Composite construction in cable-stayed bridge towers

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Summary

This paper presents an application of composite construction to solve the problem of anchoring the cables to concrete towers in cable-stayed bridges. The use of steel saddles made out of perforated plates is proposed as a compact, economical and fatigue resistant solution to this problem. Design and analysis of such a solution is presented as applied to a cable-stayed bridge with a 203 m long central span.

1. Introduction

A well known design problem in cable-stayed bridges consists in anchoring the cables in the tower. As concrete towers are usually the most economic alternative to transmit predominantly axial compression loads to the foundations, many problems arise to transmit horizontal cable force components from front stays to backstays. Moreover, in moderate spans it is difficult to fit both anchorages inside the tower. Several solutions have been proposed to this problem [1,2] although many of them are not fully satisfactory.

Basically the available alternatives consist either in overlapping anchorages (which may lead to awkward arrangements of tubes, anchorages and cables) or designing an internal gallery inside the tower (which means complicated prestressing arrangements as well as important dimensional constraints both for the size of the tower and to manage the jacks) or, finally, anchoring the cables in steel elements which may adopt very different shapes and sizes [2-5].

The solution which is presented here consists in the use of steel saddles which are embedded in tower concrete and which support both anchorages. Adherence between the saddles and concrete is obtained by means of perforated plates. This solution allows a very slender design of the towers as well as a quick positionning of the saddles (thus avoiding usually lengthy positionning of stay tubes).

The design of the saddles has to take into account both their intrinsic resisting properties and the force transmission to concrete. The rationale of the design as well as the analysis of such elements is presented in this paper to show that this is a very valuable alternative for anchoring cable stays in concrete towers.

2. Tower concept

The project for which we have developped this saddle concept is the Papaloapan bridge (Mexico), which was opened for traffic in 1995. This is a cable-stayed bridge with concrete deck and a 203 m long main span (Fig. 1). Because of the very flat landscape, the overall design concept was aimed at minimize the transverse dimensions of all the elements of the bridge; the depth of the deck is only 1.44 m for 23.4 m width in order to raise the bridge profile as little as posible to reduce the approach embankments. The stays are arranged in two vertical planes, on both sides of the deck.

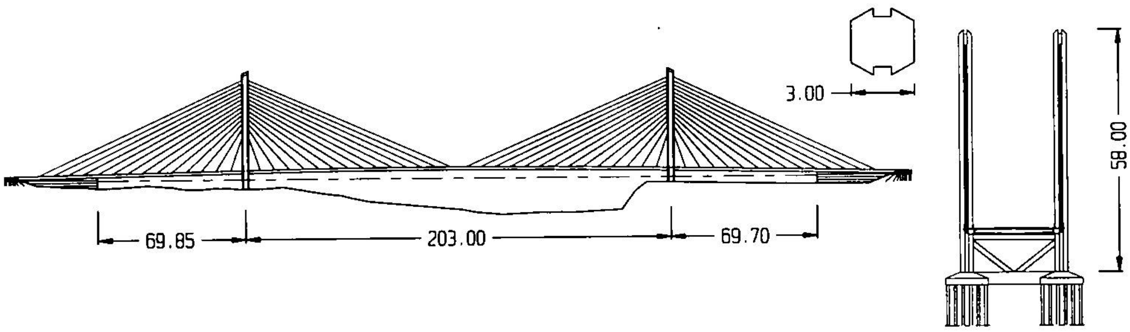

Fig. 1. Papaloapan cable-stayed bridge (bridge and tower elevations; shaft cross section)

The towers are made of two independent shafts which are only linked together by a triangular bracing under the deck. The sense of verticality of these shafts is enhanced by the symmetry of the cable arrangement, the aspect ratio of the shafts (1/19) and their octogonal constant cross-section.

Then, as result of these conceptual ideas, it was very important to maintain symmetry in all possible aspects of the design and to avoid any interruption in the vertical contour lines of the tower shafts. Cable overlapping in the tower was precluded and there was no room for an interior gallery in the shafts. As a consequence, the natural choice seems to be the use of steel connecting elements between front and back anchorages. The need to maintain vertical contour lines leaves no other choice than embedding these steel elements inside the shafts.

3. Saddle concept

The steel elements which are designed to connect front and back anchorages will be called saddles from now on since their concept is somewhat similar to the saddles of suspension bridges. Their design has to take into account two main constraints: size has to be reduced such as to fit them inside the tower and fatigue resistance has to be excellent since there will not be any possibility for inspection as they are embedded in concrete.

Both reasons made us consider the perforated plate as a suitable concept since it was originally proposed as a good alternative to the stud connector concept for its fatigue strength [6,7]. Nevertheless important differences may be found between the loads which are applied on a perforated plate shear connector as described previously and our present saddles.

Shear connection is only part of the static problem in the saddles. They also have to transmit tension forces between both stays and vertical compression forces to the concrete tower. A typical design (Fig. 2) consists of a vertical 60 mm thick plate with a double array of 100 mm diameter circular holes, two horizontal 40 mm thick horizontal plates and some additional stiffening plates. Design changes slightly as a function of the slope of the cables. Both anchorages are connected to the vertical plate by means of an intermediate tapered butt-welded cast steel plate whose cross section changes from rectangular to circular in order to be screwed to an annular connector holding the cable anchorage.

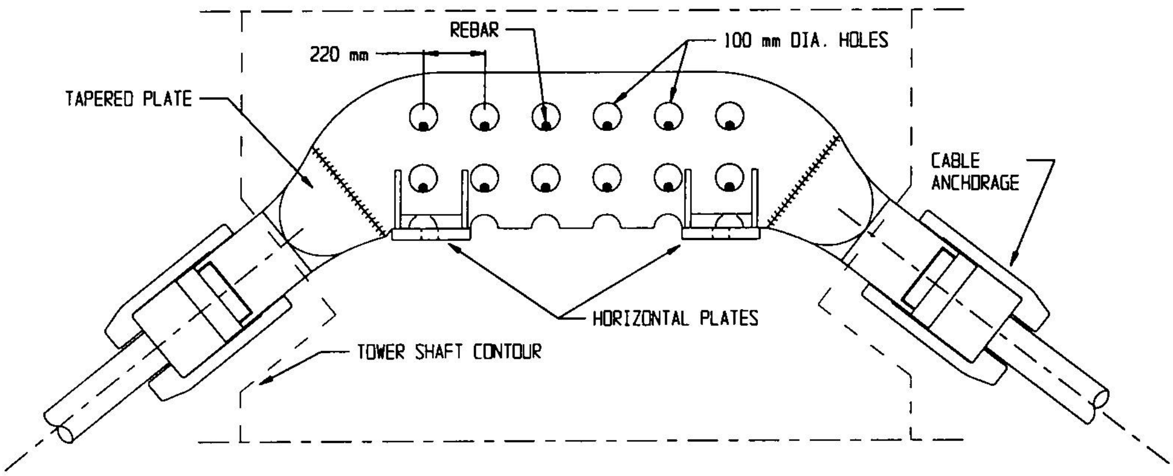

Fig. 2. Saddle elevation.

As compared with perforated plates for shear connection [6,7], these plates are thicker and the holes are also larger. The size of the holes is increased for a number of reasons including the need for large shear strength, the interest in having rebars across the holes and leaving wide enough space for concrete aggregates as well as a better control of stress concentrations in the steel plates. Thickness is much larger than what would be necessary for a shear connector because the vertical plate has to withstand very large tension forces from both stays. All steel elements are made of AH-55 steel (roughly equivalent to AE355) and were stress-relieved by means of a thermic treatment after welding.

4. Saddle design and analysis

4.1 Statical schemes

Among the load cases which have to be considered there are two limit situations which define the resisting properties of the saddles:

- The symmetric load pattern corresponds to roughly similar cable forces on both sides (Fig. 3). In this case the saddles are supporting tension forces and for such load case the stress concentrations around the holes may be very significant. The saddles are also transmitting vertical components to the concrete shaft through the horizontal plates as well as through the shear connectors.

- The unsymmetric load pattern corresponds to unbalanced cable forces; the limit case would be that which happens during erection when only one cable may be stressed (Fig. 3). In this case the saddles are mainly shear connecting devices since they transmit horizontal and vertical forces to the concrete shaft by means of the circular holes as well as through the horizontal plates (in the case of vertical components).

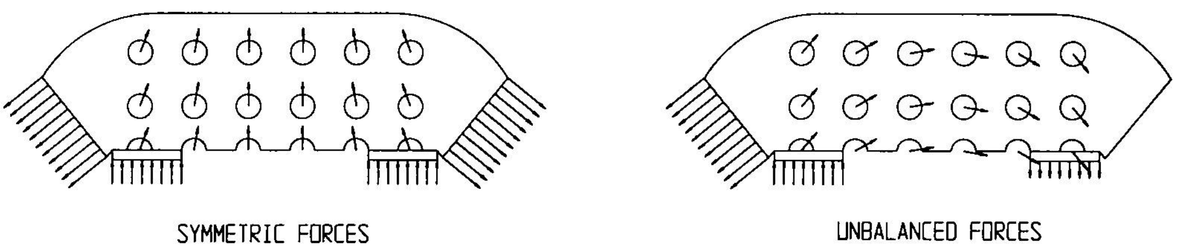

Fig. 3. Basic statical schemes.

Statical analysis of the saddles could be performed by means of a complete 3D finite element model (Fig. 4). In this model concrete is supposed to be sliding against the steel plate and shear is only trasmitted through the concrete dowels. Nevertheless, as complicated concrete cracking patterns make such analysis very unstable, it is difficult to get ultimate strength values. Then such model is only used for elastic analyses. Consequently the static problem has been divided into three parts (the shear connection, the steel plates stresses and concrete stresses) as a way to get a reliable and safe design.

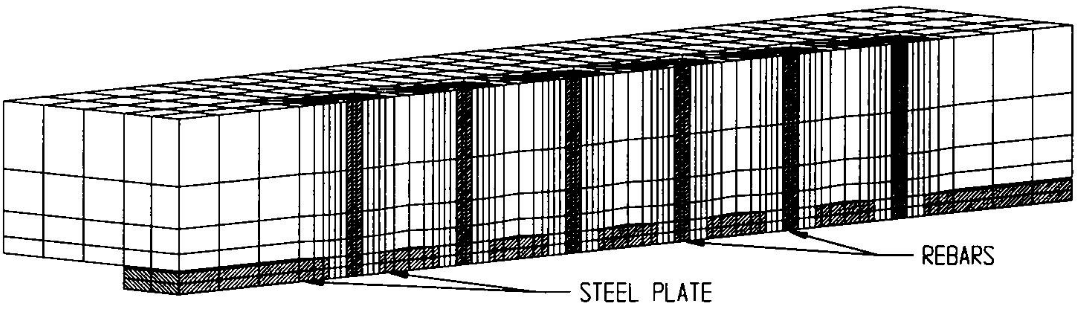

Fig. 4. Three-dimensional finite element model.

4.2 Shear connection

Although previous analytical and experimental studies are available [6,7], this kind of connection is not yet standard and there are no generally accepted formulas to define its ultimate capacity. If we forget about the plate itself, shear capacity is a function of concrete shear strength (with many limitations because of the complicated geometrical configuration) and of transverse reinforcement as in any standard shear analysis.

This case is somewhat different from the perforated plate with a single array of holes since in that case shear may be transmitted in a fully three-dimensional pattern (normal and parallel to the plate). The multiple array of holes forces shear transmission to concrete only in the normal direction to the plate. The problem is then bi-dimensional and, in some sense, simpler. Several methods have been considered for the design.

If we consider each external plane of the plate as a shear joint and we apply standard design rules such as CEB-FIP [8], Eurocode [9] or ACI [10], the contribution of concrete will be computed as the product of a shear stress and a reference area. Depending on the choice of both factors this contribution could range between 10 and 30 kN for a 100 mm diameter hole and a C40 concrete. The contribution of reinforcement bars is also subjected to different interpretations specially with respect to the coefficient of friction to be applied to the shear transmission plane (it may vary between 0.7 if we consider the contact between the steel plate and concrete and 1.4 for monolithical concrete). A safe estimation of this coefficient of friction (0.9) would give a 608 kN contribution for a 32 mm diameter rebar (f_{yd}=420MPa). Then transverse reinforcement is by far the most important factor in defining shear capacity of the connection.

Another limit state which has to be checked corresponds to cracking at inclined angles but this is a more standard computation and it happens to be not as demanding for transverse reinforcement as the previous one. The capacity per hole in the same conditions as before would be 677 kN for the conservative assumption of having cracks at 45°.

Finally the connection has been checked against the dowel action failure. This point might be controversial since reinforcement is embedded in the 100 mm diameter concrete cylinder which is monolithical with the whole shaft. This check gives the most conservative estimation of the connection shear capacity. According to CEB-FIP Model Code [8], the computed capacity would be 217 kN per hole.

Andrä's formulas [7] were also used as an estimation of shear capacity but they have to be taken with care since they have only be shown to be valid for smaller holes and for a three-dimensional shear transmission pattern. Moreover they give the global capacity of the connection including plate failure and our design does not maintain the same scale factor for all geometrical dimensions. As applied here they would predict a 400 kN capacity with a 36 mm rebar per hole which would could be converted into 316 kN for our 32 mm diameter rebar.

We finally used the dowel action model as the most conservative estimation. Nevertheless this design process shows how interesting it might be to carry an extensive testing program to define precisely the shear capacity of this kind of connection.

4.3 Steel plate analysis

The second link in the transmission of forces between the cables and the shaft is the plate assembly. To check its state of stress a finite element model has been analyzed (Fig. 5). The most interesting feature of the model may be found in the modelling of the holes. These holes were considered to be filled with concrete and elastically supported at their center. Horizontal plates were also elastically supported at their base. In this way it is possible to obtain a reasonable reaction distribution among the different holes and plates and a reliable stress distribution in the steel plates. Spring constants were evaluated from the three-dimensional finite element analysis which was mentioned earlier (Fig. 4). Resulting value was expressed as $3.1G_cA/D$ (where G_c is the concrete shear modulus, A is the area of the hole and D its diameter) to emphasize the concrete dowel effect which is the origin of this stiffness.

Stress results were checked according to AASHTO Standard [11] in service load design (allowable stress is 0.55 f_y) and against fatigue (stress variation at any point is limited to a value ranging from 27 to 110 MPa depending on the detail). According to finite element results, maximum stress is by far the governing design criterion; fatigue is not conditioning in any case the design of the saddle (maximum stress variation is only 9 MPa). This result is important since it shows, the interest in using this type of connection for a fatigue sensitive structure.

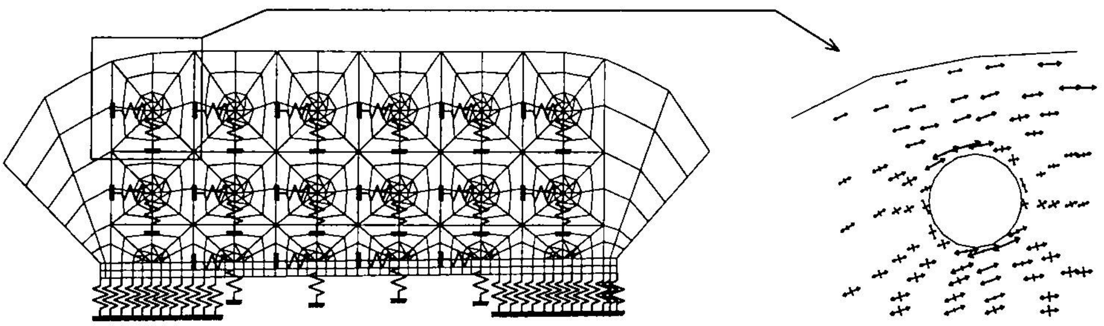

Fig. 5. Finite element model of the plate and detail of stresses.

The stress analysis of the saddles shows the importance of the holes as stress concentrators and the fact that filling with concrete these holes somewhat reduces the stress concentration. If the analysis is repeated after leaving one of the holes empty, maximum von Mises stress is increased by 27%. Then many important reasons support the need for a very careful concrete casting process in order to make the saddle work as it is assumed in the analyses.

With respect to the fatigue strength of this connection it has to be emphasized that the areas of the plate which show a certain fatigue risk are the stress concentrations around the holes. Since the holes are machined and stress relieved afterwards it is very unlikely that residual stresses may be present a these points. Then the fatigue process may be very well controlled through the static analysis of the plate.

4.4 Concrete analysis

Previous analyses lead to reaction forces at all the elastic supports. These reaction forces are input to a new finite element model of the shaft to check concrete stresses. This new analysis does not discover any new aspect of the connection since all the results may be obtained by equilibrium conditions and standard design rules. This analysis shows that tensile stresses develop in the neighbourhood of the saddles due to strain compatibility. Although these stresses are not important, transverse prestressing was arranged in the direction of the saddles to reduce cracking and further increase the fatigue strength of the connection. This transverse prestressing (4 no.36 mm diameter bars per saddle) is in any case much less than what it would be necessary to prestress a hollow shaft to fully transmit cable forces.

5. Conclusions

The perforated plate concept has been selected as a compact and fatigue resistant device to be used simultaneously as a link to transmit tension forces and as a shear connector to absorb unbalanced forces to a concrete structure. Its use in cable stayed bridge towers simplifies design and erection. They are specially interesting for short and medium span bridges where the size of the towers does not leave much space for other connecting devices. A design methodology has been presented but it is too conservative. Extensive experimental testing should be performed to define precisely the strength of such connection.

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