VSL

THE INCREMENTAL LAUNCHING METHOD IN PRESTRESSED CONCRETE BRIDGE CONSTRUCTION

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1. Introduction

1.1. General

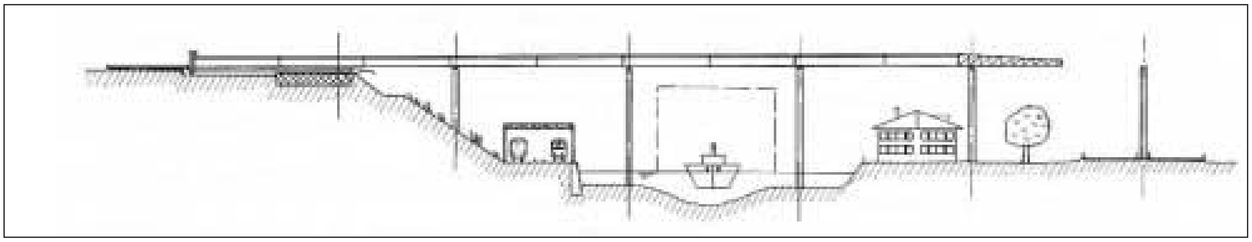


Fig. 1: Principle of construction

The incremental launching method is one of the highly mechanised erection methods used in bridge construction. The method consists of manufacturing the superstructure of a bridge by sections in a prefabrication area behind one of the abutments; each new unit is concreted directly against the preceding one and after it has hardened the resultant structure is moved forward by the length of one unit (fig. 1). This principle has already been used for many years in the construction of steel bridges. This is hardly surprising, in view of the equal strength of steel in tension and compression since, provided the design is suitable, the alternating stresses which occur when the bridge is slid forwards can be accepted without difficulty. This is not so with concrete, which can withstand only small tensile stresses without damage. Special measures are therefore necessary to enable concrete bridges to be slid forward by steps; the skilful use of prestressing is the most important of these measures.

One may ask why it was that the essential features of the now well-known incremental launching method were only used for the first time in a prestressed concrete bridge, when the bridge over the Rio Caroni in Venezuela was built in 1962 (fig. 2). The incremental laun

Fig. 2: Bridge over the Rio Caroni, Venezuela

ching method as applied today for prestressed concrete bridges was first used in 1965 at the Inn Bridge Kufstein, Austria. After the Second World War, bridges were designed on the principle of the minimum consumption of materials. Later on, the labour component of the construction costs became increasingly large, as a consequence of the complication of formwork and falsework, so that construction methods which were less labour-intensive came to the fore, in which a certain excess consumption of materials was more than compensated by savings in the labour costs. These conditions are especially predominant in the incremental launching method. The development of teflon and related products which enable sliding of units to be carried out with a low coefficient of friction finally provided the conditions in which the method could be used with success.

A major part in the development of the incremental launching method for prestressed bridge construction was played by Prof. Dr. F. Leonhardt and his partner Willi Baur. These engineers carried out the basic design for the Caroni Bridge and the Inn Bridge Kufstein and since then have designed in detail and been responsible for the construction of many other projects. In 1967 a patent (No. 1237603) was granted to them for the method in the Federal Republic of Germany. The Ziiblin Group obtained a similar patent (No. 451227) in Switzerland in 1968. The CITRA Company also obtained a patent in France in 1970 (No. 1588840). At present, no information is available about further patents in other countries. Before the method is adopted, however, it is advisable to clarify the patent situation.

The incremental launching method is generally economical for bridges of spans of 30 to 60 m and already for quite small projects of lengths exceeding about 150 m.

By the end of 1976 about 80 bridges, having a total area of about 300,000 m^2 (equivalent to a total of 25 km of bridges of 12 m width) had been constructed by the incremental launching method. The method has therefore proved eminently successful.

1.2. Preconditions for use of the method

The incremental launching method can be used for straight bridges, or where the superstructure has a spatial curve of constant radius throughout the length. This means that it is even possible to construct bridges which are curved both horizontally and vertically, provided that the radii are constant.

The superstructure should consist of a beam of constant section, for which the slenderness ratio, that is the span-to-depth ratio, is not more than 17 when completed. Normally, the ratio lies between 12 and 15, the first value applying to larger, the second to smaller spans. It is of advantage, with regard to design and detailing, if all the spans except the end ones are equal or almost equal in length; the length of the end spans should not exceed 75 0% of that of the standard

spans.
The most suitable cross-sections are the single-cell box section or the double T-beam; double-cell box sections have also been used, but their construction is somewhat more complicated in respect of shuttering and supports.
It is evident that a sufficient area of suitable loadbearing ground must be available behind one abutment for the construction yard. If the bridge has a longitudinal gradient, it is preferable for the construction yard to be behind the lower abutment, so that no braking equipment is necessary during launching.
If some of the preconditions for the use of the incremental launching method do not already exist, the modifications required are frequently quite small. It is however to be hoped that in the future increased attention will be paid at the design stage to the possible use of the incremental launching method. The VSL Organisations will be glad to provide advice in this connection.

1.3. Features of the incremental launching method

The method is characterised by the following features:

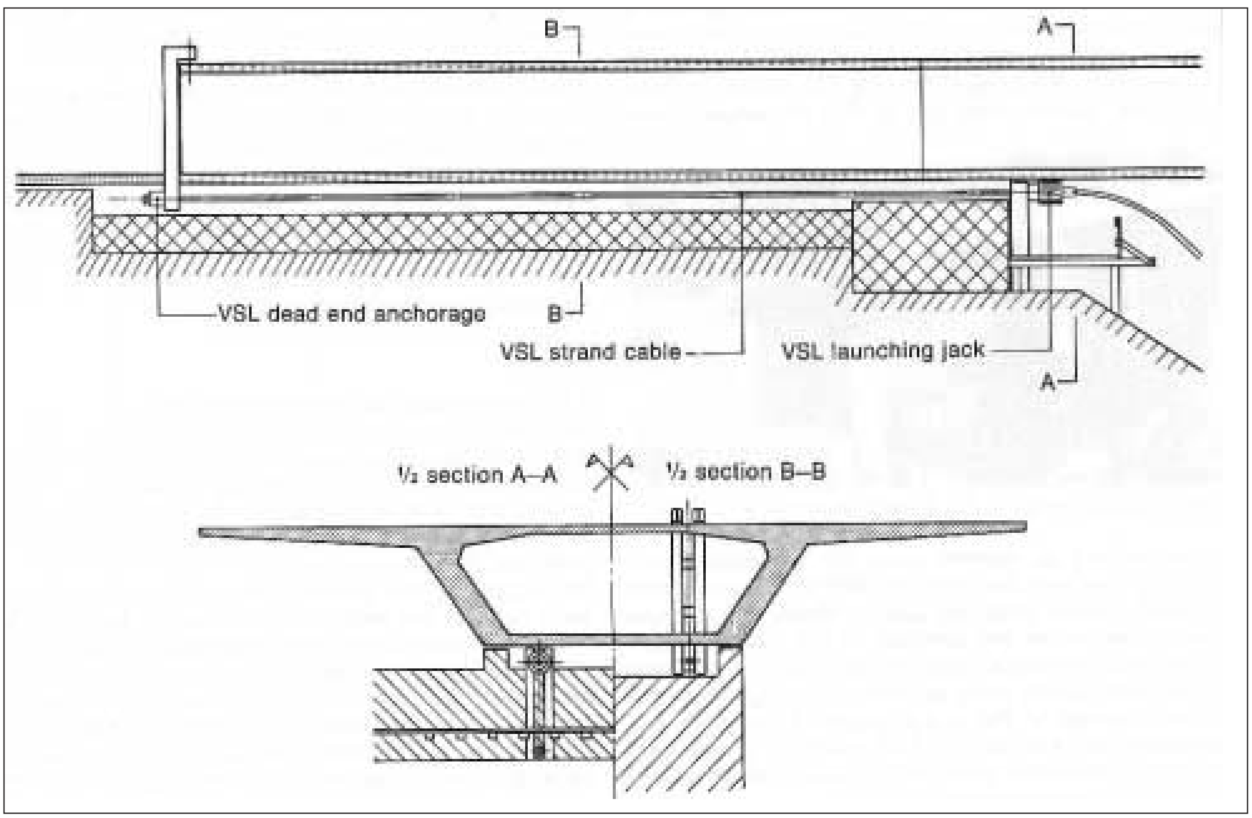
- Construction is carried out completely without falsework, so that there is no problem in passing over obstacles below, such as roads, railways, rivers, buildings or conservation areas (see also fig. 1).
- The fabrication yard is stationary and located behind one abutment, which makes accurate construction possible. The concentration of plant in one area also keeps the site investments and overheads relatively low and the transportation distances extremely short.
- The superstructure is made up of units of 15 to 25 m length, each completed in one week; there are no joints, since each unit is concreted directly against the preceding one.
- During the construction stage the superstructure is centrally prestressed, to limit the tensile stresses produced by the bending moments. Small tensile stresses should be permitted (partial or limited prestressing), even if such stresses are not permitted in the completed structure; they considerably improve the economics of the method, without detracting from the safety of the structure.
- A lightweight nose is fitted to the cantilever end of the superstructure to reduce the cantilever moment during launching.
- A hydraulic jacking device for launching is located at the abutment.
- The bridge supports are equipped with special sliding bearings.
- Auxiliary supports may be incorporated between the piers for long spans and/or where the span/depth ratio is high.

1.4. Features of the VSL launching method

The use of the VSL system in the incremental laun
ching method has the following special characteristics:

- The jacking equipment has been developed from the prestressing equipment for the VSL strand post-tensioning system, so that the pumps and various other components can be used for either purpose.
- The operating of the equipment is simple.
- Equipment of large capacity (up to 5800 kN [580 t] per jack) is available.
- The pulling elements (strand cables) are light and flexible.
- The jacking equipment is concentrated at one abutment.

Fig. 3: VSL jacking equipment



2. Suggestions for structural design

2.1. General

In most countries it is still not usual for the incremen tal launching method to be specified for bridges (thE particular exception is the Federal Republic of Ger many). In those cases, therefore, in which the use o the method appears possible, the preparation of an alternative proposal will be necessary. For this purpose the following documents are required:

- the general specifications
- a general site plan (1 : 2,500)
- a site layout drawing (1 :500)
- a plan view
- a longitudinal section
- the dimensions of the cross-section, piers and abutments
- details of the bearings.

The relevant standards must also obviously be known.

2.2. Loading cases

In addition to those loading cases which must in gene ral be considered, the erection conditions are of espe cial importance to the structural design when the in cremental launching method is to be used; these con ditions influence both the superstructure and also the piers and abutments.

During launching the superstructure is subjected to continually alternating bending moments (fig. 4). Each cross-section moves from regions of positive moment: into regions of negative moments and vice-versa, so that tensile stresses occur alternately at the botton and top parts of the section. The use of central pre stressing reduces the tensile stresses to the permissiblE value.

After the superstructure has been completely launched it must be raised successively by 5-10 mm at each pier by means of jacks, so that the final bearings car be installed. This, however, does not constitute a spe cial loading case, since the influence of differentia settlements at the supports must in any case be investigated. Regular check level readings are taken to ensure that the figures assumed in the design are not exceeded. It may therefore be necessary to provide adjustment devices, to permit wedging up of the bearings.

When the horizontal jacking equipment is installed only at the abutment (the VSL equipment is of this type), the bridge piers will be subjected to a horizontal force in the direction of launching at the level of the bearings due to friction. This must be allowed for by appropriate design and reinforcement (possibily prestressing) or by guying or anchoring of the piers.

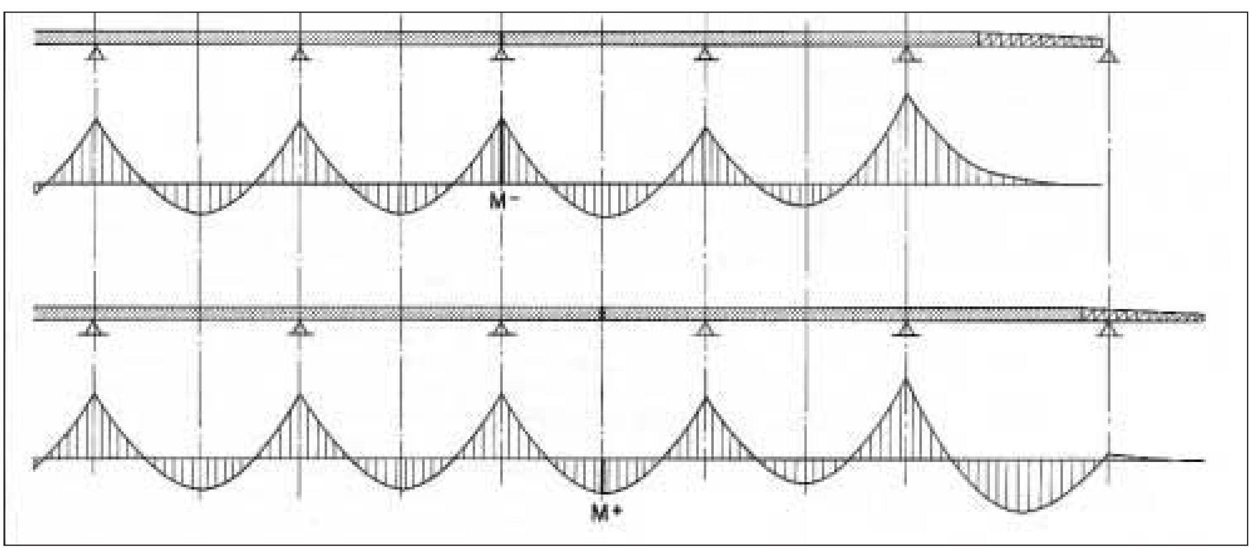
The abutment which takes the thrust of the jacking equipment must frequently be capable of accepting very large horizontal forces towards the end of the launching operation, caused by the frictional resistance. If the abutment alone is not capable of withstanding these forces, it is usually possible to design the foundations immediately behind it in the fabrication area to provide the required additional resistance. If this is not sufficient, the abutment must be strengthened or secured with ground anchors.

2.3. Prestressing

In contrast to all other construction methods, a central prestress is required during the construction stage in the incremental launching method. As already mentioned in this section, this is due to the alternating bending moments. What however does central prestressing really mean? Central means that prestressing cables are so arranged that the resultant compressive stresses at all points of the cross-section are equal and therefore it makes no difference whether the tensile stresses produced during launching occur in the upper or lower parts of the section.

This type of prestress is, of course, quite incorrect for the pattern of moments in the completed state and moreover cannot be subsequently adapted to that pattern. (This was in fact done in the first bridge con

Fig. 4: critical moments during construction



structed by this method but the expense proved to be too great). By adopting a relatively low span/depth ratio, however, it is possible to keep the central prestressing low and economical. The arrangement of the central prestressing cables is such that, in conjunction with the reinforcement, they provide the necessary factor of safety against rupture during construction.

When the bridge superstructure has been completely launched, the continuity tendons are pulled or pushed through and stressed. Their lay-out is designed according to the bending moments in the completed state in which they supplement the central prestressing, which, of course, remains active. In planning the stressing programme, careful consideration is given to the changes in forces and stresses which will be produced.

2.4. Auxiliary equipment

It has already been mentioned that the conditions during erection have a very great influence upon the economy of construction of the superstructure of a bridge built by incremental launching. Various auxiliary equipment is necessary during construction. To reduce the cantilever moment as the superstructure is pushed forward a temporary nose (fig. 5) is fitted to the front end of the superstructure or alternatively the front end may be guyed from a mast (fig. 7).

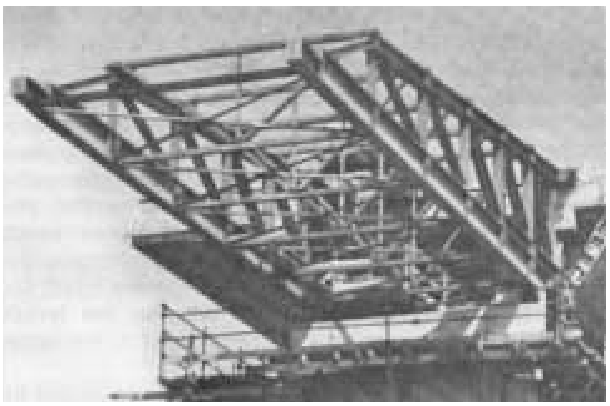
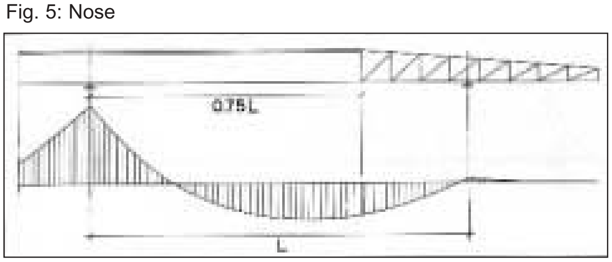


Fig. 5: Nose

The nose usually consists of two braced main girders (lattice or solid web girders) of steel. Its length is 60 to 65 0/o of the span of the bridge in the construction stage. Its weight ranges between 1 and 2 t/m, or more if the length is over 30 m. Noses of prestressed concrete have also been used, but they are of course somewhat heavier; they may however be economical in certain circumstances, for example where steel is expensive or transportation difficult. The use of a nose does not, however, enable the cantilever moment to be reduced to the value of the inner support moment for which the central prestress must be designed. In theory it would be possible to achieve this with a longer nose, but it

Fig. 5: Nose



would not be economic. Temporary additional prestressing proves to be cheaper.

For designing the nose, the determining factors are firstly the maximum positive moment at the point of fixing of the nose to the superstructure and secondly the maximum bearing reaction applied to the bottom flange. The maximum positive moment occurs when the nose projects sufficiently far beyond the pier for the superstructure to occupy about 75 % of the preceding span (fig. 6). Lateral forces due to wind and possibly also due to oblique forward pushing («bearing force,, at the lateral guides) also act upon the nose.

It is also possible to reduce the cantilever moment by guying from a mast instead of by using a nose. This, however, requires constant adjustment to the forces in the guys during forward movement, whereas the nose requires practically no attention.

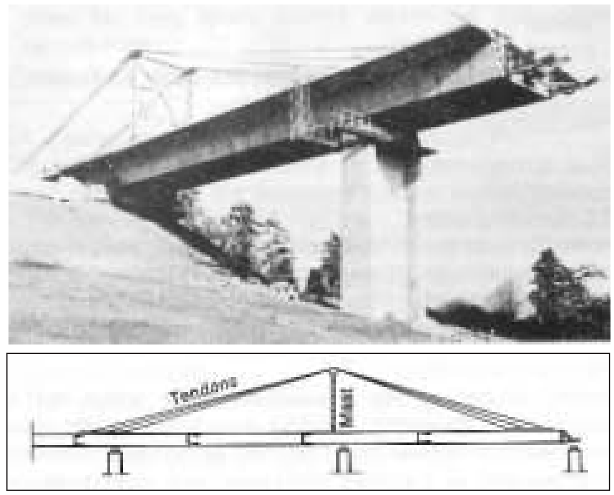


Fig. 7: Guying from a mast

For bridges with spans exceeding 50 m it may be of advantage to use auxiliary piers during erection. This permits a reduction in the central prestress which, in such large spans, would not only lead to high stresses but also require a considerable quantity of material. If possible, the auxiliary supports should be of reinforced concrete, since steel is relatively elastic and temperature-sensitive. Combinations of concrete and steel have also been used, for example in the form of concrete walls braced with steel or steel tubes filled with concrete. Above a height of about 40 m auxiliary piers are economical only in exceptional cases.

2.5. Piers

The piers are usually designed for the final loading condition, in which the loadings and static system are very different from those during construction. In the construction stage, the piers have a larger buckling length and the horizontal force in the direction of forward push is larger, whereas the bearing forces are smaller. If the loading cannot be accepted by the crosssection of the pier, either the cross-section must be increased or the piers must be temporarily guyed. The latter solution is normally the more economical. If the piers are very high, the horizontal force produced during forward jacking can be eliminated by using jacking equipment mounted directly on the piers, so that the action and reaction cancel out.

3. Construction suggestions

3.1. General

When the incremental launching method is used, the advantages of in-situ and precast concrete construction are combined. The fabrication area is stationary and often covered to make it independent of the weather, and the transport distances are very short. This concentration of equipment permits almost factory conditions of construction and correspondingly high quality.

The fabrication area (fig. 8) includes the formwork, concrete mixing plant, a rail-mounted tower crane, storage areas for reinforcing and prestressing steel and the jacking equipment. If the prestressing tendons are made up on site, space for tendon assembly is also required.

The jacking equipment at the abutment and the temporary bearings on the piers and in the fabrication area must be accessible for the forward jacking operations.

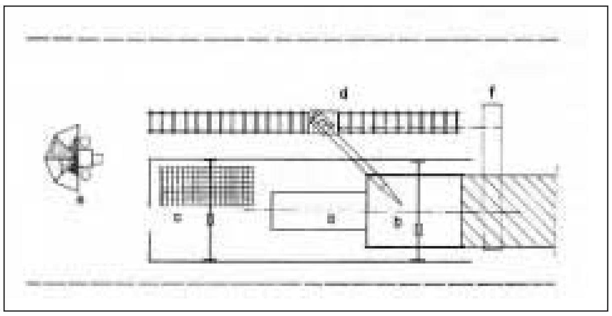


Fig. 8: Fabrication area

3.2. Formwork

For a box-section structure, construction is usually carried out in two phases: in the rear part of the fabrication area, the bottom slab is cast, while the webs and deck slab are cast in the forward part. Consequently, there are two formwork units (fig. 9). The rear formwork, where accuracy is of especial importance so that the sliding surface shall be clean and the launching direction accurately maintained, consists of the bottom formwork and of side forms each about 0.5 m high. It should be possible to lower these forms, to eliminate additional friction during forward jacking. The bottom slab in each case is concreted in the preceding cycle, so that in the second phase it can support the internal formwork and part of the weight of the concrete of the deck slab. The internal formwork is collapsible and movable, enabling it to be reinstalled with minimum expenditure after each cycle. The two external forms in the second region are in a fixed position and can be hydraulically lowered.

The construction of a superstructure having a double T-beam section naturally requires only one formwork unit, in which both the external and internal formwork are in fixed positions and can be hydraulically lowered outwards and inwards respectively.

All formwork is of steel, provided the number of cycles is sufficiently high (in excess of about 25), otherwise it may, for instance, be of plastic-coated timber.

3.3. Sequence of work

A particular advantage of the incremental launching method is that the separate working operations recur in regular cycles, so that even with a relatively inexperienced team it is possible for high quality and output rates to be attained. By training the team (usually about 15 persons) and as a result of the wide use of mechanisation, a high rate of progress can be attained.

In the incremental launching method the working rhythm is co-ordinated to the construction of one unit per week. The individual operations are as follows (for a box-section structure):

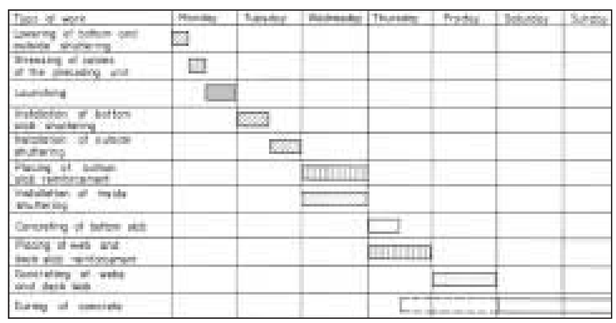
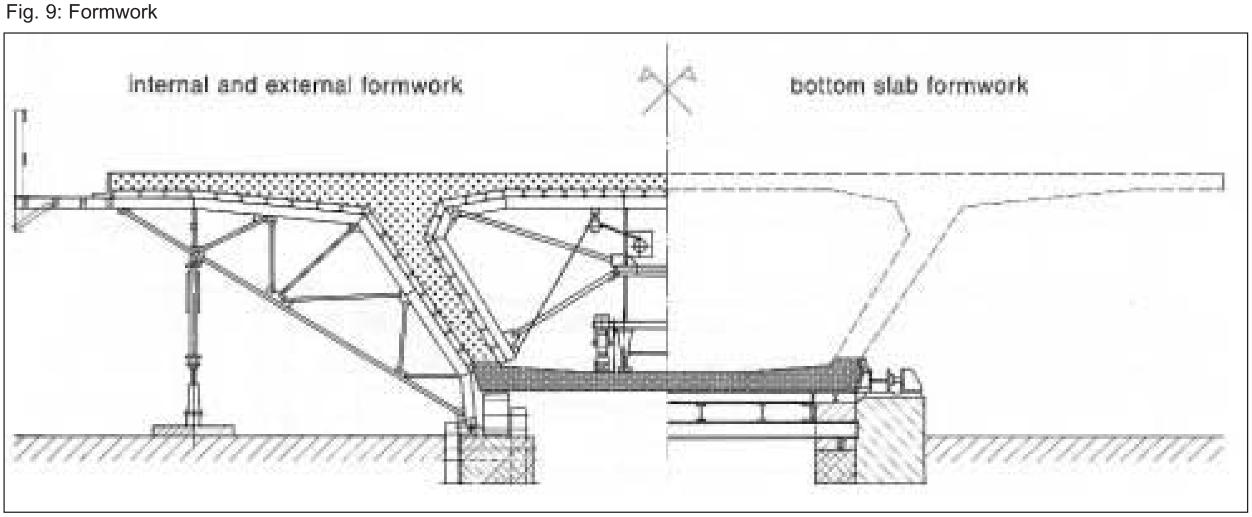


Fig. 9: Formwork



The length of one increment depends firstly upon this programme and secondly upon design and cost considerations. From the design viewpoint it is desirable for the construction joints to be located at sections of low stress, that is near the points of zero moment. This means, however, that the pier diaphragms must be installed later. If this is to be avoided, the subdividing of the units must be done in such a way that each pier diaphragm is located at the front of the increment to be concreted. This however involves a departure from the principle of locating the joints in lightly stressed cross-sections. To use the repetition effect to maximum advantage, a whole number of increments should fall in one span. The length of a unit is finally influenced also by the costs, since the total costs of formwork and forward jacking should be a minimum; this is the case when the equation $B = \sqrt{Kv \cdot Kb^{-1} \cdot L}$ is satisfied, in which B = length of formwork in m, Kv = cost for one forward jacking (including the costs for hire and operation of equipment, operation of temporary bearings, joint forming, couplers for the central prestressing tendons and stressing of them), Kb = cost per metre run of formwork (including costs for foundations, formwork facing, supporting structures and pulling strands), L = total length of bridge to be constructed.
The satisfying of all these conditions often results in the length of a unit being a half-span; depending upon the size of cross-section and length of span, it may be necessary to choose an increment smaller than this (for example $1/3$ or $1/4$ of span). In the normal case, the length of an increment is from 15 to 25 m.

3.4. Prestressing

It has already been mentioned that we must distinguish between central prestressing and continuity prestressing.

3.4.1. Central prestressing

The tendons for central prestress are usually located in the deck slab and bottom slab (or in double T-beams at the bottom of the web) in a quantity inversely proportional to the distances between centroid of tendons and centroid of section. The prestressing tendons are usually made up of small units, of up to about 1000 ki`I (100 t) working load (that is up to VSL 5-7), since the dimensions of the two slabs permit only small anchorages to be used. The tendons in the upper slab are coupled and stressed only in each alternate unit and those in the lower slab in every third (possibily every second) unit alternately, which results in an economic sequence of work for the stressing team, resulting in reduced costs.
The assembled tendons for the upper slab may be reeled onto drums and the drums suspended from a trestle (fig. 11), to enable other operations to continue unimpeded.

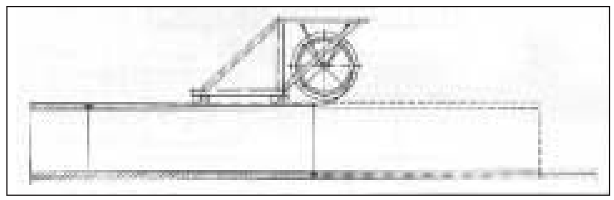
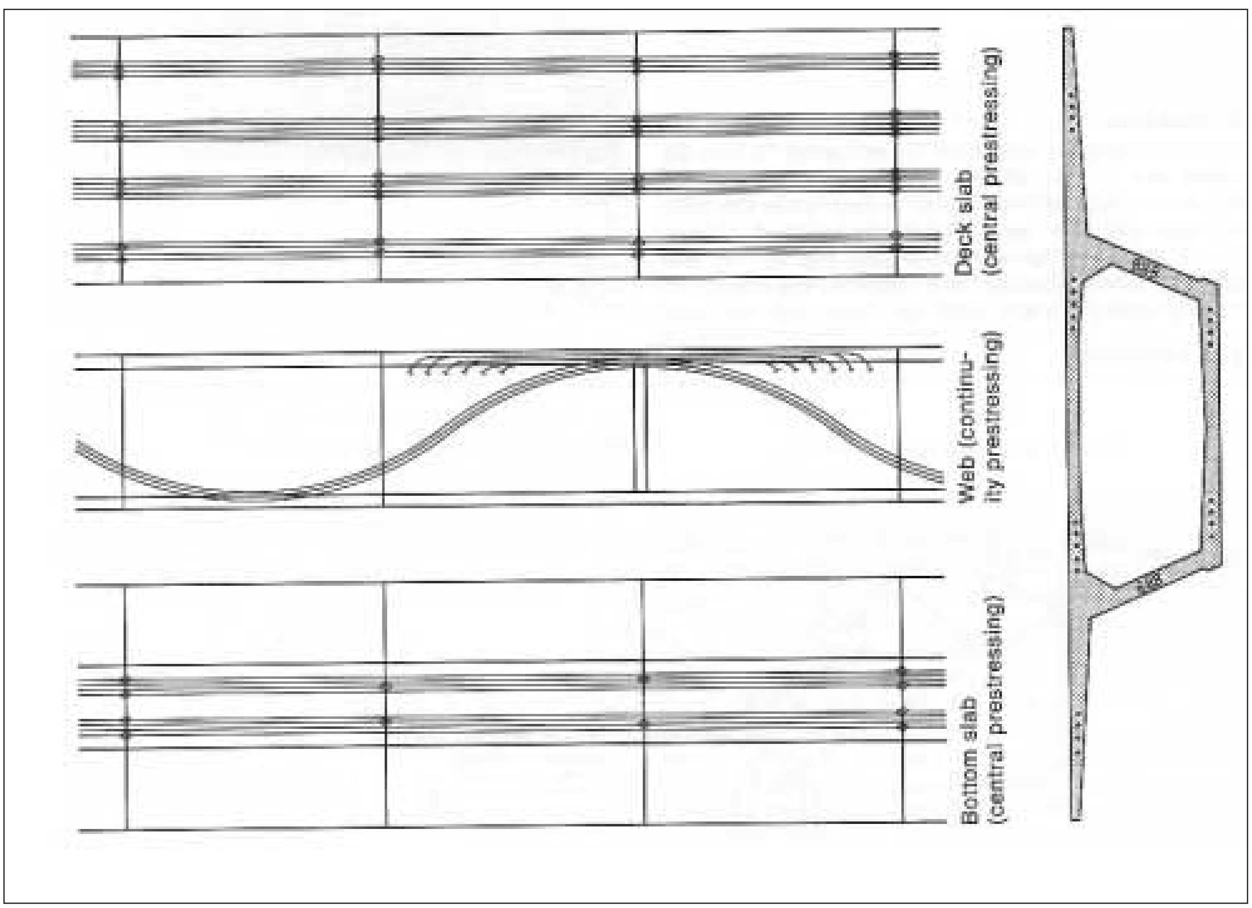


Fig. 11: Trestle for tendons for deck slab

Fig. 10: Cable arrangement (central and continuity prestressing)



In the past, bar systems have often been used instead of cables for the central prestress. The bars were then lengthened at each unit by couplers; they were stressed, however, only at every second or third increment.
In exceptional cases, the central prestress is provided by cables which are not concreted in but are located outside the concrete cross-section. Anchoring buttresses are then used as the stressing points. This arrangement is adopted when it is necessary to remove the cables for the completed state, to prevent the stresses becoming too high. This case can arise with large spans and a relatively shallow bridge superstructure.

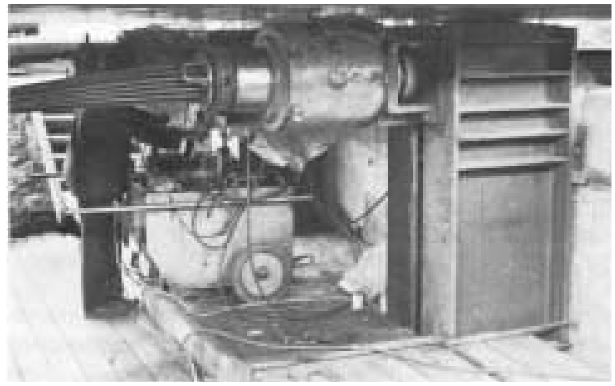


Fig. 13: VSL-jacks with suporting structure

3.4.2. Continuity prestressing
The continuity tendons, which are normally located in the webs and terminate at anchoring buttresses on their inner faces, are plulled or pushed through later and are not stressed until the bridge has been completely launched. These cables are larger units and usually have stressing anchorages (in the VSL system: type E) at both ends. This does not however mean that they must be stressed at both ends, since single-end stressing may be sufficient depending upon the friction conditions. For the latter case, the VSL prestressing system offers an alternative solution by the use of the dead-end anchorage type H (fig. 12): The strands of the continuity tendons are pushed through one by one from the stressing anchorage, as soon as the unit in which the H-anchorages are situated is in the formwork. The dead-end anchorage is formed in-situ and concreted in. Since stressing cannot be carried out until some time later, temporary corrosion protection is necessary.

3.4.3. General remarks
With the incremental launching method, the total quantity of prestressing steel is in general some 40 to 60 0/o higher than for bridges constructed on falsework, due to the provision of both central and continuity prestressing. The resultant additional costs are, however, more than compensated by savings in formwork and labour costs.
Double T-beam bridges require more prestressing steel than box-section bridges, but are very simple to construct.
In addition to the longitudinal prestressing, transverse and vertical prestressing may also be necessary. Prestressed cables are also used for attaching the temporary nose.

3.5. Auxiliary equipment
Various auxiliary equipment and components are necessary for the incremental launching method. A pulling device rather than a pushing device is more suitable for applying the forward movement; this principle is used, for instance, in the VSL system (see also fig. 3). The VSL jacks bear against a steel support structure, which in turn is anchored to the abutment (fig. 13). Before each forward movement, a further auxiliary structure of steel beams (termed pick-up beams) is placed at the end of the just completed unit, to which the strand cables are attached. At the abutment the cables pass through the centrehole jacks and can be easily withdrawn after each launching operation. The speed of forward movement depends upon the types of jacks and pumps used and is from 3 to 6 m/h.
Further auxiliary components include the temporary sliding bearings (or special sliding surfaces on the permanent bearings) and the lateral guides (fig. 14).

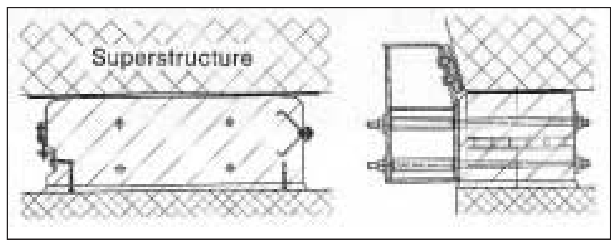
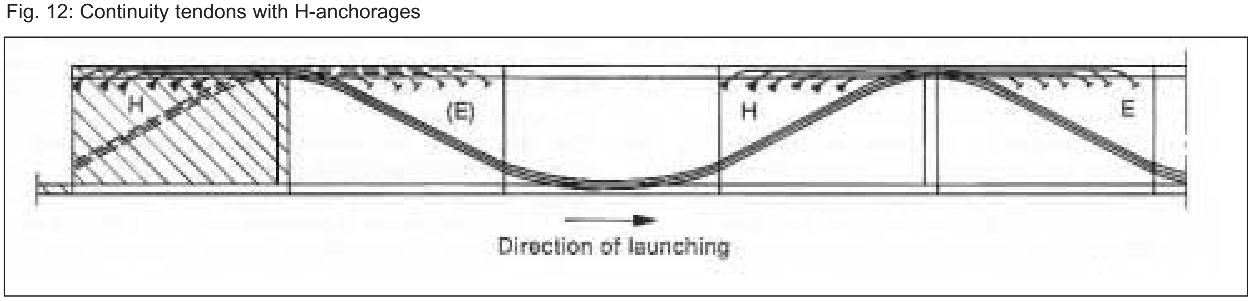


Fig. 14: Temporary sliding bearing with lateral guide

The temporary bearings usually consist of a high-quality concrete block covered with a stressed chrome-steel plate. The surface of the block must be of such a shape that the sliding plates can be inserted without difficulty. It must be remembered that the sliding plates, which are of steel-reinforced neoprene with a teflon coating on one face, become slightly compressed under the load.
In order to remove the temporary bearings when transferring the bridge onto the permanent bearings or in order to remove the sliding plates and chrome-steel plate (when the bridge is launched over the permanent bearings), jacks are used. The jacks are positioned alongside the temporary bearings beneath the webs or beneath the pier diaphragm, depending upon which of these locations will provide sufficient space for the jacks and the necessary operating access to them.

Fig. 12: Continuity tendons with H-anchorages



The forward end of the temporary nose is so designed that, in spite of the deflection, it can ride gently and reliably onto the pier. Either the underside bearing surface is rounded vertically at the forward end or there is an upward hinging end piece which, when the nose meets the pier, is pressed into the horizontal position by jacks. Even when the cantilever moment is relieved by mast guying, a short nose with a rounded running surface is fitted to the forward end of the structure to ensure that it lands without difficulty on the pier.
The design of the temporary piers will of course depend upon their height, but the principal factor influencing it is whether they are to be completely demolished when removed or whether they are so designed that most of their component parts can be reused. Short supports of relatively small dimensions can be easily removed. If they are of steel, then some parts can certainly be used again; if they are of reinforced concrete, they are demolished after use. High temporary piers can with advantage be built up from prefabricated components, and their subsequent removal therefore presents no special problems. This also permits a certain amount of reuse. Some examples of auxiliary pier designs may be seen in fig. 15.
The auxiliary supports are automatically relieved of load when the continuity tendons are stressed and the temporary bearings can be removed and the piers dismantled.
The guying or staying of piers and auxiliary supports, that is anchoring back of the heads of the piers, can be carried out by two basic methods: by inclined guys or, where the spans and length of bridge are small, by horizontal anchoring back (fig. 16). In both cases, these guys can be constituted of individual prestressing strands or of complete prestressing cables.
The method which uses horizontal anchoring to the abutment has the advantage that the abutment is relieved of load by the tying-back forces. Each pier head must, however, be tied back individually, since otherwise the pier deformations would be cumulative and become too large. Inclined guys are anchored either in the base of the adjacent pier or directly to the ground by ground anchors.

Fig. 15: Auxiliary piers

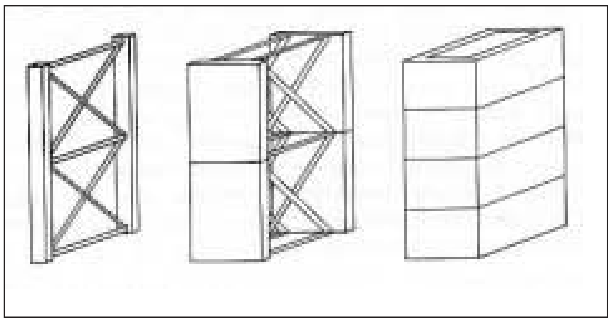
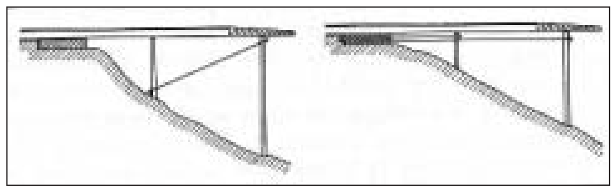


Fig. 15: Auxiliary piers



4. VSL service range

4.1. Extent

The VSL Organisations can offer a comprehensive service for a bridge to be constructed by incremental launching; the extent of the tender will depend upon the particular circumstances. The VSL services consist essentially of:

- the drawing up of a preliminary scheme for a bridge not originally designed for incremental launching,
- the supply, placing, stressing and grouting of the prestressing tendons (central and continuity prestressing),
- hiring and operating of the horizontal jacking equipment,
- supply of pulling cables,
- design and supply of the auxiliary structures,
- design and supply of the temporary sliding bearings (including sliding plates),
- design and supply of the steel or prestressed concrete nose or mast guying system,
- hiring and operating of jacks for transferring the bridge from the temporary to the permanent bearings,
- design and supply of any necessary pier guys.

Some of the VSL Organisations are also able to offer the slipform for the piers as well as bridge bearings and expansion joints.

4.2. Personnel requirements

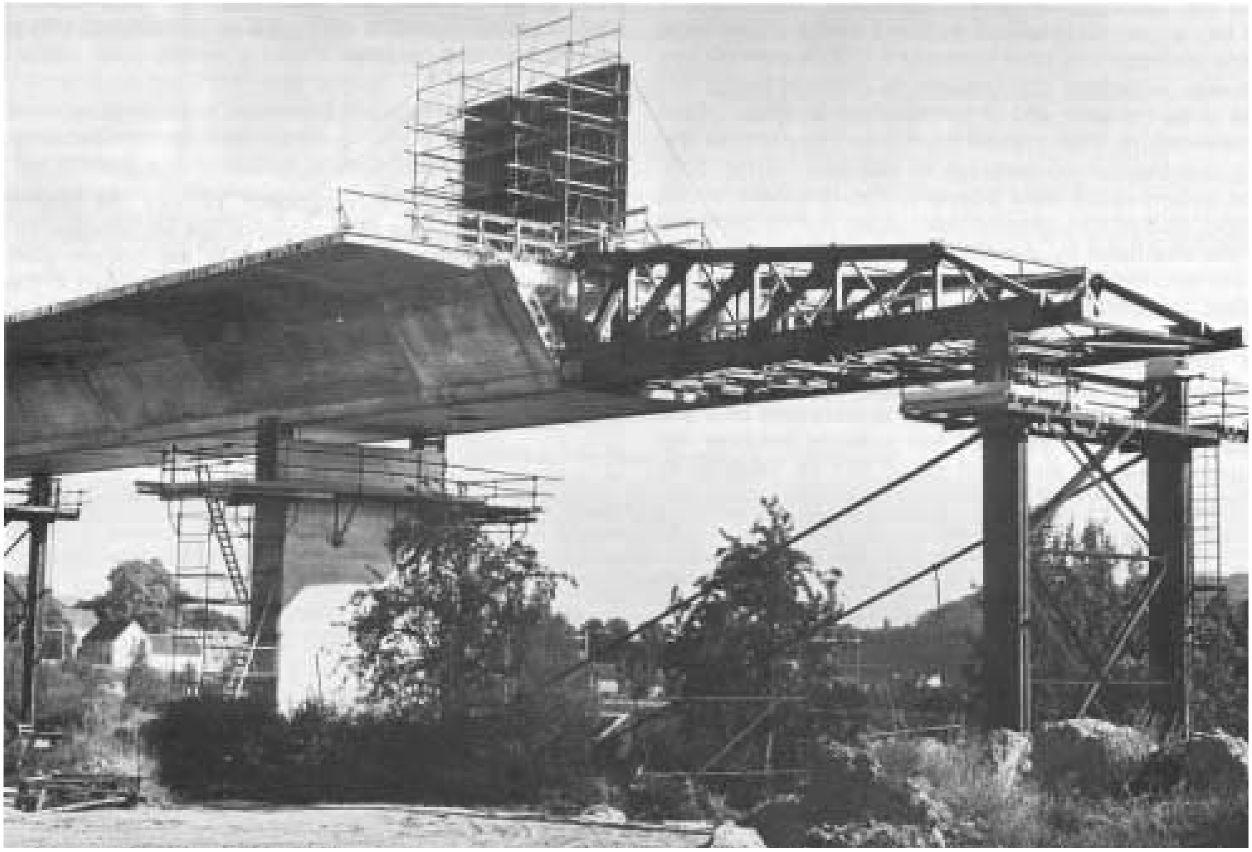
The prestressing and incremental launching operations are provided as a combined tender wherever possible. By using VSL personnel for both classes of work appreciable savings in cost can be achieved since the crew can be kept continuously employed. During the launching operations, the main contractor is asked to provide a crew for manning the sliding bearings, since these people would hardly have any other work to do during this period.

4.3. Preparation of a tender

Detailed drawings and specifications are an essential basis for a tender. In addition discussion between the main contractor and the VSL Organisations is desirable, in order to clarify the possibilities and extent of the VSL services.
The tender for the incremental launching operations comprises the transporting, installation, provision and hiring, dismantling and operating of the equipment and auxiliary components. A drawing indicating the detailed constructional arrangements is also included with the tender.

5. Examples of completed structures

5.1. Ravensbosch Viaduct, Netherlands



Client:	Provinciale Waterstaat Limburg, Maastricht
Engineers:	Bouvy, van der Vlugt, van der Niet, Scheveningen
Contractor:	Joint Venture Internationale Gewapend Betonbouw (IGB), Breda Societe Belge des Betons (SBB), Brussels
Post-tensioning:	Civielco B. V., Leiden
Launching:	VSL INTERNATIONAL LTD., Berne

Introduction

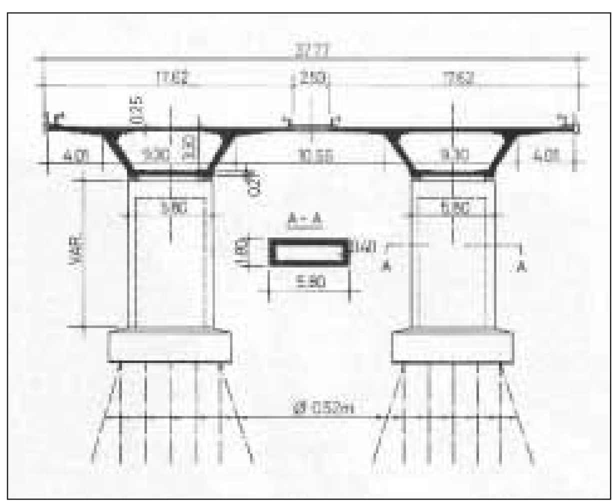
The Ravensbosch Viaduct forms part of the motorway linking Maastricht and Heerlen in Southern Netherlands. It spans the valley of the Strabekervloedgraaf near Valkenburg at a height of about 25 m. The structure consists of two parallel box girders topped by a common deck slab of 37.77 m width. Its length of 420 m is divided into eight spans of 42, 6 x 56 and 42 m. The viaduct is uniformly curved with a 2000 m radius.

Choice of the Construction Method

Due to the importance of the structure and in order to find the most economical construction method, three different designs were prepared for tender:

- The basic design with spans of 42, 6 x 56 and 42 m to be executed according to the (Incremental Launching Method»,
- Alternative I with spans of 45, 5 x 66 and 45 m to be carried out with prefabricated segments,
- Alternative II with spans of 35, 7 x 50 and 35 m to be conventionally built.

Additionally the contractors had the opportunity of tendering with a design of their own. Eleven prequalified contractors (six Dutch and five from abroad) were invited to tender. After a period of three months eighteen offers were received, i. e. ten for the basic design, two for alternative I, four for alternative II and two for other designs. The joint venture IGB/SBB offered the lowest bid with its price for the basic design and it was awarded the contract worth nearly 7.5 million Dutch Florin. The time for the execution of the viaduct - the first in the Netherlands built according to the Incremental Launching Method - was limited to 26 months.



The Structure in Detail

The main dimensions of the structure are given in the preceding figure.

The lay-out of the section was, of course, specially adapted to the use of the Incremental Launching Method and some typical details show that: The height of the box girder corresponds to about 1/17th of the main spans whereas this ratio normally is 1/20th or less. The aim was to reduce the quantity of post-tensionihg cables to be installed and to increase the stiffness of the superstructure. This second point was important as during construction no previous compensations for longterm deformations were possible. The thickness of the bottom slab is also greater than usual; it was governed on the one hand by the dimensions of the anchorages of the tendons incorporated, on the other hand by the weight transferred by the inside shuttering when the deck slab was concreted. Finally it should be noted that the dimensions of the whole section are constant throughout the length of the bridge whereas normally the thickness of the webs at least is increased near the supports. But here this was not possible because the shuttering was of steel and could not be adapted to different sections.

Construction Sequence

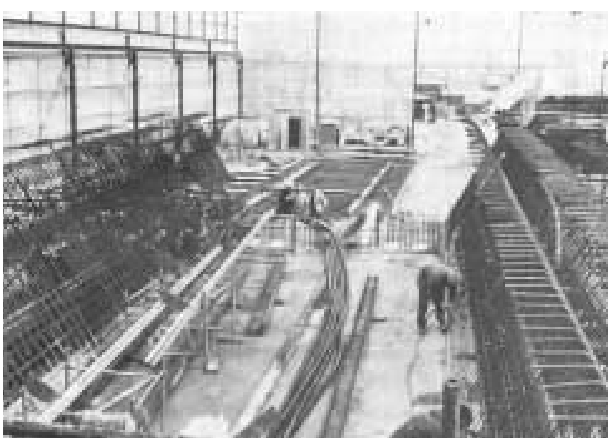
The construction yard of the Ravensbosch Viaduct was located behind the eastern abutment. This side was chosen in view of the launching operations as the viaduct has a downward inclination towards the west of 1 0% .Thus the friction was accordingly reduced. For the construction yard an aera 75 m long and 25 m wide was necessary. This gave room for two casting yards, a storage area for reinforcing and prestressing steel, a runway with a tower crane and a concrete mixing unit. The casting and storage yards were protected by a roof.

The increments of about 19 m length were executed in three stages. In the first casting yard the bottom slab was built. Then the webs were cast in the second yard, followed by the deck slab. By this method the bottom slab had an age of one week when it came into the second yard. It was therefore capable of supporting the inside shuttering and the weight of the concrete of the top slab.

The construction cycle for one segment was as follows:

Monday morning:	stressing of the cables of the segment cast the week before
Monday afternoon:	launching
Tuesday:	construction of the bottom slab
Wednesday:	construction of the webs

Construction of the webs



Thursday and Friday: construction of the deck slab Saturday and Sunday: hardening of the concrete

Special attention had to be given to the accuracy of the shuttering. Indeed a deviation of 1 mm at one end of the 19 m segment would have resulted in a cumulated error of 100 mm over the total length of the bridge. It was, however, possible to install the shuttering with a precision of 1/10 mm!

Prestressing

In the case of the Ravensbosch Viaduct the central prestressing consists of tendons of 828 kN ultimate capacity; eight cables are arranged in the bottom slab, eighteen in the deck. They induce a central stress in the concrete of about 1.5 N/mm2. Temporary piers helped to keep the central prestress small. In front of the structure a steel truss of 15 m length and 20 tonnes weight was installed. How much it reduced the cantilever moment can be seen by the weight of a corresponding part of the superstructure which was about 375 tonnes for a 15 m legth.

Continuity cables are made of VSL tendons EE 6-12 (ultimate capacity 3100 kN). Each web contains a group of six cables arranged in such a way that over the supports the groups of two adjacent spans overlap. Thus on each side of a pier diaphragm six cables are anchored in block-outs at the top of the web. The cables were pulled into the ducts only after completion of launching and then fully stressed.

The deck of the viaduct is also post-tensioned. VSL cables 6-4 at 330 mm c/c are used for this purpose. One cable in four has a fixed anchorage type U at one end and was

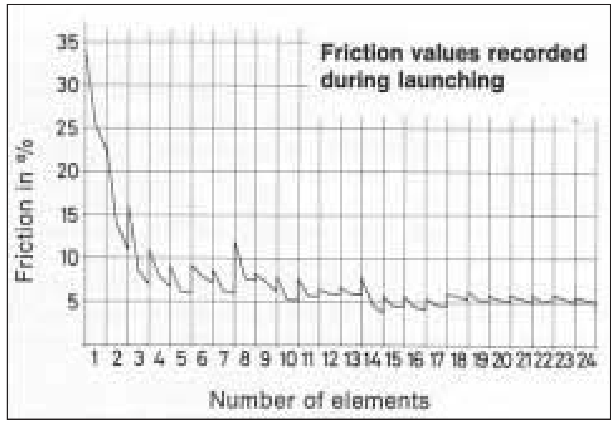
Placing of transverse cables

Launching

For the execution of the launching the VSL strand system was used. Two jacks SLU-330 were fixed to steel girders placed in front of the eastern abutment. Each jack pulled a cable 6-31 (breaking load approx. 8000 kN) anchored to a pair of steel girders specially installed at the end of every increment. The total weight to be launched near the end of the operation was about 11 500 tonnes. The stroke of the jacks being 200 mm, launching over a distance of 19 m - the length of a segment - took about six hours.

During the construction stage all permanent and tem

porary piers were provided with special bearings consisting of a block of concrete Grade 60 (60 N/m m2 at 28 days) covered with a stressed sheet of chrome steel. In order to keep the friction as low as possible steel/ neoprene/teflon plates were introduced between the advancing box girder and these bearings. The same method was used to guide the bridge laterally. Lateral guides were placed on both sides at every permanent pier and on the inner side only at the temporary piers. After completion of launching the special bearings were replaced by permanent ones.
The friction was recorded at each jacking operation; as can be seen from the figure on the right, it was rather high at the beginning but then became constant at about 5 %. This value corresponded to the assumption made at the design stage.



5.2 Highway bridge over the Lech at Landsberg, Federal Rapublic of Germany

Client:	State of Bavaria, Road Construction Office of Weilheim
Engineer:	Bung Consultants, Memmingen
Contractor:	Joint venture Bilfinger+Berger, Munich Wayss & Freytag, Augsburg
Prestressing:	VSL GmbH, Garching-Hochbruck

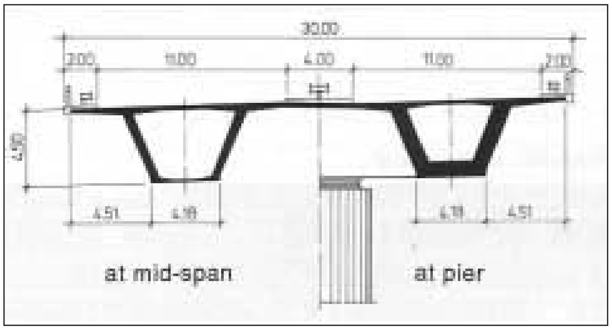
Introduction

To bypass Landsberg, about 60 km to the west of Munich, the Federal Highway B 12 was moved to the northern perimeter of the town and designed as a portion of the future highway from Munich to Lindau (Lake Constance). This work required the construction of a new bridge over the Lech. The structure, 264 m in length and 30 m wide, crosses the river at a height of about 30 m and has a curvature of 20,800 m radius in the vertical plane and 1,687 m radius horizontally. Its maximum longitudinal gradient is 3.3 0/o.

The structure

The bridge had originally been intended as a steel bridge with three clear spans of 88 m each. In 1972, construction of the corresponding abutments and piers was commenced. Difficulties then arose in connection with the constructional activities for the Olympic Games at Munich, so that construction of the bridge was temporarily stopped. As a consequence of the increase in steel prices which occurred at this period, the project was finally dropped and a new design was prepared, providing for a superstructure of prestressed concrete. The already existing abutments and piers were retained, with the spans of 88 m.

The structure was now designed as two box-section girders of 4.50 m depth, connected together by a common deck slab. In view of the possibility of flooding of the Lech and the height of the superstructure above the river, the incremental launching method was selected for construction.



Construction

The following are some of the special features of the construction of the bridge, partly as a result of these circumstances:

It was neccessary to construct the formwork, which was of timber, for the 17.60 m long increments in front of the lower (west) abutment, instead of in the more usual position behind the abutment. Since the intermediate piers had been constructed as central, round supports of 6 m diameter, it was necessary to build special temporary structures of concrete-filled steel tubes for the launching condition. During the construction stage, it was obviously not possible to bridge the gaps of 88 m without intermediate support; auxiliary supports of reinforced concrete were therefore provided in the centre of each span and additionally in the upper end span 11.60 m in front of the abutment. A steel nose was fitted to the forward end of the superstructure, to reduce the bending moments. The two box-section girders were slid forward in succession, since as usual only one formwork unit was used; the southern box section was first constructed, followed by the northern section. After launching of both bridge halves, the two parts were joined together by a 7.10 m wide central strip of the deck slab. The support diaphragms and end diaphragms also provide a cross-connection.

After a commencement period, in which two weeks were necessary for the construction of an increment, it was possible to construct one increment per week. Each of the girders was subdivided into 15 increments. Erection of the bridge had commenced in December 1975 and the work was substantially completed in March 1977.

The VSL prestressing

For the transverse and continuity prestressing the VSL system was employed. The transverse prestressing comprises the cables in the deck slab and also the tendons in the support and end diaphragms. For the deck slab, 361 kN (36.8 t)-cables of type VSL EE 5-4 were used, all 29.30 m long and at an average spacing of 0.49 m. When the two box-section girders were erected, only the empty cable ducts were built in. The cables were not installed until the central strip had been concreted. They were installed strand by strand using the VSL push-through machine, which was operated for this purpose on the north side of the bridge from an overhanging platform. The cables were stressed at one end only, but alternately

VSL push-through machine

The eccentric or continuity prestressing consists of VSL tendons EE 5-16. Some of these are in the webs and some in the bottom slab (the span cables). A certain number of web cables terminate at stressing buttresses, which had been concreted later onto the webs. All the tendons (244 in total) were pushed into the ducts strand by strand using the push-through machine after the incremental launching had been completed. The longest tendons, which are located in the central span and extend at both ends beyond the supports, are 124.3 m in length. The majority of longitudinal cables had to be stressed at both ends, in order to keep the friction losses due to cable curvature as low as possible.

For the central prestressing of the first flour increments, in addition to the small prestressing tendons, VSLcables 5-16 were also incorporated in the webs and bottom slab, to keep the stresses at the cantilever end during launching within the acceptable limits.

5.3. Highway bridges Bonn-Ramersdorf, Federal Republic of Germany

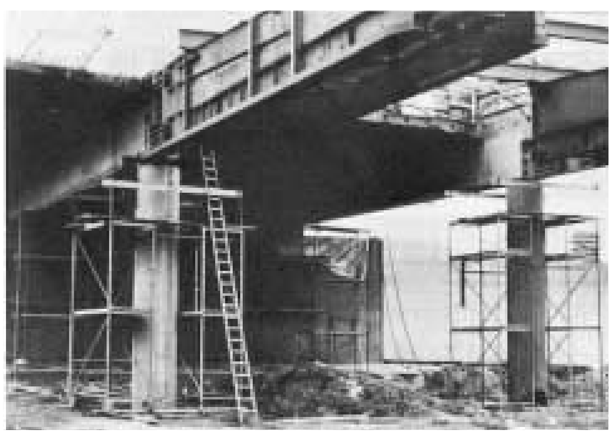
Client:	Landschaftsverband Rheinland, Highway Construction Office, Bonn
Engineer:	Wayss & Freytag, Cologne/Frankfurt
Contractor:	Joint venture Wayss & Freytag, Cologne Beton- and Monierbau, Cologne
Prestressing:	VSL GmbH, Garching-Hochbruck

Introduction

Between the highway intersection Bonn-Ramersdorf and the Konrad-Adenauer-Rhine Bridge, two parallel bridges carry the A 56 over the tracks of the Bonn Suburban Tramway (Siebengebirgsbahn) and over the Konigswinterer Hauptstrasse (B 42). Both structures are about 145 m long and 19.28 m wide. In the region of the bridges, the line of the highway is at a maximum gradient of 1.9 0/o and a curve of 2,400 m radius, also with a vertical curve (radius 39,000 m). Each half bridge has seven spans ranging between about 15 and 22 m.

Details of the bridges

Each half bridge consists of a double T-beam, without diaphragms, of 1.80 m total depth, with a distance between web centre lines of 11 m. The webs rest on circular individual piers of 4 to 6 m height and 1.10 m diameter, founded upon large piles. The cross-section of the superstructure was adopted from a special proposal of the contractor, who bid on the basis of the incremental launching method. It should be noted in this connection that only a few double T-beam bridges have so far been erected by this method. It is for bridges of relatively shallow depth, however, that double T-beam sections are simpler than box-sections and the formwork is therefore less costly.



Erection

Each half bridge was constructed in nine sections of 13.65 to 16.20 m length in a wooden form. The form was erected behind the upper abutment. After a somewhat slower initial period, the construction of one increment required one week, so that forward jacking could be carried out each Monday. To permit sliding over the individual piers, the bearings were constructed in such a manner that they could be used as sliding bearings during erection and as permanent bearings in the completed state. Each pier on the inner side of the curve was equipped with a temporary guide device in addition.

A steel nose was fitted to the cantilever end of the bridge structure

Bearing with sliding plates

the bridge structure in the usual way. In this case, it was possible to use an existing nose from an earlier site, adapting it by means of a few new parts to the changed conditions.

The two parts of the bridge were constructed one after the other, commencing with the north structure. The erection period lasted from September 1976 to May 1977.

The VSL prestressing

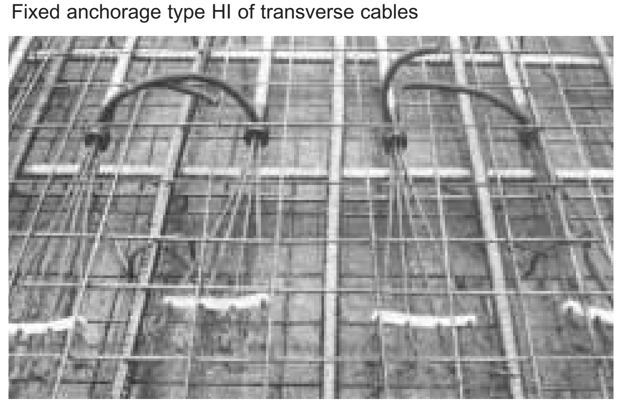
In bridges constructed by incremental launching, three types of prestressing are usually found: the central, the eccentric and the transverse prestressing. On this project, the cables of the second and third groups were by the VSL system.

For the continuity prestressing, only the empty ducts (80/85 mm diameter) were laid during construction. After the bridges had been completely launched, the strands were pushed in with a push-through machine, from one end over the entire length of 145 metres. In each web, there are four cables of type VSL EE 5-16 which, on account of their length and in view of the fact that they extend over seven spans, had to be stressed at both ends.

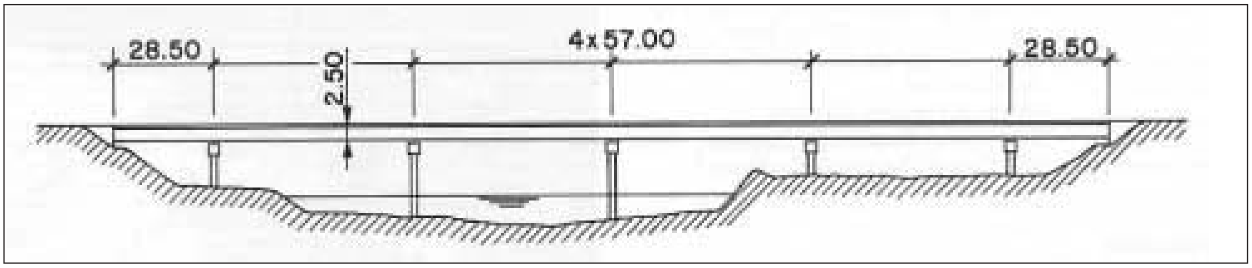
For the tranverse tendons, VSL cables of type EH 5-4, of 17.30 to 19.50 m length, were used. They were made up directly on the site and installed as complete, ducted cables. In total, 1,158 transverse tendons had to be manufactured, laid at an average spacing of 0.24 m. To ensure symmetry, the stressing anchorage was alternately at the left and right edge of the slab.

At the abutments, the two bridge structures were closed by prestressed end diaphragms, which had been concreted against the bridge structure afterwards.

Fixed anchorage type HI of transverse cables



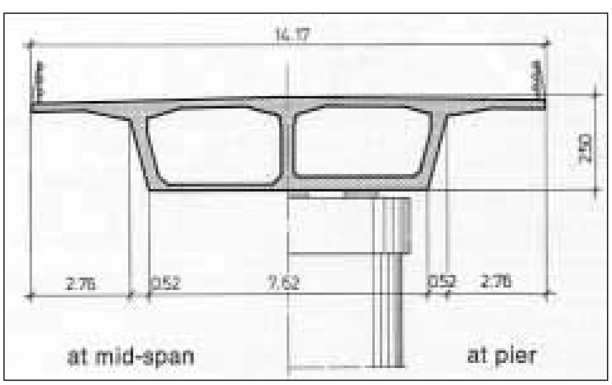
5.4. Bridge over the Wabash River, USA



Client: Indiana State Highway Commission
Engineer: VSL Corporation, Los Gatos, California
Contractor: Joint venture
Weddle Brothers Construction Co.,
Rogers Construction Co., both of Bloomington, Indiana
Prestressing and launching: VSL Corporation, Los Gatos, California

Introduction
This is the first bridge built by incremental launching in the USA. It carries the re-located US road number 136 over the Wabash river in the vicinity of Covington, Indiana. The structure is straight both in elevation and in plan and without any gradient. Its length is 285 m with a width of 14.17 m, and it spans the river at about 11 m above the mean water level. When in spate, however, the Wabash River rises considerably, so that construction of the bridge on falsework was out of the question.

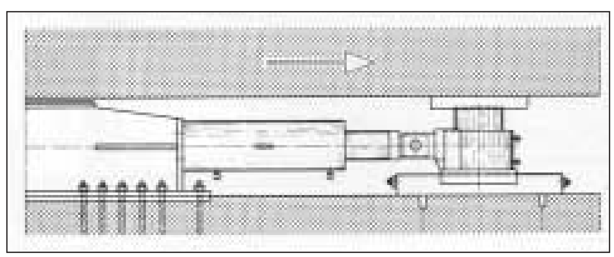
The Tender
In the tender documents it had been envisaged that the superstructure would be built of prefabricated segments by the cantilever method. This would have necessitated the use of mobile and floating cranes, to enable the elements to be placed. The Client, however, permitted alternative proposals, on the condition that the superstructure cross-section and arrangement of supports would be retained. This led the VSL Corporation to offer a variant, in the incremental launching method, to interested contractors. In fact, this bridge presented ideal circumstances for the use of this method, possibly with the exception of the double-cell box-section.
The latter circumstance, however, presented no obstacle, since double-cell box-section bridges had already been erected by the incremental launching method.
The Weddle/Rogers joint venture, which had submitted the cheapest bid and obtained the contract on that basis, decided in the middle of October 1976 to adopt



the proposal of the VSL Corporation and to entrust this Corporation with the redesign of the bridge and the provision of the launching equipment, the steel nose, the temporary sliding bearings and the reinforcing steel, and also with the execution of the prestressing operations.

Details of the bridge
The superstructure of the Wabash River bridge is 2.50 m in depth and is subdivided into four central spans of 57 m and two end spans each of 28.50 m. It rests on solid-wall piers of 6.10 m width and 1.52 m thickness. They are founded directly on the solid rock and are at an angle of 80° to the axis of the bridge, since the bridge does not cross the river at right angles. The abutments, however, are orthogonal.

Erection
The superstructure was constructed in 20 increments each 14.25 m in length. Each section was built in two steps, the bottom slab being first constructed, followed by the webs and deck slab. Two forms were therefore necessary, a rear form for the bottom slab and a forward form for the remainder of the cross-section. Using this method, one section could be constructed every week and forward jacking could be carried out every Monday.
Two 3000 kN (300 t)-jacks were used for the forward jacking. These were, however, not VSL jacks of the SLU range but a sliding equipment differing from the VSL system and already used a number of times in Europe; this equipment is shown below:



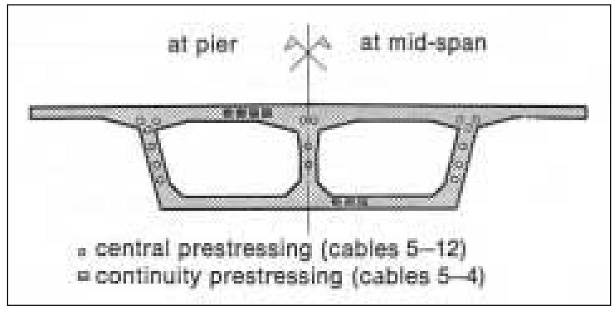
This type of sliding equipment can be used where the applied force is sufficiently large in relation to the sliding force required.
The principle of operation is as follows: the horizontal (sliding) jack is fully retracted before sliding commences. Then the vertical jack is extended and the superstructure is slightly lifted. The sliding jack then pushes forward the vertical jack, which rests on a special sliding bearing, and with it the entire superstructure, by the length of one piston stroke. The vertical jack is then released, the piston of the sliding jack is retracted and the cycle commences again (with the lifting of the superstructure), and is continued until the superstructure has bee-n advanced by the length of one increment.

An insert possessing a high coefficient of friction is placed between the piston of the vertical jack and the superstructure, thus ensuring that no relative movement between the piston and superstructure can occur.
In order to limit the bending moments in the superstructure during the erection stage, auxiliary supports of steel were installed between the permanent piers. Since these temporary supports were relatively light, they had to be anchored back by VSL cables type 5-4 to the rearward main piers. The lower slab of the superstructure, with its width of 7.62 m, is wider than the piers, so that here again auxiliary structures had to be provided, capable of carrying the temporary sliding bearings. A steel nose, 17.70 m in length, consisting of two cross-braced solid-web girders, was fixed to the cantilever end of the superstructure to reduce the bending moments. After launching had been completed, the bridge was transferred from the temporary bearings to the permanent bearings.

The prestressing

Both the central and the eccentric prestressing consists of VSL tendons. For the central prestressing, 16 cables 5-12 were used; contrary to the usual practice, however, these tendons were placed in the webs. Six each of these tendons were placed in the outer webs and four in the middle web. They were stressed when the concrete had reached a minimum strength of 24 N/mmz (245 kg/cmz). The tendons of the eccentric prestressing are of type 5-4 with a flat duct and cast-iron anchorage.

They are located exclusively in the bottom slab (in the span region) and in the deck slab (over the supports). The use of flat ducts enabled bundles of two or three superimposed tendons to be formed. The tendons terminate at small buttresses on the lower and upper side respectively of the slabs. They were stressed strand by strand.



The unusual arrangement of the cables was due to the constraint that the cross-section dimensions of the original design had to be retained. The use of small cables for the eccentric prestressing had, however, the advantage that the stressing buttresses were very small and a small, light jack, very easy to use for overhead work, could be used for stressing.
In addition, VSL tendons were also used for the prestressing of the 1.77 m wide support diaphragms, for anchoring the temporary structures to the piers and for attaching the temporary nose.

6. Representative list of incrementally launched bridges post-tensioned and/or launched with the VSL-system

Pipeline bridge SNAM across the Po River, Italy

Bridge for oil pipelines near Pavia, executed 1968/69

Owner	SNAM S. p. A.	
Engineer	Dr. Giancarlo Giuliani, Milan	
Contractor	Presspali S. p. A. and EDIM SRL, Milan	
Post-tensioning }	VSL Italia S. p. A., Milan	
Launching	(formerly Beton	Precompresso S. p. A.)
Total length	1362 m (6 parts: 4 x 215 m and 2 x 251 m)	
	Alignment: straight, slope 0.3 °/o	
Spans	38 x 35,84 m	
Width	4,60 m (U-shaped section)	
Depth	2,12 m	
Pier heights	10 to 15 m	
Tendons	temporary (central)	156 cables EE 5-12
	final (eccentric)	152 cables EU 5-7
		12 cables EU 5-3
Launching equipment	6 VSL-Monojacks	

Ravensbosch viaduct, Netherlands

Bridge of the Maastricht-Heerlen Motorway, executed 1972/74

Owner	Provinciale Waterstaat Limburg, Maastricht
Engineer	Bouvy, van der Vlugt, van der Niet, Scheveningen
Contractor	Joint Venture IGB, Breda/SBB, Brussels
Post-tensioning	Civielco B. V., Leiden
Launching	VSL INTERNATIONAL LTD., Berne
Total length	420 m, Alignment: horizontal radius 2000 m, slope 1%
Spans	42/6 x 56/42 m (auxiliary piers during construction)
Width	37,77 m (2 box girders)
Depth	3,30 m
Pier heights	6,50 to 23,50 m
Tendons	longitudinally 192 cables EE 6-12
	transversally 1152 cables EE/EU 6-4
Launching equipment	2 SLU-330 with 2 cables 6-31

Dal bridge Avesta, Sweden

Bridge of the National Highway No. 70 over the Dal River, executed 1972

Owner	Statens Vagverk
Engineer	ELU-Consult, Stockholm
Contractor	Nya Asfalt AB, Stockholm
Post-tensioning	Internordisk Spannarmering, Stockholm
Total length	350 m, Alignment: horizontal radius 3500 m, vertical radius 25'000 m, max. slope 2,9 °/a
Spans	40/6 x 45/40 m
Width	16,50 m (1 box girder)
Depth	3,20 m
Pier heights	15,80 to 24,80 m
Tendons	48 cables EE 5-12 1 = 56,7 to 77,1 m

Horomoi bridge, Japan

Highway bridge near Muroran (Hokkaido), executed 1973

Owner	Hokkaido Prefecture
Engineer	Osaka Consultant, Osaka
Contractor	Taisei Corporation, Tokyo
Post-tensioning	
Total length	170 m, Alignment: straight, horizontal
Spans	52,50/63,00/52,50 m (auxiliary piers during construction)
Width	10,00 m (1 box girder)
Depth	3,00 m
Pier height	38,5 m
Tendons	8 cables EE 5-31 l = 172,5 m

St. Isidore viaduct, France

Bridge of the Motorway A 8 near Nice, executed 1975/76

Owner	Departement des Alpes Maritimes
Engineer	SFEDTP, Paris
Contractor	SONEXA, Nice
Launching	Joint Venture VSL INTERNATIONAL LTD., Berne / VSL France, Paris
Total length	228 m, Alignment: straight, slope 6%
Spans	28,24/4 x 40,10/39,30 m
Width	10,50 m (1 box girder)
Depth	2,50 m
Pier heights	18 to 23 m
Launching equipment	2 SLU-330 with 2 cables 6-31

Trichtenbach bridge, FR Germany

Bridge of the Stuttgart-Singen Motorway near Trichtingen, executed 1975/76

Owner	Highway Department Baden-Wiirttemberg, Rottweil
Engineer	Leonhardt & Andra, Stuttgart
Contractor	Joint Venture Wayss & Freytag/Karl K6bler
Post-tensioning	VSL GmbH, Garching-HochbrOck
Total length	147 m, Alignment: horizontal radius 2500 m,slope 0,6%
Spans	46/55/46 m
Width	30,50 m (2 box girders)
Depth	3,20 m
Pier height	31 m
Tendons	28 cables EE 5-16 l = 147 m

Untere Muehle bridge, FR Germany

Bridge of the Stuttgart-Singen Motorway near Trichtingen, executed 1975/76

Owner	Highway Department Bade n-Wurttemberg, Rottweil
Engineer	Leonhardt & Andra, Stuttgart
Contractor	Joint Venture Wayss & Freytag / Karl Kiibler
Post-tensioning	VSL GmbH, Garching-HochbrOck
Total length	119 m, Alignment: horizontal radius 2500 m slope 0,6 %
Spans	37/45/37 m
Width	30,50 m (2 box girders)
Depth	3,20 m
Pier height	22 m
Tendons	12 cables EE 5-16 4 cables EE 5-12

Mako bridge, Senegal

Bridge of the RN 7 over the Gambia-River, executed 1975/76

Owner	Republic of Senegal
Engineer	SFEDTP, Paris
Contractor	
Post-tensioning	VSL INTERNATIONAL LTD., Berne
Launching ,	
Total length	190 m, Alignment: straight, horizontal
Spans	16/3 x 21/2 x 16/3 x 21/16 m
Width	8,50 m (slab)
Depth	0,88 m
Pier height	6,50 m
Tendons	48 cables EE 5-12 l = 95 m
Launching equipment	2 SLU-90 with 2 cables 5-7

Lech bridge Landsberg, FR Germany

Highway bridge over the Lech River, executed 1975/77

Owner	State of Bavaria, Road Construction Office Weilheim
Engineer	Bung Consultants, Memmingen
Contractor	Joint Venture Bilfinger+Berger, Munich/ Wayss & Freytag, Augsburg
Post-tensioning	VSL GmbH, Garching-HochbrOck
Total length	264 m, Alignment: horizontal radius 1687 m, vertical radius 20 800 m, max. slope 3,3 0%
Spans	3 x 88 m (auxiliary piers during construction)
Width	30,00 m (2 box girders)
Depth	4,50 m
Pier height	approx. 30 m
Tendons	longitudinally 244 cables EE 5-16 transversally 545 cables EE 5-4 168 cables EP 5-16 (diaphragms)

Boivre viaduct, France

Bridge of the Motorway A 10 near Poitiers, executed 1976

Owner	COFIROUTE, Paris
Engineer	SGE, Chevilly-La Rue
Contractor	SOCASO, Chevilly-La Rue
Launching	Joint Venture VSL INTERNATIONAL LTD., Berne / VSL France, Paris
Total length	286 m, Alignment: straight, slope 0,5 %
Spans	35,70/5 x 43,00/35,70 m
Width	9,50 m (1 box girder)
Depth	2,50 m
Pier heights	15 to 29 m
Launching equipment	2 SLU-330 with 2 cables 6-31

Sarugaishi bridge, Japan

Railway bridge of the Tohoku New Express Line, executed 1976/77

Owner	Japan National Railway, Tokyo
Engineer	
Contractor	
Post-tensioning	Taisei Corporation, Tokyo
Total length	390 m, Alignment: straight, slope 1,1%
Spans	13 x 30 m
Width	12,20 m (1 box girder)
Depth	2,30 m
Pier heights	17 to 25 m
Tendons	cables type EE 5-19

Bridges Bonn-Ramersdorf, FR Germany

Highway bridges over road and railway, executed 1976/77

Owner	Landschaftsverband Rheinland, Highway Construction Office, Bonn
Engineer	Wayss & Freytag, Cologne/Frankfurt
Contractor	Joint Venture Wayss & Freytag/Beton- and Monierbau, Cologne
Post-tensioning	VSL GmbH, Garching-HochbrOck
Total length	145 m, Alignment: horizontal radius 2400 m, vertical radius 39 000 m, max. slope 1,9 °%
Spans	15 to 22 m
Width	38 60 m (2 double T-beams)
Depth	1,80 m
Pier heights	4 to 6 m
Tendons	longitudinally 16 cables EE 5-16 l = 145 m transversally 1158 cables EH 5-4 1 = 17,30 to 19,50 m 28 cables EH 5-16 1 = 17,80 to 19,55 m (end diaphragms)

Indre viaduct, France

Bridges of the Motorway A 10 near Montbazon, executed 1976/77

Owner	COFIROUTE, Paris
Engineer	SGE, Chevilly-La Rue
Contractor	SOCASO, Chevilly-La Rue
Launching	Joint Venture VSL INTERNATIONAL LTD., Berne / VSL France, Paris
Total length	184,70 m, Alignment: straight, slope 0,7%
Spans	35,70/43,00/43,00/40,00/23,00 m
Width	19,84 m (2 box girders)
Depth	2,80 m
Pier height	approx. 6 m
Launching equipment	1 SLU-330 with 1 cable 6-19

Neckar bridge Beihingen, FR Germany

Bridge of the Motorway A 81 over the Neckar River and Canal, executed 1976/77

Owner	Highway Department Baden-Wurttemberg, Stuttgart
Engineer	Leonhardt & Andra, Stuttgart
Contractor	Joint Venture Wolff & Muller / Friedrich Jag KG
Post-tensioning	VSL GmbH, Garching-HochbrOck
Total length	301 m, Alignment: vertical radius, max. slope 3 °/o
Spans	52,70/59,03/69,38/59,90/59,99 m (auxiliary piers during construction)
Width	18,50 m (1 box girder)
Depth	2,80 m
Pier heights	9 to 15 m
Tendons	108 cables EE 5-16 l = 25 to 158 m

Wabash river bridge, USA

Bridge of the US Road No. 136 near Covington (Indiana), executed 1977

Owner	Indiana State Highway Commission
Engineer	VSL Corporation, Los Gatos, California
Contractor	Joint Venture Rogers Construction Co. / Weddle Bros. Construction Co., Bloomington, In- diana
Post-tensioning	VSL Corporation, Los Gatos, California
Launching	
Total length	285 m, Alignment: straight, level
Spans	28,50/4 x 57,00/28,50 m (auxiliary piers during construction)
Width	14,17 m (1 double-cell box girder)
Depth	2,50 m
Pier heights	6 to 11 m
Tendons	cables 5-4 and 5-12
Launching equipment	2 3000 kN [300t]-jacks

Gaddvik bridge, Sweden

Highway bridge over the Lulealv River in Northern Sweden, execution 1977/78

Owner	Statens Vagverk
Engineer	ELU-Consult, Stockholm
Contractor	ABV-Nya Asfalt AB, Stockholm
Post-tensioning	Internordisk Spannarmering, Stockholm
Total length	614 m, Alignment: horizontal radius 7000 m, vertical radius 60000 m, max. slope 1 °/o
Spans	37/12 x 45/37 m
Width	13,50 m (1 box girder)
Depth	2,45 m
Pier height	16 m
Tendons	central cables EK/KK 5-7 l = 30 m eccentric cables EE 5-12 l = 80 m

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