Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 79 (1998)

Artikel: Assessment and strengthening of two cantilever-type concrete bridges

Autor: Teigen, Jan / Fjeldheim, Steinar

DOI: https://doi.org/10.5169/seals-59876

Assessment and Strengthening of Two Cantilever-Type Concrete Bridges

Jan TEIGEN
Principal Eng.
Det Norske Veritas
Hovik, Norway

Jan Teigen, born 1951, received his civil engineering degree from Norwegian Institute of Technology in 1974, Master of Science degree form Univ. of Colorado in 1990 and Dr Phil. degree from Univ. of Oslo in 1994. He is currently Principal Engineer in Section for Fixed Structures at Det Norske Veritas.

Steinar FJELDHEIM
Principal Eng.
Norwegian Public Roads Adm.
Oslo, Norway

Steinar Fjeldheim, born 1944, received his civil engineering degree from Univ. of Denver in 1968. He is currently Principal Engineer in Bridge Maintenance Division of Bridge Department at Norwegian Public Roads Administration.

Summary

For both Puttesund and Sørsund Bridge, excessive sag has developed over time resulting in a severe slope-discontinuity at the hinge in the centre of the main span. Both bridges are also found to have insufficient capacity in some regions of the main span, Puttesund basically related to shear and Sørsund mainly to bending. The purpose of this paper is to present some of the design work and evaluations carried out for various alternatives to raise and strengthen the bridges. Generally, it proved difficult to find efficient and economic methods for raising the girders to a significant proportion of the existing deflections. Thus, for the final upgrading procedures priorities were instead given to weight saving actions, improvements of traffic comfort and, of course, the necessary strengthening of the girders. Analyses were based on nonlinear FE-technique. Also in situ measurements of concrete stresses have been carried out.

1. Introduction

Puttesund Bridge (Fig. 1) was completed in 1970. It is a prestressed concrete single cell box girder bridge built by the cantilever construction. The main span of 138 m is counterbalanced by two 28 m rock-filled abutments. Sørsund Bridge (Fig. 2) was completed in 1962. It consists of two approach spans and three cantilever constructed main spans with single cell box girder of lengths 50 m + 100 m + 50 m. In the centre of the (largest) main span of both bridges is located a hinge that transfers shear and torsion and allows for axial movements. In common is also the weight saving opening in the bottom slab on adjacent sides of the hinge. Over the years the main span has deflected about 0.4 - 0.5 m on the bridges resulting in a severe slope-discontinuity at the hinge. On Sørsund some remedial actions to improve the traffic comfort have been taken in the form of adding further surfacing in the region of the hinge, the maximum thickness being 18 cm. This substantial increase in weight has again amplified the deflection, and today the rotation capacity is exceeded implying the joint is permanently closed at the top. This malfunction has however probably prevented the arms from further sagging. On Puttesund measurements have shown a steady increase in deflection over the last 15 years, but at a somewhat decreasing rate. Also cracks have been recorded. Around the quarter-points of the span inclined cracks at the upper part of the webs are observed, indicating a potential shear capacity problem.

2. Program review, modelling considerations and types of analysis

Since the computer program used initially on Puttesund gave poor agreement with the observed behaviour of the bridge, it was decided to employ the state-of-the-art *computer program DARC*. This is a nonlinear finite element program that originally was developed as a part of a doctoral study by the first author. The key ingredient of the program is a new 3D shear-beam element that allows for analysing the response of reinforced and prestressed concrete beams in arbitrary combinations of the axial, bending, shear and torsion modes. All significant nonlinear material and time dependent effects are taken into account like cracking, crushing, creep, shrinkage and ageing of the concrete and yielding and relaxation of the reinforcement. Also the segmental construction capability is included. The 3D element mentioned is formulated on so-called hierarchical form, which makes it possible to suppress the shear effect so that the axial/bending behaviour can be represented alone. For Puttesund having a shear capacity problem, this option became particularly useful since it clarified at an early stage the maximum effect by shear-strengthening the bridge.

Generally the element mesh of the girders is chosen in accordance with the division of casting segments used in the construction. Each element is based on the dimensions at the midlength of the corresponding segment. The bottom slab is however sloped to achieve the arch effect in girders of variable height, still using only one element per segment. Due to symmetry, only half the bridges were initially analysed based on free-edge boundary condition at the hinge. For Sørsund, part of the corresponding approach span was also omitted, giving a 120 m model of the total 408 m bridge. The assumed symmetry is valid only for symmetric configurations of traffic loading. For Puttesund the whole system had to be taken into account in a final analysis when maximum shear in the critical segment was studied.

Two kinds of analysis are performed. The *time analysis* simulates the bridge behaviour from start of construction and up to the current year of the analysis ('today') when usually some kind of remedial action is introduced. Finally the analysis continues for another 25 years. The construction sequences and corresponding times are based on drawings and other documents available. Unless otherwise noted only permanent loading is considered (dead weight, prestress, etc.). The purpose of the time analysis is primarily to investigate the deflection-state of the bridge 'today' and in the future with or without the various remedial actions for raising and strengthening the girder. Material properties are based on characteristic code values or best estimates in absence of such. The *capacity analysis* is a short-term analysis under combination of permanent and traffic loading. The purpose of the analysis is to determine the load carrying capacity of the bridge at a given state. The prestress level for the state is taken from the time analysis. Material properties and loading are based on factored ULS design values. The current class of traffic loading is used in the analysis, although both bridges originally were designed according to a lower load class. Instead, the required factor of traffic loading is now reduced to 0.87, which is two third of the current code value of 1.3. The analysis is carried out by first applying the permanent loading up to its factored load level, usually with load factor 1.2 except for 1.0 on prestressing. Then the traffic loading is gradually introduced until the failure load of the bridge is reached.

3. Puttesund Bridge

The *time analysis* of the **existing bridge** gave for the situation of 'today' per Sept. 1995, a resulting position of the cantilever tip about 29 cm below the prescribed road line. For further 25 years ahead the predicted sag will exceed 32 cm. In these figures the precamber used for construction has been subtracted. In comparison, the observed sag of 'today' is about 41 cm, thus 12 cm more than computed. Four years after completion of the bridge, in 1974, this difference was about 2.5 cm, while in 1980 it had increased to 4.6 cm. From this trend it is good reason to expect that the additional sag for the bridge in the next 25 years will become a lot more than the computed 3 cm. Among reasons for this discrepancy may be cyclic effects from heavy vehicles, impact loading at the hinge due to the growing slope-discontinuity and uncertainties in long-term material properties. Only permanent loading is considered in this analysis. To investigate the effect of one occurrence of heavy traffic loading, an additional analysis was performed where the characteristic traffic loading was applied shortly after completion of the bridge and then removed. This gave an immediate permanent contribution to the tip deflection of 2 cm, but no additional long-term effect was detected. Also an analysis with the purpose of investigating the sensitivity with respect to long-term material properties was carried out. Here the assumption was made that the 28-day code values were not reached before 'today'. Using the same ageing function as before implies quite weaker concrete at the early years. The impact on computed sag was dramatic, now becoming about 55 cm 'today', compared to 29 cm formerly. Finally, the contribution to the sag from shear deformations was investigated. At the tip this was about 15 %. Getting closer to the abutment the shear contribution increases, eventually becoming dominant. The *capacity analysis* was run with prestress level corresponding to the state of 'today' as taken from the time analysis. Failure occurred by yielding in the stirrups accompanied by crushing of concrete in the lower part of the webs in the segment closest to the quarter-point of the span, where also the inclined cracks are observed. At failure the factor of traffic loading was 0.75, thus below the required value 0.87.

An obvious way of raising the girder is by application of **external prestressing along the bridge**. Here a total of six cables are placed at the soffit of the bridge deck flanges (Fig. 3), each with a tensioning force of 2 MN applied behind the abutments. In the *time analysis*, the tensioning performed 'today' gave an immediate uplift of the cantilever tip of 8.5 cm. After three months 1 cm additional uplift was gained. However, for the next 25 years the tip gets almost the same additional long-term sag as if no external prestressing is applied. Also a time analysis including extra weight in the hinge area to improve traffic comfort was made, reducing the uplift to 6 cm. The *capacity analysis* was run with prestress level corresponding to 25 years ahead of 'today' and with the extra weight included. The analysis gave a factor of traffic loading at failure of 0.70, thus slightly less than obtained without longitudinal external prestressing and extra weight. Since failure is governed by shear, longitudinal prestressing has very little influence on the capacity. Also an identical analysis, but with shear effect suppressed, was carried out. The factor of traffic loading at failure then became 1.45, expressing the capacity of the bridge in the axial and bending modes. This indicates that the bridge may pass the required safety level with ample margin using an efficient shear strengthening method. A preliminary analysis confirmed that a promising remedy then is vertical prestressing of the webs.

A less traditional alternative for raising the girder is by means of **pretensioning the bottom slab**. This may be obtained by jacking compression into steel tubes placed on the slab (Fig. 4). Besides giving uplift, the method will also reduce the shear forces due to the inclination of the bottom slab. All work is carried out inside the box girder undisturbed by the traffic. Analysed here are four steel tubes with a total jacking force of only 2.4 MN. Due to the wide opening in the bottom slab, this is about the maximum tensile force that can be transferred to the cross section without any strengthening. The *time analysis* gave an immediate uplift of the cantilever tip of 3 cm. The uplift is now more concentrated to the hinge area, giving an almost horizontal tip tangent and thus eliminating the demand for extra traffic comfort. The *capacity analysis* showed very little impact from this solution. However, with increased pretensioning the method is still interesting, either as a separate means or in combination with others.

Since the methods considered so far did not give the desired uplift without substantially increasing the prestressing forces, it was decided to investigate the effect of a **pseudo cable stayed solution** (Fig. 5). A simple steel tower, supporting two cables anchored 9 m and 33 m from the cantilever tip, is placed at the front of each counterbalance abutment. The proportion between the forces in outer and inner cable is 1.0 : 0.7, which means that the vertical components will be almost of same magnitude. A simplified analysis was made based on treating the cable forces as external loading on the existing bridge. The intention was to find the approximate magnitude of the cable forces that would bring the arms back to their original level. The result was somewhat surprising, since cable forces of only 4.0 MN and 2.8 MN in outer and inner cable respectively, became sufficient. At this stage the shear force in the critical segment is almost zero. Thus, the solution appears to elegantly solve both the sag and shear problem. However, since a more comprehensive analysis was not run, some unforeseen problems may come forward.

An **evaluation of the various alternatives** may be summarised as follows:

- Use of longitudinal external cables will require a substantial increase in prestressing force if significant uplift shall be achieved. This means that the superstructure will require more strengthening locally in the anchoring zones of the cables and thereby also a corresponding increase in weight and costs. The prestressing will not help to obtain the desired load class, but is first of all a means to improve the traffic comfort and make a better appearance.
- Pretensioning the bottom slab has much the same features as using longitudinal external cables.
- The pseudo cable stayed solution appears to solve both the sag and the shear problem, but is found too expensive.
- Application of vertical prestressing improves the shear capacity but does not give any uplift.

Since the **upgrading procedure** had to solve the shear capacity problem, it was decided to optimise further the solution for transverse prestressing. The various means to actively raise the girder were all abandoned because of their high costs. Instead a relatively simple way of improving the traffic comfort was called for, which basically consists of replacing the sidewalks of concrete with aluminium, removing existing asphalt and building up a wearing course of variable thickness in the hinge area using leca concrete. To avoid bumps, the transition curve is based on the existing deflection taking into account the minimum allowable vertical road radius for the design speed in question. On top is placed a 3 cm layer of high quality asphalt. The final weight saving on each arm becomes 21 MN, which is advantageous in view of keeping future deflections at a minimum. Also the temporary removal of the guardrail makes it possible to adjust its alignment in the sagging area.

So far, all attention has been given to the analysis of the bridge in the longitudinal direction. Since shear was found critical in a region of each arm, it was deemed necessary also to carry out a *transverse analysis*. The web moments from traffic and weight of bottom and top slab will utilise the same reinforcement as required by the global shear forces, and thus possibly amplify the shear capacity problem. The transverse analysis showed that the web moments occupied a substantial part of the stirrup-capacity. Based on this, **vertical prestressing** equal to 2 MPa in the webs was assumed along the whole bridge span. In order to have the prestressing effective over the whole height of the webs, the bars are anchored at the soffit of the bottom slab and as close to the top surface of the deck as possible (Fig. 6). The spacing of the bars is determined so that the concrete stresses become uniformly distributed in the longitudinal direction. Since DARC is based on a beam model it cannot account for the transverse effects in the box girder directly. Instead the area of existing stirrups were reduced in the model by the amount of reinforcement required to carry the transverse moments. This approach is believed to be on the safe side, and consequently also the amount of prestressing found on this basis to obtain the required shear capacity. For the **final analysis** two different locations of heavy traffic loading are now considered for the global analysis. One symmetric condition giving maximum moment over the whole span, and one asymmetric that gives maximum shear in the critical segment. In the latter case the model must comprise the whole bridge system. The analysis confirmed that vertical prestressing became necessary along the whole bridge span due to the combined effect of global shear and transverse moments. However, the strengthening changed the type of failure from shear to bending, the factor of traffic loading now being 1.35. No final time analysis was made.

4. Sørsund Bridge

The *time analysis* of the **existing bridge** showed that the cantilever tip for the situation of 'today' per July 1996, was positioned 27 cm below the prescribed road line, and for the next 25 years an additional 1 cm is predicted. In these figures an assumed precamber profile that brought the superstructure into correct position at completion has been subtracted. Again, there is a distinct discrepancy between calculated and measured values since the observed sag of 'today' is about 50 cm. However, later measurements of the asphalt layer showed that the thickness varied between 12-18 cm, which is two to three times more than assumed in the analysis. The *capacity analysis* was run with a prestress level for the state of 'today' as taken from the time analysis. Failure occurred at a factor of traffic loading of 0.30, thus quite far from the required value 0.87. The failure mode was crushing of concrete in the lower region of the girder at a segment in the main span with wide opening in the bottom slab, about 20 m from the hinge. This indicates the opening causes insufficient compression capacity, and that bending and not shear is governing the failure.

In the critical segment the transverse web moments are not pronounced because the cross section here basically consists of two individual beams due to the wide opening in the bottom slab. Consequently the stirrups can mainly be utilised for global shear transfer. For the closed box areas, a *transverse analysis* has however been made. Since bending failure is found critical, the use of longitudinal external cables may now become a relevant means not only for giving uplift but also for strengthening the bridge. However, again facing a modest budget of rehabilitation, load class requirements must have top priority in the **upgrading procedure** and actively lifting the cantilever arms is left out. The strengthening consists of casting the inner 12 m of the open bottom slab (Fig. 7). Improved traffic comfort is solved in a way similar to Puttesund.

Before running the final analysis, certain input data were correlated to **measurements made on the bridge**. One intention was to compare in situ stresses in concrete at selected points with values obtained from the analysis. The measurements of concrete stresses were carried out using the so-called stress-relief coring technique. Generally, measured stresses were found to be higher than computed in the top slab and vice versa for the bottom slab, indicating the actual prestress level may be higher than assumed in the analyses. Thus, the prestress was adjusted so that better agreement with measured stresses was obtained. In addition to stress measurements, also the concrete strength and modulus of elasticity were tested, allowing for a 33 % increase in concrete strength. Since moment and not shear is deemed critical, the original symmetry-based model was retained for the **final analysis**. The 12 m casting of the open bottom slab was introduced in the analysis by separate elements after the application of permanent loading, but before the traffic loading. Again bending became critical, but now failure took place in the corresponding region with open bottom slab in the side span at a factor of traffic loading of 1.05, confirming sufficient capacity of the bridge. No final time analysis was made.

5. Conclusions

The nonlinear analysis program DARC proved to be a successful tool to identify critical failure modes and to investigate the various strengthening and lifting alternatives for the two sag-deteriorated bridges. Both structures were found to have insufficient capacity, one related to shear failure and the other to bending. It proved difficult however to find efficient and economic uplift methods. For the chosen upgrading procedures, priorities had to be given to strengthening of the girders, weight reductions and improvements of traffic comfort. Vertical prestressing of the webs and casting a region of the open bottom slab were the adopted strengthening methods for the shear-weak and bending-weak bridge, respectively. Computed sag has generally been less than observed. It seems difficult to point out one main cause for this since long-term deformations are dependent on many factors. For bridges with a hinge in the centre of the main span, however, it is believed that the traffic impacts there due to the growing slope-discontinuity are particularly detrimental. The improved traffic comfort and weight reductions may thus become mitigating. Future performance of the bridges should be carefully recorded. To improve the analysis concept, more effort should be paid to clarify the interaction between global and local load effects.

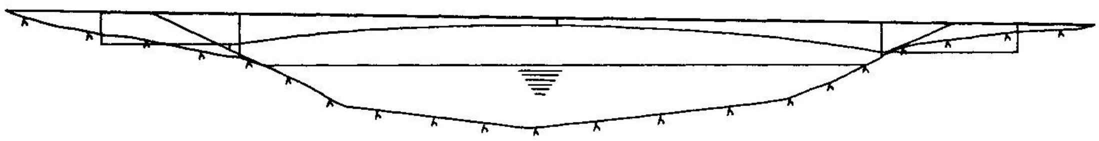

Fig. 1 Overview of Puttesund Bridge

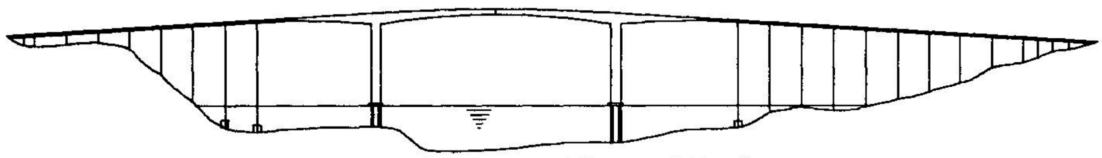

Fig. 2 Overview of Sørsund Bridge

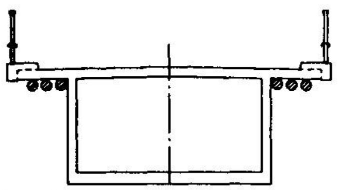

Fig. 3 Longitudinal external cables

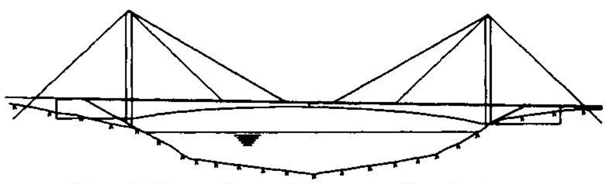

Fig. 5 Pseudo cable stayed solution

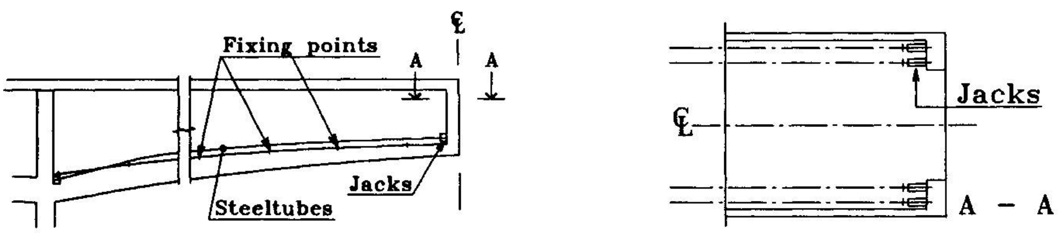

Fig. 4 Pretensioning of bottom slab

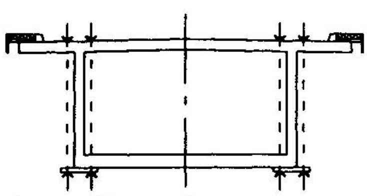

Fig. 6 Vertical prestressing

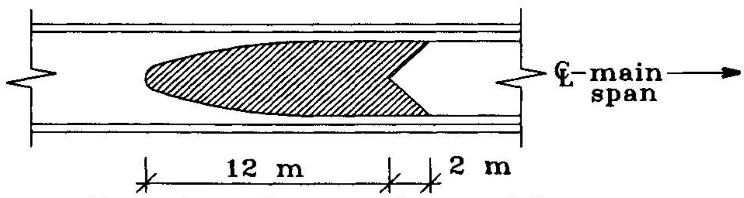

Fig. 7 Strengthening of bottom slab