

CHAPTER 4: THE CONSTRUCTION PROCESS OF SEGMENTAL BRIDGES

The following Chapter 4 presents the important techniques for erection of concrete segmental bridges. Their characteristics are outlined so that understanding of the specific nature of each of these methods can be achieved. Apart from that this chapter deals with the most important issue of construction loads by distinguishing the various types of construction loads and showing their relation to the erection method used for a specific project.

4.1 DEVELOPMENT OF PRESTRESSED SEGMENTAL BRIDGES

Application of prestressed concrete for bridge construction was developed by French engineer Eugène Freyssinet, as described in Section 2.1.6, and has spread widely thereafter. Only prestressing made the slender, long-span concrete bridges of today possible. The basic principle of prestressing is to induce an initial compressive force in the concrete that will balance tensile stresses that occur in the member under service conditions before any tensile stresses occur in the concrete and cause cracking. Menn (1990, p126) names the two methods of inducing these stresses in the structure:

- By imposed forces from reinforcing steel that is prestressed to a certain degree;
- By imposed “artificial displacements of the supports”, e.g. bearings.

The second method according to Menn (1990) is much less used because of high losses of the prestressing force due to concrete creep and shrinkage. Prestressing tendons that are used for the first method consist of high-strength steel and are fabricated as wires, strands, or bars (Nilson and

Winter 1986). For a continuous beam on several supports, most tension will occur in the lower fibers of the cross-section around midspan and in the upper fibers above intermediate supports. It is therefore most useful to place tendons in the locations where tensile stresses will occur in the structure under service. This thought naturally leads to the idea of implementing longitudinal tendons in the beam that are not simply straight but follow a curve from the top above supports to the bottom at midspan and back to the next support. In Balanced Cantilever Construction the top cables in reaching out from the cantilever base to support the cantilever dead load are called cantilever beam cables; the bottom cables in the middle of the span are called integration cables (Mathivat 1983).

Prestressed concrete, compared with normal reinforced concrete has a higher degree of sophistication and causes higher cost for labor and for the prestressing tendons; on the other hand it saves cost through more economical use of material. Only prestressing makes long and slender concrete spans possible at all.

4.1.1 Degree of Prestressing

Menn (1990) mentions that choice of the best prestressing profile for a certain project is not predetermined but is a task for the bridge designer. He further gives an overview of the degree of prestressing. Full prestressing is supposed to withstand all tensile stresses under service conditions. When “calculated tensile stresses in the concrete must not exceed a specified permissible value” (Menn 1990, p127), so-called limited prestressing is performed. The last and most common method is partial prestressed, which does not specifically limit the concrete tensile stresses. Still, calculation of “behavior at ultimate limit state and under service conditions” (Menn 1990, p127) must be calculated, also taking into account the normal reinforcement. The purpose of the normal mild reinforcement is the control and distribution of cracking. Because of the high prestressing force, less conventional reinforcement is needed in the concrete, and members can be thinner and lighter, leading to more economical structures. The reduced susceptibility to cracking gives prestressed concrete higher durability.

Some factors effectively contribute to initial and long-term reduction of the prestressing force. Immediate losses of prestress, also called initial losses, occur once the prestressing force is applied, after the concrete has been placed and cured. Loss of prestress needs to be anticipated during design. Long-term losses in concrete depend on its design mixture, curing, the environmental climate, and the member geometry. Textbooks give information on the reasons for prestress losses and provide many formulas to calculate their effect. The following Table 4-1 based on Barker and Puckett (1997, pp455-466) summarizes these effects:

Table 4-1: Influences Causing Loss of Prestressing Force

| Initial loss of prestress | Long-term loss of prestress |
|---|--|
| Slippage of strands in the anchorages (before wedges or nuts grip firmly) | Relaxation of steel strands (loss of stress under constant strain) |
| Elastic shortening of concrete member (relieves previously stressed tendons) | Creep of concrete member (plastic deformation under constant stress) |
| Friction between tendon and duct interior (“wobble effect” because of curved ducts) | Shrinkage of concrete member (volume change due to evaporation) |

4.1.2 Pre-Tensioning

Prestressing basically can be carried out as pre-tensioning and post-tensioning, referring to the time when the prestressing force is imposed with respect to casting. In pre-tensioning the tendons are anchored to e.g. a stiff frame around the casting bed and are prestressed before the concrete is placed. When the concrete has gained sufficient strength the tendons are relieved from their anchorages and stress the concrete through bond between steel and concrete. Menn (1990) notes that this method is especially feasible for precasting concrete elements because of the solid anchorages required.

4.1.3 Post-Tensioning

Post-tensioning denotes the method of stressing the tendons only after the concrete has reached a specified strength. To allow for the necessary movement of the tendons inside the concrete they

are installed in tendon ducts that are made from steel or polyethylene. The ducts need to be fixed to the normal reinforcement to prevent misalignment during casting. After post-tensioning the ducts are filled with cement grout under pressure for and protection against corrosion of the tendons. Grouting the ducts will introduce bond between the steel and the surrounding grout. Unbonded post-tensioning is less common. Very similar to prestressing tendons are the techniques used for protection of stay cables of cable-stayed bridges against corrosion, as described e.g. by Funahashi (1995).

Two different ways of construction exist for post-tensioning. The prestressing tendons can be located either inside the concrete or outside of it. External post-tensioning has the advantage of easy accessibility for inspection, maintenance works and replacement. Nevertheless problems with corrosion protection are the reason for use of interior post-tensioning in most projects.

Post-tensioned tendons need special anchorages that are cast into the concrete structure. Anchorages have the shape of cones that are sitting on the end of the duct for better accessibility to single tendon strands with the prestressing jack. Anchorages are mostly surrounded by spiral reinforcement, which serves to distribute the compressive stresses into the concrete member. Small wedges around each strand or nuts (Menn 1990) fix the strands to the front plate of the anchorage. Special anchor blocks, so-called blisters are cast into the structure to provide enough space for the anchorages, e.g. on the inside of box girder segments of the second generation (Podolny and Muller 1982). Previously, tendon anchorages were also found in the joint faces, where problems with accessibility occurred. Textbooks on prestressed concrete structures provide more information on the layout and calculation of prestressing systems.

4.2 CONCRETE BRIDGE ERECTION TECHNIQUES

Concrete segmental bridges have already been introduced in Section 3.6.1. The following sections will present the important methods that are used in erecting concrete segmental bridges nowadays and the equipment employed. Special focus is put on constructability issues, pertaining

to characteristics and requirements, advantages, and disadvantages of each method to prepare for the case study that is presented in Chapter 5.

4.2.1 Cantilevering Method

Before used in construction of concrete bridges, the cantilevering method had already been used in Asia for wooden structures of earliest times, as Podolny and Muller (1982) report. Amongst the major steel structures that were erected with the cantilevering method are the Firth Rail Bridge and the Quebec Bridge that are presented in Section 2.1.5. Erection of concrete bridges with the cantilevering principle led to development of specialized sequences that are discussed further below.

As already introduced in Section 3.6.2, cantilevering for concrete segmental bridges is a construction method where segments, either precast or cast-in-place, are assembled and stressed together subsequently like a chain to form the self-supporting superstructure. Prestressing cables located in the upper part of the segment cross-section support the cantilever. In the variant of the progressive placement method stay cables are often used to support the cantilever prior to closure of the span.

Time-dependent material behavior of the segments under successive load steps requires comprehensive calculations for all construction stages. Every segment will develop strength with increasing age of the concrete. Governing for the structural behavior of the cantilever is that every segment carries and transfers loads from all following segments and construction loads until closure of the span. From these very basic facts in conjunction with geometry and expected loads on the structure the calculation of moments and local stresses, as well as calculation of the deflections that they cause is possible. Optimization of geometry, prestressing, and camber are then performed.

Depending on the specific segment configuration and erection sequence chosen for the cantilevering method the cantilever may never be exactly balanced so that the superstructure needs to be balanced to ensure stability. It is possible to fix the supports at the piers of

cantilevering superstructures and install vertical prestressing tendons. Furthermore it is very common to make use of an additional temporary pier with vertical prestressing that is located close to the permanent one (Casas 1997). This pier helps withstanding overturning moments from unbalanced load cases on the bridge superstructure.

Several advantages have contributed to the success of the cantilevering method. Certainly the most important one is that no falsework or centering is required, leaving traffic under the spans widely unobstructed during construction. Access from the ground is only necessary for construction of the piers and abutments and in preparation for the start of cantilevering, which starts from these locations.

Only relatively little formwork is required due to the segmental nature of the superstructure. Cantilevering is a very feasible method if the bridge spans are too high above ground for e.g. economical use of falsework, and if the terrain under the spans is otherwise inaccessible or unfeasible, being e.g. a deep gorge with danger of flood events. Especially in these cases rapid construction can be achieved with cantilevering.

Fletcher (1984, p13) notes that especially in cantilevering “complete calculations are required for the construction stage[s] and these are complicated as many stressing effects are time-dependent.” In addition to this, the influence of stepwise construction needs to be considered. However, the statical system that needs to be analyzed is rather simple and in case of the cantilever prior to closure at midspan even statically determinate.

4.2.1.1 Precast Construction

Precast construction means that bridge members or segments are prefabricated at a location different than the site, transported to the site, and installed there. Mathivat (1983) gives the maximum economical span of bridges built in precast segment as about 150 m, since cost for the placement equipment increase considerably the longer the spans are. Construction with precast segments has several advantages in comparison with cast-in-place segmental bridges. Casting of the segments can be performed under controlled, plant-like conditions at the precasting yard.

This industrialized process allows easy quality control of segments prior to placement in the superstructure and saves money through reuse of the precasting formwork. Surface finishing works, such as texturing, sandblasting, painting, and coating can be performed on the ground level without scaffolding when the segments are still accessible from all sides prior to installation in the superstructure.

Another major advantage mentioned by Mathivat (1983, p212) is that the complete casting of the superstructure can be removed from the critical path of the overall construction schedule, since superstructure “segments can be precast during construction of the substructure.” Assembly of the bridge superstructure takes much less time than cast-in-place construction, as precast segments do not need to cure on site before being prestressed together. Through the early casting of segments material properties are also influenced positively. As segments are usually stored at the precasting yard or on site for a while the concrete will have gained more strength until installation than cast-in-place elements have when being loaded. The time-dependent effects of concrete shrinkage and creep will occur with reduced extent because of the increase age of the concrete segments (Mathivat 1983) and will cause smaller deflections of the superstructure than with cast-in-place construction.

However, cost for the precasting yard, storage, transportation, and installation of precast segments needs to be evaluated in comparison with cost for the form travelers for cast-in-place construction to achieve an economical solution.

The precasting yard requires investment in equipment. Adjustable formwork to form the bridge geometry and alignment needs to be installed. Lifting equipment is also required to put the segments into the storage area and later load them on truck to be hauled to the construction site.

It is common practice to use the match-cast method to achieve high accuracy in segment prefabrication. Match-casting means that the segments are cast in the formwork between a “bulkhead at one end and a previously cast segment at the other” (Levintov 1995, p46). Segment joint faces need to be clean of any dirt for match-casting.

Levintov (1995) distinguishes concrete segment prefabrication into short-line casting and long-line casting. Short-line casting would comprise formwork of the length of only one segment; with

the previously cast segment being moved into position for match-casting on a mobile carriage. Short-line casting can be carried out in the horizontal position or with the segments tilted facing upward (Podolny and Muller 1982), however, the normal horizontal position facilitates match-casting. The overall bridge alignment requires careful adjustment of the formwork prior to each concrete placement. Short-line casting does not take much workspace.

Long-line casting on the other hand means erection of formwork for about a complete bridge span. According to Levintov (1995) the formwork can be erected stationary for the superstructure soffit only, with smaller movable forms for web sides and interior formwork. This formwork will be cheaper than the flexibly adjustable formwork for short-line casting, but will require much more workspace. Levintov cautions that the long-line casting is feasible for straight superstructures or superstructures with constant curvature. Segments are match-cast progressively on the long-line formwork by step-by-step advancement of the movable formwork units and a movable bulkhead.

Phipps and Spruill (1990) describe the precasting cycle that was used in construction of the Biloxi Interstate I-110 viaduct. According to them, the freshly cast segments were steam cured in a movable shed covering the casting bed of the short-line formwork. The pretensioning strands were released by cutting them, quality control and testing of concrete samples was performed, and internal formwork units were removed from the new segment. After lifting the previously cast segment from its position for match-casting into the storage area, the new segment was rolled out of the formwork. It was positioned for match-casting according to the required overall alignment. Cleaning of the joint face and the bulkhead was done prior to casting the next segment. Reinforcement bars were preassembled in reinforcement cages to speed up placement. Pre-tensioning strands were used in the box girder segment, being stressed prior to concrete placement. After concrete placement and consolidation with vibrators the segment was screeded and given a surface finish before the curing shed was set up over the casting bed. With the sequence described a casting cycle of one superstructure segment per day could be achieved. In the final superstructure post-tensioning cables were installed to stress the precast segments together.

Precast segments have joints that require special attention. An epoxy agent is usually applied to the joint faces shortly before putting a segment into its location in the superstructure. Joints are usually only a few millimeters thin. Podolny and Muller (1982) explain the functions of the epoxy agent that is applied to the joint faces when placing precast segments. During segment placement the epoxy serves to lubricate the joint faces, which are cleaned by sandblasting and “compensate for minor imperfections in the match-cast surfaces” (Podolny and Muller 1982, p485). In the finished structure the hardened epoxy seals the joints against moisture and thus additionally protects the tendons in their ducts. Furthermore, the epoxy is able to transmit compressive forces and shear forces. Information on mixing, handling, and properties of the two main ingredients, the epoxy resin and the hardener, is provided by Podolny and Muller (1982). Interestingly, the epoxy agent can reach a higher final strength than the concrete itself.

In addition to the epoxy transmitting shear forces between segments the joint faces are given a special shaping to transmit shear. So-called shear keys are cast into the joint faces to lock the segments together. They transmit shear forces and also help in exact alignment of the segments during assembly. Segments of the so-called second generation facilitate many smaller shear keys that are located not only in the box girder webs, but also in top and bottom flanges (Podolny and Muller 1982).

4.2.1.2 Cast-In-Place Construction

Podolny and Muller (1982) provide an example for a typical casting cycle. As outlined in Section 3.6.3.1, any previously cast segment needs to have developed at least the specified strength to be prestressed to previous elements and support the subsequent one. After finishing all work on a segment the form traveler is detached from the previous position and moved forwards on rails that are mounted on the bridge superstructure. In order to remain balanced during advancement the form traveler may be equipped with a counterweight. Upon arrival at the new position it is adjusted and anchored to the existing superstructure at its rear to be able to withstand overturning moments that will occur from the weight of new concrete. The external formwork is cleaned and aligned to the required geometry of the next segment, also incorporating the desired camber.

When the form traveler has thus been prepared the reinforcement and tendon ducts for bottom slab and webs will be installed and connected with the previous ones. In cast-in-place construction it is possible to have continuous mild reinforcement in the superstructure, whereas in precast segmental construction only the longitudinal prestressing tendons will cross the segment joints.

Reinforcement can be pre-assembled into cages that are lifted into place by crane. Prestressing tendons are already inserted into their ducts prior to placement of concrete because of better accessibility. After these preparations, concrete is placed. Accessibility of the bottom part of the box girder may require that the bottom slab is cast before internal formwork for webs and top slab is advanced and aligned. After curing sufficiently for strength and durability, the tendons in the newly cast concrete segment can be prestressed. Finally, the casting cycle starts all over again to cast the next segment.

Concrete placement can be carried out by various means, e.g. with buckets that are hoisted by crane, or by pumping. While placing the concrete in lifts care needs to be taken that no segregation of the concrete mixture occurs, and that proper consolidation will be achieved. The most common method is to vibrate the concrete in the formwork by means of internal or external vibrating devices. Most important for the quality of the concrete is curing to achieve strength and durability. Upon gaining enough strength, the tendons in the newly cast segment will be stressed to some degree and the cycle starts all over again. Based on Mathivat (1983, p201) an overview of casting steps and typical values for their duration is given in Table 4-2. It should be noted that the sequence of steps given in this table is only a generic example and would be broken down into more steps for planning an actual construction project:

Table 4.2: Typical Duration of Casting Steps

| Duration | Activities |
|-----------------|--|
| 1 day | Post-tensioning tendons in previous segment Stripping formwork Advancing form traveler |
| 2 days | Placing reinforcement, ducts, and tendons |
| 1 day | Placing concrete for bottom slab, webs, and top slab |
| 3 days | Curing concrete (including Sunday) |

Mathivat (1983) also gives information on means of accelerating this process. Use of special form travelers “with lateral main beams or self-supporting carriages” (Mathivat 1983, pp201f) will leave the bottom slab widely unobstructed and make construction easier. Secondly, “increasing the length of segments” can be considered during the design phase while keeping in mind the increasing weight and cost of bigger form travelers. Stepwise construction of the box girder is also possible, with relatively simple formwork for the top slab following a few segments behind the main form traveler. With this method less concrete has to be placed in one single step of concrete placement. Yet it requires careful structural design of the vertically segmented superstructure. Finally, an example of a segmental bridge is given for combination of cast-in-place and precast segment sections. Webs of the superstructure box girder of the Brotonne Bridge in France were precast and placed into the form travelers, which were used to fabricate the remaining cast-in-place parts of the cross-section (Mathivat 1983).

4.2.1.3 Balanced Cantilever Construction

Balanced cantilever construction denotes building a bridge superstructure from both sides of the pier table in a scales-like fashion. This erection method is also known under the name free cantilever construction (Podolny and Muller 1982). Fletcher (1984) gives information that the pier table element, serving as a base from which cantilevering is begun, is usually between 6 and 12 m long. In order to balance the weight of both arms of the cantilever superstructure the segments will be about equally placed at both ends. Actual placement of new segments will hardly proceed exactly at the same times as Mathivat (1983) expresses. Therefore the pier can undergo overturning bending moments and needs to be designed accordingly. Temporary towers with vertical prestressing or counterweights can provide additional support. Figure 4-1 schematically shows a typical construction stage in Balanced Cantilever Construction.

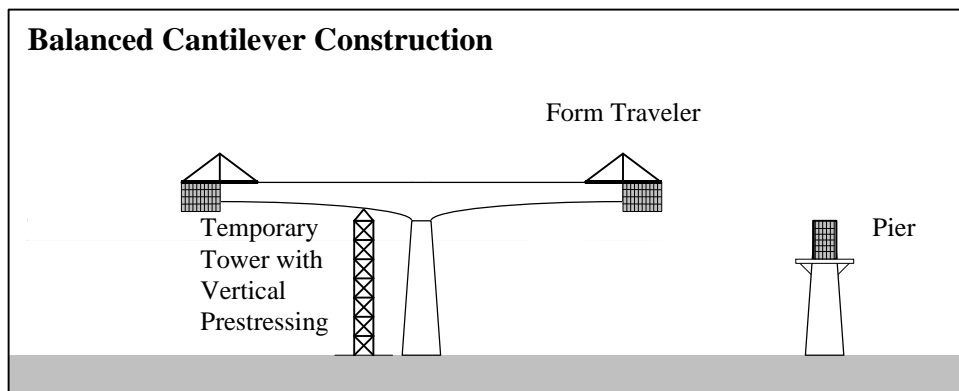


Figure 4-1: Balanced Cantilever Construction

Balanced cantilevering can be carried out with cast-in-place or precast segments. For cast-in-place balanced cantilevering a set of two form travelers is required, one for each arm of the cantilever. For multi-span bridges the form travelers can be dismantled after finishing cantilevering from one pier and can be set up for new use on the next cantilever.

In case of a bridge with variable box girder depth the pier table segment will be the most massive segment of the superstructure. This segment needs to be constructed prior to cantilevering to provide a working platform from which the two form travelers can start. It also includes diaphragms that facilitate the flow of forces from the cantilever arms into the piers. Because of size, geometry, and construction separate from the rest of the superstructure the pier table segment will take a considerable amount of time to construct. It can be put into place either with large precast segments or as cast-in-place with formwork mounted on the pier shaft.

An interesting pier design specifically feasible for cantilevering is mentioned by Fletcher (1984), who points out that a pier consisting of transverse twin walls is advantageous as it provides stability for cantilevering but allows horizontal movement of the superstructure from thermal elongation through flexing of the wall panels.

4.2.1.4 Progressive Placement Method

The progressive placement method, in comparison with the balanced cantilevering method, is a one-directional process as shown in Figure 4-2. All cantilever segments are subsequently placed at the tip of a cantilever that is built across all spans. Both cast-in-place and precast segmental construction can be used. Often stay cables from the tip of a temporary tower on the superstructure support the cantilever. With growing cantilever superstructure this support mechanism has to be advanced. Another method of support is use of temporary towers, which are mentioned in Section 4.2.3.3. According to Mathivat (1983) this method is competitive for spans between 30 to 50 m in length, whereas incremental launching and balanced cantilevering are also used for much longer spans.

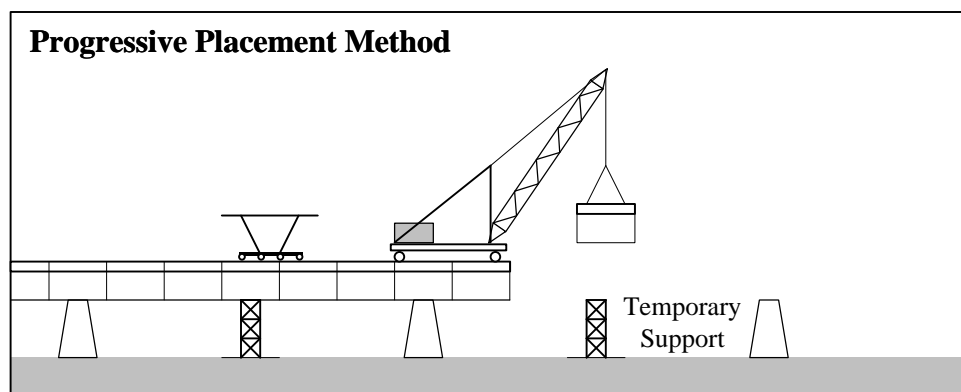


Figure 4-2: Progressive Placement Method

Progressive placement has several advantages, as Mathivat (1983) points out. First of all, the placement process does not have to switch sides as it occurs in the balanced cantilevering method. Thus process control is simplified. In addition to this, good access to the placement location is given on the already completed part of the bridge superstructure. With the progressive placement method horizontal curves can easily be accommodated.

From a structural point of view the progressive placement method is advantageous in substructure design. Only vertical forces from the dead load of the superstructure under

construction are experienced. In comparison with incremental launching and balanced cantilevering, a simpler flow of forces takes place between superstructure and the piers. No horizontal forces are introduced in the piers and no unbalanced bending moments have to be withstood by the piers. It is therefore possible to immediately install the permanent bearings (Mathivat 1983).

Some disadvantages of the progressive placement method need to be dealt with during design and construction. As construction only progresses at the tip of one cantilever, progress is slower than in balanced cantilevering. Progressive placement resembles incremental launching in that the superstructure undergoes stresses very different from the permanent service conditions, including even stress reversals. In both cases the structure needs to incorporate temporary prestressing tendons to account for these stresses. Mathivat (1983) also points at the difficulty in erecting the first span with progressive placement. Other construction methods may have to be employed for this stage.

4.2.1.5 The Linn Cove Viaduct

An interesting example for use of the progressive placement method is the Linn Cove Viaduct (Anon. 1984), shown in Figure 4-3. Its location in an environmentally sensitive area in North Carolina, the inaccessibility of the sloping site at the mountain face, and the highly curved alignment of the viaduct provided a set of difficult conditions for this project. Match-cast prefabricated segments for the highly curved viaduct were delivered by truck directly to the end of the already completed part of the structure, where they were placed and attached with temporary thread bars and tendons. Deviating from the method outlined above, no cables were used above the spans, but steel bents were installed under the spans during construction to provide support. Only drilling so-called microshaft piles for the pier foundations needed to be done directly on ground, as the piers themselves also consisted of precast segments that were craned into position from above and post-tensioned vertically. Implementation of the progressive placement method in connection with precast pier segments that were lowered into place from

above, called top down construction, helped protecting the natural environment at the site as far as technically possible.



Figure 4-3: The Linn Cove Viaduct, North Carolina, U.S. (taken from Rives 1997, p41)

4.2.1.6 Concluding the Cantilevering Process

The cantilevering process will finally have reached its end when both girders meet at midspan and need to be connected. Three different ways exist to achieve this connection in the structural system (Mathivat 1983). A hinged connection can be installed that allows horizontal movements in the superstructure. As Mathivat (1983) writes, this system is structurally relatively simple, yet the hinges are complicated details and the overall structural redundancy of the system is reduced. Podolny and Muller (1982) also mention the lower ultimate load-carrying capacity of the hinged system and the higher susceptibility to creep and relaxation phenomena. Furthermore, the two superstructure halves can have a slight angle between them as deflections occur, which is detrimental to “the appearance of the bridge and the user’s comfort” (Podolny and Muller 1982, p36).

Secondly, part of the midspan superstructure can be designed as a suspended span sitting on bearings between the cantilevers. In this configuration the deflection angle between the shorter cantilevers and the suspended span will be much smaller, and “differential settling of the supports” can better be accounted for (Podolny and Muller 1982, p38). Still, the connections require special details in the structural system.

Finally, the whole superstructure can be made continuous at midspan. Achieving this statically indeterminate system is the most common way in building cantilever bridges for several reasons. Mathivat (1983, p41) names specifically that the deflections in the stiffer continuous superstructures are “indeed far smaller than those met in hinged structures” and both visual appearance and drivers’ comfort are better than in hinged superstructures. He also notes the necessity for expansion joints in very long continuous multi-span superstructures and advises to provide expansion joints about “300 to 600 m apart” (Mathivat 1983, p45) in points of small moments in the superstructure. Horizontal movements of the bridge superstructure can be accommodated “by the flexibility of the piers themselves, or by using elastomeric bearings or sliding supports” (Mathivat 1983, p45).

Continuity is generated by casting a closure segment into the gap at midspan, through which continuity tendons, the so-called integration cables as mentioned in Section 4.1 run in the bottom part of the box girder (Mathivat 1983). Prior to casting this segment, misalignments of the two superstructure halves are corrected with hydraulic jacks. It should, however, be tried to keep these additionally imposed stresses small by paying close attention to the correct alignment including camber when casting the superstructure halves. Additionally, as mentioned in Section 3.6.2.7, the two girders are often jacked apart to compensate for future effects “of long-term creep and shrinkage of the superstructure on the substructure” as Matt et al. (1988, p37) report. They further mention casting the closure segment of their bridge project at night to avoid problems from temperature gradients in the superstructure. For casting and curing of the midspan closure segment the girders need to be fixed in their position. Finally, continuity tendons can be inserted into the newly cast segment and post-tensioned. Upon closure of midspan internal stress redistribution takes place, shifting the moments from the supports more towards midspan. The formerly free cantilevers are now restrained in deflection and rotation. Podolny and Muller

(1982) provide a sample calculation for the effect of stress redistribution with consideration of time-dependent effects.

4.2.2 Cantilever Erection Equipment

Different erection equipment is used in bridge construction. With the specific focus on cast-in-place and precast cantilever segmental bridges, form travelers and launching girders will be introduced in the following sections. Other equipment that can be used for placement of precast segments is e.g. cranes.

4.2.2.1 Form Travelers

Cast-in-place cantilever construction requires formwork that is attached to the tip of the growing cantilever for casting. The following paragraphs will deal with girder cross-sections only. As the cantilever grows the forms travel are set forth in steps. These form travelers give shape to the segment, support the weight of the newly cast concrete until it has gained enough strength to be post-tensioned to the previous cantilever segments, and transfer the segment weight to the already existing superstructure. Determining for the capacity of the form travelers is the maximum size and initial weight of the biggest segment in the bridge superstructure, including other construction loads.

Form travelers available in today's construction industry are made by specialized manufacturers and are reusable and very flexible (Levintov 1995) with respect to changing geometry of the bridge superstructure and its alignment, including camber. They can be enclosed in a heated tent to enable concrete placement and curing to proceed during adverse weather conditions, especially low temperatures. In comparison with a precasting yard, form travelers often offer the less costly solution, since transportation and storage of prefabricated segments is avoided, and they integrate all the functions of the precasting plant into a relatively small device. Fletcher (1984) notes that by use of form travelers the formwork is reused several times, while adjustments to variable

segment geometry, especially depth, remain possible with relatively little effort. Disadvantages of cast-in-place cantilevering with form travelers are discussed in Section 3.6.3.1. Figure 4-4 shows a typical view of a form traveler.

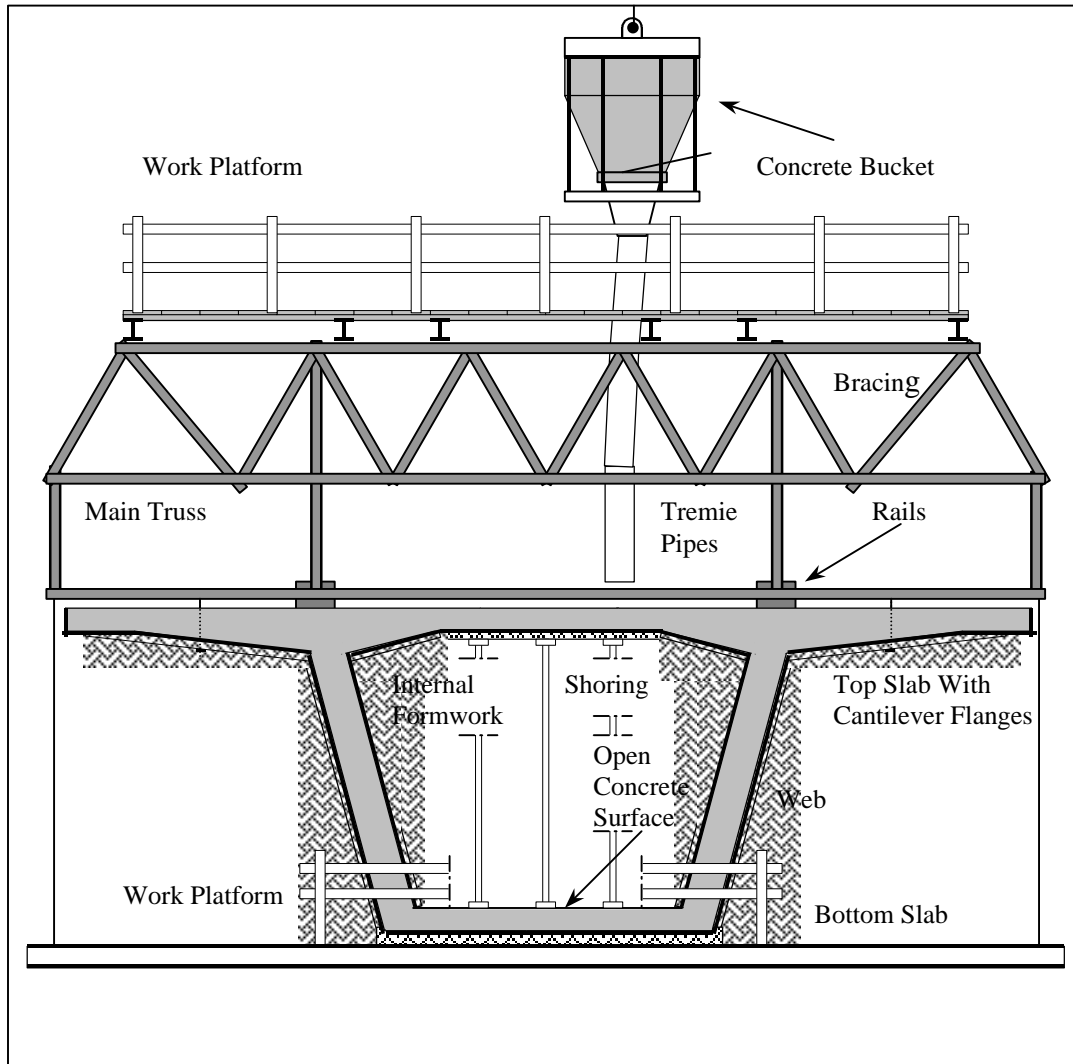


Figure 4-4: Form Traveler Front View

Form travelers consist of a sufficiently stiff steel frame to which form panels for the box girder segments are attached at the front. According to Levintov (1995, p43), the steel frame is mostly composed of two parallel “diamond- or triangular-shaped frames that are connected and stiffened

by diagonal bracing and transverse trusses at the upper front and rear.” Wheels allow longitudinal movement of the form traveler on rails on top of the superstructure. For stability of the traveler it can be held down by a counterweight at its rear end (Mathivat 1983). After advancing, the traveler is anchored down to the cast parts of the superstructure with tendons at its back, as Levintov (1995) writes. Thus, the overturning moment from the load at the traveler tip can be resisted. It is mentioned that for structural reasons the form traveler main beams are located above the webs of the concrete box girder, so that construction loads can be transferred into the main load-carrying system of the bridge directly. The longitudinal main beams of the form travelers need not necessarily be located above the webs. Form travelers are also used in configurations with the main beams in a lateral position, leaving the bridge deck free (Podolny and Muller 1982). Mathivat (1983) further distinguishes so-called self-supporting assemblies, where the stiffening effect of the form panels contributes to the stiffness of the whole form traveler.

Suspended from the traveler are not only the adaptable forms for exterior and interior of the concrete segment, but also working platforms on different levels that can be accessed from above.

4.2.2.2 Launching Girders

Apart from various types of cranes that can be used to place precast segments, launching girders are widely used for this purpose. Levintov (1995) mentions limited access under the cantilever and great height of bridge superstructures above ground as reasons why launching girders would be used. They are very feasible for bridges with several spans, as due to their length they can be advanced over gaps that are still to be bridged. During construction they are moved forward on rails whenever a major part of the bridge superstructure has been completed.

Launching girders, also called launching gantries, are large trusses that are placed longitudinally on the bridge superstructure. One or more movable crane devices for transportation of the precast

segments can be attached to them, running along the chords of the girders. Precast segments are delivered to the girder by special heavy-duty vehicles.

If launching girders are built of high strength steel their weight can be reduced considerably. At the same time, however, larger deflections occur that are limited by additional support of the girders with a king post system with stay cables (Mathivat 1983).

Launching girder trusses can have triangular or rectangular cross-sections and can be constant in depth or higher towards the middle. They can be disassembled into parts that are connected with high-strength friction bolts (Mathivat 1983) for transportation, very similar to tower crane booms.

Most launching girders are overhead trusses that have three leg supports. The three legs are called rear, central, and guide leg. Some of these legs, often the guide leg, are not permanently fixed to the girder to allow the advancing movement as will be described below. Very often these legs form a bent above the superstructure, leaving space for the precast segments that are turned 90° sideward to be moved through the gap. Pivoting the whole launching girder around the rear support leg (Mathivat 1983) accommodates bridge superstructure curves in the horizontal plane.

A major feature of launching girders is their length in comparison with the span length of the bridge superstructure. Levintov (1995, p45) writes that launching girders composed of “single or double trusses may range from slightly longer than a span length to slightly longer than twice a span length.” Erection sequences for these two extremes shall be briefly described in the following paragraphs.

4.2.2.2.1 Launching Girder Slightly Longer Than One Span

Construction of the bridge superstructure with a launching girder about a span long is performed as follows. After advancement the girder rests with its rear leg on the cantilever tip at midspan and with its center leg on the next pier. Around this pier new segments will be placed with balanced cantilevering, filling the remaining half-span behind the pier and advancing the

cantilever to the next midspan. Afterwards, the next pier table segment is placed and the cantilever advances half a span so that it comes to rest on guide leg and rear leg. The guide leg will remain on the pier as the girder advances further. When the central leg arrives at this pier and the rear leg is at midspan the next placement position has been reached. Figure 4-5 shows the construction sequence with a launching girder that is slightly longer than one span.

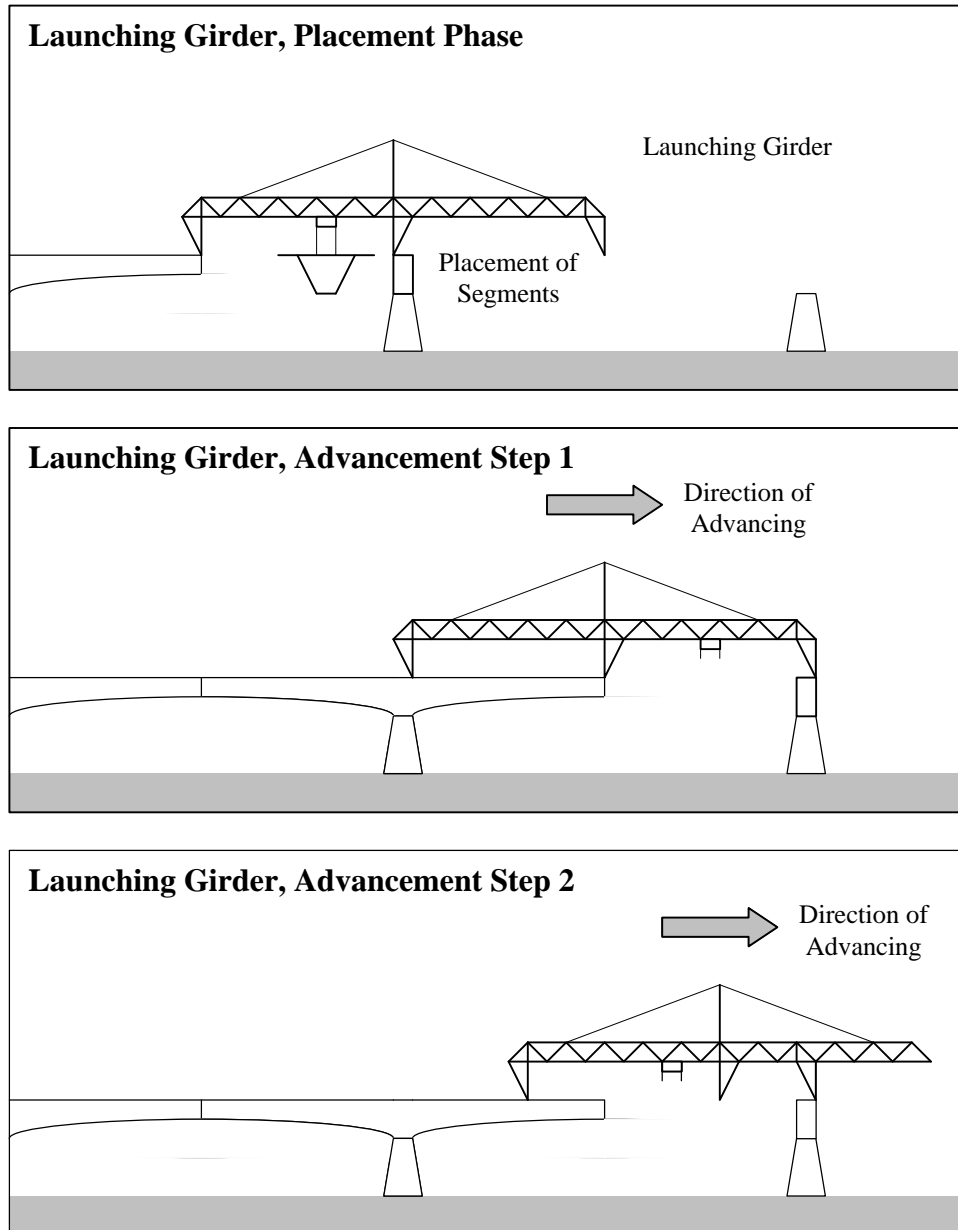


Figure 4-5: Working Scheme of Short Launching Girder

A major drawback of this method is found in the previous description. The bridge superstructure will take considerable loads during construction, since the heavy launching girder rests with one leg at the midspan cantilever tip.

4.2.2.2.2 Launching Girder Slightly Longer Than Two Spans

Construction of the bridge superstructure with a launching girder about two spans long does not incur the aforementioned detrimental load condition. At all stages the load-carrying girder legs will be located above piers. This is shown in Figure 4-6. In the normal placement position the launching girder rests with its rear leg above a previous pier and with its central leg above a free pier. It is easily possible to also support it at the guide leg once the third pier table has been placed. Placement of the segments will then proceed on both sides of the pier table in the middle of the girder. To speed up construction, the girder can be equipped with two crane devices to place segments on both sides simultaneously. After the remaining gap in the superstructure has been closed and the cantilever has grown into the next span, the girder is advanced one complete span. During advancement the girder rests on the guide and central leg that remain on the newly finished pier and the one that lies ahead. This second way of employing launching girders is the more recent technique (Mathivat 1983).

Even longer launching girders have been used in construction, as reported by Mathivat (1983). Due to the long span required for launching girders and the mechanical parts, such as crane and advancement devices, launching girders can be quite costly. If possible, launching girders should be adaptable for reuse or should be rented. Launching girders can reach lengths of more than 150 m, depending on the requirements of the bridge spans, and weights of up to about 400 t (Mathivat 1983). In addition to these specialized, expensive and heavy pieces of construction equipment it is also possible to use simple lifting devices that are located at the cantilever tip. In the case of the Linn Cove Viaduct, which has been presented in Section 4.2.1.4, a derrick was mounted to the bridge superstructure that placed segments as they were delivered by truck (Anon. 1984). Other types of deck-mounted equipment are imaginable and mentioned by Levintov (1995, p44), e.g. “a longitudinal beam fitted with lifting tackle and winches.”

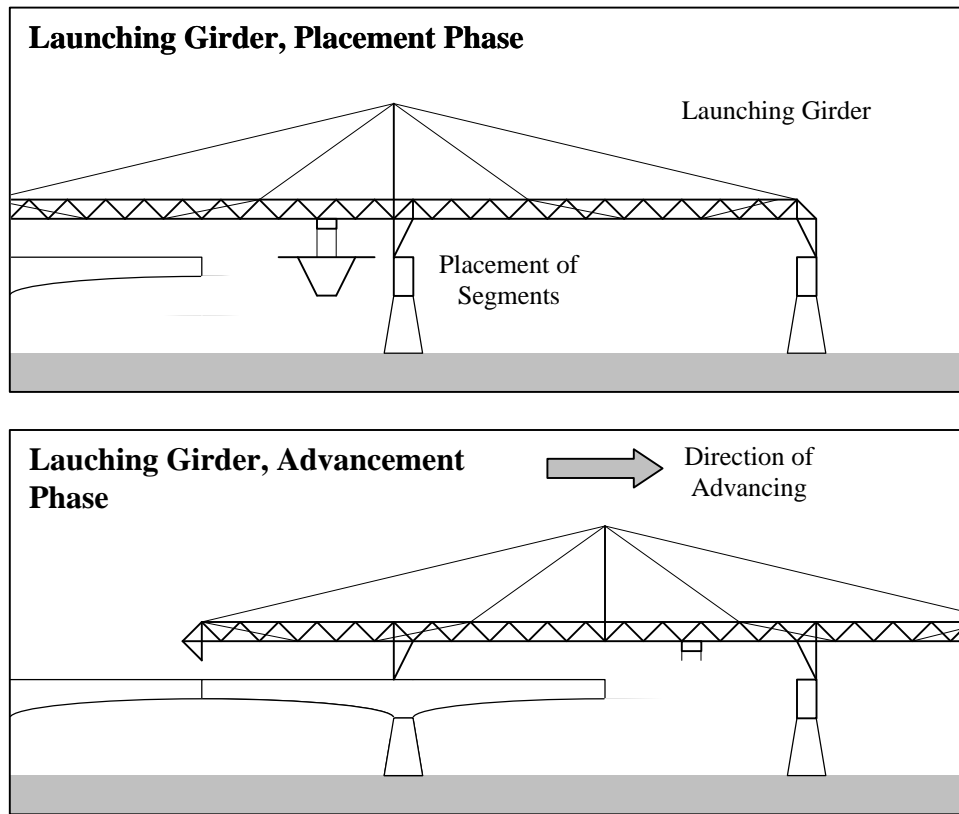


Figure 4-6: Working Scheme of Long Launching Girder

4.2.3 Incremental Launching

Incremental launching was developed by the German engineers Fritz Leonhardt and Willi Baur for the Rio Caroní Bridge in Venezuela (Podolny and Muller 1982), which was built from 1962 to 1964. The incremental launching technique, as opposed to other methods presented in this chapter, consists of casting a continuous chain of segments at one particular location on site and then pushing the growing superstructure out over site to be bridged. A casting bed with adjustable formwork for the superstructure segments is set up. This casting bed can also be enclosed in a heated tent so that controlled casting and curing conditions are achieved. The normal cycle time, regardless of segment length is one week. Segment lengths according to Liebenberg (1992) typically range between 15 and 30 m.

Two different techniques for launching the bridge superstructure from the casting bed exist. Hydraulic jacks can pull the superstructure with steel rods, as it was done for the Rio Caroní Bridge (Podolny and Muller 1982). The second, more common method is to employ a pair of hydraulic jacks acting vertically and horizontally. Continuous repetition of lifting the superstructure off the abutment and then pushing it forward as far as the jack allows will achieve the launching in incremental steps. Figure 4-7 shows the launching process schematically. Podolny and Muller (1982) caution to design the jack capacity for more than the usual friction coefficient of 2 % because of imperfections that can occur during construction.

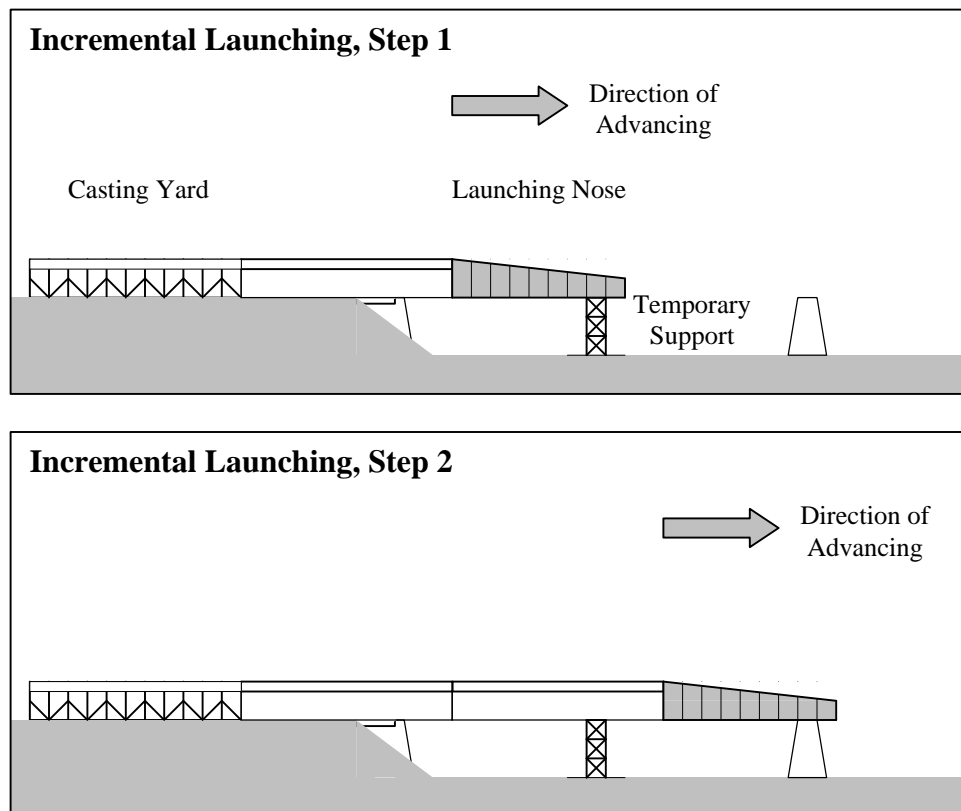


Figure 4-7: Incremental Launching

In front of the cantilevering superstructure a lightweight steel launching nose is attached with tendons that reaches the next support before the bridge superstructure itself arrives. Its purpose is

to keep the bending moments in the superstructure smaller. Mostly the launching nose has a length of about 60 % of the bridge spans (Podolny and Muller 1982). Another way of reducing the bending moments is to implement temporary towers between the bridge piers. These towers need to be able to take the horizontal forces that arise from launching.

On top of all supports, including abutments, piers, and temporary towers temporary sliding bearings are installed during construction that will later be replaced with the permanent ones. Stainless steel plates are installed on the bearings. While the superstructure is advanced, Neoprene pads coated with Teflon and reinforced with steel plates are inserted between concrete and steel to reduce friction (Liebenberg 1992). Very low friction coefficients of 2% or less can be achieved with this method.

Several advantages make incremental launching a very competitive erection method. As with any cantilevering method it leaves the site below completely unobstructed during construction. Only for very long spans temporary towers or cable stays from above as supports are needed. Except for these the equipment necessary is reduced to the jacking mechanism, the adjustable stationary casting bed, and temporary sliding bearings, all of which may possibly be reused, which reduces the capital investment considerably. Podolny and Muller (1982) furthermore mention the cost savings due to avoidance of segment transportation and heavy construction equipment. They also point at less maintenance cost due to the higher prestressing of the superstructure. The controlled casting and curing conditions allow steady and quick construction progress.

Bridges that are erected with the incremental launching method should, according to Podolny and Muller (1982), have a constant cross-section, especially in depth, and have a straight superstructure. It is possible to accommodate small variations in alignment and horizontal and vertical curvatures provided that they have a constant radius. Close control of the bridge geometry during casting and launching is very important. Sloping grades at the bridge site are also accommodated, in this case “the launch is usually in the downward direction”, more than 2 % slope would require a retarding mechanism to stop the movement of the superstructure (Liebenberg 1992, p165).

Liebenberg (1992, p164) also gives a very clear statement of the main difficulty of the incremental launching method: “During launching, the section undergoes complete stress reversals as it progresses from a cantilever to the first support and thereafter over the following spans to its final position.” Clearly, this erection sequence generates a bending moment envelope in the structure depending on the span lengths that needs to be accounted for in designing the cross-section properties and the amount of reinforcement and prestressing tendons. The stresses due to the aforementioned high bending moments require much longitudinal prestressing both at top and bottom of the cross-section. Another disadvantage is the large workspace that is needed for the casting bed at the abutment and the adjacent storage areas (Podolny and Muller 1982).

The Aichtal Bridge in Germany that was built mainly between 1981 and 1983 serves as a good example of the incremental launching method. According to Basse et al. (1985) this bridge with its total length of 1,161 m is the longest one ever built with incremental launching. It crosses two valleys at a maximum of 48 m and 50 m above ground, respectively. A fixed bearing is located at the pier between the two valleys. At the same location a second jacking system was installed for use in later construction stages.

The normal pier spacing for the 21 spans of the Aichtal Bridge is 51 m, reaching a maximum of 80 m and 84 m respectively at the deepest parts of the valleys. These wide spans required use of temporary towers that were braced with stay cables from the ground to resist the horizontal forces from launching. The whole bridge superstructure consists of two parallel single cell box girders that are 3.50 m deep, 5 m wide at the soffit and carry 13.50-m wide decks. After completion of one girder all construction equipment was relocated for the second box girder.

An enclosed 25.50-m long casting bed with adjacent assembly yard for the reinforcement cages was erected behind the abutment with the launching jacks. For winter construction work another 50-m long tent with large heaters was set up for proper curing and the piers were built with thermally insulated climbing formwork. Casting of segments was done in a weekly cycle. Both longitudinal and transverse limited prestressing was implemented. Basse et al. (1985, p23) give information on the sequence of casting steps that is compiled in Table 4-3:

Table 4-3: Sequence of Casting Steps for Aichtal Bridge

| Day | Activities |
|------------------------|--|
| Monday | Post-tensioning, stripping of formwork, incremental launching of segment |
| Tuesday | Placement of reinforcement cages for bottom slab and webs, Installation of ducts and tendons, installation of interior web formwork |
| Wednesday | Completion of formwork installation, concreting of bottom slab and webs |
| Thursday | Removal of web formwork, installation of interior top slab formwork, Placement of top slab reinforcement, ducts, and tendons |
| Friday | Concreting of top slab |
| Saturday and Sunday | Curing of concrete |

Launching of the cured and post-tensioned segments required many personnel for supervision of all sliding bearings. An overall longitudinal slope of the bridge reduced the jacking forces necessary for launching. An overall curvature with radius 1,500 m in the horizontal plane and a constant 3.5 % cross slope of the bridge superstructure induced stresses due to restrained deformations during launching. Furthermore, the changing span lengths had to be considered in coming up with the prestressing program to optimize the tendon profile and prestressing forces.

A surveying program had been prepared to control geometry during all construction steps. Several fixed surveying points were located along the site. Overall, tolerances for deviations from the planned bridge geometry during casting and launching were less than 1.5 mm (Basse et al. 1985).

4.2.4 Falsework

Falsework has been used in construction since ancient times, when Roman bridge builders erected their semicircular stone arches for bridges, aqueducts, and vaults on wooden centering. Falsework provides continuous support for the formwork that gives shape to the superstructure. In most cases falsework is used for cast-in-place concrete structures. It requires firm, relatively even ground on which it can be erected.

Apart from custom-built timber structures a wide range of steel elements and modules for falsework is available in the construction industry. Liebenberg (1992) gives a range of up to 300 m in length and about 10 m in height for bridges to be built with this method. He also specifically points at the necessity for stable foundations of the falsework, sufficient bracing of the falsework structure, and consideration of the deflection of the falsework in the overall superstructure camber.

Falsework, either stationary or traveling, can also be configured as casting girders that hold the formwork into which the concrete is placed. In that, these girders resemble the erection girders of the span-by-span method, which are used to assemble precast segments. Liebenberg (1992) further distinguishes the girders depending on their location to the bridge superstructure as overhead or supporting it from below, or combination of both. He also reminds that use of major pieces of equipment, such as casting girders needs to be considered carefully because of the high capital investment that is necessary.

4.2.4.1 Stationary Falsework

Stationary falsework is the simplest method of erecting the bridge superstructure. Advantages mentioned by Liebenberg (1992) are that stationary falsework can be erected by less specialized workers to any desired shape. Use of modern standard elements of which the falsework is put together allows uncomplicated erection. There are, however, several disadvantages related to stationary falsework. A lot of material is required for stationary falsework, which requires much time and manual labor to be spent on its erection, in addition to the cost of purchasing or renting the materials themselves. Therefore, Liebenberg (1992) concludes that it needs to be erected some spans in advance to keep up with rates of placement of concrete that can be achieved.

Today, falsework is competitively used e.g. for the complex alignments of highway interchanges, as Cassano (1987) reports for the example California. He states that apart from the feasibility for the highly curved superstructures, use of major falsework systems also allows placement of very large volumes of concrete at the same time and thus speeds up construction. Cassano (1987)

mentions that all falsework in California has to be designed and built according to the Falsework Manual of the California Department of Transportation and is reviewed and inspected by qualified engineering personnel.

4.2.4.2 Traveling Falsework

Traveling falsework alleviates some of the problems associated with stationary formwork. This type of falsework, including formwork, is assembled to larger units that can be moved to the next span to be cast. As with stationary falsework, this method requires the site to have relatively level and firm ground to allow movement of the falsework on a wheel assembly. In case the ground conditions are less favorable, using the span-by-span method might be advisable, where the superstructure segments are assembled on erection girders. Span-by-span erection is introduced in the following Section 4.2.4. Traveling falsework is shown in Figure 4-8.

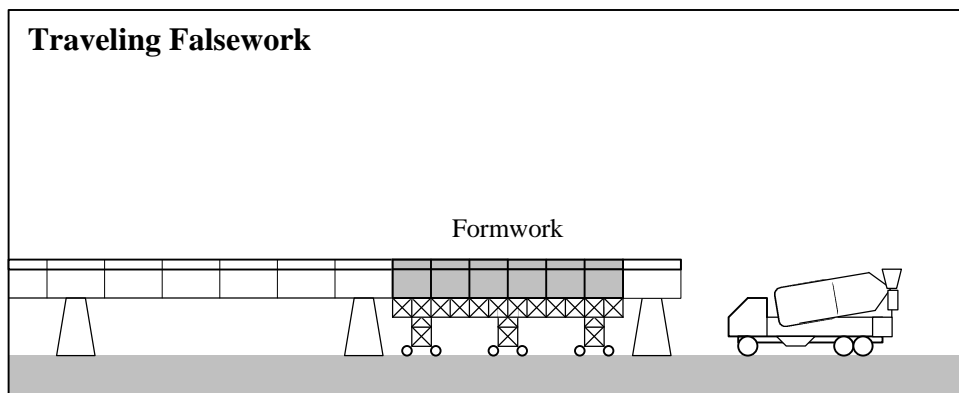


Figure 4-8: Traveling Falsework

4.2.4.3 Temporary Towers

A kind of falsework used frequently in construction is the use of temporary towers to support the superstructure that is under construction at intermediate positions. These towers can e.g. be used

in the balanced cantilevering method to stabilize the structure against tipping over from construction loads. They can be additionally strengthened with prestressing rods to withstand the forces that are imposed on them during construction. Other applications are found during incremental launching, where together with the launching nose they serve to keep the range of bending moments small. Instead of using temporary towers as supports Liebenberg (1992, p156) also mentions stiffening girders “by means of prestraining with a king post and adjustable inclined ties.”

4.2.5 Span-By-Span Erection

Levintov (1995, p47) lays out the characteristics of the span-by-span erection method as assembling all segments for a span in a set, which is “then aligned, jointed, and longitudinally post-tensioned together to make a complete span.” The principle of span-by-span erection is shown in Figure 4-9. Span-by-span erection is typically limited to bridges that consist of box girders with constant depth. The actual construction can have several variants, the segments can be assembled on the ground and lifted in place as a group by a heavy-duty crane or they can all be put into their final position on erection girders along the spans to be completed. The second method was e.g. used for constructing the Biloxi Interstate I-110 Viaduct. In this project different types of erection girders were used. The authors report that on some spans triangular trusses were implemented, and on the other hand steel box girders came into use, which left more clearance for traffic underneath (Phipps and Spruill 1990). Erection girders were supported at their ends “by steel falsework resting on the footings at each pier” (Phipps and Spruill 1990, p130). After completion of a span the erection girders were set forward to the next span and adjusted. By then, the precast segments for this span had already been supplied and would be lifted in place by crane. Fine adjustments of the segments on the erection girders were possible by means of variable individual supports. Finally, post-tensioning would be performed to link all the segments together to form a complete span. The structure became self-supporting after casting of the closure joints with the pier table segments had been done and the longitudinal prestressing force was induced. With the method described, an erection speed of one span per about 3.5 days could be achieved (Phipps and Spruill 1990).

Erection girder need not rest on the ground, but can also be supported by already existing substructure or superstructure, e.g. the piers of a span that is to be constructed. Project specific design of substructure and superstructure and considerations as e.g. for traffic clearances set the boundaries for erection with erection girders.

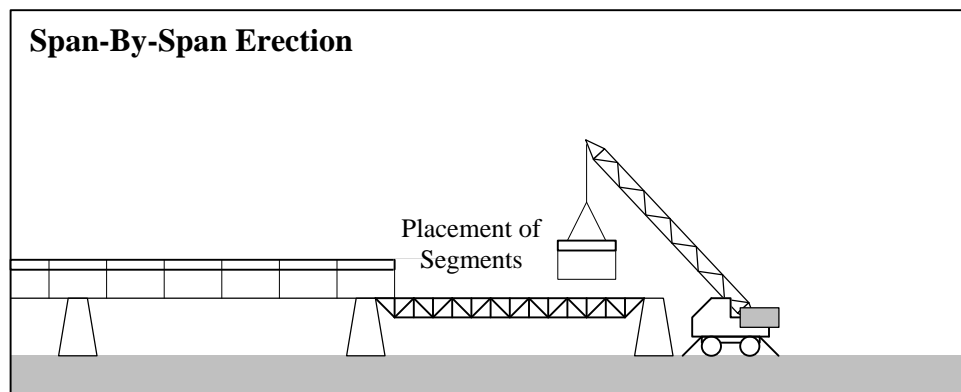


Figure 4-9: Span-By-Span Erection

4.3 CONSIDERATION OF CONSTRUCTION LOADS AND STRESSES

The following sections deal with the relationship between construction loads and the stresses that these induce in structures and the structures themselves while they are under construction and still awaiting completion. The central issue for all considerations is structural safety, meaning failure against structural failure. The generic concept of resistance R that is greater than the most unfavorable combination of load S that induces stresses has been introduced in Section 3.5.2.

Codes require that all construction influences will be properly taken into account during design. Even a professional code applicable for bridges (ACI 1995, p51) in its Section 5.3 only points out that “Consideration should be given to temporary loads caused by the sequence of construction stages, forming, falsework, or construction equipment and the stresses created by lifting and placing precast members.” It assigns the responsibility for the construction scheme, which imposes stresses on the structural members, to the contractor. It is further pointed out that

the stability of precast members and prestressing should be taken into account. “Environmental loads... should be considered during construction using an appropriate return period or reduced severity. Lower load factors may be used to account for the acceptability of higher temporary stress levels” (ACI 1995, p51). The aforementioned load factors are provided in tables in the same code.

How the actual process has to be accomplished in detail remains the structural engineer’s task. Close cooperation of designer and contractor in development of the construction sequence contributes to quality.

For better understanding of the relationship between structures under construction and actions influencing them it is useful to get a clear definition of the technical terms related to this topic. The nature of *actions* has been explained in Section 3.5.3 as any loads or restrained deformations that can cause stresses within the structural system. The term *construction loads* should in this context be understood as the broader sense of any *actions* that occur during construction of the structure prior to normal service conditions.

Then again, the concept of construction loads needs to be extended by consideration of the uncompleted structure during construction, which may not have reached full resistance to imposed actions.

The term *erection method* denotes the physical means of putting the foundations, bridge substructure, and especially its superstructure into place. Every type of erection method requires certain equipment and site installations to carry out the work tasks. The erection methods goes along with specific limitations imposed on the flow of work tasks to be scheduled, e.g. that bridge piers have to be finished prior to begin of incremental launching of a bridge superstructure.

The *construction sequence* is the specific succession of work tasks for one particular project developed under consideration of the erection method chosen for its conditions and restraints for economic construction.

Construction stages are notable steps within the progression of work tasks from the initial site operations startup to the finished structure. These steps can be distinguished by the appearance of the structural system, loading conditions, or other factors.

Load steps can be defined as specific sets of loads on a structure in its current construction stages. These load steps or load cases are combinations of actions that are anticipated to occur at the same time with a certain probability, and are incorporated into analytical calculations with partial factors of safety.

4.3.1 Types of Construction Loads and Influences

Generally, construction loads are by nature of relatively short duration in comparison with the overall planned service life of a structure. Construction loads may influence a structure over a brief time only, e.g. from equipment or material for a new segment that is temporarily stored on an already completed part of the superstructure of a bridge. Construction loads can affect the structure in very unfavorable conditions, e.g. when a crane is located at the tip of a cantilever to place segments. Thus resulting stresses in the structure can even exceed stresses due to permanent and dynamic loads under service.

Real loads are generally distinguished into two classes, dead loads and live loads. During construction the structure has to carry its own weight, its dead load, and the superimposed dead loads of bridge parts that are not structurally important but necessary for service, as e.g. the bridge furniture, the so-called accessories.

Many different live loads influence the structure. In most of all cases, live loads are idealized either as uniformly loaded areas on the superstructure or as singular loads from larger pieces of equipment. Live loads can result from erection equipment, e.g. the launching nose in incremental launching and lifting devices placed on the structure such as cranes and launching girders. Forces are also imposed on the structure through restraints from fixed bearings during construction, e.g. on piers for cantilevering. Along with these structural details, the boundary conditions can still change; e.g. considerable settlements can occur when the soil is initially loaded. Temporary

supports, e.g. additional temporary towers for long spans or stay cables also generate stresses in the superstructure. Another factor to be considered is the prestressing tendons that are installed in the concrete members to withstand stresses during construction and under service. More longitudinal forces can be caused by the erection itself, e.g. horizontal jacking forces during incremental launching. Formwork and supporting installations, e.g. the forms and frames in a launching girder also impose live loads, as well as the fresh concrete that it carries.

Finally, environmental influences also create changing loads on the structure, e.g. wind, snow, and temperature gradients. Extreme events, such as floods, storms, and earthquakes can also hit a structure during construction and may need to be considered in the calculations. Apart from these Acts of God, accidents may happen. Podolny and Muller (1982) note that to prevent a scenario such as falling of a form traveler during cantilevering, inspections are necessary. They also note that critical fixtures, e.g. suspension rods that reach through the superstructure and anchor bars that hold the traveler, need to have a large safety margin and may be provided in double numbers. In general, “cast-in-place cantilever construction has established an extremely good safety record” (Podolny and Muller 1982, p482).

The aforementioned extreme load cases are mostly considered with a lower factor of safety than for service conditions (ACI 1995). The reason for this approach lies in the reduced probability of occurrence during the relatively short construction period in comparison with the total duration of service for which the bridge structure is designed. More explanation for this rationale lies in the fact that although the resistance is reduced during construction the structure produces less danger for the general public, as it has not been opened for traffic by then. Furthermore, the bridge under construction is under direct control of engineering personnel on site that can immediately take appropriate measures if necessary to ensure safety of further construction works.

When looking at the structure and its behavior during construction, a striking similarity with the model presented in Section 3.5.1 appears. In fact, when looking at any structure the four main elements are geometry of the structural system, structural details and restraints from boundary conditions, material properties, and loads. For analysis of how a structure behaves while being under construction the same elements need to be taken into consideration.

After having considered the loads and restraints, i.e. all actions, two elements remain to be discussed. The geometry of the structural system itself is developing continuously with progress of construction depending in a manner that depends on the erection method and needs to be considered. Incomplete structures, e.g. the cantilever beam prior to midspan closure for continuity are inherently weaker than in the finished state because of less structural redundancy. All these different construction stages need to be analyzed as a combination of load cases and the resistance of the structure at that construction stage for structural safety.

Apart from that structural resistance the material resistance may also be weaker than in the final state. Especially for cast-in-place segmental bridges the still young concrete usually has not developed its full specified strength when it is being prestressed and loaded with more segments for quick erection. The structural resistance and material resistance can also be understood as the two components of structural safety, namely strength and stability, as outlined in Section 3.1.3.

Summarizing, stresses induced by construction loads may be higher than those from service loads as the incomplete structural system is mostly different and weaker than finished structures, concrete has not gained full strength, and the boundary conditions may be different from the service state. In other words, the great importance of construction stages lies in the criticality that results from the still low structural and material resistance, while loads may be actually more adverse and boundary conditions different.

4.3.1.1 The Zilwaukee Bridge

Underestimating the construction loads and their effects on the unfinished structure caused several accidents and failures of bridge structures under construction in the past. An example for an accident of a bridge under construction is provided by Anon. (1988), who describes construction of the Zilwaukee Bridge in Michigan. The new bridge provided a replacement an old drawbridge on Interstate I-75 that was necessary because of growing traffic. According to Anon. (1988, p69), the Michigan Department of Transportation (MDOT) made plans for both

alternatives of a steel plate girder and a precast segmental concrete bridge “to foster competition and thus reduce the project cost.”

Construction of the concrete structure began in 1981 after a longer bidding process. The dimensions of the two parallel box girder superstructures are 2.4 km in length with a 38-m high navigation clearance and a width of 22 m. Balanced cantilevering under a launching girder was chosen for erection of the 1,592 precast segments. A heavy-duty truck delivered segments weighing about 110 to 145 metric tons on the bridge. They were placed with the 287-m long, 1,540 metric tons heavy launching girder. “Thus, during construction, the new bridge is subjected every day to far heavier loads that it will ever experience after it is opened to traffic” (Anon. 1988, p70).

During construction all expansion joints were blocked against movements “by inserting temporary high-strength concrete blocks”, which are fixed with tie-downs (Anon. 1988, p71). In late 1982 an accident occurred when these blocks failed in one of the joints, which caused the respective superstructure to tilt and sag at one end, damaging bearings, joints, and a pier footing that was forced out of the vertical. The subsequent investigation concluded that the main cause for this incident “was construction loads at the particular location that were too heavy” (Anon. 1988, p71). Difficult repair works were undertaken to realign and reinforce the pier at its base. Adjacent soil was temporarily frozen for stabilization and a major concrete counterweight was placed. Afterwards, pile holes were drilled and filled with concrete, on which the reinforced pier base was erected. “A structural steel frame was fabricated and erected on top of the new footing, and fitted with 12 hydraulic jacks” (Anon. 1988, p72) that should help realigning the superstructure. After new bearings and connecting cables to the counterweight had been installed the superstructure was brought back into alignment and construction work commenced again until opening in 1988.

4.3.1.2 *The West Gate Bridge*

A much more severe failure of a bridge under construction occurred in 1970 in Australia, when a 112-m long steel box girder span of the West Gate Bridge, crossing the Lower Yarra River in the Melbourne area collapsed during construction (Royal Commission 1971). The description of the circumstances that led to the total failure and loss of 35 lives reads like a thriller, i.e. like a compilation of how not to perform a bridge project. Again, the construction influences were underestimated.

In this case, the whole project throughout the time before the collapse had been plagued with errors and omissions in overall structural design, in detailing, and in preparation and checking of field operations, as the Royal Commission (1971) concludes from its investigation. According to their report, already the design lacked proper consideration of the unusual erection method that had been proposed. Safety margins for the box girder that was divided into *longitudinal* halves were insufficient even despite strengthening that was added after failure of the Milford Haven Bridge in Wales some months earlier. Even earlier, in 1969 the Forth Danube Bridge in Vienna had suffered major buckling in the lower parts of its steel box girder (Royal Commission 1971).

Lack of open communication between project participants, especially between the engineering consultants and the contractor's personnel, as well as other managerial disputes and strikes of union workers further contributed to the unhealthy atmosphere under which the novel structure was to be constructed.

Direct cause of the collapse was removal of bolts that connected the box girder halves in an attempt to straighten out buckling that had occurred. Matching the seams between the two halves of the steel box girder in the lifted position had already earlier proved to be very difficult, despite efforts of jacking them together and using heavy kentledges to match the camber lines of the spans. The asymmetric trapezoidal halves tended to bow out of shape when being jacked up at both ends. In its conclusion, the Royal Commission (1971, p97) once again stressed the need for continuous reviews, checks, and improvement particularly in bridge engineering:

“Engineers engaged on the design of major bridges cannot stand still. It is part of their duty, not only to their clients but to the community as a whole, to advance, to

develop new concepts of design, to adopt new methods of calculation such as the computer and to encourage the production and use of improved materials, as high tensile steels and pre-stressed concrete. (...) It is however necessary to emphasize that when leading designers are working as pioneers, only just within the bounds of the engineer's knowledge, some slight misjudgment, or failure to appreciate every aspect of a new problem may prove disastrous and bring fatal and tragic results. Under these conditions, it is more than ever necessary to employ really adequate margins of safety and to ensure that they are not eroded by various unexpected and accidental factors, including of course, imponderables and human fallibility.”

4.3.2 Influence of Erection Method and Construction Sequence

Typically, every erection method with its succession of construction stages brings about a characteristic stress development in the structure. Examples from different erection methods that are presented in more detail in Section 4.2 illustrated this point.

Wide use of segmental bridge construction, which is described in Section 3.6.1 makes it possible to clearly distinguish steps whenever a new segment is placed. Computer software is used to model these steps and analyze them to ensure that limit stress values are not exceeded at any time.

Goñi (1995) describes how calculations for the Chesapeake and Delaware Canal Bridge were carried out. This cable-stayed bridge was modeled with plane frame analysis software that accounted for the stepwise construction and time-dependent material properties as needed by the structural engineers. Input data that were used for processing are given in the following (Goñi 1995, p31):

- *“Material properties of each segment including their creep and shrinkage characteristics.*
- *Data of casting and erection of each box girder segment.*
- *Material properties of each tendon including area, modules of elasticity, relaxation characteristics, and the friction and wobble parameters that affect the force along the length of the tendon.*

- *Tendon dimensions and layout.*
- *Properties of each cable stay including area, modulus of elasticity, and location of their connections to the pylon and deck.*
- *Coordinates of the model nodes and definition of the segments.*
- *Definition of all the construction loads applied during erection.*
- *Definition of the support and restraint conditions of all of the structural elements.*
- *Definition at every construction phase of segments and stays to be assembled, tendons to be stressed, construction loads to be applied, and boundary conditions to be implemented.”*

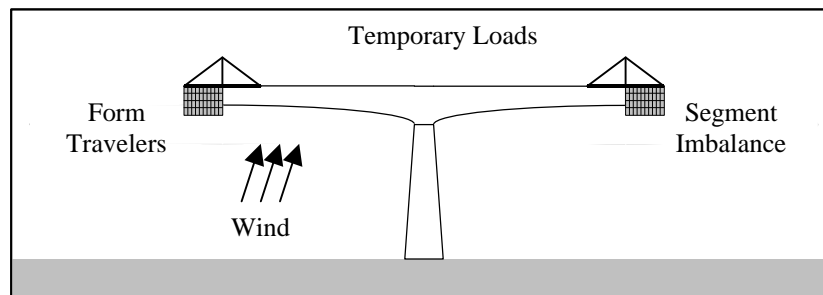
Incremental launching basically functions by incrementally pushing the superstructure from the shore where it is produced over the bridge piers to cross the obstacle. The tip of the cantilever with the launching nose will consecutively be free cantilevering in the spans and be supported by the piers. This erection method causes complete stress reversal in the girder, which makes considerable top and bottom prestressing necessary. If the range of bending moments from this launching process is depicted along the superstructure, a moment envelope results, which is e.g. provided by Basse et al. (1985) for the example of the Aichtal Bridge. Reinforcement and prestressing tendons need to withstand these stresses at the respective locations.

Balanced cantilevering needs to be safe against overturning moments from construction loads as described in the previous Section 4.3.1 until closure of span. The stresses within the cantilevering girder do not change their sign, but with advancement of the form travelers to cast a new segment or placement of a new segment the cantilever stresses will increase. Ways to alleviate danger of overturning are use of additional temporary towers with vertical prestressing to withstand vertical compression and tension from unbalanced cantilever arms, and temporarily fixing hinged bearings that are located on top of the piers. Mathivat (1983) gives an example of vertical prestressing within the pier table segment to rigidly attach the superstructure to the pier. The piers need to be designed strong enough to withstand bending moments that might occur from the most unfavorable combination of actions on the growing cantilever. A brief list of possible causes for overturning moment is taken from Mathivat (1983, pp159f) and adapted in Table 4-4 for easier reading:

Table 4-4: Causes for Cantilever Imbalance

| Cause for Imbalance | Example |
|------------------------------|--|
| Non-coincident work progress | Concreting or segment placement faster on one side |
| Construction inaccuracies | Weight difference between cantilever arms |
| Site temporary loads | Material stored on cantilever arm |
| Wind loads | Gales attacking the structure at an angle |
| Construction accidents | Falling of concreting or placement equipment |

Figure 4-10 illustrates some of the aforementioned causes for cantilever imbalance during construction.

**Figure 4-10: Causes for Cantilever Imbalance**

Falsework has characteristics very different from the previously presented erection methods. It provides the structure that is erected on it with approximately continuous elastic support. The deflection of the formwork and falsework itself as well as settlements of soil on which it is founded need to be considered in stress and deflection calculation. The superstructure camber needs to be adjusted accordingly.

4.3.3 Existing Codes and Regulations

A variety of codes and regulations have to be taken into account when designing a bridge. The following paragraphs give an overview of these codes and regulations.

4.3.3.1 American Concrete Institute

The American Concrete Institute (ACI) issues so-called “Committee Reports, Guides, Standard Practices, and Commentaries” (ACI 1995, p1) that are developed and updated by expert committees. Apart from the general building code ACI 318 (ACI 1999), the ACI 343R-95 “Analysis and Design of Reinforced Concrete Bridge Structures” (ACI 1995) is of prime importance for bridge structures. It covers functional requirements, including even a short section on bridge aesthetics, economics, and erection, materials, basic considerations for construction works, load cases, design aspects, characteristics of prestressed and precast concrete, substructure and superstructure, and reinforcement details. Initial sections on requirements for bridges and construction considerations provide a brief overview of topics related to this study. ACI 343R-95 also gives reference where applicable to other institutions that issue codes and regulations. ACI 345-82 “Standard Practice for Concrete Highway Bridge Deck Construction” is referenced.

4.3.3.2 American Association of State Highway and Transportation Officials

Another important organization is the American Association of State Highway and Transportation and Highway Officials (AASHTO). AASHTO issues the “Standard Specifications for Highway Bridges” (AASHTO 1996) and “Guide Specifications for Design and Construction of Segmental Concrete Bridges” (AASHTO 1998b), along with other manual on construction practices and specifications for bridge and highway projects (AASHTO 1980), (AASHTO 1985). Furthermore, AASHTO provides engineers with detailed information on sampling and testing of materials and their properties for use in transportation projects (AASHTO 1998a). Concrete and its ingredients, structural steel, steel bolts, metal alloys, reinforcement bars, soil, and timber are amongst the materials covered. In addition to these topics, testing equipment and procedures are also described in the comprehensive volume.

In particular, the “Guide Specifications for Design and Construction of Segmental Concrete Bridges” (AASHTO 1998b) have been developed with respect to the growing number of segmental bridges to provide information to bridge engineers. The very first section of this guide (AASHTO 1998b, p6) states that normally “the provisions of the Sixteenth Edition of the

AASHTO Standard Specifications for Highway Bridges (AASHTO 1996) are intended to apply to the design of segmental concrete bridges.”

These “Guide Specifications for Design and Construction of Segmental Concrete Bridges” provide information specifically devised for segmental bridge construction. The manual comprises two main divisions, dealing with design specifications and with construction specifications.

Section headings of Division I—Design Specifications include general requirements and materials, analysis, design, detailing, substructure design, and special provisions for bridge types, specifications, contract drawings, and alternate construction methods (AASHTO 1998b). In particular, the subsections under these sections briefly deal with analytical methods, provide loads and applicable load factors, deal with seismic design, provide information on stresses, prestress losses, flexural strength, shear and torsion, fatigue stress limits, and deal with the specifics of anchorage zones. Furthermore, constructive details, such as ducts, couplers, concrete cover and reinforcement spacing, bearings, expansion joints, and cross-sectional dimensions for box girders are covered.

Subsection headings of Division II—Construction Specifications include a variety of materials, such as concrete, mild reinforcement and post-tensioning materials, and guidelines on construction procedures that are specific to the various methods of segmental bridge construction, e.g. installation, stressing of post-tensioning tendons and grouting of their ducts, application of epoxy as a joint sealant, geometry control, shop drawings, bearings, construction tolerances, and repair of minor defects. The last subsections deal with cast-in-place segmental construction, precast segmental construction, and incremental launching individually (AASHTO 1998b).

The AASHTO Standard Specifications for Highway Bridges (AASHTO 1996) is a very comprehensive compilation that provides information for a wide range of issues pertaining to design of construction of all kinds of highway bridges. The two main divisions, titled design and construction, respectively, deal with structural systems of bridges, materials and their properties and testing as well as with details, such as prestressing tendons and anchorages, bearings, joints,

bridge decks, retaining walls, and railings (AASHTO 1996). Seismic design is covered in a separate section.

In particular, the AASHTO Standard Specifications for Highway Bridges provide information on the following actions that may affect the structure: Dead load, live load, impact or dynamic effect of the live load, wind loads, other forces, such as longitudinal forces, centrifugal force, thermal forces, earth pressure, buoyancy, shrinkage stresses, rib shortening, erection stresses, ice and current pressure, and earthquake stresses (AASHTO 1996, p19). However, no reference is given to erection stages in this context. Section 8.15.3 of Division II—Construction is dealing with construction loads and states the following (AASHTO 1996, p493):

“Light materials and equipment may be carried on bridge decks only after the concrete has been in place at least 24 hours, providing curing is not interfered with and the surface texture is not damaged. Vehicles needed for construction activities and weighing between 1,000 and 4,000 pounds [0.454 and 1.814 metric tons], and comparable materials and equipment loads, will be allowed on any span only after the last placed deck has attained a compressive strength of at least 2,400 pounds per square inch [16.55 N/mm²]. Loads in excess of the above shall not be carried on bridge decks until the deck concrete has reached its specified strength. In addition, for post-tensioned structures, vehicles weighing over 4,500 pounds [2.041 metric tons], and comparable materials and equipment loads, will not be allowed on any span until the prestressing steel for that span has been tensioned.”

Finally, the provisions of the AASHTO Standard Specifications for Highway Bridges give actual information on how to treat erection methods (AASHTO 1996, p493):

“Otherwise, loads imposed on existing, new or partially completed portions of structures due to construction operations shall not exceed the load-carrying capacity of the structure, or portion of the structure, as determined by the Load Factor Design methods of AASHTO using Load Group IB. The compressive strength of concrete ($f'c$) to be used in computing the load-carrying capacity shall be the smaller of the actual compressive strength at the time of loading or the specified compressive strength of the concrete.”

Table 3.22.1A of Division I—Design provides coefficients for different combinations of loads, including the aforementioned Load Group IB for both Load Factor Design and Service Load. The following Table 4-5 gives the values of the aforementioned table particularly applicable for construction loads, i.e. Load Group IB within Load Factor Design (AASHTO 1996, pp30f):

Table 4-5: Coefficients γ and β for Load Factor Design, Group IB

| Group | β | | | | | | | | | | | | | |
|-------|----------|-----------|-----------|-----------|-----|-----------|---|----|---|----|----|-------|----|-----|
| | γ | D | $(L+I)_n$ | $(L+I)_p$ | CF | E | B | SF | W | WL | LF | R+S+T | EQ | ICE |
| IB | 1.3 | β_D | 0 | 1 | 1.0 | β_E | 1 | 1 | 0 | 0 | 0 | 0 | 0 | 0 |

(Taken from AASHTO 1996, Table 3.22.1A)

with

g = load factor, see Table 3.22.1.A;

b = coefficient, see Table 3.22.1.A;

D = dead load;

L = live load;

I = live load impact;

E = earth pressure;

B = buoyancy;

W = wind load on structure;

WL = wind load on live load—100 pounds per linear foot [1.459 kN/m];

LF = longitudinal force from live load;

CF = centrifugal force;

R = rib shortening;

S = shrinkage;

T = temperature;

EQ = earthquake;

SF = stream flow pressure;

ICE = ice pressure”

and

$(L + I)_n$ - Live load plus impact for AASHTO Highway H or HS loading

$(L + I)_p$ - Live load plus impact consistent with the overload criteria of the operation agency”

(AASHTO 1996, p30).

Values for the coefficient for earth pressure β_E and the coefficient for dead load β_D are provided in the explanations for Table 3.22.1A. Applicable in the case of members in bridge superstructures would be $\beta_E = 0.5$ to 1.3 for different cases of earth pressure and $\beta_D = 0.75$ to 1.0 “For Column Design” or “ $\beta_D = 1.0$ for flexural and tension members”, respectively (AASHTO 1996, p30).

4.3.3.3 American Society for Testing and Materials

The American Society for Testing and Materials (ASTM) also provides comprehensive specifications on the manifold aspects of materials testing and gives information on properties of construction materials, e.g. structural steel, reinforcement bars, and concrete and its ingredients (ASTM 1996).

4.3.3.4 American Welding Society

The American Welding Society (AWS) provides standards for welding of reinforcing bars, including information on material characteristics, limit stresses, splice details, quality of workmanship and inspections, and on the different welding techniques (AWS 1975).

4.3.3.5 Concrete Reinforcing Steel Institute

The Concrete Reinforcing Steel Institute (CRSI 1996) gives detailed information on formwork systems as well as reinforcement bars and welded wire fabric.

4.3.3.6 Other Institutions

Other notable institutions are the American Institute of Steel Construction (AISC), the Prestressed Concrete Institute (PCI), the Portland Cement Association (PCA), and the Transportation Research Board (TRB) of the National Research Council. They publish manuals and professional journals on concrete and steel technology that present state-of-the-art design methods and construction projects.

4.3.3.7 Federal Highway Administration

The Federal Highway Administration (FHWA) is a branch of the U.S. Department of Transportation and is located in Washington, D.C. with so-called field offices all over the nation. It is fulfilling its mission of creating and maintaining high-quality transportation systems in several ways. The following brief summary is based on information provided by the Federal Highway Administration on its site on the World Wide Web (FHWA 1999).

A main area of FHWA activities is provision of technical expertise for design, construction, and maintenance of transportation projects through close cooperation with regional and local authorities. To support this work research projects are undertaken, e.g. in advanced technologies for supervision of traffic. Secondly, the FHWA supervises federal funding for transportation projects and sets up regulations for such projects. In case of natural disasters and other major incidents in the transportation systems the FHWA assists with investigation and repair works. The FHWA also develops regulations and guidelines for construction projects that are related to public transportation.

Apart from regulatory work the FHWA is also active in the field of training and educating professionals in transportation issues, such as safety, economy, and protection of the environment. An example for training is seismic bridge design, which the FHWA conducted to improve safety of bridge structure in case of earthquakes. The very large variety of programs that the FHWA sponsors shows the involvement of this institution in bridge construction.

4.3.3.8 Owner Specifications

Finally, the owner of a construction project usually provides detailed information in form of plans (VDOT 1997b) and specifications (VDOT 1998) that comprise the documents for advertisement of bidding. For bridge projects the owners will predominantly be the State Departments of Transportation, e.g. the Commonwealth of Virginia Department of Transportation (VDOT), which issues “Metric Road and Bridge Specifications” (VDOT 1997c). These include two major parts, the General Provisions that provide general rules for contractual issues and the Specifications, which deal with different topics such as materials, roadway construction, bridges and other structures, incidental construction works, roadside development, and traffic control devices.