Full Dynamic Model of Golden Gate Bridge

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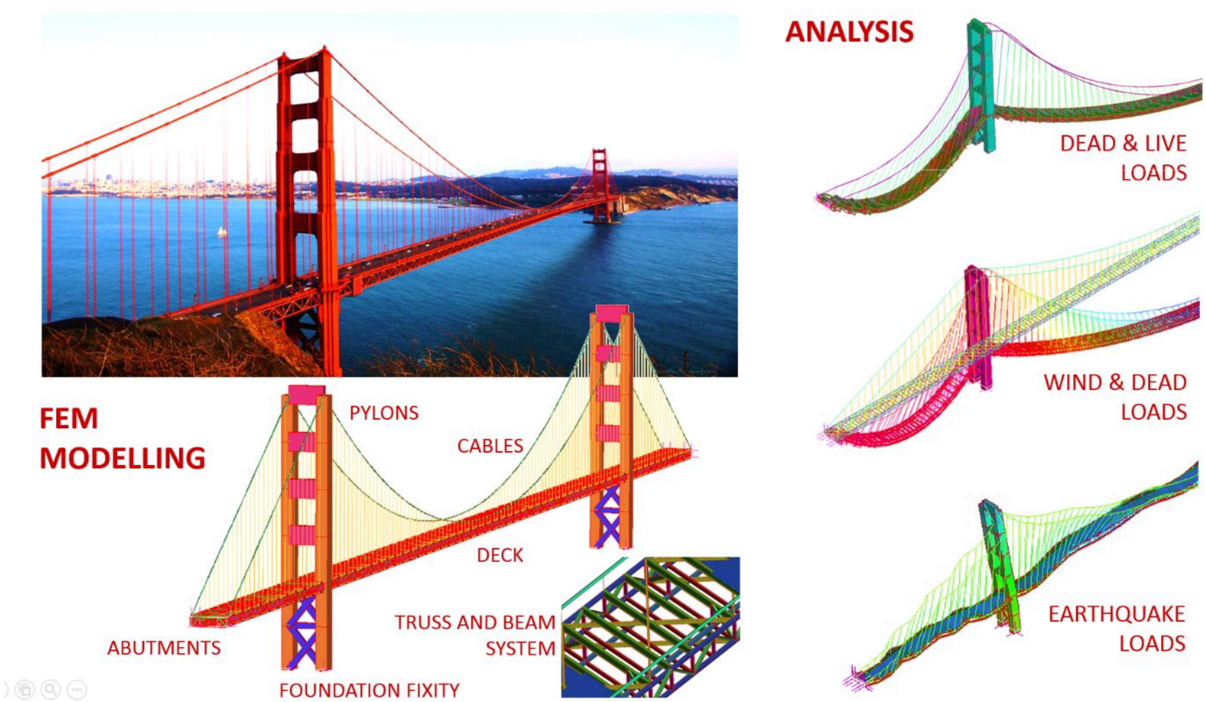
Abstract

An investigation into the structural systems of the Golden Gate Bridge when subject to dead, live, wind and earthquake loading was carried out using finite element modelling. This investigation was carried out using Strand7 and was verified through analytical calculations. This report begins with a study into the structural elements of the actual bridge which includes a summary of the member and section sizes and dimensions. From this study a finite element model was produced. This report outlines the modelling techniques, element types and analysis solvers used in modelling and analysing the structure. This report then considers the member sizes used in the model and outlines any variations in member sizes required for a successful analysis. Finally, this report discusses this results produces by the analysis and verifies the results through simple hand calculations.



INTRODUCTION

The Golden Gate Bridge is a 6 lane suspension bridge that crosses the Golden Gate Channel, connecting the San Francisco Peninsula to Marin County. After its conception in 1916, the bridges construction was a controversial issue, and the final decision to proceed with construction was not passed until 1930. (Golden Gate Bridge Research Library, May 2012) The bridge was opened in 1937 and held the title for being the longest main span suspension bridge for 27 years. (Golden Gate Bridge Research Library, May 2012) The 2332m long catenary cables are the longest bridge cables ever made. These cables used an innovative process to bind thinner wires together to make one large cable which allowed for the construction of the record breaking main span. (Longsworth, L., Loeterman, B., 2004) The bridge crosses the 1.6km wide channel which is well known for having high winds and it is situated adjacent to the San Andreas Fault line, a very active transverse fault. (Golden Gate Bridge Research Library, May 2012) The Golden Gate Bridge is one of the world's most spectacular and well known bridges. Having been declared one of the wonders of the modern world by the American society of Civil engineers, this bridge provided a very interesting case study for this investigation. (Golden Gate Bridge Research Library, May 2012) The analysis focused on the main span of the bridge as it was the most complex part of the bridge, and was a ground breaking structural engineering feat at the time of construction.

TABLE 1. Bridge Details (Highway and Transportation District, 2012)

Location: San Francisco	**Year of Construction Completion:** 1937
Architects: Leon Moisseiff & Irving Morrow	**Approximate Cost**: $35 million
Structural Engineers: Joseph Strauss	**Main Span**: 1280.2m
Function: Motorway bridge / freeway bridge	**Occupied Area**: approximately 2,000 m2
Structure: Suspension Bridge	
Number of Lanes: 6	

STRUCTURAL MEMBERS

The Golden Gate Bridge consists of a bridge deck, supported on a system of beams and trusses. This truss system spans between the bridge pylons and is hung from vertical cables at 15m intervals. These vertical cables are supported by two major catenary suspension cables which pass over the pylons and into anchors at either ends of the bridge. The loads applied to the bridge deck in service are transferred into the deck truss. Majority of this load in the truss is carried by the vertical cables, placing the vertical cables into tension. This tension load is then passed into the main cables which carries the load in tension into the abutment anchors at either end of the bridge. Some portion of the load in the truss is also carried by the pylons. The main function of the deck truss is to transfer loads applied at points not near a vertical cable intersection, into an adjacent cable. There is also a stiffening effect of the entire deck system against deflections. Below is the description of the main members of the structure:

Main Deck: Comprised of 4 major components;

- Concrete Bridge Deck.
- Permanent Corrugated Steel Formwork
- Cross Girders
- Deck Truss System

The deck truss shown in Figure 1 is a complicated system of members, most of which are made up of trusses themselves. The size for which were determined using drawings and scaled images, and recorded in Table 2. This truss system is attached to the Steel Cross Girders, which run perpendicular to the bridge's direction of travel, and sit on top of the truss. These cross girders are Deep UB sections, designed to carry flexure, and transfer load into the truss below. Sitting on top of this is the permanent steel formwork, which works both to distribute the load of the deck onto the cross girders, as well as act as tensile reinforcement for the slab on top.

Catenary Cables: There are two "main" cables which are 2332m in length, each. These are made up of 27572 galvanised steel wires, with a total diameter of 0.92m. These cables have a yield stress of 1100MPa, and a suspended length between the two pylons of 1280m.

Vertical Cables: There are 250 pairs of vertical cables with a diameter of 68.3m and a yield strength of 1100 MPa.

Pylons: 227m tall and constructed from structural steel. 300 MPa yield stress. Base dimensions at footing are 10m x 16m. These are rigidly connected to the deck truss via a riveted plate.

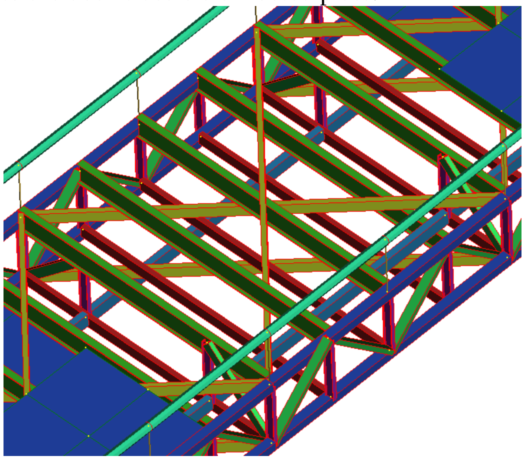


Figure 1. Internal View of Deck Truss

TABLE 2. Structural Element for the Section 1 (Bridge – Projects)

Details of the Structural Elements	Available Structural Element Sizes	Revised Member sizes for Modelling
Green Cross Girder	2500UB3650	2500UB3650
Red Cross Girder	1200x500x50 RHS	2000x2000x200 SHS
Blue Top and Bottom Chords	1400x700x75 RHS	2000x2000x200 SHS
Yellow Cross Bracing	1200x500x50 RHS	1200x500x50 RHS
Green Cross Bracing	500x250x50 RHS	1000x500x50 RHS
Blue Main Girders	1000x1000x100 SHS	1400x700x75SHS
Pink Vertical Members	500UB667	1500UB1590
Vertical Cables	0.126m Diameter	0.3m Diameter
Caternary Cables	0.92m Diameter	1.2m Diameter
Pylons	16000x10000	16000x10000
Pylon Diagonal Bracing	2500x2500	5000x5000
Pylon Cross Bracing	8000x4000	16000x8000
Bridge Deck Slab	0.5m Thick	0.5m Thick

STRUCTURAL SYSTEM

The Structural System Used to Resist Vertical Load

The system used to resist vertical load consists of the following elements:

- The reinforced concrete bridge deck which is loaded directly by traffic
- Cross girder I-beams which directly support the bridge deck
- A truss system which supports both the cross girder beams and bridge deck
- Vertical cables connecting the truss system to the catenary cables
- Two primary catenary cables which pass over the pylons and are anchored at the abutments
- Anchor abutments at either end of the main span which anchor the catenary cables and transfer load to the ground
- Two pylons which support the catenary cables and allow the cables to maintain a catenary shape. The pylons also locally support the bridge truss system at their intersection.
- 4 Cross beams above the deck and 4 diagonal members below the deck connect the pylons and stabilise the vertical structures
- Very deep pile foundations resting on bedrock which support the pylons

Vertical loads applied to the bridge deck are carried by the deck through bending. The deck is a reinforced concrete deck and employs the same design principles as a reinforced concrete slab in a building, i.e. under vertical downward load, the concrete mass in the top of the slab is in compression while the reinforcing steel at the bottom of the slab is in tension. The deck is supported every 5m by cross girders. Thus the deck acts as a continuous slab with one-way action.

The cross girders supporting the deck span across the deck and are deep I-beams with multiple web stiffeners along their length. These cross girders carry load from the deck to the truss system below the deck through bending. The I-beam experiences tension in the bottom flange and compression in the top flange. This beam could be designed using standards such as AS4100. The web stiffeners act to increase the deep beams shear capacity and decrease the chance of shear buckling in the web.

The system of trusses below the deck support the cross girders and the bridge deck at the deck edges. This system of trusses is actually made up of smaller trusses. Therefore, in modelling the bridge, the trusses were modelled as rectangular hollow sections to simplify the design. Figure 2 shows the complex truss system below the bridge deck and the simplified truss system used in modelling the bridge. These trusses carry the load from the cross girders and bridge deck and spread it along the main longitudinal truss. This main longitudinal truss spans between the vertical cable supports and carries the load through bending. This bending is carried within the truss by compression in the top chord, tension in the bottom chord and axial forces in the diagonal members.

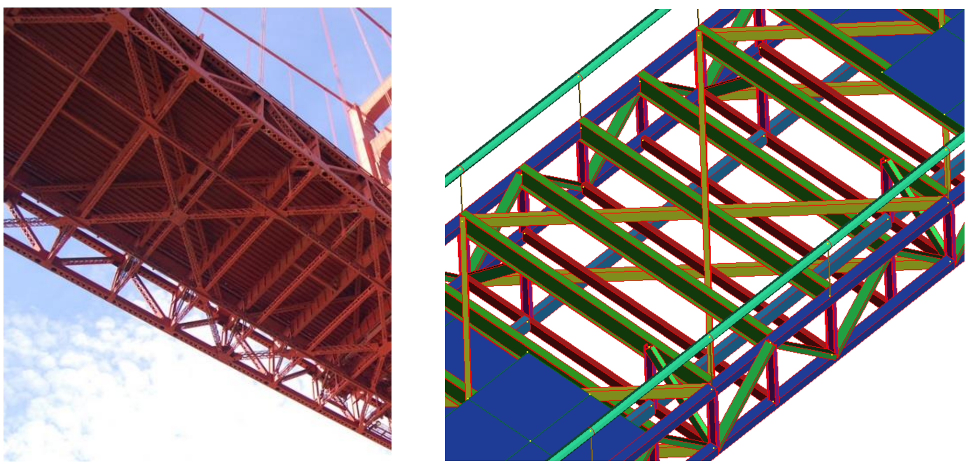


Figure 2: Truss system below bridge deck in reality and simplified for Strand7 model.

The vertical cables connected to the bridge truss every 15m pick up the load carried through the longitudinal truss. These cables carry this load from the bridge deck up to the catenary cable through tension. These vertical cables are able to carry load according to the cable cross sectional area and the tensile strength of the steel.

The catenary cables pick up the tensile forces in the vertical cables and carry the force up over the pylons and down to the anchors at either end of the bridge. The catenary cable acts in pure tension due to its catenary shape and is able to carry tensile load according to the cable cross sectional area and the tensile strength of the steel. The catenary cable is in fact made up of 27,572 galvanised steel cables which are grouped into 61 cable groups which are then bunched together to form the 0.92m diameter cable. These main cables are anchored at the abutments to keep them in tension and to pass the tensile load into the ground through the abutments.

The anchors at the abutments receive the load from the catenary cables and transfer the tensile force into the ground. The numerous cable bunches which make up the main cable are set into the concrete anchor blocks in a splayed out pattern to distribute the load throughout the anchor. The anchors resist the pulling tension in the anchors through their own self weight and through friction as they are embedded into the ground. The anchor blocks weigh approximately 54,400,000kg and contain approximately 4,400 tons of steel reinforcement (Golden Gate Bridge Research Library, May 2012).

The pylons support the catenary cables and locally support the bridge deck. The pylons are loaded in compression by the catenary cables as they keep the cables' catenary shape and carry part of the vertical component of the tensile force in the cable. The pylon is further placed into compression as it carries load from the deck itself at the pylon deck intersection. Figure 3 shows this intersection and an estimation of the transferal of load from the deck and truss to the pylon.

Figure 3: Connection of deck and truss system to pylon

The pylons rest upon deep foundations which lie on bedrock below. The northern pylon rests on solid diabase or basalt rock which has a high bearing strength and is able to easily dissipate the load down into the ground. This northern foundation uses wall friction around the foundation periphery as well as end bearing of the foundation on the bedrock to dissipate load into the ground. (Sedgwick, A. E., 1931, February). The southern pylon rests on serpentine rock which consists of sedimentary formations of sandstone and clay. This southern foundation has a reduced bearing strength and is designed to dissipate load to the sounding ground through wall friction alone. (Sedgwick, A. E., 1931, February). As a result, this southern tower is embedded 34m deep. (Golden Gate Bridge Research Library, May 2012).

The Structural System Used to Resist Lateral Load

The major lateral loads acting on the bridge are wind and earthquake loads, although the action of earthquake loads are significantly more pertinent due to the bridge's proximity to the San Andreas Fault. Therefore, this section of the report will focus on the bridges capacity to resist lateral loads from earthquakes. After a 7.1 magnitude earthquake in 1989 the bridge has undergone a series of retrofitting for seismic activity. (Overview of Golden Gate Bridge Seismic Retrofit Construction Project, 2013, February) this process began with the retrofitting of the northern and southern approach spans, while the retrofitting of the main span remains incomplete. Therefore, in analysing the bridges capacity to resist earthquake loads, we will consider the bridge as it was at the time of the 1989 earthquake which the bridge withstood successfully.

Earthquakes act on the bridge through the foundations of the two pylons and the anchorages. (Ichiro Konishi & Yoshikazu Yamada, 1960) According to Ichiro Konishi & Yoshikazu Yamada (1960), the connection between the towers and bridge deck and the towers and cables are not a significant concern when considering the movement of the bridge under earthquake loads. It is suggested that the movement and subsequent forces acting in the bridge towers or pylons are of primary concern. These loads are primarily dependent on the dimensions and configuration of the pylons and the foundations on which they rest. This response of the bridge is dependent on the inertia and stiffness of the pylon system as well as the conditions and stiffness of the foundation system. (Ichiro Konishi & Yoshikazu Yamada, 1960). Therefore the primary system used to resist lateral load consists of the following:

- The two pylons which support the bridge deck and rest on the bedrock below. These towers are braced with 4 Cross beams above the deck and 4 diagonal members below the deck. These cross members connect the pylons and provide lateral stability. These cross members will act in tension and compression as the earthquake pushes and pulls the pylons on either side of the bridge deck
- The deep foundations resting on bedrock below the river, supporting the pylons acts of resist lateral load as they dissipate the lateral movement of the pylons back into the ground after earthquake has stopped.

LOADS

Dead Load

The dead loads acting on the structure were based entirely on self-weight as there are no significant loads that act on the structure in addition to its self-weight. The Strand7 model included the members' sizes and the material densities and thus the combination of the two allowed the mass of the structure to be determined. The input of gravitational acceleration allowed Strand7 to work out the dead loads on the structure and they were included in the analysis by specifying the acceleration due to gravity.

Live Loads

The live load applied to the structure was in accordance with AS5100.2 SM1600 Loading. An extract of the loading has been included in Figure 4.

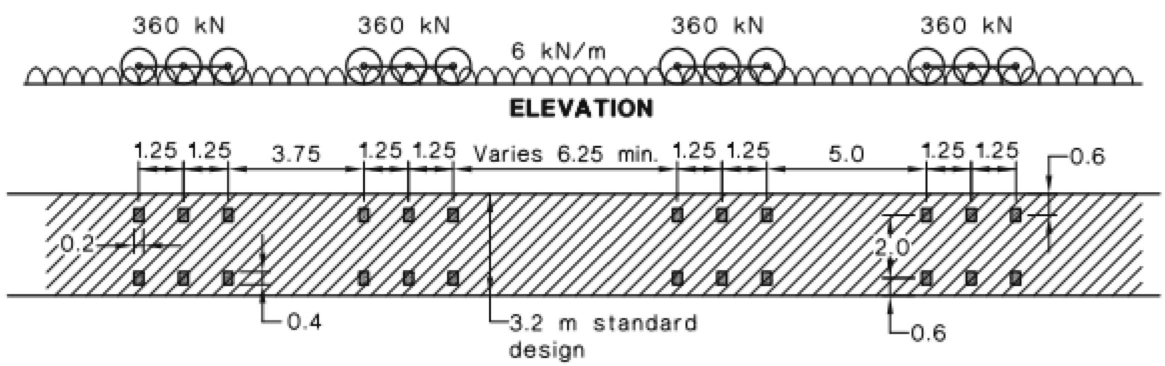


Figure 4: M1600 Traffic Loading AS5100.2

To simplify the computation, the live load was applied as a pressure load. The total load specified by AS5100.2 was added together and averaged as a pressure load over the road deck. Since there are 6 lanes of traffic this load was multiplied by six. Live loading is applied for the worst case. In light of this the bridge was assumed to be fully loaded with the worst case truck as specified in the above standard. On a global level this simplification provides almost no significant change in the results determined. Had there been more time allocated to the task and more computing power available the live loading would be calculated using the load path tool. The load path tool uses the loading taken from AS5110.2 and applies it over a lane. The load influencer solver is then used to work out the critical position for this loading case. This is performed by selecting primary members within the structure and calculating the worst case position for the traffic loading with respect to these elements. Once the load path and load influence tool have been used a position is determined with translates into a static load applied to the structure.

Static Wind Loads

The design wind speeds were determined by analysing wind speed data from the National Oceanic and Atmospheric Administration's National Climatic Data Centre (NCDC). The data gathered from the bridge as it exist today as well as the data gathered from a buoy just west of the bridge in the middle of the Golden Gate were analysed. The design wind speeds for the structure were also determined in accordance with the American Society of Civil Engineers, Minimum Design Loads for Buildings and Other Structures (ASCE 7-10) standard for a risk category IV structure. The two design wind speeds were compared and it was found that the ASCE 7-10 design wind speed matched well with the 500 year design wind speed derived from NCDC data. The ASCE 7-10 design wind speed was 115 mph (51.4 m/s). The design wind speed was multiplied by the width of the members and applied as a UDL to plate and beam members. For truss members the tributary lengths of the beams were found and an equivalent point load was applied to the nodes at the end of each truss member. Only lateral wind loads were considered from the westerly direction since it is the most critical. The westerly direction is the most exposed to winds coming in from the Pacific Ocean.

Dynamic Earthquake Loads

The bridge is located in a very active seismic region and thus it is important to use location specific earthquake spectra. The design spectrum of the applied earthquake was derived from the 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design for 37.815°N, 122.48°W. The soil of the site was identified as Site Class B – "Rock" and the highest importance category, IV was assigned to the structure. The spectrum for the site was derived from United States Geological Survey (USGS) data, the USGS design report may be found in the Appendices. The seismic load spectrum was applied to the structure as a base acceleration from the X-direction. That is, it was assumed that the earthquake ground accelerations would arrive from the East which is accurate given that the San Andreas runs North-South and is east of the bridge.

Dynamic Wind Loads

Wind loads present a significant risk to the structure given its long span and its exposure to relatively strong winds. Dynamic wind loads are a function of the aerodynamic interaction between the structure and the flow of air past the various structural elements. However, whilst an analysis of sufficiently detailed wind data (not available for the Golden Gate Bridge) can be used to determine the turbulence buffeting loads on the structure, the dynamic wind effects caused by the interaction between wind flow and the structure such as flutter, wake excitation etc. cannot be determined without wind tunnel testing or computational fluid dynamic modelling. Such loads are also beyond the scope of bridge and wind design codes which recommend "specialist advice" be sought. As a result of the lack of information in this regard, dynamic wind loads were not modelled in this analysis. The authors acknowledge that the wind loads are significant, but have chosen not to model these loads due to a lack of sufficient data

NUMERICAL ANALYSIS

Mesh and Mesh Quality

Plate elements were used to model the bridge deck while truss and beam elements were used to model the remainder of the bridge. Only the plate elements within the model required meshing as truss and beam elements do not require meshing. The mesh refinement of the bridge deck was governed by the node positions of the truss below the bridge deck as nodes of the deck had to align with the nodes of the truss supporting the deck. This resulted in relatively coarse mesh, however this did not impact on the analysis as the deck was not a critical component to be analysed in the model.

Element Types

Bridge Deck

The bridge deck was be modelled as a plate elements which had the properties of concrete. It was deemed that these plate elements would provide a suitable approximation of the actual reinforced concrete bridge deck system. The cross girders supporting the deck were modelled as beam elements. In reality these cross girders are I-beams with numerous web stiffeners along their length and as such may have be more accurately modelled using plate elements. However, it was determined that this modelling would not provide any additional helpful information on the bridge and would only serve to overcomplicate model. Therefore, modelling the cross girders as beam elements provided a sufficient level of accuracy.

Cross Girders

The cross girders are supported by a system of primary trusses, which in turn are made from smaller secondary trusses. The primary truss system was modelled using B2 beam elements with beam properties, while the secondary trusses which made up this primary truss were simplified to be square hollow sections (SHS) in the model. The dimensions of the rectangular hollow sections (RHS) used in the model were determined by finding a RHS section which had a similar second moment of area to the estimated second moment of area value of the truss members. It was assumed that this approximation would provide a sufficient representation of the real structure.

Pylons

The pylons and cross beams connecting the pylons were modelled using B2 beam elements.

Catenary and Vertical Cables

The Catenary cable was modelled using truss elements which spanned between nodes with coordinates that mimicked the catenary shape of the cable. These node coordinates were determined via both a theoretical analysis using catenary curve equations and through Strand7 modelling. Strand7 contains a function to determine the deflected shape of cables using B2 cable elements. The catenary cable was modelled separate to the main structure and by assigning the appropriate Cable Free Length the deflected shape of the cable was easily found. The node locations of the deflected cable was exported to a spreadsheet and the points were compared to the theoretical values. It was found that the theoretical values derived from the theoretical equations underestimated the amount of sag by a small amount. This was to be expected given that material strain is not taken into consideration by the catenary equations. The nodes were then imported into the final Strand7 model for the entire bridge. The nodes were then joined by B2 Truss elements.

The catenary was modelled with truss elements as this allowed for a string groups to be used at the point where the cables pass over the pylons. It should be noted that string groups can only be used with truss elements. This use of a string group was an important feature of the model as the string group allowed the cable to act as if it could slide over the top of the pylon. The sting group forces the tension to stay constant over the truss elements. This effect emulated the realistic cable attachment as found on the Golden Gate Bridge and ensured that the pylons where not excessively loaded by the catenary cables. The string group and trusses elements on either side of the pylon achieved this sliding effect by allowing the truss elements to lengthen and shorten concurrently on either side of the pylon. Truss elements were also used since Strand7 concentrates the mass of cable elements at the nodes when running an analysis whilst truss elements allow the mass to be distributed along the length of the cable.

The vertical cables were similarly modelled using truss elements, under static load cases in which they were under no compressive loads. However, under the earthquake loading scenario, cut-off bars with no compressive capacity were used since these elements may come under compressive loads under the dynamic action.

Material Properties

Concrete components were given the properties of AS3600 32MPa concrete. All steel elements were given the properties of structural steel. Young’s moduli were not altered, however the yield strengths were. The yield strength of all cable elements were made to be 1100MPa, as previously mentioned. All other steel members were taken to have a yield strength of 300MPa, typical of the era.

Geometric

The geometric properties of all of the sections and members were measured from scaled photographs of the bridge. These geometric properties were then used to model the members themselves. The actual section dimensions were either replicated in the model, or, where the member configuration was very complex, the actual dimensions were used to find the dimensions of an equivalent member which was then used in the model to provide a simplification.

Nodal Constraints

The fixity at the base of the pylons was completely fixed in all directions of translation and rotation that is DX, DY , DZ as well as MX, MY, MZ were all fixed. This is similar to the real structure which has very deep foundations that do not allow any movement.

The fixity at the end of the bridge span, i.e. at the anchors, was the same as at the base of the pylons - DX, DY, DZ as well as MX, MY, MZ were all fixed. Again given that these are giant anchor blocks designed to prevent any movement it was sensibly assumed that these nodes would be fixed.

Solvers Used in the Analysis

Different solvers were used to analyse the bridge under differing load conditions. The stresses in the members of the bridge and deflections of the bridge under dead, live and wind loads were all found using a **Non-linear Static Solver with the non-linear geometry option**. This solver was used as it allows for the cables of the bridge to lengthen/move and increase in stiffness progressively as the tensile forces in the cables develop as the load is applied. A linear static solver was not used because it would produce skewed results as the linear static solver would not allow the cables to become fully loaded as they are in reality. This would result in incorrect member stresses and the deflections of the bridge would greatly amplified. Non-linear materials were not used. Member sizes were adjusted to ensure that the stresses in the members were below the yield stress of steel and thus the need for non-linear materials were eliminated.

To determine the natural frequencies of the bridge a **Natural Frequency Solver** was used. In order to run this natural frequency solver the model was input with the mass of the structural elements and the acceleration due to gravity. This solver was required to solve up to 250 natural modes of the structure in order to get sufficient mass participation of the bridge. This large number of modes was required because the structure is so large and heavy.

Once the natural frequency modes of the structure were found, a **Spectral Response Solver** was run to determine the mass participation of the bridge and the deflection of the bridge when subject to an earthquake spectra. This analysis required the input of the earthquake spectra and scaling factors which were established using the American Association of State Highway and Transportation Officials (AASTO) seismic code.

STRUCTURAL DESIGN

In Designing Structural Members for the Golden Gate bridge, the capacity of the members used to model the bridge have been checked against the loadings found to be acting on each respective member, given from the results file outputted by Strand7. The most critical area for each member has been identified as the area experiencing the most stress, moment or deflection (depending on what is being considered), and all members have been designed for this case. Prior to comparing a section's capacity to the max loading experienced, the sections yield stress has been factored by 90%, and the reduced strength has been used for redesign.

Cable Design

Design of Cable Members has been carried out by finding the maximum stress in each element type, and looking at the response of the selected member under these stresses. Members were then redesigned if they were found to be insufficient to carry the applied loading. The maximum stresses were considered by looking at the highest stress values for an element under all loading conditions, and then considering the max of these.

TABLE 3. Redesigned Cable Sizes

Member Type	Member Dimensions	Maximum Stress.	Redesigned Dimensions
Vertical Cables	0.3m Diameter	3.28×10^5 kPa.	0.172m Diameter
Main Cables	1.2m Diameter	1.14×10^6 kPa.	1.288m Diameter

Since the Cable elements are in Tension, the design of these is simply based on their tension yield capacity. As aforementioned the grade of steel used in the cables is 1100MPa. The maximum stress was divided by the area used in the model to find the maximum tensile loading on the element. This load was then used in conjunction with the yield stress to determine the required cross sectional area of the Cable.

Vertical Cables;

$$3.28 \times 10^5 \times \left(\pi \times \frac{0.3^2}{4}\right) = 23184.96\ kN$$
$$N^* = \varphi \times f_y \times A$$
$$A = 23184.96 \div (0.9 \times 1100000)$$
$$A = 0.0234\ m^2 = \pi \times \frac{D^2}{4}$$
$$D = \sqrt{4 \times \frac{0.0234}{\pi}} = 0.172m$$

Main Cables;

$$1.14 \times 10^6 \times \left(\pi \times \frac{1.2^2}{4}\right) = 1289309.6\ kN$$
$$N^* = \varphi \times f_y \times A$$
$$A = 1289309.6 \div (0.9 \times 1100000)$$
$$A = 1.302\ m^2 = \pi \times \frac{D^2}{4}$$

$$D = \sqrt{4 \times \frac{1.302}{\pi}} = 1.288m$$

After redesign of the members, the new dimensions indicate failure of the modelled main cable, but sufficient strength in the modelled vertical cables. It should be noted however that both of these dimensions are larger than that calculated to be on the structure in reality. The main cable has a diameter of 0.9m, and the verticals have a diameter of 0.126m, both of which prove to be insufficient in the loading applied by this model.

For elements governed by their flexural strength, the maximum bending moment was considered instead of the axial stress. These were the Pylons and part of the deck system. For the deck system, all members were considered separately and the maximum moment was considered in the members deemed to be experiencing flexural loading. Given the system is a truss, each member was also considered on the axial force acting through it. Similar calculations to above were carried out in the below spreadsheet.

Flexure was considered by looking at the maximum bending moment, and comparing that to a design bending moment based on a sections yield stress and section modulus. A sample hand calculation has been performed for the pylons, but was encoded into the spreadsheet below.

TABLE 4. Redesign of Truss Members based on Capacity

Truss Members										
Geometry					Loading	Capacity				% Cap
Member	h	b	t	A	Max Load	F_y	φ	Factored Strength	A_{yield}	
Bot Cross Girder	2	2	0.2	**1.44**	24000	300000	0.9	270000	**0.0889**	**6%**
Top/ Bot Chords	2	2	0.2	**1.44**	288000	300000	0.9	270000	**1.0667**	**74%**
Diag Top/Bot Cross Bracing	1.2	0.5	0.05	**0.16**	15200	300000	0.9	270000	**0.0563**	**35%**
Side Cross Bracing	1	0.5	0.05	**0.14**	57000	300000	0.9	270000	**0.2111**	**151%**
Blue longitudinal	1.4	0.7	0.075	**0.293**	36400	300000	0.9	270000	**0.1348**	**46%**
Vert Side Bracing	1500UB1590			**0.203**	114000	300000	0.9	270000	**0.4222**	**209%**
Flexural Members										
Geometry					Loading	Capacity				
Member	I	y	A	Z	Max BM	F_y	φ	Factored Strength	Z_{yield}	
Top Cross Girder	0.3277	1.25	0.465	**0.2621**	19000	300000	0.9	270000	**0.0704**	**27%**
Top/ Bot Chords	0.7872	1	1.44	**0.7872**	738800	300000	0.9	270000	**2.7363**	**348%**
Bot Cross Girder	0.0276	0.6	0.16	**0.0460**	1422	300000	0.9	270000	**0.0053**	**11%**

Table 4 shows the geometric properties, as entered into the model, as well as the calculated required capacity of the section to withstand yield under maximum loading. Looking at the required capacity returns a new value for the geometric property considered. This property is then compared with that inputted into the model. In the value for the % Capacity column, it can be seen which members have sufficient capacity and which do not. For Values <100%, the proportion given describes the proportion of the area used in modelling, actually required to resist the applied loading. For sections with % cap >100%, the section has yielded and so an increase in section size is required. For the Truss members, this means scaling the area by the proportion given. For the Flexural Members a new member should be selected such that the section modulus is sufficient to withstand applied moments.

Pylon Design

The pylons were initially modelled at solid beams 16m by 10m. It is assumed that the bridge pylons are actually hollow steel members, but due to a lack of technical drawings of the structure the above solid dimensions have been used in the model. To check the validity of this assumption the following simple calculations have been conducted to quantify the area necessary to withstand loading.

The first case that was considered was bending. An I value for the section was determined at the stress at the outermost fibre determined.

Bending

$$I = \frac{bh^3}{12}$$

$$I = \frac{10000 * 16000^3}{12} = 3.4133 * 10^{15}\ mm^4$$

$$M_{Max} = 6.22 * 10^6 kNm = 6.22 * 10^{12} Nmm$$

$$f = \frac{My}{I} = \frac{6.22 * 10^{12} * 8000}{3.413 * 10^{15}} = 14.6MPa$$

Compression

Following the flexural calculation axial compression was considered in the member. The maximum design stress was determined from the Strand7 model.

$$N^* = 5.08 * 10^6 kN$$

$$f_y = 300MPa$$

It is assumed that given the sheer size of the pylon that it does buckle and therefore has complete section capacity.

$$\phi N_c = 0.9 * 300 * 16000 * 10000 = 43.2 * 10^6 kN$$

$$\therefore Sufficent\ Axial\ Capacity$$

Redesign

$$A_{new} = \frac{5.08 * 10^6}{0.9 * 300} = 18814814mm^2$$

Proposed Dimensions 7000mm x 2687mm

$$f = \frac{6.22 * 10^{12} * 3500}{\frac{2687 * 7000^3}{12}} = 283MPa$$

$$\therefore The\ new\ proposed\ dimension\ has\ sufficient\ capacity$$

Considering both axial compression and flexure it was found that axial compression induced the most critical stress in the pylon. It was also determined that the area of the pylon could be reduced by approximately 88% and still satisfy a stress lower than the yield stress of 300MPa. This confirms the assumption that the pylon is in fact hollow.

RESULTS

Vertical Deflection along bridge Centreline

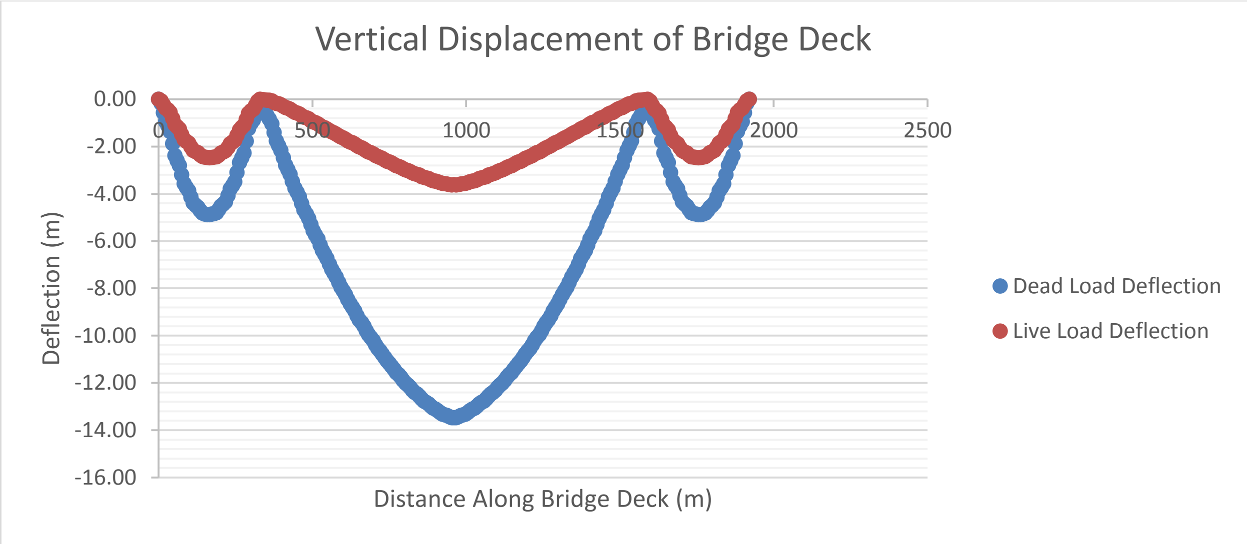


Figure 5: Vertical Deflection along bridge centreline

The bridge was initially constructed with a negative concavity or camber. This upward camber was built into the bridge such that the deflection of the deck under its own dead load would bring the bridge to a relatively 'level' or 'horizontal' position. The bridge camber during construction is shown in the photograph in Figure 6. The model was not able to account for this pre cambering of the bridge deck because the complex nature of the cambered construction meant that modelling it would have been unrealistic. Therefore, it has been assumed that the deflections due to dead load, as shown in figure 5, will have been taken up in the pre camber of the bridge and would act to return the bridge to a neutral flat position. Therefore the dead loads need not be considered when looking at the serviceability deflections of the bridge under serviceability loads.

Having accounted for the dead load by assuming it will return the cambered bridge to a neutral horizontal position, only the live load and wind load need to be considered when looking at the serviceability deflections. The serviceability load case for live load applied to the bridge was 1.0 time live load. Under this serviceability live loading the largest vertical deflections were 3.6m at the midspan of the bridge. This deflection under the live serviceability load corresponds to the maximum deflections expected for the actual golden gate bridge structure which is estimated to be 3.3m (Golden Gate Bridge Research Library, May 2012). We conclude from this that the deflection under live load falls within the serviceable limits.

The vertical deflection under wind load has not been included in the graph because the vertical deflection due to wind is comparable to the deflection under live and dead load alone. This is because of the large restoring or inertial action of the bridge dead load itself which acts against the wind load and the deflections due to wind. Therefore, it is concluded that the deflection due to wind do not governing the serviceability requirements of the bridge.

Figure 6: Golden Gate Bridge under construction showing

Lateral Deflection along bridge Centreline

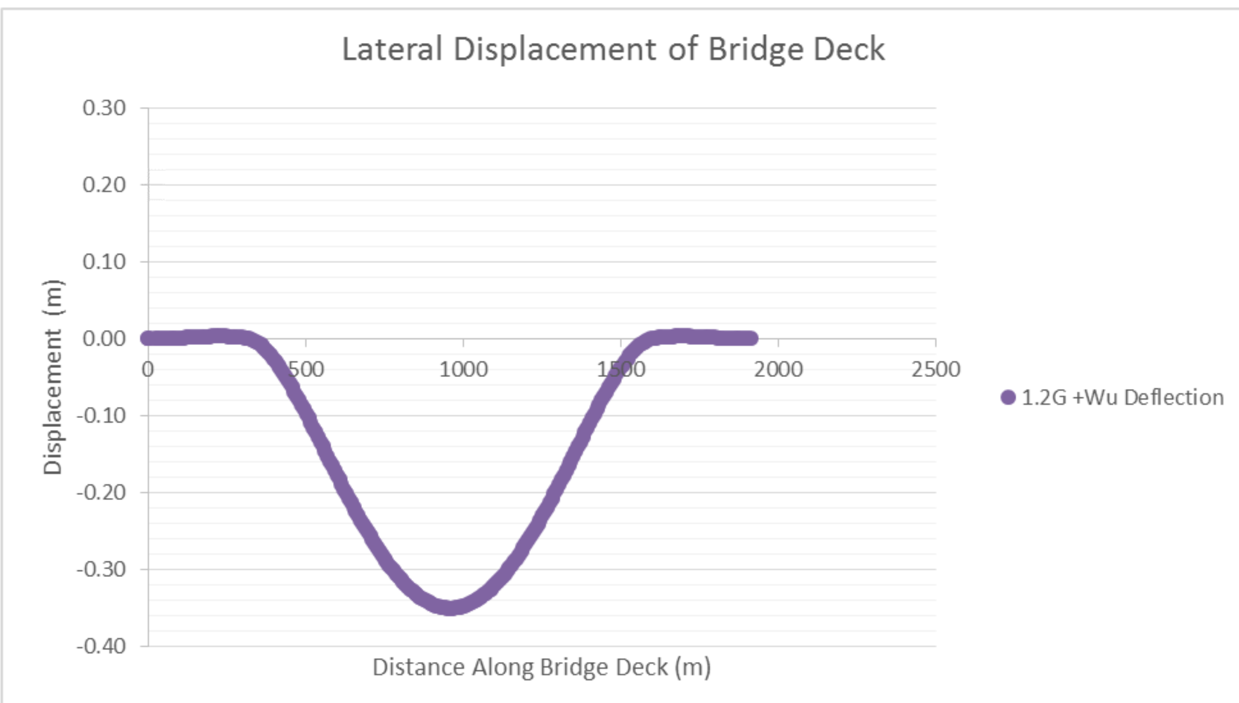


Figure 7: Lateral Displacement of Bridge Deck

Limited deflections were observed laterally in the bridge deck. A maximum displacement of 0.35m was observed under the load combination including wind actions. This deflection may be small due to the solver used. The load combination including wind action was run with the non-linear static solver, and this doesn't enable loadings to be run excluding the gravity load. As such, the wind was not able to be considered independently. As a result of this, the bridges own weight applies a restorative moment to that felt by the wind in the lateral loading. The bridge's inertia stops the bridge from deflecting as much as would have been observed if this load had been considered independently. Since the bridge's weight is obviously effective in reality, the situation emulated is in nature, more accurate then considering wind alone, with the downside being wind's effects not being able to be accurately quantified as its own entity.

Vertical Deflection and Longitudinal Stresses at the Bridge Midspan

As expected when compared to the main span the differential deflection transversely across the bridge deck is minimal. Most notably the deflection graph for wind load involves a twisting of the bridge deck as seen in Figure 8. As restraint from the cables is only applied one way in the vertical direction the bridge twists under the load. This twist is further accentuated given that the graph is taken at the bridge midspan far away from the support provided from the pylons. Twisting is observed to occur about the centre node of the bridge deck given that this node undergoes minimal displacement between the dead and wind load case.

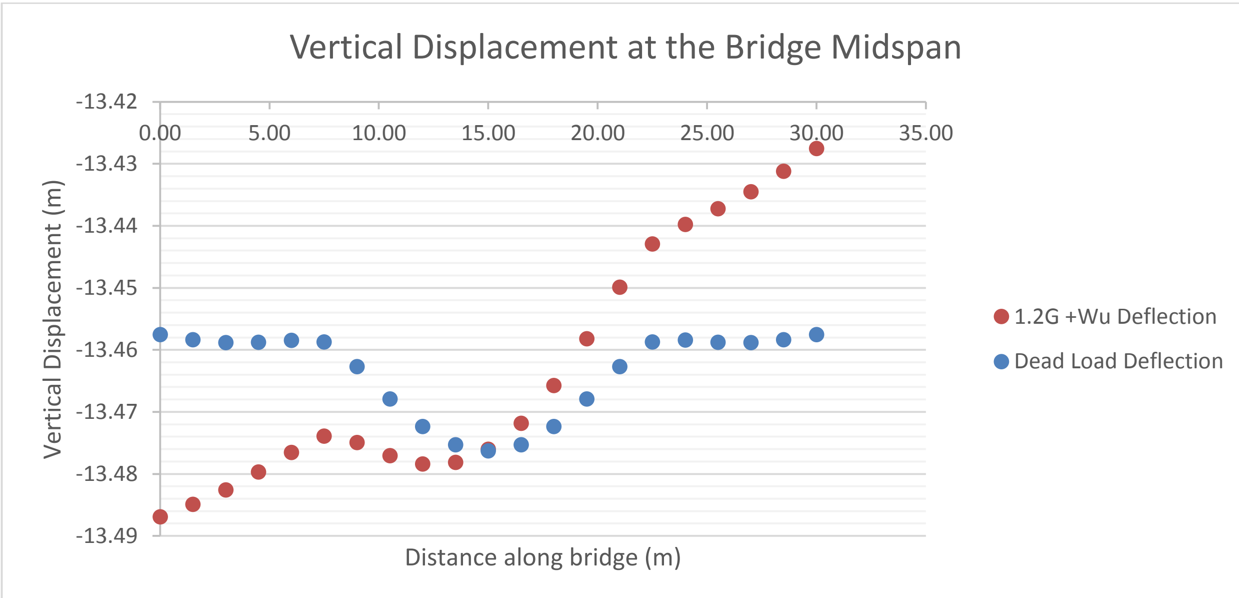


Figure 8: Vertical Displacement at the Bridge Midspan under Dead and Wind Load

Longitudinal Stresses along the Bridge Centreline

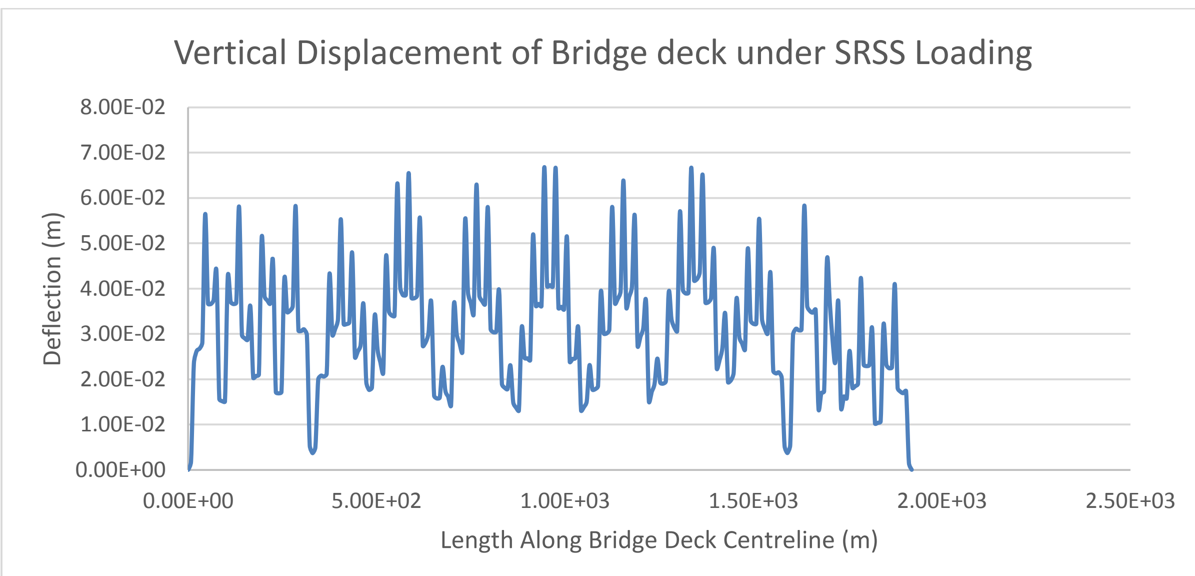


Figure 9: Vertical Displacement of Bridge deck under SRSS Loading

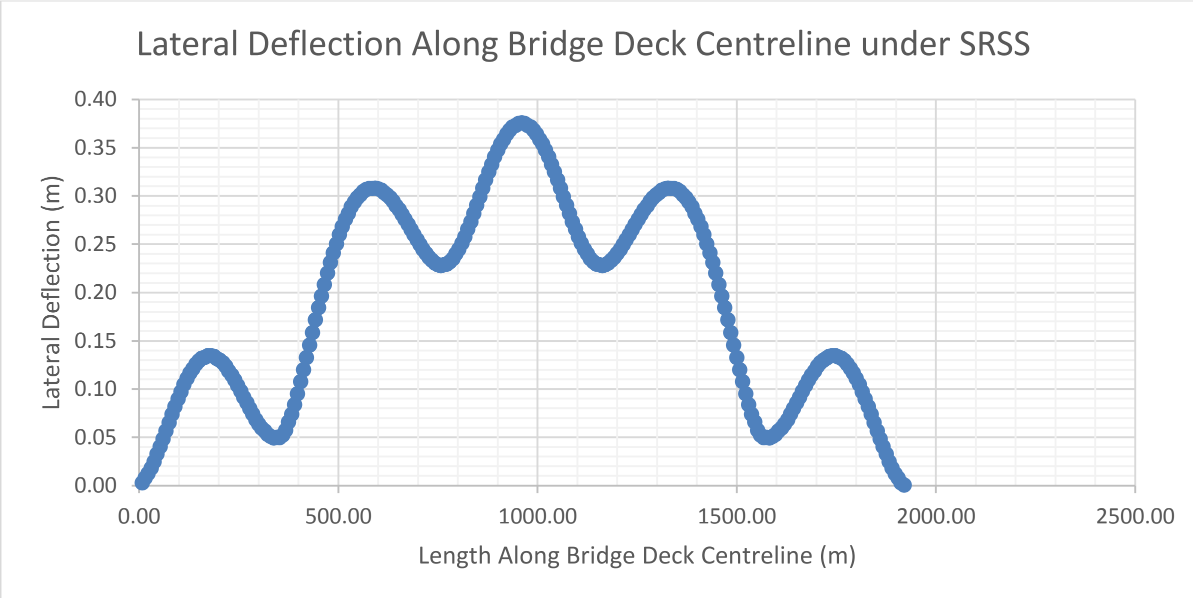


Figure 10: Lateral Deflection along Bridge Deck Centreline under SRSS

Only a small amount of deflections were observed under the earthquake case, both vertically and laterally which can been seen in Figures 9 & 10. Due to the very low natural frequency of the structure (given its substantial size and weight), compared with the very high natural frequency of the applied earthquake, very minimal engagement is observed. Many modes had to be considered to get any meaningful mass participation, to support this phenomenon. Observing the graphs above it can be seen that the deflection laterally in the bridge deck under earthquake loading are of a higher magnitude that the vertical deflections.

Validation of stresses in elements found using Stand7 through hand calculations

With the aim to rationalise the results produced by Strand7 and to verify the validity of the model a series of hand calculations have been provided as below. The various members considered in this section will be considered under the 1.2G+1.8Q load combination as this was the governing load combination.

Cross Girders

Forces acting in a cross girder due to the load combination 1.2G+1.8Q
The cross girders are assumed to be simply supported by the truss system and spanning 30m

$$G = Selfweight\ of\ concrete\ deck = 1.1 \times 0.5m \times 25kN/m^3 = 15kPa$$
$$Q = Traffic\ load\ accoridng\ to\ SM1600 = \ 1.8 \times 10.16kPa = 18.3$$
$$Tributary\ width\ of\ cross\ girder = 7.5m$$

$$UDL\ acting\ on\ cross\ girder = 7.5 \times (15 + 18.3) = 249.75\frac{kN}{m}$$

$$M^* = design\ bending\ moment = \frac{249.75 \times 30^2}{8} = 28{,}097\ kNm$$

This design bending moment found through simple hand calculation is of a similar magnitude to that which was determined from Strand7, 19000kNm. The difference observed is likely due to the differing support condition used in the model and in the hand calculations. The hand calculations assume that the cross girder is simply supported and thus there is not moment at the connections. In comparison in the model moment is carried through the connections to the vertical members. The transfer of moment through the connection lowers the maximum moment in the section.

Vertical Cables

Load acting on Vertical Cable due to Load combination 1.2G+1.8Q

N* = 1.2 x Concrete Deck Self Weight

$$selfweight\ of\ concrete\ deck = 0.5m \times 25kN/m^3 \times 1.2 = 15kPa$$

TABLE 5. Self-weight of Structural Members

Element	Cross Sectional Area (mm2)	Length (mm)	Volume (mm3)	Unit Weight (kg/m)	No.	Weight (kN)
Green Cross Girder		30000		3650	3	3222.585
Red Cross Girder	760000	30000	22800000000		3	1755.7938
Blue Top and Bottom Chords	760000	15000	11400000000		4	877.8969
Yellow Cross Bracing	82500	42426	3500145000		1	269.5409162
Green Cross Bracing	27500	10607	291692500		2	22.46280189
Blue Main Girders	151875	15000	2278125000		2	175.4349891
Pink Vertical Members		7500		1590	2	23.85
					Weight	6347.564407

Pressure load is equal to:

$$\frac{6347.56}{30 * 30} = 7.05kPa$$

$$Tributary\ width\ of\ vertical\ cables = 15m$$

As calculated traffic load is equal to:

Q = 10.416kPa

$$N * (self\ weight\ of\ concrete\ deck + 1.1 * Selfweight\ of\ Truss\ system) \times 30 * 15 + 1.8 * Q * 15 * 30$$

$$N = (15 + 1.1 * 7.06) \times 30 * 15 + 1.8 * 10.416 * 30 * 15 = \ 18682kN$$

This is then divided by two as there is one cable on each side of the bridge.

$$N^* \ = 5131.72 * 0.9 = 8407kN\ per\ cable$$

The actual load felt by the vertical cables modelled in Strand7 is around 23000kN. The difference is likely to be caused by the distribution of load into each of the cables. Under a non- linear analysis the deflected shape changes the stresses felt by each element.

CONCLUSIONS

In summary, the Golden Gate Bridge has been modelled to emulate the various loading conditions imposed on it in reality. The bridge's weight is evidently extremely substantial, given its large size and span. For a bridge of this magnitude to span the distance it does, massive structural members need to be utilised, which then add to the structures weight. Even when compared with scaled up, worst case traffic loads, design wind loads and dynamic earthquake loading, the self-weight of the structure alone was the biggest contributing factor to stresses and deflections observed in the bridge.

The initial model using the structures real dimensions was insufficient to carry the loadings applied. Therefore, every member was increased in size to allow the model to run. Ultimately, this lead to the review of structural member sizing, by checking them against the maximum stresses and moment applied, and suggesting their geometrical properties for efficient and sufficient capacity against yield. Some members were found to be loaded at 3.5 times their yield stress, while others as little as 5%. This suggests some inconsistencies in the way loads are transferred through our model – either in the modelling phase (and member sizes) or the way the bridge has been loaded.

Due to simplifications made, as well as assumptions about the process of construction, some of the data for stress and deflection does not accurately represent the magnitude of that felt by the structure in reality. However, the trends observed, and critical sections identified do appropriately match what is to be expected in reality. In future, a dynamic analysis of the wind loading would greatly improve the confidence had in the results, given that gusts are a large issue in the geographic location, however due to the complex nature of this analysis they have been neglected by this modelling exercise.

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