Consistent Design of Segmental Concrete Bridges

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ABSTRACT

The segmental method is an accepted and economic construction technique, however, the related design task is extremely demanding and technically ambitious. Generally it requires sophisticated structural analyses, where all properties influencing the deformation behavior are properly taken into account. These requirements include inter alia the consideration of structural non-linearity, creep and shrinkage behavior, pre-camber and deformation control during erection.

Using the segmental technique for cable-supported structures such as cable-stayed or post-tensioned bridges aggravates the complexity of the design task and yields additional challenges. The paper shows a comprehensive computational approach where difficult problems such as pre-camber and deformation control, optimization of stay cable stressing and taking into account creep and shrinkage effects are handled in a consistent manner.

Keywords: Segmental Bridge, Pre-cast Segments, Casting Machine, Cable Stayed Bridge, Pre-camber, and Geometry Control.

INTRODUCTION

The segmental method is a widely applied construction technique and well suited to improve the construction process of concrete bridges. However, the proper design process for such bridges requires accounting in detail for many influences, which can be considered by tough estimations in the design of traditional bridge types. These are in fact detailed investigations of the deformation behavior of the bridge in all intermediate stages during erection. Major topics in this respect are taking into account any structural non-linearity and accurately modeling the creep and shrinkage behavior, besides the prerequisite considering in detail the actual erection process in time.

Special problems arise in the design of Cable Stayed Bridges, where the design concept for achieving the appropriate tensioning procedure is often based on finding the forces in the individual cables that give rise to certain allowable structural displacements, moments or stress distributions in the girder and the pylons at the end of construction. The stressing forces and the sequence of stressing for all the cables needs to be optimized to meet these pre-defined requirements as closely as possible.

The paper specifically highlights some of the topics, which a modern comprehensive design analysis tool must be able to consider within the standard analysis process and without additional intermediate external calculations and data exchange processes, which often are sources of errors and cause tedious additional work in case of any structural model changes during the design process. Practical examples illustrate, how these topics are handled in a commercially available structural analysis package.

The specifically addressed topics are

- Optimizing the stay cable stressing sequence in order to allow for slender superstructure cross-sections of Cable Stayed Bridges.
- Properly considering the stage-wise erection and the related deformation behavior
- Properly considering the creep and shrinkage behavior in order to get trustworthy deformation predictions
- Accurate pre-camber requirements and deformation control during erection
- Computational techniques for easily handling any required correction measures in case of deviations from the predicted shape in intermediate stages.

CABLE STAYED BRIDGES – CABLE TENSIONING OPTIMIZATION

The standard bridge design process begins with preparing a structural model, defining the loads and the construction schedule (construction stages). The engineer will then “run” a computer analysis. Two result types of the computer analysis are of main interest: The loading case result, and the envelope result. The loading case result represents the structural state at a defined point in time. The envelope result provides information about maximal/minimal peaks of a given result together with other corresponding results. Based on the results, the design criteria can be checked and optimization can be started. A simple example is given to demonstrate the principle of optimized cable tensioning: In Fig.1 the bending moment diagram is shown for the final stage without optimization of the cable tensioning process.

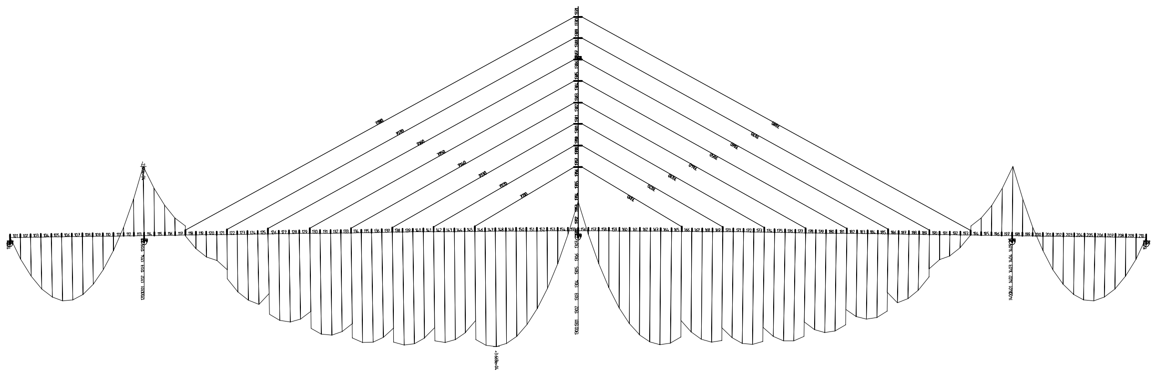

Fig.1 Bending Moment Mz at final stage without optimization

Optimizing the stay cable tensioning sequence can considerably changing the stressing state in the superstructure and therefore allow for using much more slender superstructures. The weight reduction results in savings with respect to both material consumption and lifting energy consumption. Fig.2 the shows for the same structure typical results after optimization.

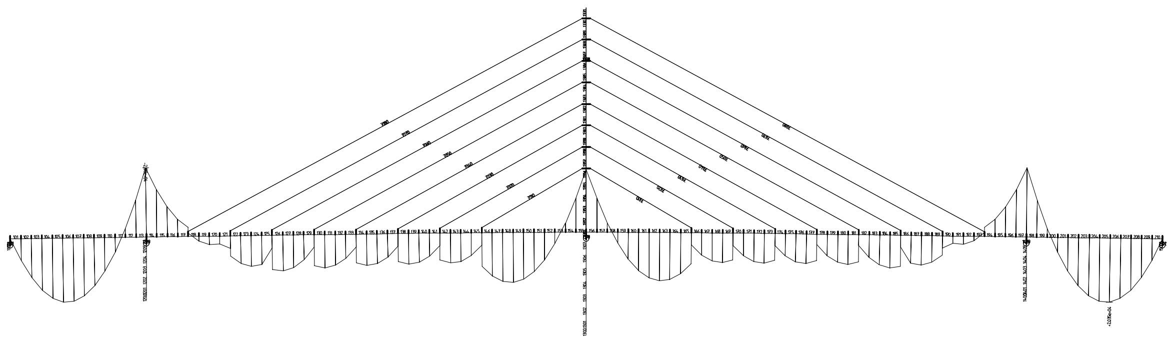

Fig. 2 Bending Moment Mz at final stage after optimization

However, for several reasons, finding the optimum tensioning strategy can be an extremely tedious and time-consuming process. Reasons are for instance:

- Tensioning one cable affects the forces in all the other cables.
- Cables cannot withstand compressive forces but stressing an adjacent cable may apparently cause this condition.
- Stressing of the stay cables is an expensive procedure, therefore, the number of individual stressing actions must be kept to a minimum.
- Definition of the tensioning strategy is interrelated with the chosen erection method and the accurate simulation of the erection procedure can be very complicated.
- The deck girder and pylon systems must behave reasonably in all construction phases. I.e. deflections should be neither excessive nor incompatible with the type of construction.
- Creep and Shrinkage greatly complicates the analytical process.

- "Uplift conditions" could exist at the temporary supports, which further complicates the analysis. Whilst special "Non-tension" members could be used, this greatly increases the degree of indeterminacy and hence the speed of analysis.

The above-mentioned reasons bring about the need for a consistent, standard, non trial and error type approach to the solution for these complex structures. The solution implemented in the structural analysis package RM, the heart of Bentley's recent BrIM initiative [1], is called AddCon method, or Additional Constraint Method. It is a further development of the unit load method described in detail in [2] and [3].

The method is based on the idea of formulating the engineer's considerations for achieving an optimal solution as additional constraints in the equation system solved in the analysis process. These constraints are result values such as structural displacements or bending moments or linear combinations of these values, which should be equal to or less than a predefined limit value. The counterpart of these constraints is a set of additional degrees of freedom, usually the result vectors of unit load cases with actual loading values being weighted until the constraint conditions are fulfilled.

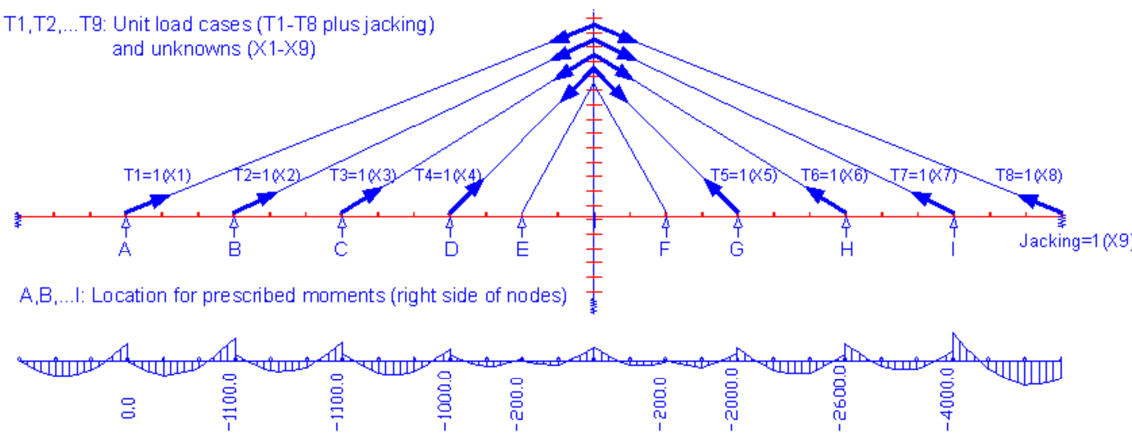

Fig. 3 Stay cable stressing and moments in the superstructure

With respect to cable stayed bridges, the concept that almost any moment diagram can be achieved in the deck and in the pylon by adjusting the following Degrees of Freedom is in line with the structural system choice:

- The tensioning forces in the stay cables and their stressing procedure
- The support movements (translation – longitudinal and vertical)
- Prefabrication shape of the deck girder and the pylon
- The erection procedure of the deck and the pylon

The actual approach is briefly described:
The result of one structural state i (e. g. results of load case i) can be written as a vector of dimension e:

$$\left\{E^{0i}\right\} = \{E_1 E_2 E_3 \ldots . E_e\}^T \tag{1}$$

Each item $E_j, j = 1..e$ in vector $\{E\}$ represents one result value of any type, e.g. displacement, internal force/moment, stress, etc, or a linear combination of such values. The significant results for the user can be written in vector form as well, where these results are calculated as linear combinations of the basic results:

$$\{E^i\} = [L]\{E^{0i}\} \tag{2}$$

Vector $\{E^i\}$ has the dimension n where $n << e$. The matrix $[L]$ has the dimension n x e and converts results from vector $\{E^{0i}\}$ to vector $\{E^i\}$. The result value for which a constraint can be defined is calculated as the linear combination of all system state results, e.g. as linear sum.

$$\{E\} = \sum_{i=1}^{m} \{E^i\} \tag{3}$$

In each design step, a new, different result vector for the same chosen result will be produced. Parts of the results $\{E^i\}$ may change; other parts may remain constant, depending on the different system parameters. For the further analysis it is necessary to split the m result vector into mv variable and mc constant results ($m = mc + mv$).

Finally we get the constraint equation, which is a system of linear equations. The solution is trivial for the special case where n=mv:

$$\{f\} = [M]^{-1} * \left(\{E^{user}\} - \{E^{const}\}\right) \tag{4}$$

It is necessary that matrix [M] be non-singular. No physical solution exists if matrix [M] is singular. This may happen if the chosen constraints are coupled. Due to numerical effects it is also possible to get solutions that have no practical meaning because matrix [M] is nearly singular.

Further on, we can also get results, which are structurally not acceptable, e.g. if compression cable forces are required to fulfill the constraint conditions. Structurally acceptable results clearly demonstrate that the chosen structural parameters are correct and also define the required tensioning and construction strategy. Structurally unacceptable results will point the way for modification of the parameters to be used in the next analysis.

Note that the system of equations is not symmetric and the diagonal coefficients may be zero. This is to be considered when solving the equations. This basic solution defines the cable forces and the jacking force for the final stage and, at the moment, does not include the effects of:

- The sequence of construction stages
- Creep and shrinkage
- 2nd Order Theory
- The non-linearity of the cables due to the sagging effects

The basic principles must therefore be extended to accommodate these effects.

CONSTRUCTION STAGE ANALYSIS

A similar system of unit loading cases can be defined for the construction stage analysis. The unit loading cases are, in this case, applied to the different structural systems, which exist at the individual construction stages. Figure 4 below, which shows a few of the construction stages, demonstrates the principle of the analysis method. The loading cases for each construction stage are combined to form the set of simultaneous equations, which must be solved to find the required multiplication factors for the unit loadings.

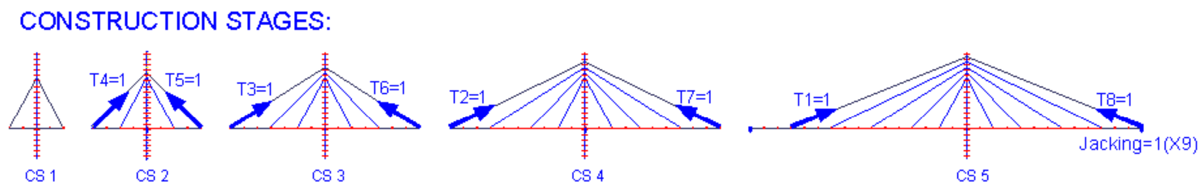

Fig. 4 Stay cable stressing in the construction schedule

Provided that the results are calculated in a linear analysis, a linear optimization solution can be expected, and for non-linear analysis a non-linear optimization solution must be applied. Unfortunately, this is not a case. Even with simple linear structural analysis, non-linear optimization methods must be applied. There are two major reasons for the non-linear components in the results: Continuous changing of the structural system and considering time effects in calculation.

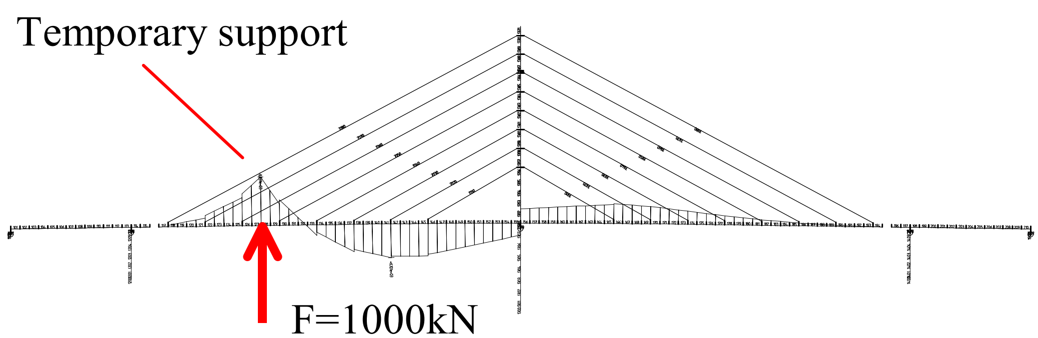

Fig 5 Bending moments on the main girder before closure (with temporary support active)

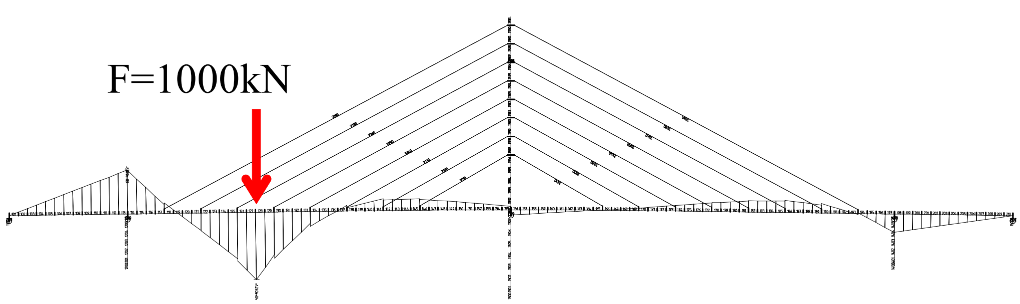

Fig 6 Bending moments after closure on the main girder due to temporary support removing

Continuous change of the structural system is a major reason for getting non-linear optimization problems. To understand the physical reason, a very simple example of using a temporary cable in the construction schedule is shown in Fig. 5 and 6. This non-linearity is different to the time-effects non-linearity, but it can be treated with the same optimization method.

CONSIDERING TIME EFFECTS

In practical engineering, time effects (creep, shrinkage and relaxation) must be considered in the analysis. In most codes, the time effects are described as quasi-linear. But these effects produce non-linear results in the final state. The reason for this is the forth dimension (time): A change of weighting factor for one loading will creep over time. These “creep results” will be increased with the same “weighting factor” if creep has “quasi superposition behavior”.

Looking at the structural state after some time, changes occur due to the initial change of loading and changes from all following creep steps up to this state. Changes due to creep are not known initially. The non-linear effects change the results only slightly, but linear optimization cannot be applied any more. In order to cover these effects in the described system, the above-mentioned matrix [M] (eq. 4) is split into a linear and a non-linear (time-effect) part.

Taking account of creep effects in accordance with the CEB-FIP model code is even more complex than it was the case in previous traditional methods. In spite of the above statements it can be shown through a series of mathematical equations that the effects of creep can be treated in a linear manner.

The following relation basically describes creep effects:

$$\{\varepsilon_e\} \,.\, \phi = \{\varepsilon_c\} \text{ (Elastic Strain * Creep Factor = Creep Strain)} \tag{5}$$

Decomposing the structure down to element level, the above equation is applied to each individual element by applying the generalized displacement method rules for calculating initial strain type loads:

- Define $\{\varepsilon_e\}$ over the element.
- Define $\{\varepsilon_c\} = \{\varepsilon_e\} \,.\, \phi$ over the element.

The member end displacements $\{\delta_c\}$ are found by weighted integration of the strain vector over the element length in the usual way. The member end forces are then calculated and the system of equations is assembled and solved for nodal displacement $\{\delta\}$ in the usual way. $(\{\delta\} - \{\delta_c\}) \,.\, [k] = \{F_I\}$ gives the internal forces due to creep. The system of analysis is completely linear up to this point.

Cognizance must however be taken of the age differences in the concrete as well as the various ages of different parts of the structure at the time of each increment of load application (ε_e is no longer constant but varies with time). A finite difference approach in time is applied here and using a linear variation over a time interval.

$$\underline{\{\varepsilon_t\}} = \{\varepsilon_0\}*\overline{(t_1 - t)}/\Delta t + \{\varepsilon_1\}*(t - t_0)/\Delta t \tag{5}$$

This equation can then be put into the basic displacement equation: at time t_0 , $\{\varepsilon_e\}$ is known and then by solving the equations for $\{\delta\}$ at time $t = t_1$ a recursive formula can be derived which results in a linear relationship at time t_1 such that the equation including the effects of creep is the same as the original equation with the exception that the modulus E is replaced by $E/(1+\phi*0.5)$.

The essence of the above statement is, that all the creep influences on the final distribution of internal forces and displacements are related in a linear manner to the elastic strain which initially caused the creep. The principles of linear superposition may, in consequence, be applied and the total creep occurring during a single time step may be decomposed into single contributions. Considering one of the prescribed ideal moment positions:

$$M_{creep} = M_p + M_{c\,t=1} \,.\, X_1 + M_{c\,t=2} \,.\, X_2 + M_{c\,t=3} \,.\, X_3 \text{............etc.} \qquad (6)$$

M_{creep} therefore consists of one part which is related to the permanent load and the other parts are related to the unit loads described above which are linearly coupled to the same unknown factors X_1 ...... X_9. As before the basic concept can now be applied; the effects of creep for permanent loads and for unit loads are decomposed into separate contributions from each time interval and then summed.

SECOND ORDER THEORY AND CABLE NON-LINEARITY

Since the element stiffness depends on the axial force (in the case of 2^{nd} Order Theory as well as for cable sag), the basic displacement method equations become non-linear. The equations defining the solutions for X_1 ...... X_9, which were proved to be linear for the creep case above, also become non-linear. An iterative approach must therefore be applied:

- A simple approach is to estimate X_1 ...... X_9, and use this estimate of the unknowns to find the variable stiffness, which on substitution hopefully gives a solution close to the final behavior. The estimate is corrected and X_1 ...... X_9 again calculated.
- A Better Approach is using the tangent stiffness for calculating the influence of the application of a small increment to each unit loading case. The equations can then be transformed to define the iterative correction for X_1 ...... X_9 and a procedure such as the Newton-Raphson method can be set up.

The cable sagging effects can be accommodated by deriving $d_S/d_{\Delta x}$ from the well-known “Peterson Formulae”, where S means the cable force and Δx is the cable extension. Convergence is accelerated and guaranteed, when using the tangent matrix with the Newton-Raphson approach as long as a real solution exists.

PRACTICAL EXAMPLES CABLE STAYED BRIDGES

The above-described optimization techniques with taking into account stage-wise erection, creep and shrinkage and structural non-linearity have been successfully used in the design of many cable-stayed bridges worldwide. Two of them shall be shortly described below.

UDDEVALLA BRIDGE

Probably the first major bridge, where these computational techniques have been extensively used, was the Uddevalla Bridge built in the late 90ies of the last century. This cable-stayed bridge is the central part of a continuous 1712m crossing over the Sunninge sound waterway between Udevallamotet north and Udevallamotet south in western Sweden.

The approach viaducts, comprising twin steel box girders with a concrete slab, are rigidly connected to the main bridge on either side and provide overall longitudinal structural stability. The cable-stayed bridge portion comprises a 414 m main span, symmetrical side spans of 179 m and two 85 m high (above the deck girder) diamond shaped concrete pylons, which anchor the fan shaped stay cable arrangement. The stay cables, which support the bridge deck on either side, are anchored at 13.32 m distance in the longitudinal direction.

Fig 7 The Uddevalla Bridge in Sweden under construction

The deck carries 6 lanes of traffic and comprises a composite, open steel grid structure with a 240 mm thick pre-cast concrete top slab, spanning longitudinally over the diaphragms.

The Uddevalla Cable-stayed Bridge construction required a 3 stage stressing procedure:

- Stage I stressing provides support for the new steel portion of the deck during assembly. The cables are initially stressed to provide support and to counteract excessive deflection before making the welded connection to the existing deck.

- Stage II stressing provides support for the whole structural self-weight comprising the steel plus pre-cast concrete top slab elements. The procedure is simple as the stressing jacks from the previous stressing operation are still connected. The first Unit load analysis to find the cable forces is carried out at this stage.
- Stage III stressing was required for counteracting the superimposed dead load and creep effects on the pylon deflection. The procedure is required because the stringent minimal pylon moment criteria preclude a sufficient pylon pre-camber. The second unit-load analysis to find the re-tensioning cable forces is carried out at this stage.

The "Ideal Moment Diagram" chosen for the initial dead load is shown below together with a general bridge arrangement. Note the unusual shape in this "Ideal Moment diagram" in the deck girder was dictated by a strict limitation prescribed for the pylon moments, which takes cognizance of the "very severe" environmental conditions for reinforced concrete weathering/corrosion. In order to comply with this stipulation, the "Ideal Moment Diagram" had to include a minimal moment condition in the pylon.

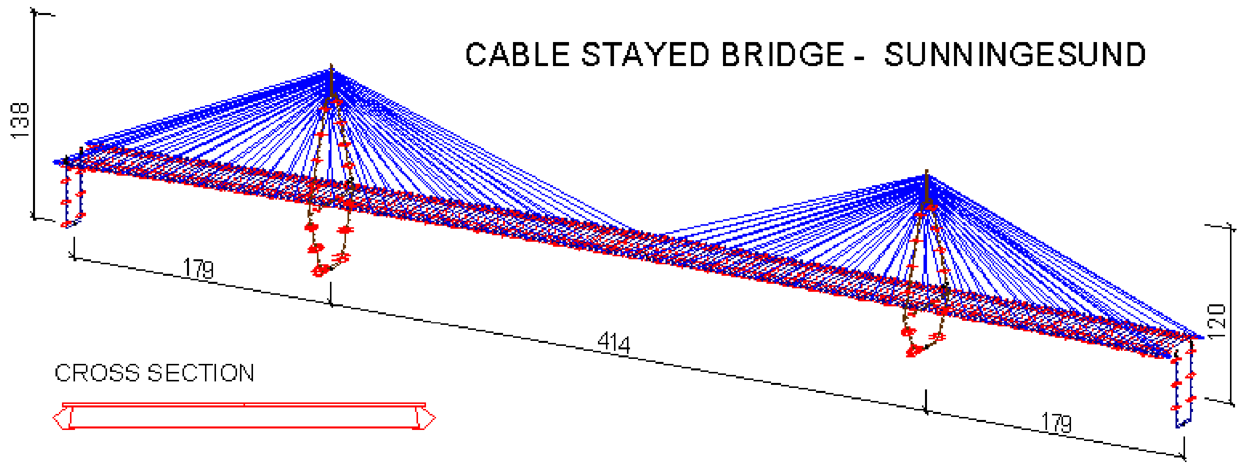

Fig. 8 Calculation model of the Uddevalla Bridge

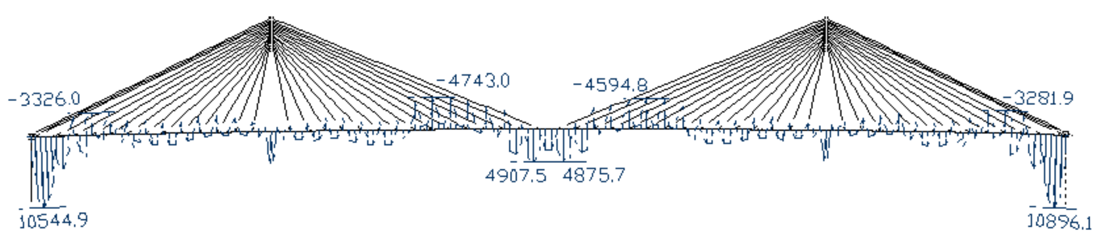

Fig. 9 Ideal bending moment diagram

VERIGE BRIDGE

Another interesting example showing the applicability on concrete box girder bridges was the design done for the Verige Bridge in Montenegro. This bridge was planned to cross the Bay of Boka Kotorska. It consists of 7 spans including the main cable-stayed part and the adjacent spans with a total length of 981m. The main span between the two pylons is 450m.

Gradis, Slowenia, performed the project engineering work in close collaboration with TDV using the mentioned software package [5] including the above-described features. Static and dynamic analyses had been carried out for all construction stages as well as for the final stage. In addition to the already mentioned features, the program package fully allows for taking into account pre-stressing of the concrete cross-sections with actual tendon characteristics and arbitrary tendon geometry layout. P-delta effects had to be considered in the design calculation of the pylon. The analysis was performed for overall 81 construction stages.

Fig. 10 Artists view of the Verige Bridge

The cable-stayed bridge was to be erected by the cantilever method starting from both pylons in both directions until the approach-bridges were reached and a monolithic connection could be established after the closure of the main span. The slender concrete box cross-section of the girder is shown in Figure 11.

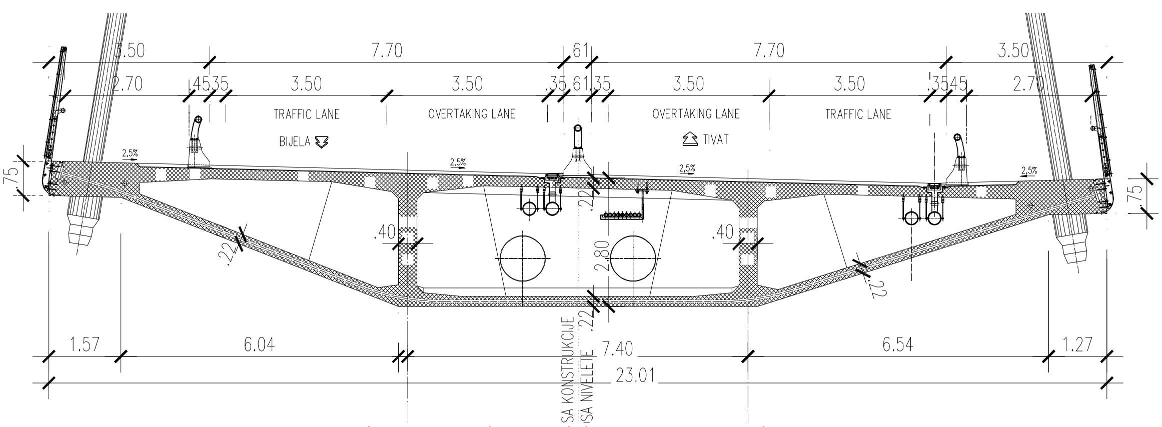

Fig.11 Verige Bridge, cross section [4]

The length of the individual cast-in-situ segments was 5.0 m. Every second segment was connected with a stay cable.

Fig.12 to Fig.14 show the bending moment diagrams for different construction stages, including all the non-linear time-dependent effects, in the last iteration cycle of the cable stressing optimization procedure.

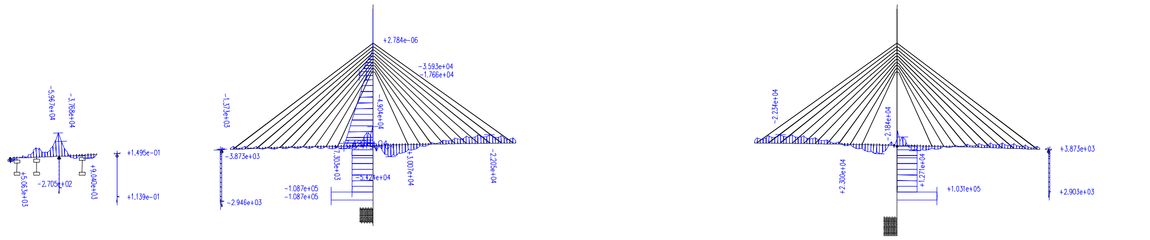

Fig.12 Construction stage 42 after stressing, bending moments in the superstructure

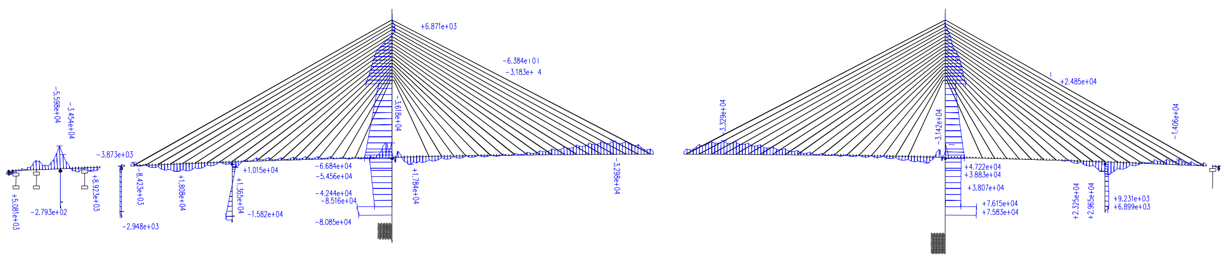

Fig.13 Construction stage 71 after closing of the approach spans 2 and 3, bending moments in the superstructure

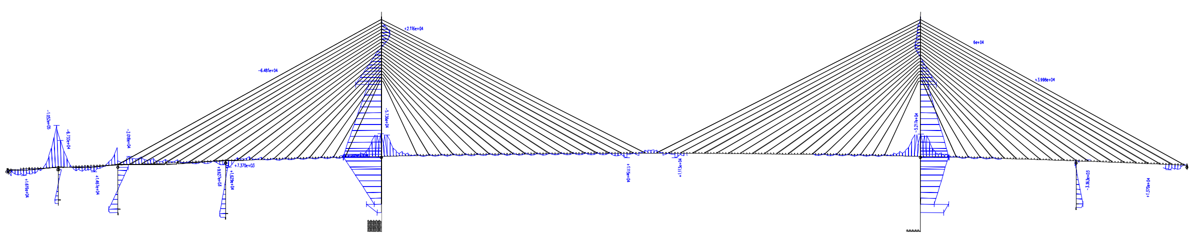

Fig.14 Construction stage 81 after applying permanent loads, bending moments in the superstructure & pylon: moments are as defined in the ADDCON Method

PRE-CAMBER AND ERECTION CONTROL

A further major problem in stage-wise construction techniques is the accurate consideration of deformations arising during erection in order to achieve at the end the intended design state of the bridge. This task requires a structural engineering tool, which gives the building contractor and consultant the ability to determine the effects – in terms of both deformation and force – of constraining the structure into a certain pre-defined position in any stage of construction.

A precise structural analysis in construction stages is required to seriously control the erection geometry. In the design process however, the optimization of internal forces under permanent loads is the primary engineering task and the choice of erection situations are steps to reach it. The analysis is performed with a given starting geometry - usually the structure in its desired final geometry. Pre-camber is the geometry of the bridge required for assembly to reach the final geometry under given loading conditions.

Significant displacements occur during the construction of bridges. They essentially depend on the erection sequence. In order to obtain the desired end-geometry of the bridge, these displacements are often compensated for with pre-camber and specific fabrication shapes of girder components. As the deformed structure is the start position for any new segment; full account of the current location in space must be taken for the analysis. This functionality has been fully provided in the used software package as described in detail in [7]. The stress-free fabrication shape is therein applied as a loading, acting on the currently active structure.

Fig.15 Prefabricated Segments waiting for Erection

Since the fabrication shape is applied as loading, this function can also be used to control and optimize the forces and displacements in the structure. Structural assembly with the option to automatically correct the kink at the segment face can accurately simulate that each newly active element is fully constrained with face-to-face connection to the currently active structure in its displaced position. It is also possible to control face-to-face connection by changing the fabrication shape of new segments or to use kink correction on the connection face.

Engineers may find, during the bridge construction, that the current position of the segment after installation does not correspond with the expected position. The expected position from the design calculation is a postulated position, assuming that the material behavior and length of time taken for construction up to that time are exactly in line with the assumptions.

USAGE OF CONSTRAINTS

In classical analyses it is assumed that elements are connected to common nodes. The common nodes consist of chosen degrees of freedoms (like displacements, strains, stresses or forces) and neighbor elements share all nodal DOF's. In reality, the structural segments are connected "face to face" and common nodes are only used to discretize the structural system. This concept works excellent if no changes occur in the "face to face" connection between the segments, free lengths between cable anchorage points and the segment geometry.

The difficulties arise immediately if “face to face” connection is manipulated during the construction or if the segment geometry or the total displacement of the structure is deviating from the design state. On the one hand, the unassembled segments are not fitting into displaced geometry of the structure any more and on the other hand, the engineer has to find and optimize necessary future correction steps in the erection procedure in order to come close to the designed set of forces and displacements in time infinity.

$$\left\{\delta^{I}_{Elem}\right\} = \left\{\delta^{I-1}_{Elem}\right\} + \left\{\delta^{I-1}_{Node} - \delta^{I-1}_{Elem}\right\} + \left\{\delta^{Kink\_Correction}_{Elem}\right\} \quad (7)$$

Due to such changes in geometry the computer model of the structure based on “common nodes” must be updated as shown in eq. 7. It is obvious that in consequence traditional program codes based on common node displacement cannot be used any more. Element stiffness properties, including both linear and non-linear geometric terms, have to be updated [8], and the local coordinate system of structural elements (defining local forces) is changed. The displaced geometry of the already assembled structure together with the geometry of the segments going to be assembled in the next erection step has to be geometrically updated.

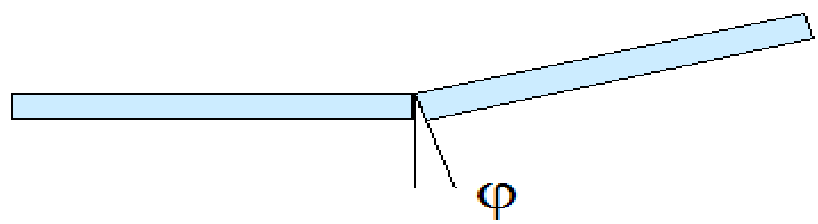

Fig. 16 Kink correction between structural segments

If the stress free geometrical shape of the new assembled elements does not fit into the geometry of already assembled structure and the structural system is fully or partially constrained, this will result in additional forces and stresses. This is an indication that additional equipment, such as a hydraulic press, is necessary to assemble those elements on site.

PRINCIPLES OF THE ERECTION CONTROL MODULE

The main purpose of the erection control module is to accurately control the position of the segments in a bridge structure built with using the stage-by-stage construction method. The procedure is repeated iteratively until the convergence criteria are achieved. Using the erection control facility, the stress free fabrication shape is applied as a load to the structure. In general, applying a stress free fabrication shape of a segment to the currently active structure will produce forces, stresses and deformations. The special case where the stress free shape exactly matches the position of the existing structural parts (i.e. is in accordance with the precamber calculation) will produce no forces stresses and deformations.

The program provides various procedures for compensating the error and applies the correction to subsequent construction stages – on a smear basis – spreading the error compensation to the pre-camber over all the subsequent construction stages up to the end of the construction.

APPLICATION EXAMPLE

The below application example is the Kanawha River Bridge in West Virginia. When complete in 2010, the 760-foot main span of this bridge will be the longest concrete span in the United States. Finley Engineering Group supplied engineering assistance in the design phase, and will provide full construction engineering services during erection of this bridge.

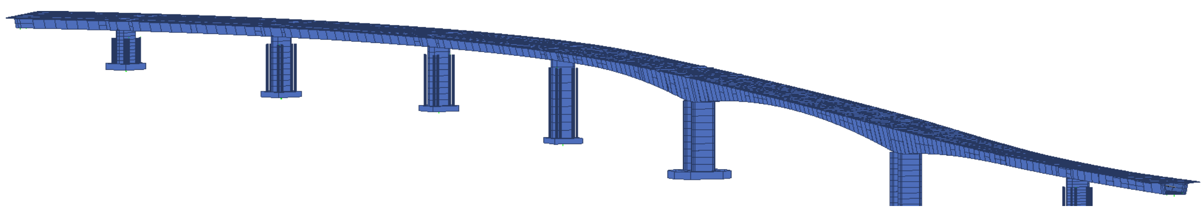

Fig. 17 Calculation model of the Kanawha River Bridge in West Virginia

The project design includes the construction of seven piers – five on land and two on the edge of the river. In addition to the 760-foot main span over a navigational waterway, the bridge includes 460-foot and 540-foot side spans and five additional approach spans ranging from 144 feet to 295 feet.

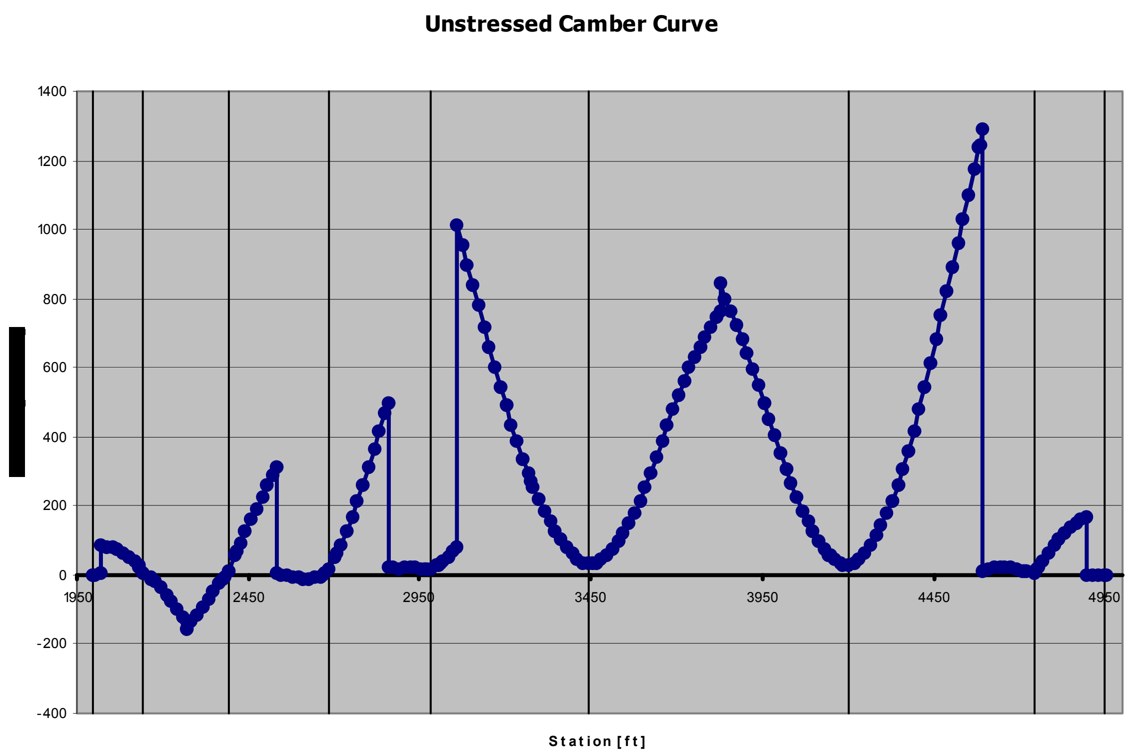

Fig. 18 Typical camber curve for the Kanawha River Bridge

Full camber analysis has been performed for the Kanawha River Bridge for different construction schedule variants investigated in the design process. The resulting design camber line for the most favorable variant is shown in Fig. 18. The mathematical model is kept active during erection and will be used for erection control analyses and defining any required compensation measures in case of deviations from the theoretical curve.

CONCLUSIONS

The design process of segmental concrete bridges requires a lot of sophisticated calculation tasks in addition to the standard analysis based on a traditional approach. These include detailed modeling of the erection process, accurately considering the creep and shrinkage behavior, taking into account structural non-linearity, adopting optimization techniques and controlling the deformation behavior throughout the erection process. Using independent specialized tools for these tasks is tedious and error-prone due to multiple data transfer.

Combining all these tasks in a single program for comprehensively modeling bridge structures and the construction sequence can considerably improve and accelerate the design procedure. A method to find the optimal tensioning strategy for the construction of cable-stayed bridges has been especially addressed. The paper describes the respective optimization method called “AddCon Method”, and explains how non-linear and time-dependent effects can be included. The Uddevalla Bridge and the Verige Bridge serve as practical examples.

Pre-camber and erections control is a further major topic in the design of segmental bridges. Including the respective functionality in a general design tool even allows for not only using the once established mathematical model in the design phase but also in the construction engineering phase. This allows for accurately knowing the fabrication requirements very early in the overall process and for taking the appropriate measures if any deviations occur during construction.

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