN \) (273MPa). The
critical mode is therefore likely to be an intermediate vertical mode.

Further expansions arise from the passing of a train, resulting in a movement of ±300mm at either shore. Long term seismic action, as mentioned above, has the potential to stretch the bridge by 600mm. Wind and seismic actions result in further movements at the shore which require consideration; a particular issue with these is that the former requires a stiff bridge for aerodynamic stability, while the latter requires a flexible, dampened structure to help dissipate energy.

A two-stage approach should be taken to account for expansion effects. These are to reduce the potential movement of the bridge, particularly where the deck connects to the land, and then to accommodate the remaining potential expansion with expansion joints and bearings. The first stage is required to minimise wear and tear on the bearings, thus reducing costs and bridge downtime.

For the first requirement, a buffer system has been developed, located below the deck on each tower. This system acknowledges that the inner rail box is a lot more flexible in plan than the outer road boxes due to its location to the neutral axis. Thus, there are two expansion joints per road box per tower (Figure 10), connected by a pin, allowing movement about the tower. Under normal conditions, an isolator locks this system, which only allows longitudinal road box expansion and maintains the bridge’s stiffness. During seismic events (i.e. longitudinal force >10MN) the isolator is released, permitting horizontal and rotational movement of the deck. An attached spring and damper system dissipates the energy.

![Figure 10: Tower-deck interaction](image)

This buffer system also acts to dissipate excessive movement from passing trains. The end of bridge expansion requirement is reduced to ±6/-4mm, equivalent to an accumulated annual expansion joint movement reduction from 65km to 1.6km, substantially reducing wear on the bearings.

Expansion joints at either end of the bridge allow for a total expansion of ±3.4m. This is clearly more than sufficient for the potential expansions described. It is likely that computer analysis has quantified further seismic and wind strains, and that factors of safety have also been applied. In calculations below, it shall be assumed that forces due to expansions have been mitigated unless otherwise stated.

### 4 Loading, strength and serviceability

Current plans of the bridge shall be assessed for their ability to carry the loads to be imposed upon them. This will be performed to BS5400-2:2006. The total bridge length is 3666m, with two towers 3300m apart. The bridge is assumed to act in isolation of its connecting road and rail links.

Loads not mentioned are beyond the scope of this paper. Only BS5400 load combinations the author feels are critical will be analysed. Construction loads are not considered to be critical.

#### 4.1 Nominal loads

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<td>Deck rail box</td>
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<td>Deck cross-beams (total)</td>
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<td>Main cables (total)</td>
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<td>Hanger cables (estimated total)</td>
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<td>Towers (each)</td>
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<td>Superimposed dead loads: (Ref. [1])</td>
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<td>Total, including rail tracks, paving etc.</td>
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<td>Wind (Eq. 1-2)</td>
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<tr>
<td>Deck (assuming pitch=0°, C_D=0.1 and C_L=-0.07 (Figure 9))</td>
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<td>Drag force F_Ds (1)</td>
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<tr>
<td>Lift force F_Ls (acting down) (2)</td>
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<td>Notional lanes (NL) = 2×4NL</td>
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</tr>
<tr>
<td>Knife edge load (KEL) per NL</td>
<td>120kN</td>
</tr>
<tr>
<td>Reduction factor from third NL</td>
<td>0.6</td>
</tr>
<tr>
<td>HB loading</td>
<td></td>
</tr>
<tr>
<td>Assume 45 units of HB loading</td>
<td></td>
</tr>
<tr>
<td>May act over 1 NL</td>
<td>450kN/axle</td>
</tr>
<tr>
<td>RU loading</td>
<td></td>
</tr>
<tr>
<td>UDL/track</td>
<td>80kN/m</td>
</tr>
<tr>
<td>Point loads/track</td>
<td>4×250kN</td>
</tr>
<tr>
<td>Dynamic factor (moment analysis)</td>
<td>1.06</td>
</tr>
<tr>
<td>Deck analysis (continuous 30m spans)</td>
<td>1.0</td>
</tr>
<tr>
<td>Global analysis (3300m span)</td>
<td>1.0</td>
</tr>
<tr>
<td>SW/0 loading</td>
<td></td>
</tr>
<tr>
<td>UDL over 2×15m/track</td>
<td>133kN/m</td>
</tr>
<tr>
<td>Dynamic factors as RU loading</td>
<td></td>
</tr>
</tbody>
</table>

**Table 2: Bridge loads (values in MN act over 3666m)**

#### 4.2 Material properties

<table>
<thead>
<tr>
<th>Component</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deck</td>
<td>Mainly S460 steel Outer surfaces 8mm, flanges 5mm</td>
</tr>
<tr>
<td></td>
<td>Box depth assumed 3.016m</td>
</tr>
<tr>
<td>Hangers</td>
<td>Assumed strength f_u = 1600MPa</td>
</tr>
<tr>
<td>Main cable</td>
<td>f_y = 1860MPa 41148 wires of 5.33mm diameter</td>
</tr>
<tr>
<td></td>
<td>Area steel = 0.92m² per cable</td>
</tr>
<tr>
<td>Towers</td>
<td>S460 steel Plate thickness 30-85mm</td>
</tr>
<tr>
<td>All steel</td>
<td>E=210GPa</td>
</tr>
</tbody>
</table>

**Table 3: Basic bridge material properties**
4.3 Deck road box design

To design for moment capacity of road box girder, consider high local loading on a road box with 4NL to get maximum hogging at supports (Figure 11). Supports assumed rigid, loads and factors as per Table 1, Ref. [18], load combination 1.

Using moment distribution method, $M = 14381 \, \text{kNm}$. Design moment $M_{\text{end}} = \gamma_w M = 1.10 M = 15819 \, \text{kNm}$.

Assuming values in Table 2 and $\gamma_w$ of 1.05, the deck requires $I=0.05 \, \text{m}^4$. The current design is estimated to provide $I=0.90 \, \text{m}^4$. This apparent error is the result of numerous effects that require additional strengthening of the deck. These include designing against fatigue in normal use to achieve the 200 year life, and designing for dynamic wind and seismic loads. Additional reinforcement will also be required to carry deck concentrated loads and to resist local buckling, which are not considered here.

Figure 11: Road deck box design scenario

4.4 Hanger design

Assuming load combination 2 [18], HA loading only, worst case loading for torsion, 52.4m separation of main cable pairs, 30m between hanger groups along the bridge length, 3 hangers per group (plus one redundant), and values in Table 2:

Figure 12: View through deck section

6839mm$^2$ of hangers are required per group, based upon $R_e$ being 10.4MN per 30m. This is equivalent to each hanger being 54mm diameter. This may be rounded up to a standard size, such as 75mm. Total hanger weight is thus 44.8MN, verifying the suitability of the hanger weight assumption made in Table 2.

The actual maximum force per hanger group may be increased by local load concentrations from HB loading and SW/0 loading. Calculation of this depends upon the elasticity of the cables, which is best solved computationally.

4.5 Main cable tension

Usually with suspension bridges, the main cable is only a small fraction of the suspended weight, meaning that the load is distributed fairly evenly along the bridge plan length and that the cable forms a parabola. The Messina Bridge is unique in that the main cable is comparable to the sum of all other suspended loads (both dead and live), so the cable shape will be between a parabola and a catenary. It shall be seen that this immense cable is largely required just to carry its self weight, and that if in future any longer spans are required for other bridges then stronger, lighter materials may have to be sought.

Firstly, considering the four cables supporting their self weight only, the cables’ horizontal component $T_0$ is found by numerically solving Eq. 5:

$$ R = \frac{T_0}{\gamma} \left( \cosh \frac{L}{2T_0} - 1 \right) $$

(5)

where $H=\text{sag (300m)}$, $L=\text{span (3300m)}$ and $\gamma=\text{weight per unit length (0.28MN/m)}$. $T_0$ is found to be 1284MN, or 349MPa (no factors applied). Adjacent to the towers, the cables’ self weight contributes to the cables’ tension as a function of cable length $S$ (Eq. 6) to a tension of 1368MN or 372MPa. It is worth noting that this is already well beyond the yield stress of standard steel.

$$ S = \frac{2T_0}{\gamma} \left( \sinh \frac{H}{2T_0} \right) $$

(6)

Meanwhile, considering all other plan length loads and that the main cable is weightless, the contribution of these forces to the cables’ tension can be approximately calculated. The horizontal component of this force is:

$$ T_0 = \frac{wL^2}{2H} $$

(7)

When $w=397.6 \, \text{kN/m}$ (unfactored load), $T_0=1804 \, \text{MN}$ (490MPa) and the tension at the towers is 1920MN (522MPa). This gives a total tension at the towers of 522+372=894MPa. When loads are factored to BS5400, these values respectively rise to 953+433=1386MPa. This compares satisfactorily with the factored cable strength of 1860/1.05=1771MPa. Remaining strength may be used for dynamic loads.

It is surprising that these calculations suggest that factored deck load is much greater than the factored cable load as this contradicts information given by the bridge’s present consulting engineers. This may be due to a combination of factors, such as working to different standards. Due to the bridge’s immense length, lower live loads may have been established; the rail live load (220kN/m) constitutes a third of the total deck load (660kN/m), and it may have been judged that this is overly conservative. Any equivalent of BS5400 factors to quantify inaccuracy in modelling may not have been used due to the extensive computer modelling that has taken place.

It is unlikely that steel cables significantly stronger than 1860MPa will be developed. Longer bridges may
have to consider other materials, such as carbon or glass fibre composites.

4.6 Serviceability

Much of the research behind this bridge aimed to minimise the deflections and movement of the bridge in order to maintain user comfort and to ensure that the bridge is open every hour of the year. The buffer system, lateral deck stiffness and deck aerodynamics ensure that deflections under SLS conditions are within ±9.9m and that associated accelerations are acceptable.

It is imperative that the main cables do not relax once in service. A strain of just 0.1% will see the centre span drop by 9m, which will mean that the deck begins to impinge on the 65m navigational clearance requirement. The manufacture of ultra high strength cables incorporates pre-stretching cycles which aim to eliminate this possibility.

5. Geotechnics

The area is characterised by numerous faults, both active and inactive, and poor soil conditions, particularly on Sicily where strong bedrock is about 400m underground. Figure 13 shows the area geology.

![Figure 13: Site geology, where arrows are tower locations, red blocks are anchorages and the large green band (H) is a layer of bound rock fragments, which is the highest layer of reasonably strong material. Within the ground, the red lines are active faults and green are inactive faults. Values on the left are height above mean sea level in metres. [17]](image)

5.1 Anchor blocks

The Sicilian anchor block will be set in Messina Gravels (E, Figure 13) while the Calabrian will be set in Pezzo Conglomerate (H), which is a soft bedrock. The poorer ground conditions on the Sicilian side means that larger anchor blocks are required than on the Calabrian side – 315000m³ and 220000m³ respectively. This is unavoidable since the Mediterranean Sea continues to the left of Figure 13, while to the right of the Sicilian anchor block (C) is low strength coastal plain deposits. The optimum position of the anchor block is also a function of the tower it serves.

The block design is dominated by static loading. The size of each block is vast at about 90m in the longest direction, yet this is necessary to resist the upwards and sliding actions from the cable tension calculated above. For instance, the pull-out resistance of the Sicilian block was calculated to be 9235MN, giving a realistic factor of safety (FoS) of 2.8 based upon the tensile force calculated above.

Anchor block movement is dictated by earthquake action. Computer analysis showed that, using the current conservative anchor block design, the worst case block movement under the design earthquake is 120mm laterally on each shore [17]. This will not cause unacceptable permanent movements of the bridge.

5.2 Tower foundations

The load carried by each tower’s foundation is 4137MN; wind loads also add a significant 2714MNm moment. However, the foundation design is controlled by earthquake loading which, for instance, add the equivalent of 5126MN vertically and 31766MNm in the bridge’s weak axis [17]. To accommodate this load, each Sicilian tower leg will sit on a 57m diameter pad 15m below ground level. Assuming a factor of safety of 3, the total load to be carried is 27789MN, equivalent to 5445kN/m² at the pad level. This is well above the ground strength, necessitating ground treatment. The selected method in critical regions is concentrated jet grouting, where columns of ground about 35m deep are removed with pumped water and replaced with a grout mix. In less critical regions surrounding but not directly below the pads, the ground will either be jet grouted with columns more sparsely separated, or it will be tampered. This will improve ground strength sufficiently to carry dispersed forces.

The Calabrian tower will be of a similar construction. However, since the jet grouting will reach the stronger Conglomerate layer (unlike the Sicilian foundation, which sits entirely in Messina Gravel) the foundation pads may be slightly smaller at 50m diameter.

Geotechnical surveys have confirmed that with the ground treatment methods, liquefaction due to seismic action is no longer a concern under the towers.

6. Construction

By far the greatest problem with the construction of the Messina Bridge is not of any single concern, but the whole raft of issues created by the huge scale of the project.

The general construction scheme of events will be quite typical of the many other suspension bridges already constructed. (As previously mentioned, this was an important factor in choosing to build a bridge rather than a tunnel.) This scheme is briefly as follows [17]:
1. Tower and anchorage geotechnical work
2. Construct anchorages
3. Build towers in sections
4. Tie back towers to anchorages
5. Place pilot lines between towers
6. Construct catwalks below pilot lines
7. Tramway installed and cables spun
8. Compress and coat main cables
9. Attach hangers
10. Lift deck sections into place
11. Construct road/rail links concurrently

The first issue with scale is the amount of material required. The 166000 tonnes of cable will absorb 3-4 years of the world’s current cable production capacity. It is likely that additional facilities will have to be created to meet this demand. The demanding construction schedule of 6 years also requires that the main cable is spun in 12 months. This is equivalent to 14000 tonnes per month – by comparison, the Great Belt Bridge’s cables were installed at a rate of 4000 tonnes per month. Here is a major advantage of using four main cables rather than two: the main cables can effectively be installed at twice the speed.

The construction schedule also affects the towers, which demands that their 100 tonnes is fabricated and installed within 24 months. No such innovation exists for this other than to use the tower’s large plan to increase the workforce and equipment on the towers. Up to 4000 workers will be involved with the bridge’s construction in all [16].

7. Conclusion

The background, research, design, development and potential construction and use of a crossing over the Messina Strait has been analysed. A suspension bridge of an unprecedented 3300m main span was selected. Hurdles, despite not being the ideal country to build such an ambitious project. It will set the precedence for other ultra long span bridges, and lessons learnt from it will continue to push the limits of bridge design ever further.

8. References