VSL

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VSL REPORT SERIES

EXTERNAL POST-TENSIONING

Design Considerations
VSL External Tendons
Examples from Practice

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Preface

The purpose of this report is to discuss the principles and applications of external post-tensioning and to present the VSL External Tendons. It should assist engineers in making decisions regarding both design and construction. This document does not represent a complete manual for detailed design and practical construction of structures with external tendons. In this respect the reader is referred to the relevant technical literature (see bibliography in Chapter 7). Furthermore, it must be mentioned that the emphasis is clearly on the applications for bridges. Where appropriate, however, reference is also made to other applications such as in buildings and circular structures.

There are many similarities between external tendons, stay tables and permanent prestressed ground anchors. In fact, regarding many aspects there is hardly any difference. Reference is therefore made to the report on VSL Stay Cables for Cable-Stayed Bridges [1] and the VSL documentation on ground anchors (e.g. [2]).

The VSL Organizations will be pleased to assist and advise you on questions relating to the use of external post-tensioning. The authors hope that the present report will help in stimulating new and creative ideas. The VSL Representative in your country or VSL INTERNATIONAL LTD., Berne, Switzerland, will be glad to provide you with further information on the subject.

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1. Introduction

1.1. Historical developments

The idea of actively compressing structural elements with a high tensile material such as steel is very old. Everyone is familiar with timber barrels and timber wheels stressed together by steel hoops. In ancient Egypt, the same technique was used for shipbuilding.

In the history of modern engineering, Farber may first be mentioned. He was granted German patent DRP 557,331 in 1927. In essence, this patent describes a prestressing system in which bond with the surrounding concrete structure is prevented by covering the prestressing steel with a bond-breaker such as paraffin. It is not known whether Farber's idea was actually applied in practice [3].

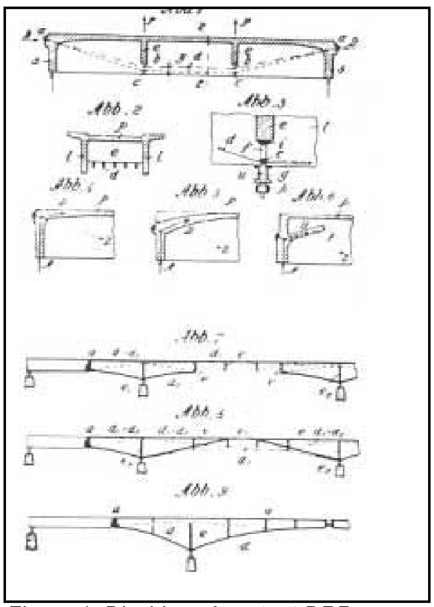


Figure 1: Dischinger's patent DRP 727,429

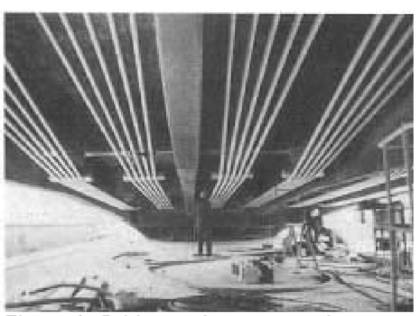


Figure 2: Bridge at Aue; external prestressed bars of the drop-in span

In 1934, Dischinger was granted his patent DRP 727,429 (Fig. 1). It contains the innovative idea of post-tensioning reinforced concrete girders with external tendons. For the determination of the magnitude of prestressing, he proposed the concept of concordant prestressing, which later became known as the "load-balancing" method. Dischinger's main concern was the long-term deformation due to the time-dependent, visco-elastic behaviour of the concrete. He was aware of the pioneering work of Freyssinet and his classical experiments, carried out in the years 1926 to 1929. While Freyssinet clearly recognized the nature of concrete with regard to creep and shrinkage [4], it was Dischinger who first proposed a valid mathematical model in 1939 [5]. Thus, in the absence of a sound theory in the mid-thirties, it was quite logical for Dischinger to opt for external post-tensioning. He wished to retain the possibility of restressing the tendons should undesirable deflections occur. Furthermore, Dischinger specifically mentioned in his publications the longer life of such tendons resulting from the reduced influence of fatigue loadings and the system-inherent possibility of replacing tendons, even under traffic, should this be required.

In the years 1936 and 1937, these ideas were applied in practice for the now quite well known bridge crossing the valley basin and the railway lines at Aue, Saxony, now the German Democratic Republic (Fig. 2, 3). For the main spans (25.20 - 69.00 - 23.40 m), external tendons consisting of smooth bars with a yield strength of 520 N/mm² and a diameter of 70 mm were used [6]. Due to World War II and its aftermath, the originally planned restressing operations were not performed until 1962, together with other repair work [7]. In 1983, the original bar tendons were again stressed [8]. Today, the bridge has been in service for more than 50 years. Some years ago the German Democratic Republic listed this remarkable structure as one of its technical monuments.

In the late thirties and early forties, Dischinger designed other road and railway bridges with spans of up to 150 m. The construction of the Warthe Bridge in Posen (today Posnan, Poland) with spans of 55.35 - 80.50 - 55.35 m was stopped because of the war. The external tendons, consisting of steel ropes ⌀ 65 mm, were already on site. They were, however, more urgently needed as external tendons for post-tensioning the heavy reinforced concrete trusses carrying traveling cranes in a large steel mill [9]. Dischinger also conceived composite bridges with external post-tensioning [10].

Based on Freyssinet's ideas, Wayss & Freytag AG designed and constructed in 1938 the bridge over the Dortmund-Hannover Autobahn in Oelde, FRG, where for the first time high tensile prestressing steel arranged inside the concrete section was used for 4 simply supported girders of 33 m span. The pre-tensioning method was applied and the

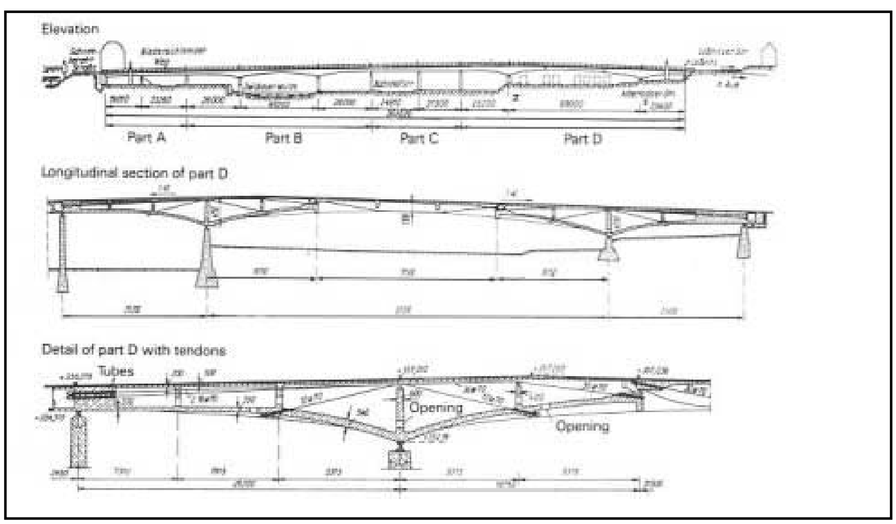


Figure 3: First prestressed concrete bridge: Bridge over valley basin and railway at Aue, German Democratic Republic (designed by Dischinger)

Figure 4: Bridge over River Aare at Aarwangen, Switzerland

prestressing steel was therefore bonded to the structure [ll]. This was actually the first bridge in what is now called conventional prestressed concrete.

In the same year, Finsterwalder developed his concept of the “self-stressing ” concrete beam, which was put to the actual test for the bridge over the same Autobahn at Rheda-Wiedenbrück, FRG (a simply supported girder of 34.50 m span). The external bar tendons ∴ 65 mm with a yield strength of 520 N/mm^2 were stressed by the self-weight of the superstructure using a hinge at mid-span and a precamber of 272.5 mm. The bar tendons were later encased in concrete [12].

In the years 1938 to 1943, Haggbohm designed and built the Klockestrand Bridge (Fig. 5), near Stockholm, Sweden [13]. For the main spans (40.50 - 71.50 - 40.50 m) the Dischinger concept was applied. The main span superstructure was prestressed with a total of 48 bars of ∴ 30 mm having a yield strength of 520 N/mm^2.

It is worth mentioning that these four bridges, of which three utilize external post-tensioning, are still in service today after more than 50 years of use.
Why was external post-tensioning virtually discarded in the succeeding years?

Under the influence of Freyssinet and other prominent engineers, the advantageous characteristics of structures with bonded tendons were emphasized. These characteristics include the higher utilization of the prestressing steel under ultimate bending moment (both with regard to achievable tendon eccentricities and stress increase up to yield strength) and the “free-of-charge ” corrosion protection by the surrounding concrete.
In 1949, Dischinger also was “converted” and became an advocate of the bonded concept. Despite this pronounced trend, external post-tensioning did not disappear completely. Several externally post-tensioned bridges were constructed in France [14], Belgium [15], Great Britain and a few other countries. Not all of these projects were successful. In some cases the corrosion protection system chosen did not fulfill the required purpose and tendons had to be replaced.

Also during this period, the first applications of external tendons for the strengthening of existing structures can be found. An early example is the two-span steel truss bridge (48-48 m) over the River Aare at Aarwangen, Switzerland (Fig 4). This bridge was built in 1889 and was no longer capable of supporting modern traffic foads. In 1967 the bridge was strengthened with two locked-coil strands ∴ 63 mm having an ultimate strength of 1,370 N/mm^2 [16].

A rebirth of external post-tensioning can be observed from the mid-seventies onward. Freeman Fox and Partners

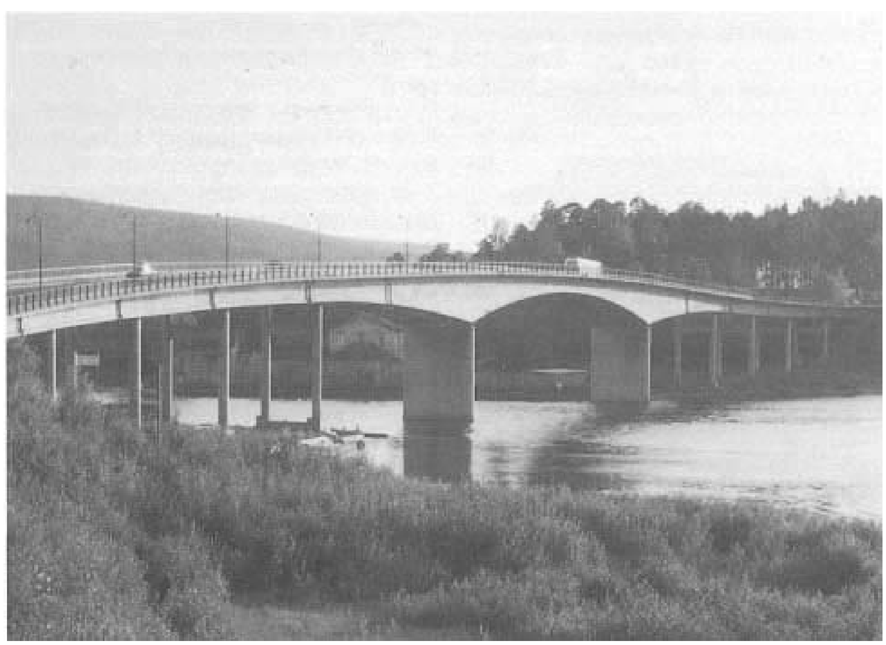


Figure 5: Klockestrand Bridge near Stockholm, Sweden

designed the Exe and Exminster Viaducts in England (see para. 6.2.1.) using external tendons each consisting of a bundle of 19 greased and plastic-sheathed strands Ø 13 mm. The prime objective was to minimize the weight of the superstructure to overcome difficult soil conditions.

The main developments certainly came from French engineers. In 1978/79 Muller introduced external post-tensioning in the United States, for the Key Bridges in Florida [17]. His main goals were speed of construction and economy. Many other important structures followed [18]. Since 1980, many bridges have been designed and built in France under the auspices of Virlogeux of SETRA (State design office of highway authority) using either external tendons or a combination of internal and external tendons [19]. At present several sizeable bridges using external post-tensioning are in planning and under construction in Switzerland and in the Federal Republic of Germany (see para. 6.2.9.).

It is true that external post-tensioning is primarily applied in bridges. There are, however, applications for other types of structures such as large-span roofs [20] and for the strengthening of buildings, silos and reservoirs [21].

1.2. Areas of application today

External post-tensioning can be used for new structures as well as for existing structures needing strengthening. As Will

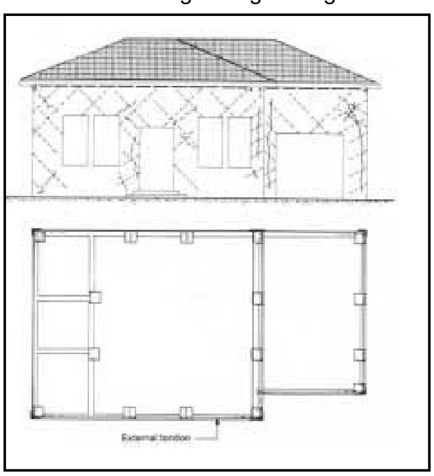


Figure 7: Strengthening of a villa damaged by the Friuli earthquake in Italy, 1987 [23]

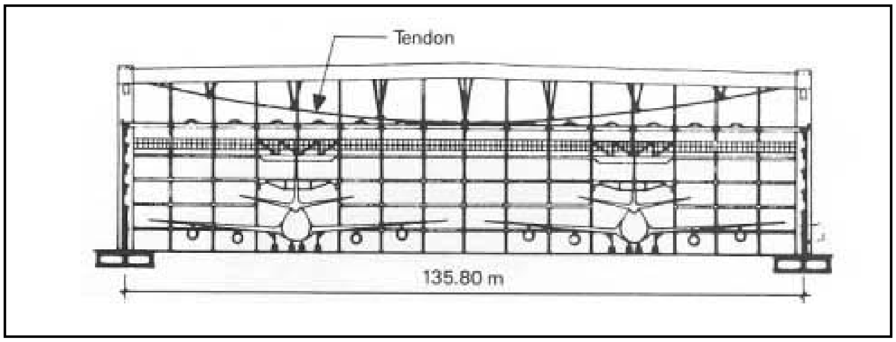


Figure 6: Hangar at Belgrade International Airport, Yugoslavia; roof with external tendons

be shown, the application is by no means restricted to concrete structures. Any material with reasonable compression characteristics can be combined with external tendons. Thus, applications in structural steel, composite steel-concrete, timber and masonry structures are known. As mentioned earlier, the technique has been used for various types of structures such as:

- Bridge superstructures

- Girders in buildings (see para. 6.5.2.)

- Roof structures (Fig. 6)

- Circular structures such as silos, reservoirs and large masonry chimneys (see para. 6.3.1. and 6.5.1. and [21], [22])

- Buildings with masonry walls (Fig. 7).

In the following chapters, the explanations are limited to the application of external post-tensioning to bridges, which at this stage represent the main field of activity.

In designing a new bridge superstructure, a designer may opt for internal or external tendons or a combination of both. Whereas for many years internal tendons were selected almost exclusively, there are a number of good reasons for deciding on external tendons.

It is interesting to note that some of the arguments used previously to promote internal, bonded tendons are now weighed differently. There is no doubt that most bridges with internal, cement grouted tendons behave very well. In some cases, however, substandard concrete (exhibiting high porosity, excessive carbonation etc.), missing or deteriorated bridge deck insulation (allowing free attack by de-icing chemicals), badly cracked concrete (resulting from inadequate design, insufficient provision of minimum reinforcement etc.) and incomplete filling of the tendon ducts by cement grout etc. have contributed to the rapid degradation of the prestressing steel by corrosion attack. Since no reliable, non-destructive inspection methods for internal tendons yet exist, it is very difficult to properly assess the degree of deterioration. Furthermore, internal tendons can neither be detensioned nor removed.

On the other hand, external tendons provide desirable features, such as the possibility of controlling and adjusting the tendon forces, inspecting the corrosion protection and replacing tendons, should this become necessary. This is, however, possible only if the tendon system together with its anchorages and saddles is designed accordingly.

Other advantages of external post-tensioning include:

- The absence of tendons inside a web means that pouring of concrete is made easier; there is no weakening of the compression area due to ducts. In this way a minimum web thickness is achievable.

- A polygonal tendon layout allows angular deviations to be concentrated at carefully designed saddle locations, thus eliminating the influence of unintentional angular changes (wobble effect).

The difference in mechanical behaviour between internal, bonded and external tendons is discussed in Chapter 2.

With regard to material quantities, it should be mentioned that with external tendons the concrete dimensions can normally be reduced. However, due to reasons such as the reduction of the available tendon eccentricity, the amount of prestressing steel generally needs to be slightly increased (Fig. 8).

1.3. Types and components of external tendons

1.3.1. General

A great variety of tendon types has been used and is described in the technical literature. It is outside the scope of this report to discuss all possibilities at length.

Essentially, an external post-tensioning tendon consists of the following elements:

- prestressing steel as tensile members,
- mechanical end anchorage devices,
- corrosion protection systems.

In the case of deflected tendons:

- saddles at points of deviation are also required.

1.3.2. Prestressing steel

At present, most material standards for prestressing steel distinguish between smooth and ribbed bars, wires and strands. No official worldwide statistics of the market shares are available. Unofficial figures, however, suggest that today the total built-in tonnage of steel consists of 75% strands, 15% wires and 10% bars. Whereas strands and wires can be applied more or less universally, bars are normally limited to short, preferably straight tendons of up to 20 m length. Table I compares the various characteristics of the most commonly used prestressing steels (based on official approval documents from the Federal Republic of Germany [24]). The figures are self-explanatory. Furthermore, the excellent groutability and the favourable ton per force price of the strands may be emphasized. It therefore becomes clear why strands have taken such a large share. This trend is continuing. Usually seven-wire strands of ∴ 13 mm (0.5”) or ∴ 15 mm (0.6”) with low relaxation properties (stabilized material) are used.

The question arises whether one day prestressing steel will be replaced by other materials, such as glass, aramide or carbon fibres. Despite the fact that in recent years engineers and researchers have made serious efforts [25], [26], the authors believe that for many years to come the application of such materials will be limited to demonstration research projects and applications with special conditions. One of the major reasons preventing a rapid transition to other materials is the much higher costs compared with prestressing steel.

1.3.3. Tendon anchorages

Until very recently, external tendons were anchored with the same mechanical devices as those used for ordinary internal, bonded post-tensioning. Under the prevailing economic circumstances, under which the suitability of a structure is judged primarily on the basis of initial construction costs, this seems to be the normal choice.

From a technical point of view, it should be remembered that the anchorages for external tendons must withstand the tendon force plus any potential subsequent force increase during the

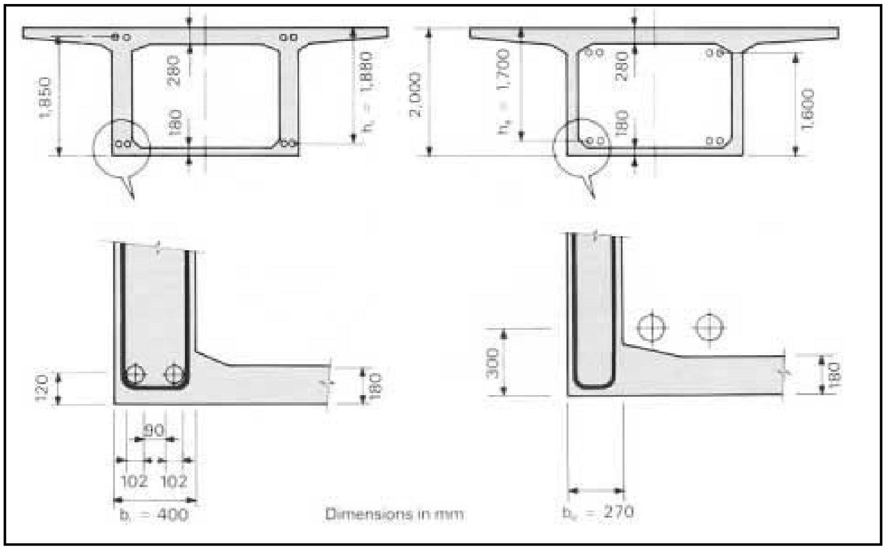


Figure 8: Comparison of box girder geometry with internal and external tendons

		Bar Ø 36 mm hot-rolled, cold-worked and tempered, ribbed	**Wire Ø 7mm** cold-drawn stabilized	**Strand Ø 13 mm** cold-drawn stabilized
Ultimate tensile strength	N/mm²	1,230	1,670	1,770
Yield strength	N/mm²	1,080	1,470	1,570
Min. elongation at rupture	%	6	6	6
Relaxation from 0.7 f_{pkt} after 1,000 at 20°C	%	3.3	2	2
Modulus of elasticity	N/mm²	$2.05 \cdot 10^5$	$2.05 \cdot 10^5$	$1.95 \cdot 10^5$
Fatigue amplitude (N/mm²) : 2 . 10^6 load cyclesat upper stress of 0.9 f_{py}	max.	210	430	250
	min.	210	265	205
Min. diameter of curvature at max. allowable stress of f_{py}	m	6.83	0.98	0.85
Friction coefficient σ#		0.50	0.17	0.19

Table I : Characteristics of prestressing steels (according to German approval documents) lifetime of a structure. For anchorages meeting the requirements of approval regulations such as those published by FIP [27], the effectiveness is normally given.

As mentioned earlier, external tendons can provide additional features such as the possibilities of monitoring, adjustment, replacement etc. These are increasingly attracting the interest of maintenance-conscious bridge authorities. Such operations are not possible with typical post-tensioning anchorages of bonded systems. Specially designed devices are therefore required and have been developed accordingly (see Chapter 3).

Evironmental class	Environmental conditions
1 Modest	Structural elements always dry or under water
2 Moderate	Structural elements subject to moist conditions
3 Severe	Structural elements subject to permanent humid conditions and / or changing wetting and drying conditions
4 Aggressive	Structural elements subject to aggressive conditions

1.3.4. Corrosion protection systems

It is known that prestressing steel needs careful protection against the various types of corrosion attack. For internal, bonded prestressing this protection is provided by the alkaline environment of the cement grout and the surrounding concrete. Experience has shown, however, that there are several aspects to which attention must be paid, in both design and construction, to make the protection really effective.

In [28] a corrosion protecting strategy is proposed, which is summarized in Table II. It is in line with more recent recommendations in various national standards. In addition to the given design measures, adequate materials and good workmanship are needed. From comparisons with the practice of the past and experience gained with existing structures, it has been recognized for some time that improvements are necessary with regard to concrete quality, detailing and the amount of reinforcement.

For external tendons, other means of protection are required. However, as for internal tendons, it seems advisable to apply a corrosion protection strategy which is based primarily on environmental conditions and also safety considerations (e.g. with regard to fire, strand failure, etc.).

Many different solutions have been adopted in the past [14]:

a) Zinc coating: Its corrosion resistance depends upon the type of galvanization and the applied thickness. Zinc coated prestressing steel has been used in France on several occasions. There is doubt, however, as to whether zinc coating provides a permanent corrosion protection. It seems to be durable only under very favourable environmental conditions. As reported in [14], coatings have been damaged during handling and installation. Another problem arose when zinc accumulated in the stressing anchorage inside the wedges.

b) Polymer coating: This technology, in which polymers are bonded to the steel by fusion, has been developed in the United States primarily for the protection of reinforcing steel. Polymer coated strands have also been available for some time [29] and a number of applications are reported (see para. 6.5.2.). It remains to be seen whether this

Environmental class (see above)	Prestressing steel in tension zone under sustained loads		Special protection measures necessary		Allowable design crack width (mm) under sustained loads			Concrete cover (nominal values in mm)		
					Prestressed concrete		Reinforced concrete	Prestressed concrete		Reinforced concrete
	Post-tensioning	Pre-tensioning	Post-tensioning	Pre-tensioning	Post-tensioning	Pre-tensioning		Post-tensioning Sheathing	Pre-tensioning Steel	
1	Yes		No		0.2	0.1	0.4	40*)	35	25
2	Yes		No		0.2	0.1	0.4	50*)	45	35
3	Yes	Yes	No	Yes	0.2	* *)	0.25	50	* *)	45
		No		No		0.1			55	
4	Yes		Yes		*)	*)		*)	*)	
	No		No		0.2* * *)	0.1* * *)	0.25	60	65	55

*) Corrosion protection not relevant for cover of sheathing * *) Not relevant for corrosion protection * * *) Under rare load combinations

Table II: Corrosion protection strategy for internai, bonded post-tensioning tendons, pre-tensioning and reinforcing steel (as proposed in [28]) system will prove to be a viable solution for prestressing steel. Problems could occur due to the fact that only the outer strand surface is protected, the king wire and inner surfaces of the six surrounding wires having no coating. At the anchorages, the coating is locally interrupted by the indentations of the wedge teeth. It is also possible that, as with zinc coated strands, problems may occur in the anchorages. Special care must be taken to prevent damage to the coating during handling and installation.

c) Protective sheathing: The protective sheathing represents an envelope around the prestressing steel. Suitable materials are steel or plastic tubes (polypropylene [PP] or polyethylene [PE]). In order to achieve an effective protection system, proper solutions are required for coupling these tubes with each other, with the anchorages and with the saddles.

Injection of the remaining voids inside the sheathing with cernent grout has proven to be economical and reliable. In the case of restressable anchorages, cernent grout must be replaced at least focally by grease or similar soft plastic material. Particularly in France, grease and wax products have been applied, instead of cement grout, on the entire tendon length [14], [30]. Besides being rather expensive, these products are difficult to inject (e.g. preheating up to 100° C required) and special measures are needed to prevent leakages (see also para. 6.2.5.).

In this category, individually greased and plastic-sheathed monostrands offer many advantages. They are manufactured under factory conditions. The prestressing steel is therefore effectively protected against corrosion during transportation, storage on site and installation, provided that proper care is taken not to damage the sheathing. Monostrands can be used either individually or in bundles as multistrand tendons. In the latter configuration they are usually placed inside a plastic or steel tube. The remaining voids are filled with cernent grout.

1.3.5. Saddles at points of deviation

When designing saddles it is important to consider the following:

- Saddle arrangements: Various solutions have been used in practice (see Fig. 9). In most cases saddles consist of a prebent steel tube cast into the surrounding concrete or attached to a steel structure by stiffening plates. The connection between the free tendon length and the saddle must be carefully detailed in order not to harm the prestressing steel by sharp angular deviations during stressing and in service; also, the protective sheathing must be jointed properly.

  If tendon replacement is a design requirement, the saddle arrangement must be chosen accordingly (e.g. double sheathing; see Alt. 3, Fig. 9).
- Minimum radii: Limits must be respected because otherwise either the prestressing steel or the protective sheathing could suffer. Although some tests exist indicating reasonable values, which may be used for preliminary designs, more research work is required in this respect. It is therefore advisable to verify the feasibility of a particular practical solution by tests.

1.4. Future developments

The revival of external post-tensioning has been a stimulus for engineers. Further innovation may be expected or indeed can already be seen on the horizon. This will include progress in materials (e.g. corrosion protection systems), in design proçedures, in structural concepts and in construction technology.

The following few examples Will highlight what could be expected:

- Bridge superstructures with underlying external tendons: It is not an entirely novel idea to arrange external tendons underneath the bridge girder. For example, this concept has been used for steel bridges, such as the Neckar Valley Bridge at Weitingen, Federal Republic of Germany [31], (Fig. 10), and for the Bridge Obere Argen, Federal Republic of Germany [32], [33]. In both cases, the extremely difficult local soil conditions led to such a design. In [34] Wittfoht proposes underlying external tendons as a standard solution for box girder bridges for road or rail traffic. Menn describes a similar system for slab bridges, by which the feasible span range can be extended up to about 40 m [35], (Fig. 11). A comprehensive test programme for determining the structural behaviour has been carried out at the Swiss Federal Institute of Technology (ETH), Zurich, Switzerland [36] (Fig. 12).

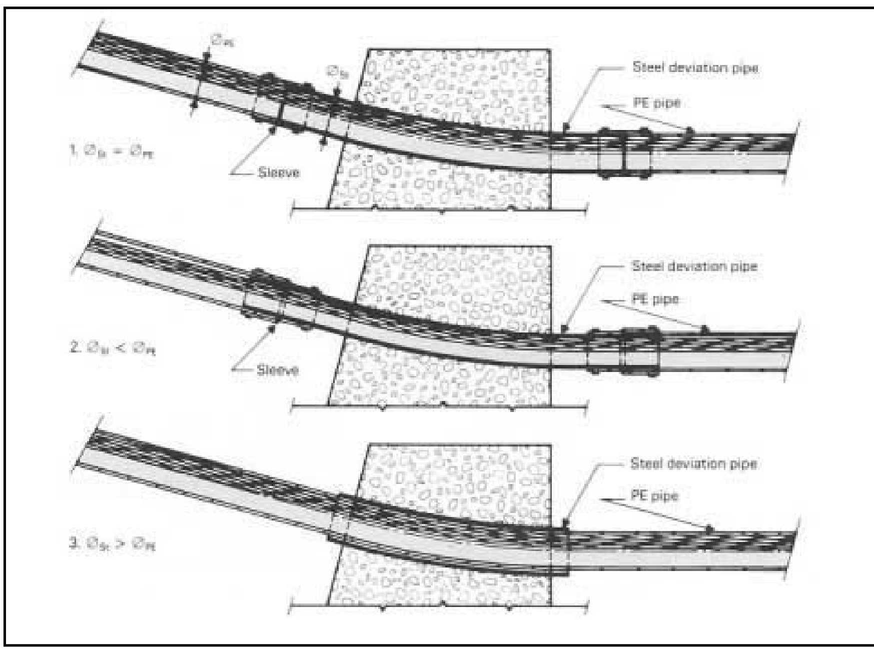


Figure 9: Various saddle arrangements

New corrosion protection systems: In related fields of application, new concepts have already been implemented. It is known that corrosion does not occur in a dry environment (relative humidity Ω 40%). This fact has been utilized in steel construction. The designers of the suspension bridge over the Little Belt (1970) and later on of the Far ∴ Bridges (1985), both in Denmark, introduced a dehumidification and ventilation system for the interior of the large steel box sections, thus protecting the inside surfaces against corrosion [37]. In Sweden, the Swedish State Power Board used a specially designed dehumidification and ventilation system for all containment tendons of Forsmark 1-3 Nuclear Power Stations. It is obvious that the conditions in a well-attended power station are more favourable than in an ordinary bridge structure. In the future, however, further developments in this direction may be expected.

Monitoring of tendons: External tendons make possible monitoring of the tendon force and of the soundness of the tendons. Refined techniques for monitoring the integrity of the corrosion protection systems and for inspecting the tendons are being developed.

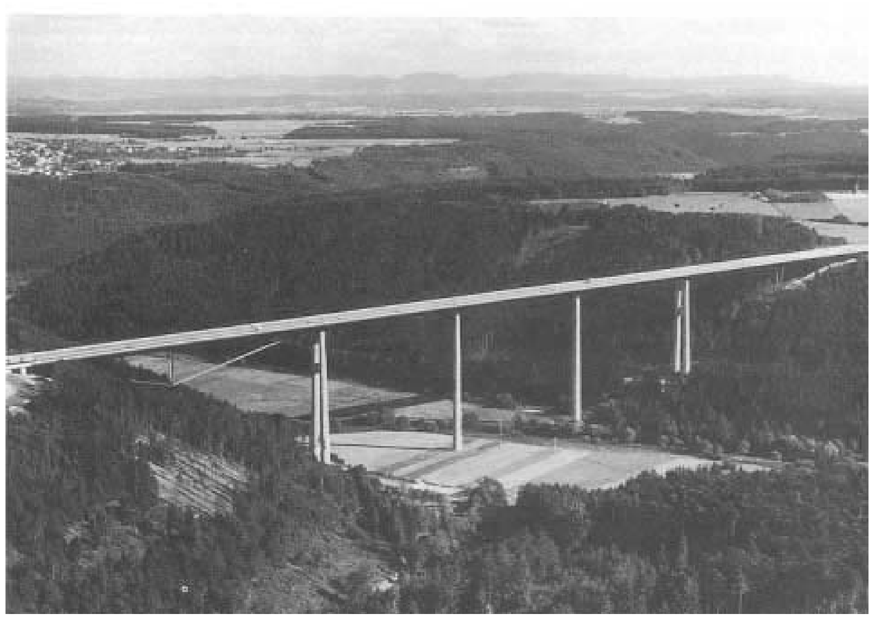


figure 10: Neckar Valley Bridge at Weitingen, Federal Republic of Germany

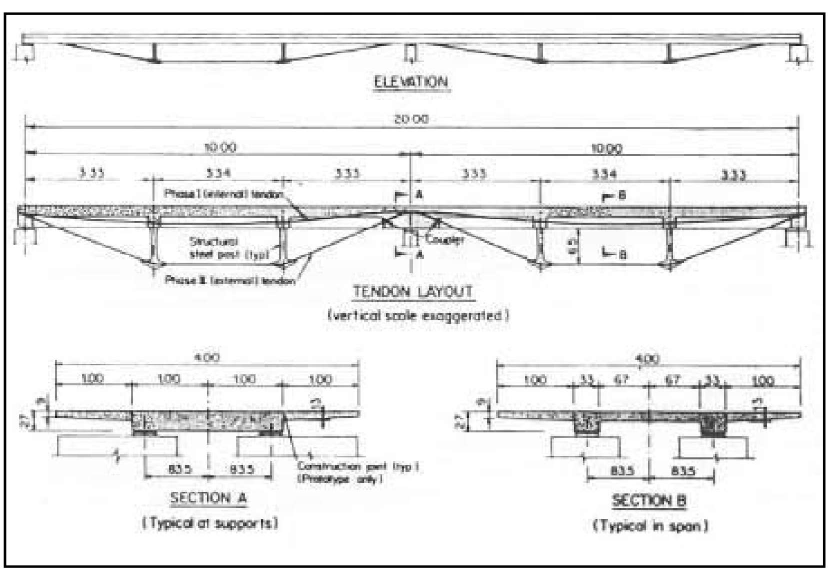


Figure 11: Details of slab bridge with underlying external tendons

Figure 12: Scale model

2. Design Considerations for Bridges with External Tendons

2.1.General

The purpose of this chapter is to highlight some special aspects that should be considered in the design of externally post-tensioned bridge super-structures.

As for any design, it is normally at the conceptual stage that the "fate" of a structure with regard to economy and durability is determined. A straightforward structural system, good detailing, and the early integration of the construction proc-ess are the major elements of a success-ful design. In this respect, bridges with external tendons are no exception.

To obtain a satisfactory behaviour of a structure both under serviceability and ultimate limit state conditions, it is essential to recognize the peculiarities of girders with external tendons. Fig. 13 shows moment-curvature curves for a typical bridge cross-section with either bonded or unbonded prestressing. For comparison, a curve for non-prestressed bonded reinforcement is also shown. The cross-sections of the reinforcement and prestressing were chosen such that all three sections would reach the same ultimate moment. As can be seen from Fig. 13, there is no fundamental differ-ence between girders with bonded or with unbonded tendons below decompression moment. The section with unbonded tendons has a larger initial prestressing force and, therefore, a higher decompres-sion moment than the section with bonded tendons. With regard to the fatigue behaviour, Dischinger [10] already mentioned the advantage that for unbonded tendons only negligible stress fluctuations occur in the prestressing steel under live load.

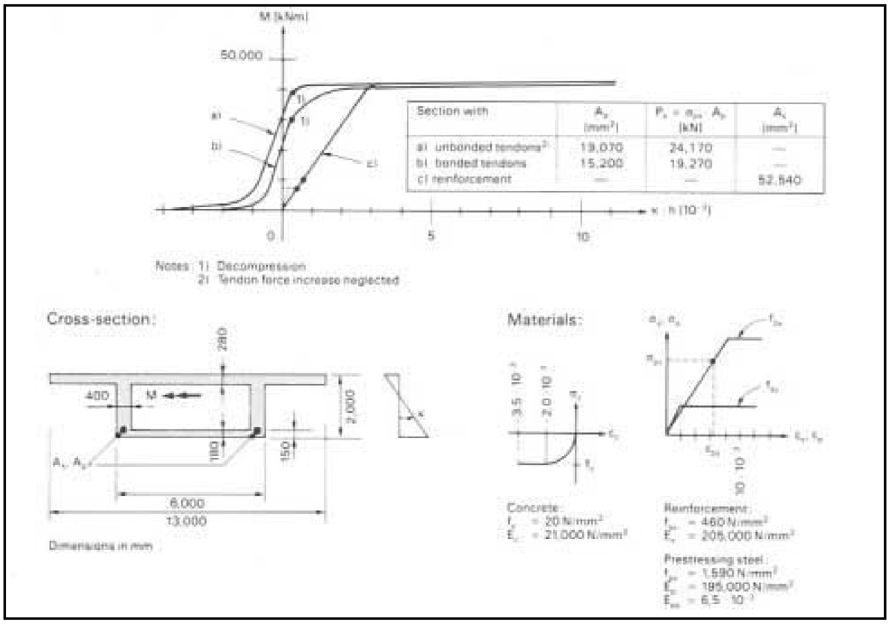


Figure 13: Moment-curvature curves for a typical bridge cross-section with bonded and with unbonded prestressing

A closer look at the behaviour of the sections is required following decompres-sion:

- Girder with bonded tendons: After decompression, the tendon force increases up to the yield strength. The tendon force increase and the associ-ated increase of the internal lever arm of the section provide a yield strength considerably larger than the decom-pression moment of the section. Owing to the bond between concrete and steef, the flexural behaviour at a section is more or less independent of adjacent girder zones.
- Girder with unbonded tendons: Because relative longitudinal displace-ments between concrete and steel are not prevented by bond, the tendon force increases only due to deformation of the entire structural system. Similar to slabs with unbonded tendons [38], the tendon force increase depends primarily on the geometry and the overall deformation of the structure as well as the tendon profile.
  For long tendons and slender struc-tures this increase will be relatively small, even for large overall deforma-tions of the system. Therefore in Fig. 13, the tendon force increase has been neglected and hence the decompres-sion moment is equal to the ultimate moment. Of course, friction at deviation points would somewhat improve the be-haviour, but unless intermediate an-chorages or partial bond at sufficiently closely spaced locations along the tendons and/or additional bonded reinforcement are provided, it shoutd be recognized that decompression essentially means ultimate.
  At any rate, the strength of an exter-nally post-tensioned girder at one par-ticular section depends on the behavi-our of the entire structural system, or at least parts of the system if intermediate anchorages are used.

Good crack distribution can only be obtained if the flexural resistance at a section exceeds the cracking moment. This principle is well known from minimum reinforcement requirements. For case a) of Fig. 13 this principle is only barely met and hence deformations may take place in just one or a few cracked sections. This may lead to an undesirable strain localization with a subsequent premature failure.

As a consequence of the described behaviour, externally post-tensioned structures are inherently more sensitive to secondary effects since, unlike bonded systems, they do not have the capability to adapt to local overloads by local yielding. Hence, while a realistic assess-ment of secondary effects is not of primary importance for bonded systems, this is quite different for externally post-tensioned systems.

In practice, continuous bonded reinforcement and partial bond of external tendons, at tendon deviations due to friction or cernent grout will contribute to increasing the ratio of flexural resistance to cracking moment and thus result in a more forgiving behaviour of the structure.

Finally, it should be mentioned that prior to grouting of the tendons a structure with bonded tendons behaves similarly to an externally post-tensioned system. In one case a bridge failed during construction [39] primarily because the special conditions of the construction stage were overlooked. This failure clearly exhibited the effects of strain localization and points to the need for a careful evaluation of all possible effects of loads and imposed deformations when designing externally post-tensioned systems.

2.2. Serviceability and ultimate limit states

2.2.1. Serviceability limit state

Usually, the amount of prestressing is selected at a relatively early stage in a project. This selection is influenced by considerations regarding serviceability and economy of the structure:

- Under dead load only, the structure shall remain substantially uncracked or existing cracks shall be closed. On the other hand, the requirement of an uncracked structure for dead and live load intcuding secondary effects might lead to undesirable long-term hogging deflections under dead load only. However, under such loading conditions cracked sections can normally be accepted if the stresses in the reinforcement are limited such that the cracks close again after removal of the load.

- From an economic point of view, one should ideally provide just enough non-prestressed reinforcement and restressing steel as necessary to obtain the required resistance.

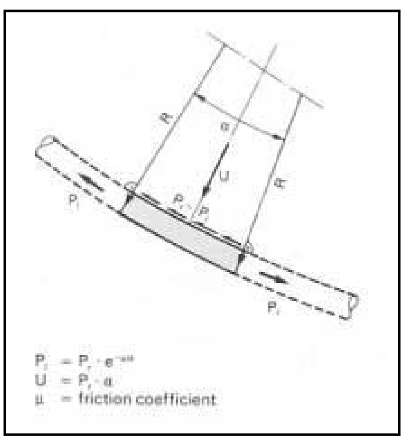


Figure 14: Forces at deviation points

A reasonable prestressing force may be estimated using the load balancing method. If a substantial part of the dead load is balanced by the prestressing, a satisfactory behaviour of the structure may be expected regarding both deflections and cracking.

As soon as the tendons and their profiles are selected, the tendon force diagram can be determined. As external tendons are generally arranged in a polygonal shape, the force diagram will have steps at the deviation points. Fig. 14 shows the forces applied to a deviation point.

Long-term losses due to relaxation of the prestressing steel, as well as creep and shrinkage of the concrete, cause a decrease in the tendon force. As long as no relative displacements between tendon and concrete occur at the deviation points, either because of the presence of high friction coefficients or because of partial bond, these losses may be estimated section by section as for bonded tendons. However, for low friction coefficients there will be some slippage between tendon and concrete and the losses may be estimated from mean axial deformations due to creep and shrinkage of the entire structure.

As mentioned in Section 2.1. structures with external tendons may be sensitive to secondary effects. Therefore, it is essential to assess tendon forces and secondary effects due to temperature, creep, shrinkage and other effects as realistically as possible when performing checks at serviceability limit states. The effects of prestressing may be considered either by the primary and secondary moment method or by the load balancing method. The first method is generally used for the final design of a structure because it allows for an easy consideration of friction losses. On the other hand, the second method is prlmarily suited for preliminary designs if friction losses are neglected.

2.2.2. Ultimate limit state

As mentioned in Section 2.1. the tendon force increase in externally post-tensioned structures will generally be rather small unless intermediate anchorages or partial bond at sufficiently closely spaced locations are provided. Hence, for ultimate timit state considerations one may opt to neglect any possible tendon force increase and use the tendon force after all losses in order to get an estimate of the ultimate resistance. Alternatively, a rigorous analysis can be performed by integrating the strain increments in the structure along the tendon axis (Fig. 15a), or an estimate of the tendon elongation can be obtained from the consideration of a rigid body mechanism (Fig. 15b).

Integration of the strain increments along the tendon axis requires an iterative non-linear analysis [14], [40]. For a given load increment and an assumed tendon force the strain increments at each section and the associated tendon force increase can be computed. Repeating the calculations with the new value of the tendon force will result in an improved estimate and after a few iterations a reasonable approximation will be obtained. Similar computations can then be made for the next load increment and so on.

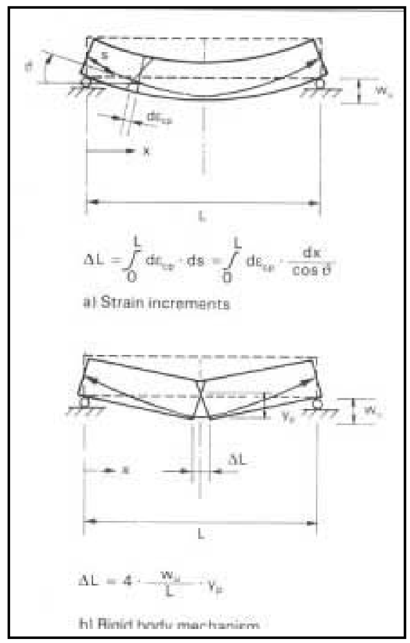


Figure 15: Tendon elongation △L

Tendon size (VSL tendon unit)	Minimum radius (m)
up to 5-19 or 6-12	2.50
up to 5-31 or 6-19	3.00
up to 5-55 or 6-37	5.00

Table III: Recommended minimum tendon radii

Rigid body mechanism considerations have been successfully applied in designing post-tensioned slabs with unbonded tendons [41], [42], [43], [44]. Typically, a nominal failure characterized by a maximum deftection of about two percent of the stab span is assumed and the resulting elongation of the tendon is determined from geometry. Knowing the elongation and the stress-strain relationship of the tendon, the tendon stress increase and the tendon force can be determined. While this procedure is well estabtished for slabs, some caution is recommended for the application to bridge girders until more information on maximum deflection values is available.

If the tendon force increase is taken into account, second order effects as exemplified by Fig. 16 have to be considered as well. However, if an appropriate number of deviation points is provided, the influence of such second order effects may be kept small.

Knowing the tendon force, the ultimate resistance of an externally post-tensioned structure can be determined using conventional methods. Similar to bonded structures, an external tendon can either be treated as part of the integral load resisting system (Fig. 17a) or it may be considered to be separated from the concrete and its action can be modelled by applying the equivalent anchorage, deviation and friction forces onto the concrete (Fig 17b). However, in contrast to bonded structures, a realistic assessment of secondary effects is usually essentiat for externatly post-tensioned systems because of their inherent sensitivity to such effects (see Section 2.1.). Hence, while a liberal attitude towards secondary effects due to imposed deformations of any sort may be assumed when designing a bonded structure, a more cautious approach is necessary for externally post-tensioned structures.

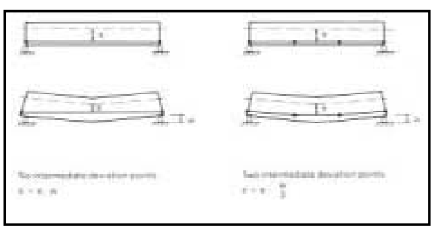


Figure 16: Influence of girder deflection on tendon eccentricity

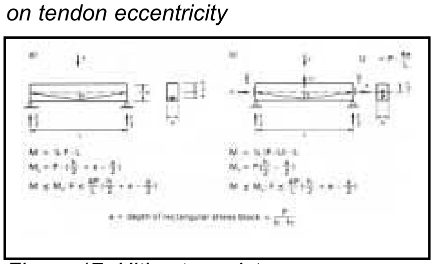


Figure 17: Ultimate resistance

2.3. Particular aspects

2.3.1. Saddles

The design and detailing of saddles at points of tendon deviations is a delicate task. An early coordination between the designer and the tendon supplier is advisable. It is of utmost importance that the forces transferred at the saddle locations are carefully evaluated. It is recommended to use higher safety factors as the proper functioning of these elements is essential for the entire structure. Typical examples of saddles are shown in Fig. 9 and 18.

2.3.2. Minimum tendon radii

Minimum tendon radii as recommended in Table Ill must be respected in order to avoid damage of the prestressing steel and the plastic sheathings as well as the outer tubing. It is also known that friction problems may occur if the tendon radii are too small.

2.3.3. Prestress losses due to friction

Similarly to conventional prestressing, the force -friction retationship can be described with the following formula:

$$P_{(X)} = P_0 \, e^{-(\sigma\# + kx)}$$

where

$P_{(X)}$ = Post-tensioning force at a distance x from the stressing anchorage

P_0 = Post-tensioning force at the stressing anchorage

e = Base of Napierian logarithms

σ# = Coefficient of friction

α = Sum of angular deviations (in radians) of the tendon in ail planes over the distance x

k = Wobble factor (inaccuracies in placing) per unit length

However, the wobble factor k can normally be neglected since the tendons are straight between the points of deviation.

Based on test results, site experience and the technical literature, the friction coefficient σ# varies as follows:

	σ#
- bare, dry strands over steel saddle	0.25-0.30
- bare, greased strands over steel saddle	0.20-0.25
- bare strands inside plastic tube running over saddle	0.12-0.15
- greased and plastic-sheathed monostrands inside plastic tube over saddle	0.05-0.07

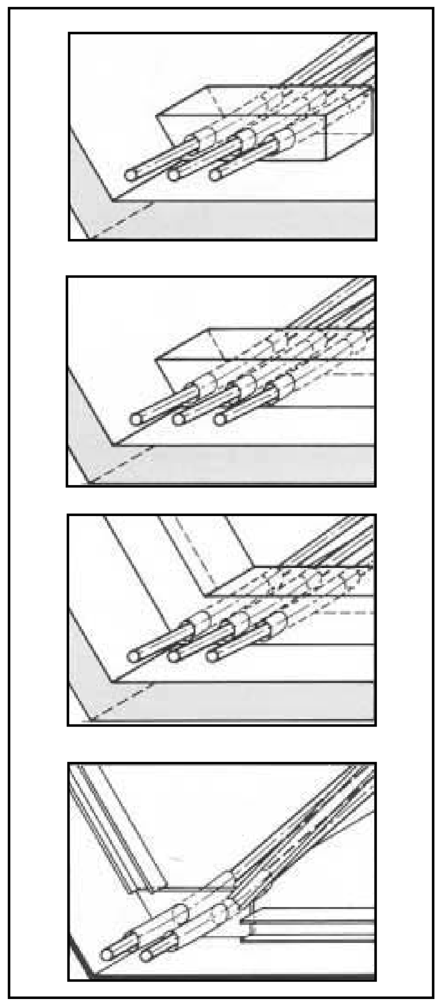


Figure 18: Examples of deviation points

3. VSL External Tendons

3.1. Introduction

The VSL External Tendons are in essence an adaptation of the well-proven VSL Post-Tensioning System [45] to the requirements for external tendons. Thus, they do not represent a completely new technology, but simply a further development of a technology relying on many years of practical experience.
The main characteristic of VSL External Tendons is the use of strands for the tensile members. As shown in para. 1.3.2., strands have a relatively high breaking strength, which results in a reduced consumption of steel. Another main advantage of VSL External Tendons is the modular principle, which enables any desired tendon size and tendon anchorage to be made up from standard units. Thus, the construction principle is always the same. On the other hand, the system is sufficiently flexible to allow for adaptation to any requirement. This means, therefore, that the information presented in Chapters 3 and 4 is merely representative and does not constitute any limitation to the possible range.

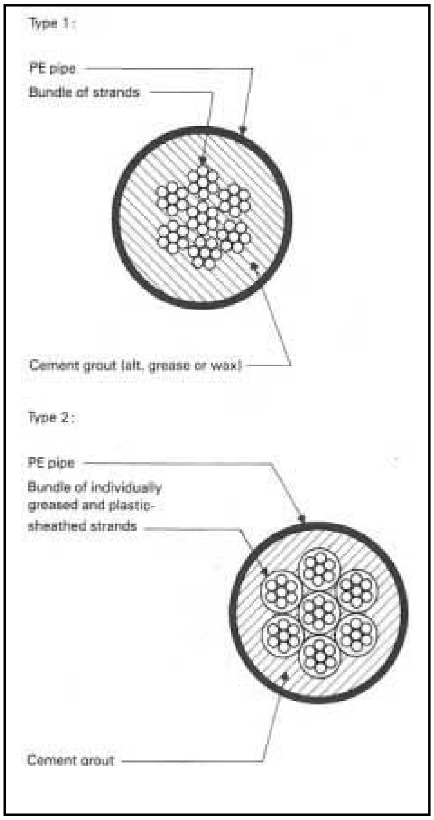


Figure 20: Cross-sections of VSL External Tendons Type 1 and Type 2

3.2. Types of VSL External Tendons and Technical Data

3.2.1. General

The VSL External Tendons consist of the following main elements (Fig. 19):

- a bundle of prestressing strands (either bare or individually greased and plastic-sheathed) as the tensile member,

- a plastic or steel tubing for the strand bundle,

- end and intermediate anchorages, and couplers,

- a grouting compound.

VSL offers two main types of external tendons (Fig. 20) which can be characterized as follows:
Type 1: Bundle of strands inside a steel or plastic tube; the grouting compound normally consists of cement grout.
Type 2: Bundle of greased and plastic-sheathed monostrands inside a steel or plastic tube; the grouting compound consists of cement grout, except in the anchorage zones, where especially suited corrosion preventive compounds are used.

3.2.2. Selection criteria

The main technical criteria for selecting the tendon type are:

- Environmental conditions and tendon exposure: as for internal, bonded prestressing, for external tendons it seems logical to select the degree of corrosion protection according to the environmental conditions and the exposure of the tendons. Based on the classes given in Table II (p. 6), it is recommended to Use Type 1 for classes 1, 2 and 3 and Type 2 for class 4.

- Need for tendon force adjustment during lifetime of the structure: in this case Type 2 is recommended.
- Tendon friction during stressing operation: as shown in para. 2.3.3., the friction with tendon Type 2 is much smaller than with Type 1. In the case of long tendons running over several spans with sizeable angular changes, Type 2 offers technical and economical advantages.

Table IV summarizes the selection criteria. It should be mentioned that other factors may influence the decision such as price, availability of materials, local practice, etc.

3.2.3. Strands

For VSL External Tendons, cold-drawn 7-wire prestressing strands of 13 mm (0.5") and 15 mm (0.6") normally of low relaxation quality are used. The geometrical and mechanical properties are given in Table V.
Whereas for tendon Type 1 bare strands are envisaged, greased and plastic-sheathed strands (monostrands) are used for Type 2. The grease has favourable characteristics with regard to

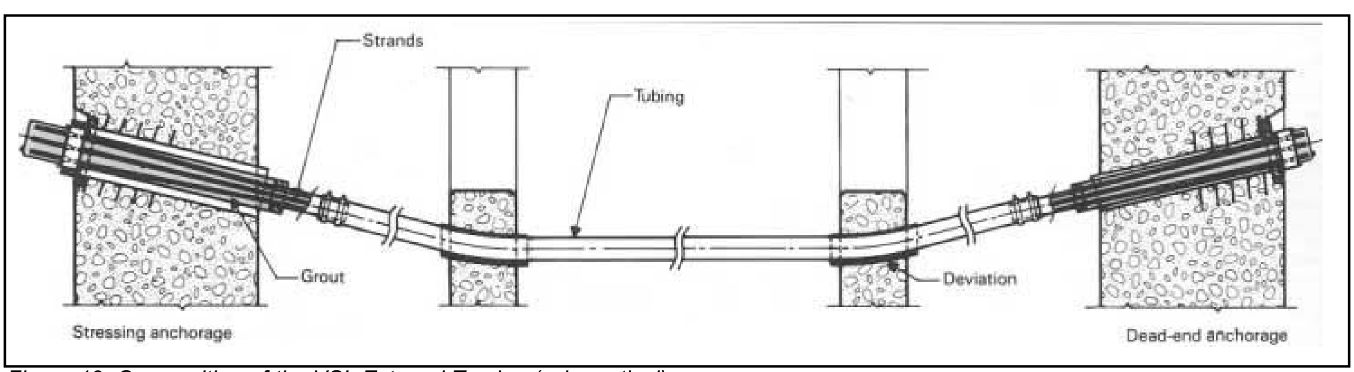


Figure 19: Composition of the VSL External Tendon (schematical)

its long-term stability and its suitability in providing corrosion protection of the prestressing steel. The sheathing is of polyethylene (or alternatively polypropylene) and has a minimum thickness of 1 mm for straight tables and 1.5 to 2 mm for curved tables.

3.2.4. Characteristic breaking loads of VSL External Tendons

Table VI gives the nominal breaking loads for the VSL External Tendons according to the four strand types as detailed in Table V. The characteristics of the strand may, however, slightly deviate from these values, depending on the manufacturer and applicable standard.

3.2.5 Tubing

The strand bundle (consisting of either bare or greased and plastic-coated strands) is usually encased in a plastic tube. Alternatively, steel tubes may be used. In certain areas, such as deviation saddles or parts of the tendon embedded in concrete, regular corrugated steel duct as normally used for post-tensioning tables may be chosen. The latter, however, is only applicable when the tendon does not need to be replaceable (see also para. 3.2.6.), and tendons Type 1 are used.

In general, the plastic material is polyethylene and meets the requirements of appropriate standards such as DIN 8074 and 8075, ASTM D 1248 and 3035 or equivalent. Alternatively, polypropylene may be used. The ratio of internal diameter to wall thickness is approximately 16:1. In general carbon black is added as ultraviolet stabilizer. This material is chemically inert against practically any foreseeable agent (see e.g. DIN 16934). It has shown excellent durability behaviour in structural applications.

In the case of steel tubes, a higher internal diameter/wall thickness ratio can be used (approx. 30:1 to 50:1). The dimensions used are primarily dictated by the availability of standardized tubes. The outer surface of the tubing is normally provided with a paint giving sufficient corrosion protection.

The plastic or steel tubing represents the prime barrier against corrosive attack. It is connected to the anchorages and the saddles, thus providing an effective and continuous envelope around the prestressing steel.

3.2.6. Anchorages

The anchorage principle of VSL Externat Tendons corresponds in essence to the VSL Post-Tensioning System. Figures 21 to 29 represent a variety of possible anchorages. Different parameters, such as required adjustability, replaceability, load monitoring, installation procedure, access to the end anchorages (e.g. for the strengthening of structures), static considerations and environmental conditions as per para. 3.2.2., influence the selection of the particular anchorage type.

The exposed surfaces of the anchorages are properly coated for corrosion protection.

3.2.7. Grouting compounds

The tubing around the strand bundle constitutes its primary corrosion protec-

	Type 1	Type 2
Environmental conditions (see Table II):		
class 1	●	
class 2	●	
class 3	●	
class 4		●
Need for tendon force adjustement:		
no	●	
yes		●
Tendon friction		
shorter tendon and small $\Sigma\alpha$	●	
longer tendon and high $\Sigma\alpha$		●

Table IV: Main technical criteria for selection of tendon type

Strand type	13 mm (0.5")		15 mm (0.6")	
	(A) Euronorm 138-79 Super	(B) ASTM A 416-85 Grade 270	(C) Euronorm 138-79 Super	(D) ASTM A 416-85 Grade 270
Nominal diameter (mm)	12.9	12.7	15.7	15.2
Nominal steel area (mm²)	100	98.7	150	140
Nominal mass per m (kg)	0.785	0.775	1.18	1.10
Yield strength (N/mm²)	1,580 80#	1#670 90#	#1,500 80##	1,670 90#
Ultimate strength (N/mm²)	1,860	1,860	1,770	1,860
Min. breaking load (kN)	186.0	183.7	265.0	260.7

1) 0.1 % proof load method 2) 1 % extension method

Table V: Strand types

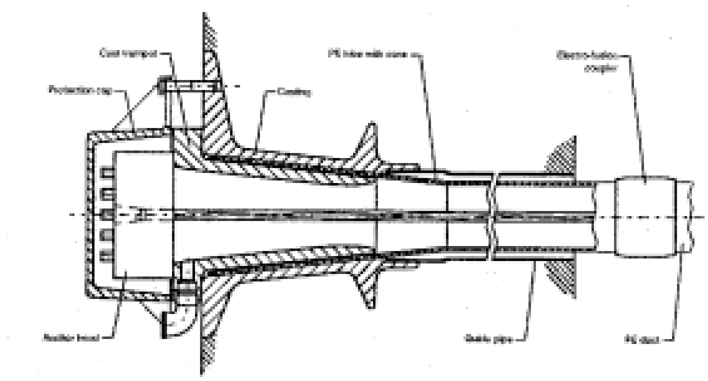
Ø 13 mm (0.5") Strand				Ø 15 mm (0.6") Strand			
		Breaking load (kN)				Breaking load (kN)	
Cable type	Max. number of strands	Strand type A	Strand type B	Cable type	Max. number of strands	Strand type C	Strand type D
5-3	3	558	551	6-3	3	795	782
5-4	4	744	735	6-4	4	1,060	1,043
5-6	6	1,116	1,102	6-6	6	1,590	1,564
5-7	7	1,302	1,286	6-7	7	1,855	1,825
5-12	12	2,232	2,204	6-12	12	3,180	3,128
5-19	19	3,534	3,490	6-19	19	5,035	4,953
5-22	22	4,092	4,041	6-22	22	5,830	5,735
5-31	31	5,766	5,695	6-31	31	8,215	8,082
5-37	37	6,882	6,797	6-37	37	9,805	9,646
5-43	43	7,998	7,899	6-43	43	11,395	11,210
5-55	55	10,230	10,104	6-55	55	14,575	14,339

Table VI: Characteristic breaking loads tion. In addition, the tendon is completely filled with a grouting compound. As mentioned under 3.2.1.. normally cement grout is used.

The grout is made from Portland cement and fulfills the same requirements as the one used in traditional post-tensioning. With its alkaline properties, it provides active corrosion protection. The grout completely fills the interstices between the strand bundle and the outer tubing.

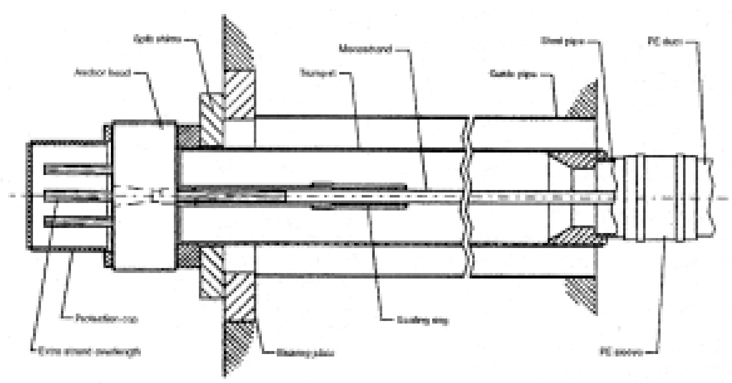
Due to the fact that the envelope reduces or eliminates the diffusion of gases and liquids, carbonation of the cement grout is inhibited.

Notes: *Bursting steel not shown for clarity.
*Figures 26 to 29 have been omitted.



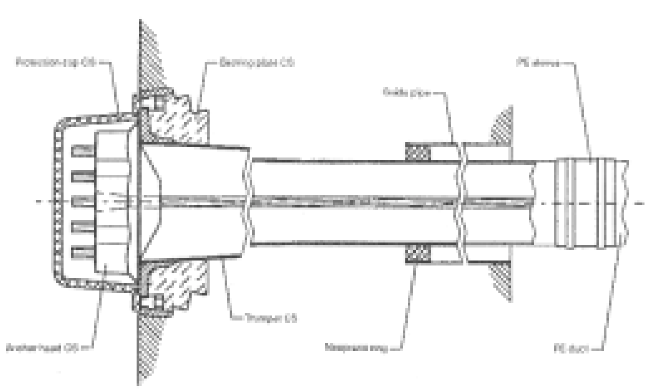
Features : Replaceable stressing or dead end anchorage where no adjustability and load monitoring is required. Available for 0.6" diameter bare strand tendons (Tendon Type 1).

Figure 21: Anchorage Type Ed



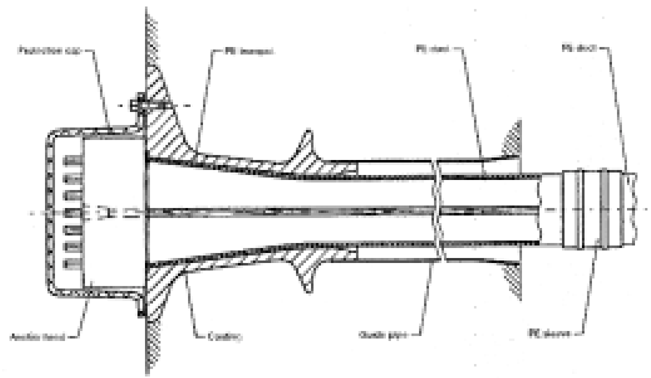
Features : Large-sized guide pipe enabling push-through for trumpet/anchor head assembly. Fully adjustable, detensionable and replaceable anchorage (Tendon Type 2). Ring nut can be used instead of split shims. Can also be used with bare strands, if only load monitoring, small adjustements or replaceability is required.

Figure 22: Anchorage Type A



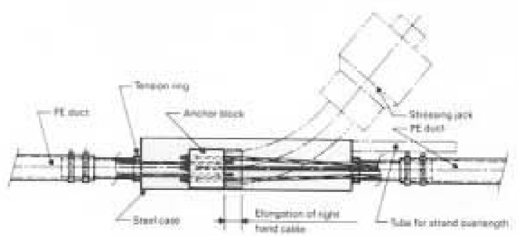
Features : Replaceable stressing or dead end anchorage where no adjustability and load monitoring is required (Tendon Type 1)

Figure 23: Anchorage Type CSd



Feature: Replaceable stressing or dead end anchorage where no adjustability and load monitoring is required (Tendon Type 1)

Figure 24: AnchorageType ECd



Composition: Anchor block with wedges, retainer plate on passive side to secure wedges, steel case. Tube for strand overlenght if detensionability or adjustability required.
Features: For tendons with insuficient access for stressing at end anchorages (e.g. strenghtening of structure) or for circular tendons.

Figure 25: Centre-stressing anchorage Type Z

Figure 30: Intermediate tendon supports

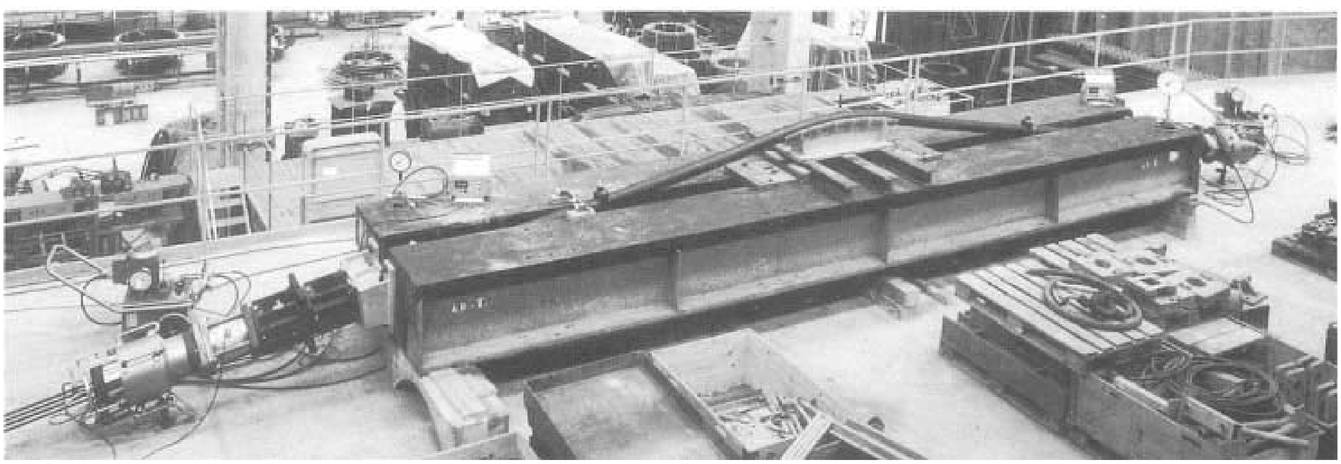


Figure 31: Test installation

3.3. Experimental evidence

Several tests have been conducted during the development of the VSL Extemal Tendons. These tests have provided valuable data for material selection and procedures, anchorage design, and friction losses in saddles. For the Bois de Rosset Viaduct project (para.6.2.9.), four tests were performed by VSL (Fig.31). The first test aimed at determining the groutability of a bundle of monostrands, especially in the saddle area. The second test involved stressing the tendon in stages up to 70% of the breaking load. To simulate actual conditions, a relative displacement of 600 mm was applied. In the third test, the tendon was slightly stressed from both anchorages prior to grouting. The tendon was then stressed to 70% of the breaking load, again applying a 600 mm relative displacement. The fourth test was similar to the third, but incorporated the improvements obtained through the earlier tests.

4. Application of the VSL External Tendons

4.1. Manufacture and installation

Basically there are two different methods used for the installation of VSL External Tendons

- Installation of completely prefabricated tendons.

- Installation of the empty tube in the final position followed by insertion of the strands.

4.1.1. Prefabrication

The method of complete tendon prefabrication is usually applied for short, light tendons where easy access on site allows the placing of the entire tendon.

Prefabrication may be carried out either in a factory or in a prefabrication area on site depending on the means of transport, the time between manufacture and installation, and the availability of adequate space on site. The standard lengths of tube are connected to achieve the required total length. PE tubes are connected by butt-welding, steel tubes by welding or with couplers.

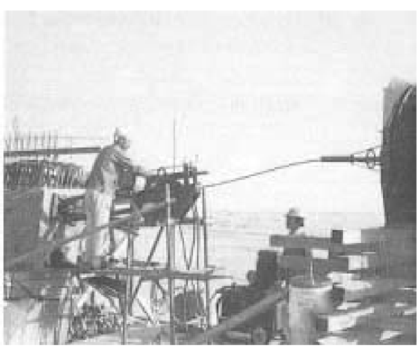


Figure 32: Pushing strand

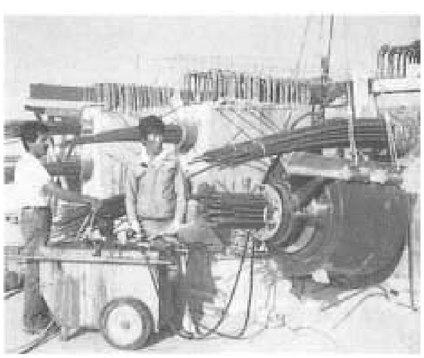


Figure 33: Stressing a VSL tendon

The strands are inserted by pushing the tube over the prepared strand bundle or by pushing individual strands through the tube.

The bearing plates or anchorage bodies and the supports at the deviation points are fixed to the structure. The prefabricated tendon is then placed into its final position either manually or by mechanical means using hoists or winches. Intermediate temporary fixings along the straight lengths are provided to keep the tendon in its correct position.

4.1.2. Fabrication in the final position

Besides the fixing of bearing plates and deviation points, it is necessary to provide temporary intermediate tendon supports along the straight length of the tendon prior to the placing of the tubes (Fig. 30).

The tube (steel or PE) is prepared in suitable sections and placed in its final position by attaching it to the previously fixed supports. The tube sections are connected by welding or by using couplers. At the ends the tube is tightly connected to the anchorages.

When the tube is securely fixed in the final position it is ready to receive the strands. The strands are inserted by pulling the prepared strand bundle (as one unit or in groups) through the tube by a winch. If the tendon consists of bare strands, the VSL push-through machine can be used by taking the strands directly from the coil and pushing them through the tube one by one (Fig. 32).

4.2. Stressing

The VSL External Tendons are stressed with the appropriate VSL multistrand jack. All the strands are stressed simultaneously but individually locked off (Fig. 33). The stressing operation normally follows the procedures established by the specifications, by local codes of practice or by the FIP recommendations.

Type 1 Tendons (bare strands) are stressed at a steady rate in one or several increments until the required stressing force is reached. Grouting is carried out after completion of the stressing operation.

Type 2 Tendons (greased and plastic-coated monostrands) are stressed in two stages. In the first stage, an initial force is applied which removes the slack in the tendon. Then the tendon is grouted. After the grout has attained the required strength, the stressing operation is continued. In the second stage, the stressing force is raised in uniform fashion to its final value.

Depending upon the anchorage type chosen, the tendon force can be checked, adjusted or released using the same multtstrand stressing jack.

4.3. Grouting

The VSL anchorages incorporate a grout connection which can be used as inlet or as outlet. Further grout connections are provided at the deviation points.

Grouting commences at the lower end of the tendon and proceeds at a steady rate until grout of the same consistency is ejected at the deviation points and finaly at the other end of the tendon. For long tendons, the grout is injected at subsequent inlets along the tendon. When using greased and plastic-sheathed strands inside a steel or PE tube (tendon Type 2), the tendon is grouted after initial tensioning by injecting cement grout into the tubing only. The anchorage zones are filled with a non-hardening corrosion preventive compound.

4.4. Completion Work

The exposed surfaces of the anchorages are properly coated for corrosion protection.

As a result of the simplicity of the construction principle, the high quality of the materials used and the excellent corrosion protection, VSL External Tendons are virtually maintenance-free.

The use of anchorages type A_m, (anchor head with thread) enables the attachment of a VSL Load Cell (Fig. 34), allowing the monitoring, checking and small adjustments of the tendon force during the whole life of the structure. Anchorages type A_s, and A_r, provide for adjusting and detensioning of the tendon force and, if required, replacing of the entire tendon.

5. VSL Service Range

5.1. General

The VSL Organizations provide a comprehensive range of services in connection with externally post-tensioned structures, including:

- Consulting services to owners, engineers and contractors
- Preliminary design studies
- Assistance with the design of externally post-tensioned structures
- Detaifed design of external tendons
- Supply and installation of external tendons
- Supply of materials, equipment, and supervisory personnel
- The use of other VSL Systems, such as slipforming, soil and rock anchors, heavy lifting, bearings, expansion joints etc.

The extent of VSL's services will usually be clarified in discussions between the owner, engineer, contracter, and the VSL Organization.

In many cases the application of several VSL Systems is possible on a single project. This enables the use of labour and material to be rationalized with consequent cost savings.

At this point, reference may be made to other VSL publications which are of importance in the construction of externally post-tensioned structures:

- Pamphlet "VSL Post-tensioning" [45]
- Technical report "Concrete Storage Structures" [21]
- Technical report "The Incremental Launching Method in Prestressed Concrete Bridge Construction"
- Technical report "The Free Cantilevering Method in Prestressed Concrete Bridge Construction".

In addition, the following VSL publications are available that may also be of interest in connection with externally post-tensioned structures:

- Pamphlet "VSL Slipforming"
- Pamphlet "VSL Soil and Rock Anchors" [2]
- Pamphlet "VSL Heavy Lifting"
- Pamphlet "VSL Messtechnik/Measuring Technique/Technique de mesure"
- Pamphlet "Life Extension and Strengthening of Structures"
- Various Job Reports
- VSL Newsletters.

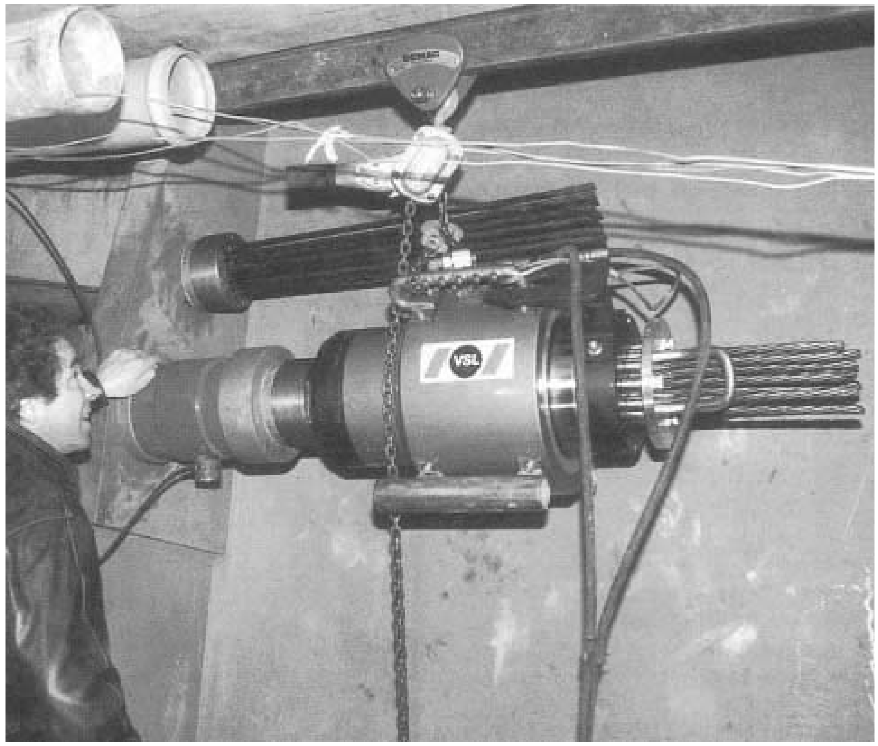


Figure 34 : Stressing of an external tendon equipped with a VSL Load Cell

5.2. Tender Preparation

The basic requirements for a tender for one or more of the above services, in so far as they concern the carrying out of detailed design, supply, installation and execution, are detailed drawings and specification documents. This applies both for structures which are about to be constructed and also for Clients proposals which require further technical development, or to which alternative proposals are to be prepared.

A VSL tender for external post-tensioning may consist of:

- Supply of material plus manufacture and complete installation of the cables, including provision of personnel and equipment, or
- Supply of material, provision of supervisory personnel and provision of equipment.

The first solution, in most cases, will prove to be the better one and therefore should be selected as a rule. The foremost reason is quality assurance. The durability over the lifetime of a structure mainly depends on quality of materials and on quality of workmanship. The experience available with VSL, whose personnel is engaged exclusively on post-tensioning, is the most suitable for the effective manufacture and installation of tendons. Another reason is economy. A specialist worker can achieve a better rate of progress both by his experience and by the advanced type of equipment he has at his disposal. He will require less time to solve unforeseen problems on site.

In addition, considerable savings are possible if a Main Contractor investigates jointly with VSL how best to use the available material, plant, equipment and methods for a specific project. VSL has, for this purpose, built up its own design engineering staff. The combination of engineering skill with detailed knowledge of the possibilities and special features of the VSL Systems has proved to be an attractive service to Main Contractors for optimizing their construction work.

6. Examples from practice

6.1. Introduction

The purpose of this chapter is to give the reader information about a number of structures in which VSL External Tendons have been used, and at the same time to illustrate in what structures external tendons are applicable. The descriptions include concrete and steel bridges, buildings, silos and other structures. The chapter is split into a first part presenting structures that have been designed for the use of external tendons from the beginning, while the second part comprises structures which had to be strengthened by means of external tendons.

The job reports show that external post-tensioning is applied in various countries; a certain predominance of France and the USA cari,, however, be observed because these countries are at the forefront of the development of this technique. Wherever available, details regarding the design of the structures and the reasons for the selection of external post-tensioning are also presented.
In both parts, the projects are listed chronologically. The designs of the older structures may differ from what is outlined in the previous chapters, but at the same time they may make evident the improvements in the state of the art achieved over the past years.

Figure 35: Exe and Exminster Viaducts

6.2. Bridges originally designed with external tendons

6.2.1. Exe and Exminster Viaducts near Exeter, Great Britain

Owner	Department of Transport, South Western Road Construction Unit
Engineer	Freeman Fox and Partners, London
Contractor	Cementation Construction Ltd., Croydon
Post-tensioning	Losinger Systems Ltd., Thame
Construction Period	1974-1976

These two viaducts are part of the motorway M5 linking Birmingham in the Midlands to Plymouth in the south. Not far from Exeter the two structures, which are separated from each other by an embankment approx. 380 m long, cross the Exe Valley. The Exe Viaduct spans the river Exe and the Exeter Canal, while the Exminster Viaduc$ carries the motorway across a double-track railway line (Fig. 35).

The Exe Viaduct has a length of 692 m and comprises eleven spans (53.50 - 9 x 65.00 - 53.50 m). The Exminster is 302 m long and has five spans (53.50 - 3 x 65.00 - 53.50 m). The superstructures of both bridges consist of two parallel, twin-cell box girders of 7.00 m width and 2.80 m depth, having 5.00 m cantilever wings on each side. Diaphragms are provided in the boxes at 7.50 m and 10.00 m distances.

The viaducts were designed with the objective of obtaining the lightest possible structure, as soil conditions were found to be poor. Therefore, the dimensions of the superstructure were minimized and the post-tensioning tables placed inside the

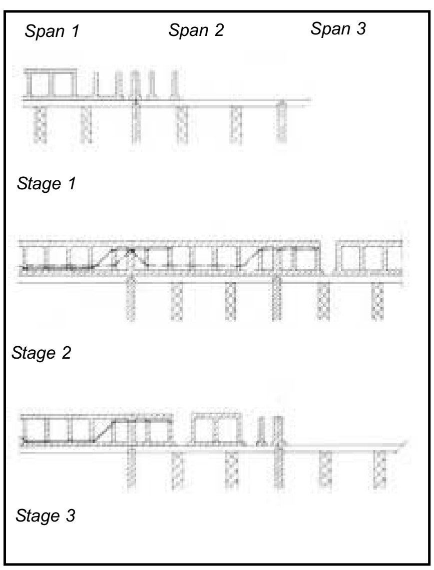


Figure 36: Construction procedure

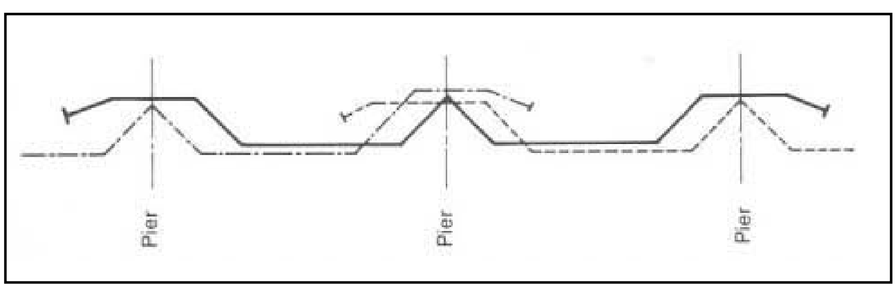


Figure 37: Tendon layout scheme

boxes. The construction procedure was chosen accordingly. Construction of the superstructure was split into two phases. First the boxes were constructed on falsework, while the cantilever slabs were added later, using movable shuttering.

The technique applied in the construction of the boxes was as follows (Fig. 36):

- Stage 1: Construction of span 1 plus 115th of span 2 by successively concreting diaphragms, base slab and webs, soffit slab.
- Stage 2: Installation and stressing of the first half of the span 1 tables, concreting of the remainder of span 2.
- Stage 3: Moving of scaffolding and from span 1 to span 3 formwork, installation and stressing of tendons covering spans 1 and 2, concreting of span 3.
- repetition of cycle.

The tendons were made up from monostrands and are not encased in any additional sheathing between the diaphragms. In each span, there are 16 VSL tendons type 6-19 dyform (breaking load approx. 5,700 kN each) having normal VSL anchorages type E at both ends. The tendons caver two spans and overlap at the piers. Thus the maximum tendon length is approx. 170 m (Fig. 37). Profiling was achieved by means of saddles cast into the diaphragms. The saddles consist of mild steel tubes of 110 mm outside diameter welded into a steel box (Fig. 38).

The tendons were prefabricated in the workshop and introduced into the box through a hole in the deck slab. Tendons near the top of the box or inclined tendons showed a considerable sag between diaphragms, due to their dead load. As this would have created problems during stressing, intermediate props were temporarily placed between neighbouring diaphragms.

Originally, stressing at both tendon ends was required to compensate for the friction losses, as in the design a friction coefficient u = 0.30 was adopted. Tests on site, however, showed an effective of between 0.05 and 0.10. A revised calculation with these values proved that unilateral stressing was therefore acceptable for obtaining the required forces. Post-tensioning work started in June 1975, and was completed in Cctober 1976. The total quantity of strand incorporated in both viaducts is 1,100 tonnes.

6.2.2. Seven Mile Bridge, Florida, USA

Owner	Florida Department of Transportation, Tallahassee, Florida
Engineer	Figg and Muller Engineers, Inc., Tallahassee, Florida
Contractor	Misener Marine Construction, St. Petersburg Beach, Florida
Erection of superstructure and post-tensioning	VSL Corporation, Los Gatos, California
Construction Period	1979-1982

The Seven Mile Bridge, which leads from south of Marathon to Little Duck Key, is the longest in the chain of road structures connecting the mainland of Florida to Key West. With a total length of 10,931 m (35,863') it is also the longest concrete box girder bridge in the world. It consists of 266 spans, most of them having the standard 41 .15 m (135') span length. The superstructure with its single-cell box section has a total width of 11.89 m (39') and a constant depth of 2.13 m (7').

The design especially aimed at speed of construction, in addition to economy. Thus some structural details are quite unusual, even somewhat bold, with regard to concept and durability. These details are:

- No gluing material to bond or seal the joints of the match-cast segments. Multiple keys only were provided to transfer shear forces.
- Segments stressed together with external tendons running inside the box and connected to pier diaphragms and deflector blocks.
- No overlay or wearing surface on the segments; the traffic runs directly on the precast concrete.

The segments of the superstructure were cast in a yard in Tampa, Florida, approx. 400 km (250 miles) north of the bridge site. The five sets of forms allowed for an average production rate of three spans (i.e. 24 segments) per week. After proper curing, the segments were barged to site. There, the segments of a span were placed aboard a shuttle barge, then winched together, aligned to the required alignment and connected with four

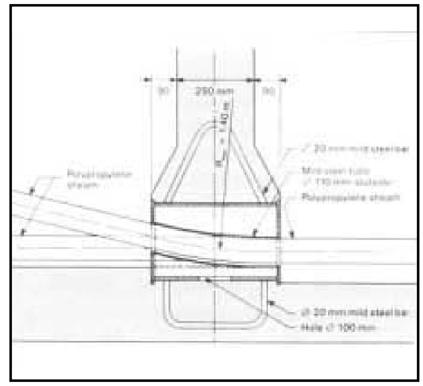


Figure 38: Saddle detail

Figure 39: Construction phase

temporary prestressing strands. The pier segment, accompanying the assembled spart, was also transferred to the shuttle barge, which then moved beneath the erection truss (Fig. 39).

The truss was a very sophisticated gantry combining innovative engineering with proven techniques and practical experience. VSL proposed to use such a truss as a variant to the construction scheme given in the contract documents. The following are the main factors that led VSL to alter that scheme:

- The segment alignment is taken off the critical path. thus increasing speed of construction and adding flexibility to the operation.
- A cleaner division is achieved between the work performed by the general contracter and by VSL.

- VSL can control all its operations overhead.

The truss, designed and operated by VSL, in a typical operation lifted the pier segment into place, then cantilevered itself forward to position its centre on that newly placed pier segment and finally raised the lifting frame bearing the preassembled span from the barge. Concrete blocks were then inserted into the gap and the post-tensioning tendons stressed to 15% of the ultimate force. After the closure concrete had reached a strength of 17.5 N/ mm² (2,500 psi) overnight, the tendons were fully stressed.

VSL started erection of the first span of the bridge on May 30, 1980. On average, three spans were installed per week, with a maximum of six spans in a six day single shift period. The structure was completed in May 1982.
Each span contains 4 VSL tendons 5-27 (breaking load approx. 4,960 kN each) and 2 of the unit 5-19 (Fig. 40). The tendons are anchored in the pier diaphragms by VSL stressing anchorages type EC. In the five central segments of a span, the tendons are deflected in deviation saddles. In the pier segment, semi-rigid duct was embedded to bring the tendon to the anchorage, while a short piece of galvanized pipe guides the tendons through the deviation saddles. Between these, the tendons are encased in plastic pipes. Plastic pipe and duct or pipe are connected with rubber boots and hose clamps (Fig. 41). All tendons were prefabricated and pulled in by hydraulic winch; they were cement-grouted for corrosion protection.

Transverse post-tensioning was also applied, in the deck slab of the pier segments only. The tables used are internal, consist of four strands ∅ 13 mm (0.5") and are provided with VSL anchorages.

Additional details about the Seven Mile Bridge and its construction can be found in [46] .

6.2.3. Châtelet Viaduct, Charleroi, Belgium

Owner I.A.C. (Intercommunale Autoroute Charleroi), Charleroi
Engineer Office J. Rondas, Brussels
Contractor Joint Venture Socol S.A., Brussels/ Ateliers de Construction Jambes-Namur; later Ateliers de Construction Jambes-Namur/ Société Pieux Franki S.A., Liège
Post-tensioning Civielco B.V., Leiden, Netherlands
Construction Period 1981-1982

In order to take the transit traffic off the city centre and to connect the industry zones at the periphery, an outer ring road was built around Charleroi, an industrial centre in southern Belgium. In the suburb of Châtelet the ring road crosses the valley of the river Sambre, which is densely built-up and through which the railway line Paris-Cologne also passes. To cross the valley a 1,097 m long viaduct had to be constructed.
The design adopted was put forward in the tender stage as an alternative to the tender design. It comprises two independent superstructures, each made up of two 3.00 m deep steel girders carrying a 16.00 m wide light-weight concrete deck. Most of the 25 spans, the lengths of which vary between 34.98 and 58.60 m (except the main span which measures 68.40 m), are simply supported beams.
In view of the bad soil conditions, the lightest possible structure was sought. For this reason the steel girders are post-tensioned with external tables. At the base of each girder, up to 6 VSL tendons type EE 5-12 are installed, providing forces from 600 to 1,200 kN. The tendons are encased in PE tubes and were grouted after stressing (Fig. 42).
As the Belgian standard in force at that time did not contain any prescriptions for this type of construction, full size model tests were performed with tendons of the above-mentioned unit in order to assess the fatigue behaviour of the tendon itself and especially of the anchorages. In this way an anchorage design fulfilling the requirements was found. Another detail checked was the tightness of the tendon system at every point, in particular at the anchorages and the saddles.
The tables being practically straight, saddles were required only near the anchorages, for reasons of space. These saddles consist of thick-walled steel tubes welded to the web of the steel girder. The PE tubes of the tendons are fitted into the saddles and the joints tightly sealed.

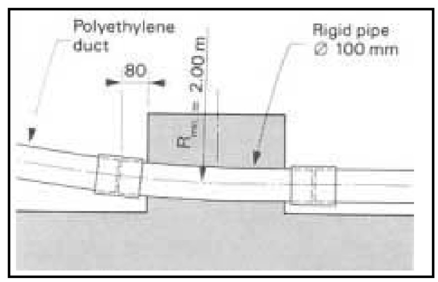


Figure 41: Detail of deviation saddle

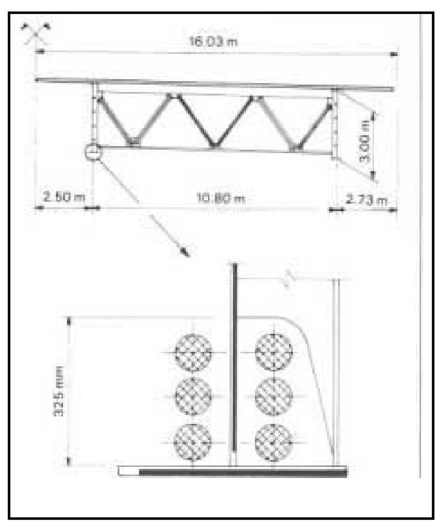


Figure 42: Cross-section and detail with tendons

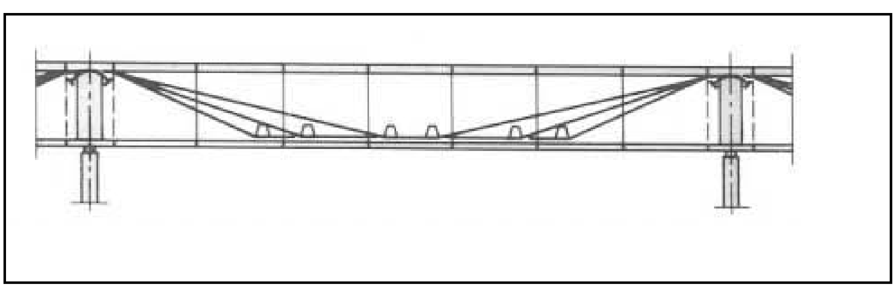


Figure 40: Tendon layout

Figure 43: General view of one bridge under construction

6.2.4. MARTA Bridges, Atlanta, Ga., USA

Owner Metropolitan Atlanta Rapid Transit Authority (MARTA), Atlanta, Ga.
Engineer Figg and Muller Engineers, Inc., Tallahassee, Fla.
Contractor J. Rich Steers Inc., New York, N.Y.
Post-tensioning VSL Corporation, Atlanta, Ga.
Construction Period 1982-1983

The two bridges described below, one designated CS-360, the other one CN-480, are the first precast segmental concrete box girder railway bridges built in the USA (Fig. 43,44). Originally the structures should have been constructed of in-situ concrete; the successful contracter, however, took advantage of a value-engineering clause and had a redesign prepared which resulted in the least amount of expenditure and saved time. Thus up to four spans were completed per week. Construction of both bridges fasted for 64 weeks.

CS-360 is 1,594-10 m (5,230') long and has spans of 21.34 to 30.48 m (70' to 100'), while CN-480 has a length of 579.12m (1,900') and spans between 22.86 and 43.59 m (75' and 143'). The single-cell box girder superstructure, designed for twin-track operation, is 2.13 m (7') deep and has a deck width of 9.22 m (30' -3").

Both structures are longitudinally post-tensioned with external VSL tendons 5-12 to 5-27 which run in the interior of the box. These are between 21.34 and 43.59 m (70' and 143') long and have EC anchorages at both ends. The strands were placed in polyethylene ducts, which were grouted with cernent mortar after stressing.

Deviator blocks are provided in every 3.05 m segment. In these and in the end blocks, the strands lie in steel pipes which are connected to the polyethylene pipes by means of rubber hoses clamped on the pipes (Fig. 45).
Transversely, the deck was pretensioned in the casting bed.

6.2.5. Loir Bridge, La Fléche, France

Owner Direction Départementale de l'Equipement de la Sarthe, Le Mans
Engineer SETRA, Bagne/ Bureau d'Etudes Dragages et Travaux Publics, Paris
Contractor Dragages et Travaux Publics, Tours
Post-tensioning VSL France s.a.r.l., Boulogne-Billancourt
Construction Period 1982-1983

The construction of a by-pass road in the town of La Flèche made a new bridge over the river Loir necessary. In view of the large areas of land liable to flooding and the bad soil conditions, a very low road level was adopted in order to avoid embankments. This fact, of course, had also a decisive influence on the design of the bridge.
A three-span bridge with the smallest possible depth of the superstructure obviously was the only solution corresponding to the given conditions. The length of the centre span was fixed at 64 m, while the lateral spans were chosen at 26 m each. In order to equalize the masses between the lateral span and half the centre span, lightweight concrete was selected for 59 m of the centre span while additional concrete was required in the last 10 m of the side spans.
For hydraulic reasons, the depth of the single-cell box girder superstructure had to be limited to 2.80 m at piers; at mid-span a depth of 1.75 m was adopted. The deck slab width is 10.75 m.
Each half superstructure was constructed parallel to the river with two 9 m long form-works, which were advanced like travellers of a free cantilever bridge. Thus segments of about 5 m length were obtained. After completion, each half was rotated to the final alignment (Fig. 46). Construction lasted from March 1982 to February 1983.
Post-tensioning consists of two families of tendons. For the quasi free cantilevering stages, internal tendons VSL type EC/ EC 6-12 were chosen, one tendon was anchored at the top of the web at each segment end. In the bottom slab of the centre span 4 VSL tendons EC/ EC 6-12 had to be placed, within the concrete section. In addition, 8 external tendons

Figure 44: Prefabrication yard

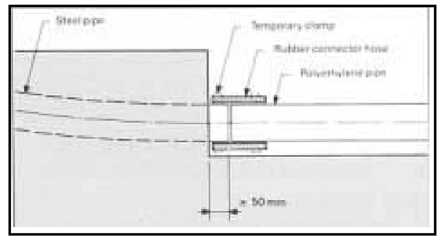


Figure 45: Detail of pipe connection at deviator block

Figure 46: One bridge half nearly completely rotated

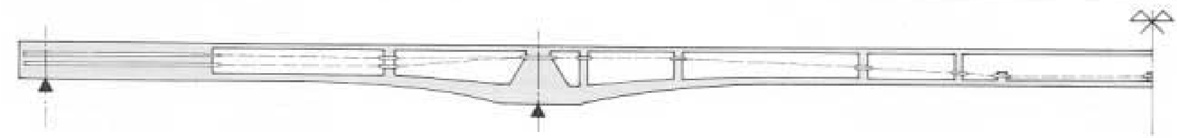


Figure 47: Longitudinal section with external tendons

VSL 6-19 run near each web. These are provided with standard VSL anchorages type EC (Fig. 47). The strands were installed by means of the VSL Push-through Method. The sheathing of the external tendons consists of steel tubes. To allow for possible force monitoring or the replacement of an external tendon, these were grouted with grease. This, however, proved to be a very expensive method due to the cost of the grease and of the steel tubes.

The external tendons are deviated in diaphragms and cross-beams provided at various sections of the superstructure. Deviation is obtained by means of a curved piece of rigid steel tube which is connected to the tendon sheathing by a connecting sleeve welded to both tubes.

6.2.6. Bridge O.A. 33, Marseille, France

Owner	Direction Départementale de l'Equipement des Bouches-du-Rhône, Marseille
Engineer	Bureau d'Etudes Dragages et Travaux Publics, Paris
Contractor	Dragages et Travaux Publics, Marseille
Post-tensioning	VSL France s.a r.l., Boulogne-Billancourt
Construction Period	1983-1985

In Marseille, the motorway A55 leaves the centre of the City northwards. Soon it crosses an industrial and railway area.

Figure 48: Bridge O.A.33 under construction

This is the location of the bridge designated O.A. 33. The bridge consists of two superstructures each having three traffic lanes. The two structures have spans of 23-29-43-2x40-43-38-20 m and 31.69-2x40.01 -38.61-2x43.02-33.01 -27.01 m respectively. They are curved both in the horizontal and the vertical plane. The single-cell box girders have a depth of 2.85 m, the width of each deck slab being 14.42 m.

Both superstructures were built using the incremental launching method (Fig. 48) as this variation offered the lowest price. Fabrication was carried out behind the Marseille abutment which is the lowest point of the structure. The pier diaphragms and internal diaphragms for the external tables were constructed at the same time as the respective increments.

In the tender a post-tensioning layout, partly or entirely external, could be proposed. For execution the following three groups of tables were selected (Fig. 49):

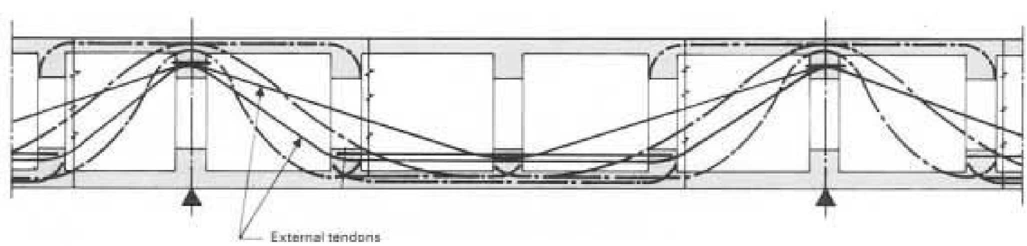


Figure 49: Tendon scheme

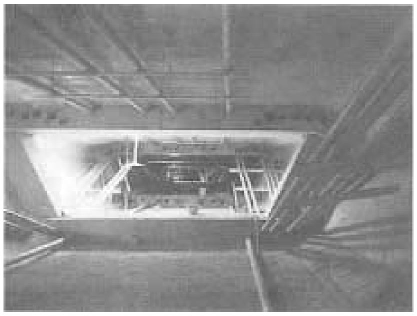


Figure 50: Tendons inside the box

- Permanent tendons which were stressed before a new increment was jacked forwards. These tables are polygonal and parabolic and are within the concrete section.
- Temporary tendons that were stressed before jacking and afterwards were destressed and removed. After completion of the first superstructure, these temporary tendons were reused in the second one. All of these tendons were external. Some were straight, while others followed a polygonal profile to give, in conjunction with the first group, a central prestress.
- Permanent tendons which were stressed after the increments were jacked forwards. They are all external and either straight or polygonal in layout.

All tendons are of the VSL unit 6-12 (breaking force 3,024 kN each) and have stressing anchorages type EC at both ends. The temporary tendons and the final external tendons were placed in polyethylene tubes, while steel pipes were used for the final internal tendons (Fig. 50). All final internal tendons were grouted with cement grout.

The external tendons are deviated in concrete frames provided in the boxes, into which curved steel tubes are placed. A short piece of PE tube is placed on the ends of the steel tubes and fixed in the concrete. To this the PE tube of the tendon is joined by means of a joint welded on both ends of the PE tubes.

6.2.7. Sunshine Skyway Bridge across Tampa Bay, Florida, USA

Owner	Florida Department of Transportation, Tallahassee, Fla.
Engineer	Figg and Muller Engineers, Inc., Tallahassee, Fla.
Contractor	Paschen Contractors, Inc., Chicago, Ill.
Post-tensioning	VSL Western, Campbell, Cal.
Construction Period	1983-1987

The Sunshine Skyway Bridge replaces a section of the bridge structure leading across Tampa Bay between St. Petersburg and Sarasota. The replacement became necessary, as in May 1980 a tanker veered out of the navigation channel and rammed one of the main piers, thus sending 400 m (1,300') of bridge into the water. In October 1982, the owner opened bids for a replacement bridge.

The winning design for the high-level and main approach spans consisted of a concrete box girder structure with a 365.76 m (1,200') table-stayed main span. It should be noted that all stays are VSL Stay Cables System 200 with up to 82 strands of ∴ 15 mm (0.6"). The rebuilt crossing was opened to traffic in April 1987 (Fig. 51).

The table-stayed part with spans of 164.59-365.76-164.59 m (540' -1,200' - 540') is adjoined on each side by three spans each of 73.15 m (240') and one span of 42.67 m (140'). The superstructure of this main part consists of a 4.47 m (14' -8") deep single-cell box girder with steeply inclined webs. The deck slab is 28.78 m (94' -5") wide. Along the central axis it is supported in the box at intervals of 3.66 m (12') by inclined struts ending at the intersection point between web and bottom slab. The main structure is followed on each side by 18 spans of 41 .15 m (135') standard length twin single-cell box girders (Fig. 52).

All box-sectional parts were made up from precast segments joined together with an adhesive, some with in-situ concrete. In the main structure, the segments were hoisted from barges and placed at alternate ends in the free cantilevering manner. In the 41.15 m (135') spans, however, the erection truss from the Seven Mile Bridge (see para. 6.2.2.) was reused after adaptation and complete spans were placed by the contracter, who had bought the truss from VSL.

The superstructure is post-tensioned longitudinally and partially transversely. The latter tables consist of VSL tendons SO/ SO 6-4 placed in the concrete section in flat corrugated ducts of high-density polyethylene. Longitudinal post-tensioning in the free cantilevered part consists of tendons within the concrete section

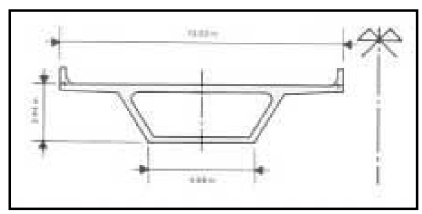


Figure 52: Cross-section of twin, single box girder structure

Figure 51: Sunshine Skyway Bridge

installed during free cantilevering while the continuity cables run inside the box. These are VSL units 5-17 and 5-27, up to 238 m (781') long. In the 41.15 m (135') spans, VSL tendons 5-19 to 5-27 are arranged inside the box similarly to the cables of Seven Mile Bridge (see para. 6.2.2.). Anchorages used were standard EC type. After stressing (double end stressing for longer tendons) the cables were cement-grouted.

6.2.8. High Bridge, St. Paul, Mn., USA

Owner	State of Minnesota, St. Paul, Mn.
Engineer	Strgar-Roscoe, Inc., Wayzata, Mn./ T.Y. Lin International, San Francisco, Cal.
Contractor	Lunda Construction, black River Falls, Wi.
Post-tensioning	VSL Corporation, Burnsville, Mn.
Construction Period	1985-1987

The new High Bridge, which replaces a structure that was built in 1889, is the first bridge in the USA to use a combination of table and steel tension-tie design for a deck-tied arch bridge. It is 839.72 m (2,755') in length with a width varying from 20.02 to 27.13 m (65' -8" to 89'). It has two river piers. The height above the river varies from 24.38 m (80') at the north end to 58.22 m (191') at the south end, making it one of the world's steepest bridges (Fig. 53).

The new bridge makes use of some innovative structural techniques. Although the main river span appears to be a traditional arch, it does not function as such. The structural loads are distributed in cantilevered action through the use of the half arches on either side of the main span and the tensioned tables beneath the deck. This unique structural system allows steel members to be lighter than conventional arches and this contributes to the graceful aesthetic qualities of the structure.

Eight VSL tendons 5-27, 166.12 m (545') long, tie the cantilevered arches together at the south pier. Similarly eight tendons of the same unit, 146.61 m (481') long, were used at the north pier. The tendons, which are straight, were anchored at both ends by means of normal E type stressing anchorages provided with a special cover cap. Each tendon was pulled from ground into a galvanized steel pipe of Ø 101.6 mm (4"). Stressing was carried out in three stages: Stage I stressing closed the gaps in the slotted connections at the ends of the wide flanges, Stage II took place after the concrete deck was cast and Stage III fine-tuned the arch to the camber and stresses desired by the engineer.

All tendons were cement-grouted for corrosion protection. A special detail was required for the trumpet to allow for structure movement during stressing while maintaining a seal capable of withstanding the high grouting pressure.

6.2.9. Bois de Rosset Viaduct near Faoug (VD), Switzerland

Owner	Département des Travaux Publics du Canton de Vaud, Lausanne
Engineer	CETP Ingénieurs-Conseils SA, Lausanne/DIC Ingénieur Conseil, Aigle
Contractor	Joint Venture of Frutiger SA, Yvonand/ Ramella + Bernasconi SA/ Reymond SA
Post-tensioning	VSL International SA Crissier
Construction period	1988-1990

The Bois de Rosset Viaduct consists of two parallel structures with 15 spans each (23.00-34.20-11x42.75-51.30-38.50 m). It has a total length of 617.25 m, a width of 2x13.0 m, and crosses a railway line at a height of approx. 10 m. The composite superstructures consist of steel trough girder sections connected to a transversely post-tensioned concrete deck slab.

The structures are longitudinally post-tensioned with four VSL External Tendons in each span. Each tendon consists of 12 individually greased and plastic-sheathed VSL Monostrands which are grouted inside a thick-walled polyethylene tube. Tendon lengths range from 196 m to 216 m, and are located inside the steel troughs, routed over a maximum of five upper deviation saddles and ten lower saddles.

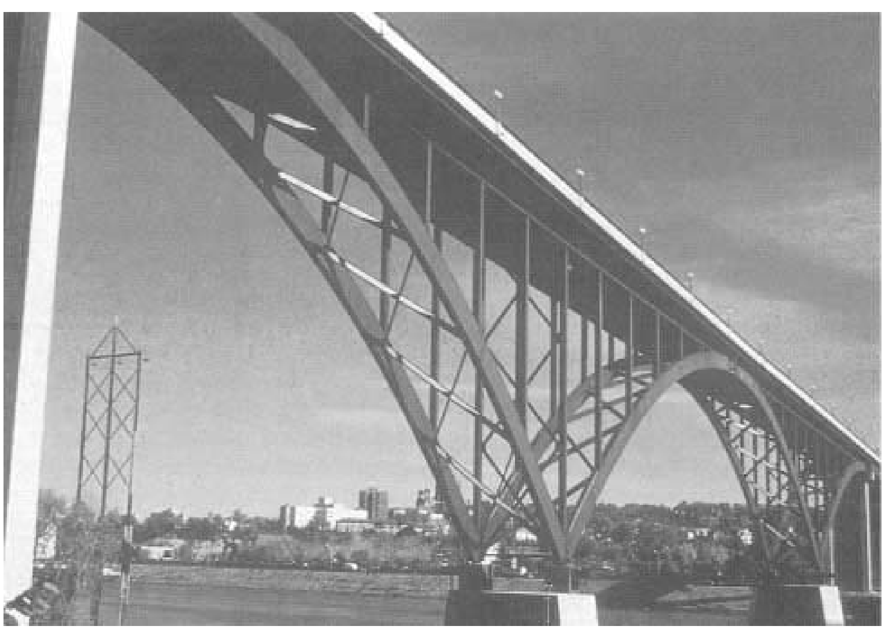


Figure 53: High Bridge, St. Paul, Minnesota

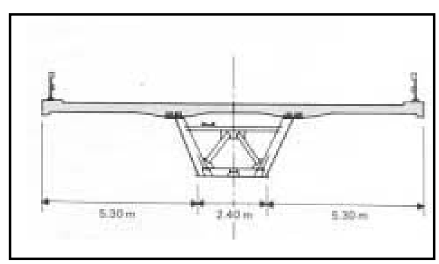


Figure 54: Cross-section of superstructure of Bois de Rosset Viaduct project

This project represents the first use of monostrand external tendons in Switzerland, and the installation is monitored as part of a long-term observation program. Four tendons are equipped with permanent VSL load cells, and all tendons are adjustable and replaceable.
As mentioned in para. 3.3, extensive testing was performed for the development of this tendon system with regaid to materials, procedures, anchorage design, and friction losses in the saddles.

6.3. Other structures originally designed with external tendons

6.3.1. Flue gas chimneys, Federal Republic of Germany

For environmental reasons, coal-fired power plants in the Federal Republic of Germany are equipped with flue gas sulphur removal systems. The purified flue gases are normally expelled through the cooling tower, together with the cooling steam. In the case of a breakdown, however, the flue gases are diverted past the desulphurization system and fed into the chimney.

The shaft of the chimney (Ø approx. 10 to 17 m) is made of reinforced concrete; inside is the flue gas pipe made of acid-resistant ceramic masonry. The flue gas pipe is surrounded by thermal insulation made of foam glass (Fig. 55). Upon breakdown of the desulphurization system, the temperature of the flue gas rises quickly by 90°C to 180°C. The shock of the sudden change in temperature leads to high compressive and tensile stresses in the heat-resistant ceramic masonry. The tensile stress exceeds the stress limit allowed in the standard DIN 1056. To ensure the serviceability of the flue gas pipe, special methods must therefore be taken.

Up to now the following methods have been used:

a) Reinforcing steel with anti-corrosive coating. The disadvantages of this solution are that the reinforcing steel does not prevent cracks and that bonding problems can occur as a result of the differing thermal expansion coefficients of reinforcing steel and ceramic brick.

b) Surrounding the masonry with stressed steel bands. The installation of these bands and the disc springs required between band segments are expensive as exterior scaffolding is needed around the chimney. The same is also true for additional stressing of the bands at a later date, which is necessary as the springs do not provide long-term compensation for creep of the gap-filling compound.

For two chimneys a new method was applied in 1986/87, in particular to avoid the above-mentioned disadvantages. This method consists of surrounding the flue gas pipe at regular intervals with external, individual post-tensioning tendons. These must fulfill the following requirements:

- easy installation,
- easy stressing operation,
- possibility of additional stressing,
- possibility of monitoring the tendon force,
- easy replacement.

The tendons used are VSL Monostrands Ø 15 mm. They rest on special bricks containing a groove into which the tendon is fitted. These bricks are thermally insulated from the flue gas pipe so that the tendons are subject to a maximum temperature of 40°C which both grease and PE coating are able to withstand without problems. Nevertheless each monostrand is additionally inserted into a protective pipe of PE in order to prevent the coating of the tendon from bearing directly onto the bricks. Each tendon entirely surrounds the flue gas pipe and is anchored in a steel buttress. To minimize the effect of friction losses, successive pairs of tendons are alternately anchored at buttresses on opposite sides. Each tendon is stressed to 100 kN. Tendon spacing is 1 m. Before reaching the buttress, one table end undergoes a deviation in a special construction, so that it can be anchored in the buttress. The deflection device and the anchor buttress are coated with an anti-corrosive paint (Fig. 56).
The installation of the special bricks, the PE protective pipes containing the monostrands, and the buttresses was carried out as part of the brickwork of the flue gas pipe. Thus, no special outside scaffolding was required. For stressing, however, a scaffold was used, which could be displaced vertically in front of the

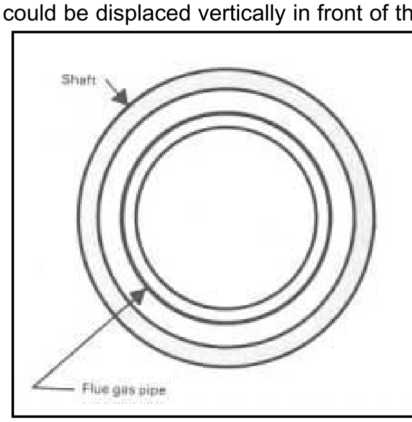


Figure 55: Cross-section of the flue gas chimney

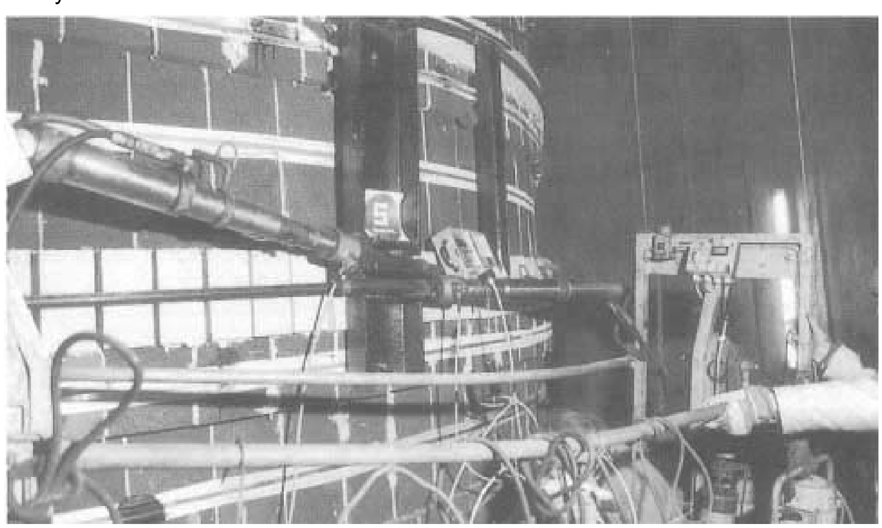


Figure 56: Deflection device with anchorages

row of buttresses. The post-tensioning force cari be checked at a later date, if required, the anchor heads being externally threaded for this purpose.

Before the system was actually applied on site, tests were carried out in order to:
- verify that the edge of the bricks would not cause any long-term damage to the PE coating of the monostrands
- test and practise proper installation, stressing and replacing of the tendons

The tests gave fully satisfactory results which were confirmed during application.

6.4. Bridges with subsequently added external tendons

6.4.1. Roquemaure Bridge near Avignon, France

Owner	Autoroutes du Sud de la France, Védène
Engineer	Etudes Ouvrages d'Art (Bouygues), St. Quentin en Yvelines
Additional Post-tensioning	VSL France s.a. r.l., Boulogne-Billancourt
Execution	1975-1976

This bridge is part of Motorway A9 Orange-Narbonne in Southern France, which it carries across the river Rhône near Avignon. The structure is 420 m long and has spans of 50-4x80-50 m. The 21.60 m wide superstructure consists of a double-T section with a depth between 5.40 m at piers and 1.80 m at mid-spans (Fig. 57). It was built in 1971 to 1974 by the free cantilevering method, with cast-in-place segments up to 6.12 m in length. In 1975 a surveillance campaign revealed the presence of major cracks (as wide as 8 to 10 mm) at the mid-span sections. After examination of the damage and after checking of the design, it was found that no temperature gradient had been considered and that the cover of the prestressing tendons was insufficient. The structure therefore had to be repaired in the shortest possible period. A complete interruption of the highway being unacceptable, the owner had to allow for the repair work to be done under light traffic. Therefore, the consultant proposed to apply external longitudinal prestressing after the cracks had been grouted with resin. In addition, the following measures had to be taken:

- Construction at either end of the bridge of a prestressed concrete cross-beam, incorporating the anchorages of the new tendons and transmitting the additional forces to the superstructure of the bridge.
- Construction of a working chamber behind each cross-beam, from which the strands would be fed into the ducts and where the cables could be stressed.
- Installation of hangers beneath the bridge deck carrying the cable ducts.

The longitudinal prestressing force required amounted to 54,000 kN after losses. VSL thus proposed to use 8 tendons of the unit 5-55 (ultimate force 9,169 kN) running from one end of the bridge to the other without any coupler (table length thus 430 m!). Four spare ducts were also installed in case additional prestressing should be needed. VSL was awarded the post-tensioning contract because, besides a reasonable price, VSL could prove its experience, in pushing through strands and it had available equipment for this method, including several high capacity jacks. Placing of the tendons was the most demanding part of the job. Placing preassembled strand bundles was excluded from the beginning because of the limited space available in the working chambers and because of the length and the weight of the tendons. Thus only the push-through method was applicable. Tests made by VSL enabled the best way of operating to be found. They showed that two intermediate pushing posts were required. According to the access possibilities, pushing sections of 135, 160 and 135 m were selected. Two pushing machines had to be placed at the first intermediate post in order to obtain the required pushing force (Fig. 58).

When a certain number of strands had been introduced, the pushing force available was no longer sufficient for overcoming the friction in the steel tubes and an auxiliary strand running between the first two posts was used, to which the strands to be installed were coupled. The first machine pushed the auxiliary strand while the second machine pulled it. After pulling, the auxiliary strand was pushed back to the first post and the operation then repeated. When the pushing operation was finished, the openings in the tubes were closed by previously mounted coupling sleeves.

Before the tables were stressed each individual strand was pretensioned to 1 N/mm² by means of a monojack to bring all strands to the same length.

Figure 58: Pushing trough strands

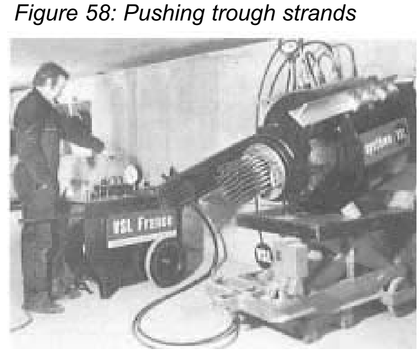


Figure 59: Stressing of additional tendons in the stressing chamber

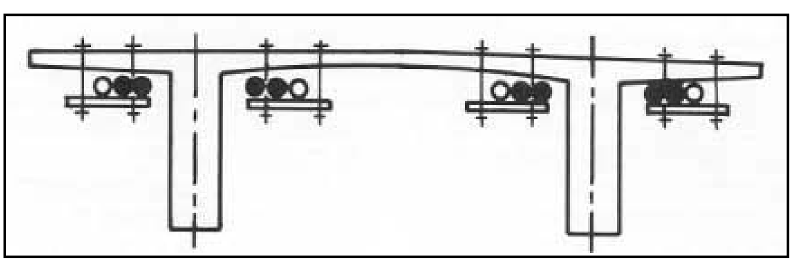


Figure 57: Cross-section of Roquemaure Bridge with added external tendons

Stressing had to be done on two cables simultaneously and at both ends for symmetry reasons. The time available for stressing 4 x (of the 8) tendons was fixed at 6 hours and therefore great mobility of the equipment was required. Five VSL jacks ZPE-1000 (one as spare) and corresponding accessories, as well as five pumps were engaged (Fig. 59). The jacks, each weighing 2.5 tonnes, were mounted on specially constructed hydraulic carriages. Stressing was done in steps of 5 N/mm². The cable extension amounted to 3,150 mm. The cables of the end cross-beams (16 No. EE 5-12 on each side) were stressed in groups of two at the same time as the longitudinal tendons.

In view of the quantity of material to be injected and of the length of the cables, the use of a special grout mix with retarded hardening and consisting of clinker and resin was required by the client. Grouting was executed in sections, which made movable equipment necessary. The grout mix was injected over a distance of 180 m before the installation had to be moved. Two tables were grouted per day, requiring 12 m3 of grouting material.

6.4.2. Ruhr Bridge Essen-Werden, Federal Republic of Germany

Owner City of Essen
Engineer
(Repair) Prof. Dr. G. Ivanyi, Essen
Contractor Polensky & Zollner AG,
(Repair) Bochum
Additional
post- SUSPA Spannbeton GmbH,
tensioning Langenfeld
Execution 1985-1986

This two-span post-tensioned concrete bridge (spans 66.40-47.00 m) has a multi-cell box superstructure with a deck width over the intermediate pier of 34.41 m. This width increases on both sides towards the abutments (Fig. 60).
In the bottom slab and in the web of the larger span, numerous cracks due to bending had developed making rehabilitatien measures necessary especially in view of the corrosion protection of the post-tensioning tables in the cracked area. Grouting the cracks (which were up to 0.4 mm wide) was disregarded since it was established that the cracks originated from temperature gradients; thus new cracks would have appeared near the grouted cracks. Furthermore, it was established that the stress variation in the post-tensioning steel considerably exceeded the allowed value.

Thus a static strengthening was required, not only corrosion protection measures. In view of the limited space available inside the box cells, which did not allow for adding reinforcement, strengthening the larger span by means of post-tensioning tables offered the best solution. Straight unbonded tendons were selected, the number of which had to be the smallest possible.

A total of 24 tendons VSL type 5-16 (breaking force 2,833 kN each) with threaded anchor heads and an average length of 75 m were required. At the abutment side these were anchored in anchor blocks added to the web prolongations, while buttresses were provided behind the pier diaphragm, which itself also had to be post-tensioned to take the additional forces.

The monostrands were placed in PE ducts which were provided with two movable joints to absorb temperature movements. As strand deviations could not be avoided and inaccuracies in the boring had to be expected and in view of the large elongations, the likelihood of damage on the strand coating due to transverse pressures caused by strand deviations was evaluated in tests. The coating remained safe in these tests.

Borings had to be carried out with high accuracy. Boring distances were 10 to 12 m (in pier diaphragm). Oblique boring (in plan view) was also required through a span diaphragm and subsequently through a web.

The strand bundles were prepared in the workshop. The diaphragm tables were placed by means of a movable crane. The longitudinal tendons were stressed at the anchorages behind the pier diaphragm. Elongations measured 440 mm on average. Post-tensioning work lasted for seven days. All cracks closed after stressing.

The rehabilitation work overall took five months.

6.4.3. Bridge over Wangauer Ache near Mondsee, Austria

Owner Republic of Austria, Federal Road Administration, Vienna
Engineer
(Repair) Kirsch-Muchitsch, Linz
Contractor Hofman u. Maculan,
(Repair) Salzburg
Additional
post-
tensioning Sonderbau GesmbH, Vienna
Execution 1987-1988

This bridge is part of the highway Vienna-Salzburg. It was built in 1962-1964. In recent years improvements have been made several times, but a thorough inspection revealed a large number of deficiencies, making a general rehabilitation necessary. In particular, a lack of longitudinal prestressing force was detected, which had become obvious in opened construction joints. Thus, rehabilitation had to include also the installation of additional tendons. Since a closure of the motorway was not acceptable, one structure was strengthened first, followed by the other.

The twin bridge has spans of 25-6x28-2x41.25-3x28-25 m, i.e. a total length of 384.50 m. Each superstructure has a double-T cross-section, 13.05 m wide and 2.20 m deep. Since the existing longitudinal post-tensioning was bonded, and no spare ducts were available, the additional post-tensioning had to be placed on the

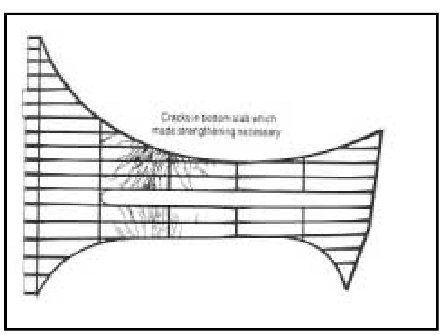


Figure 60: Plan view of Ruhr Bridge (showing cracks)

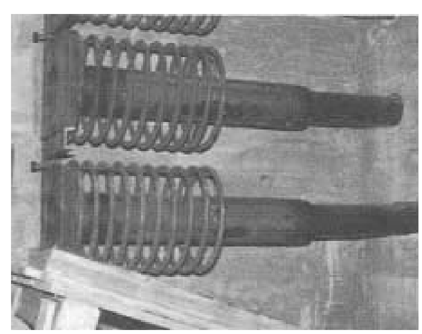


Figure 61: Anchorages inside the box prior to concreting

figure 62: View of added tendons

outside of the webs. However, the total length of 386 m was considered to be a problem.
Four VSL tendons EE 5-12 per web were selected as the additional post-tensioning. Polyethylene tubes of 90 mm diameter and 4 mm wall thickness were chosen as sheathings (Fig. 62). At the ends of the superstructures, end diaphragms, each post-tensioned by 3 VSL tendons EP 5-7, were provided.
The ducts were fixed to the webs at the quarter points of the spans by means of clamps. In between, additional cable supports were provided in order to avoid wobble. These supports were hung from the deck slab. In order to prevent the strands from abrading the polyethylene duct at clamping points, steel tubes were placed inside the polyethylene ducts at those points. These steel tubes also act as stiffeners at the joints of the polyethylene ducts.
The strands were installed by the VSL Push-through Technique. Two pushing machines were placed one behind the other and driven by a hydraulic pump of corresponding power. The strands had to be cut to length by hand before they could be pushed through. However, only the first 3 to 4 strands could be fully pushed through without squeezing. Therefore, strand installation was completed by hand from a joint opened in the duct about 250 m from the push-through machines.
Before stressing all joints were checked for tightness. Stressing was done from both ends. A friction coefficient of only 5% was observed. Grouting was provided for corrosion protection, not bond. It was performed from the lowest point in the middle of the table, towards both ends. Vent hoses were provided at distances of 40 to 50 m. Two grouting pumps were required.

6.5. Other structures with subsequently added external tendons

6.5.1. Clinker Silo, Jakarta, Indonesia

Owner	PT Perkasa Indonesia Cement Enterprise (Indocement), Jakarta
Engineer (Repair)	VSL INTERNATIONAL LTD. Berne, Switzerland
Contractor (Repair)	PT John Holland Constructions Indonesia, Jakarta
Additional post-tensioning	PT VSL Indonesia, Jakarta
Execution	1985

The silo, which is located at Tanjung Priok, Jakarta's harbour, is used for the storage of clinker awaiting export. The structure has an internal diameter of 19.80 m and a height of 30 m. Its wall is 400 mm thick. It was built of reinforced concrete in the early seventies. Because the reinforcement was inadequate, it had to be repaired.

The repair consisted of placing external tendons around the silo. In view of the high ambient temperature and the aggressive environment, it was decided to use greased and PE-coated strands (monostrands) ∴ 13 mm bundled to tendons comprising four strands. Since there were no buttresses, VSL anchorages type Z 5-4 were used (two on every tendon). In total 60 hoop tendons were required. The tables were assembled by hand and temporarily hung from steel supports. The anchorages were placed on concrete pads. After stressing, the anchorages were covered with concrete

Figure 63: View of rehabilitated clinker silo

Figure 64: View of Pier 39 parking structure

so that now the repaired structure appears to have four buttresses (Fig. 63).

6.5.2. Pier 39 Parking Structure, San Francisco, USA

Owner	Pier 39 Associates, San Francisco, Cal.
Engineer (Repair)	Bijan, Florian & Associates Inc., Mountain View, Cal.
Contractor (Repair) and Additional post-tensioning	VSL Corporation, Los Gatos, Cal.
Execution	1986-1987

This structure, which is part of a shopping centre, was originally constructed in 1978/79 in the Fisherman's Wharf area; it has space for 1,000 cars (Fig. 64). It has five parking levels including the roof, and a rectangular plan with overall dimensions of 118.90 x 63.00 m (370' x 196'); at one corner there is a square recess of 20.90 x 54.60 m (65' x 170').

The original structural system consisted of post-tensioned beams, 914 mm (36") deep and spanning 21.00 m (65 1/2'), which frame into columns to form a parallel plane frame in the transverse direction. One-way post-tensioned slabs, 114 mm (4 1/2") thick, span the longitudinal direction with spans of 5.80 m (18'). The beams contained seven ∴ 15 mm (0.6") monostrands while the slab had ∴ 13 mm (0.5") monostrands at 660 mm (26") centres. When in 1985 severe slab cracking at the roof level and substantial water leakage from the roof were noted, further inspection was carried out. Top-of-slab cracking adjacent and parallel to many beams, and beam deflections of up to 38 mm (1 1/2") in many instances were found. Also several strands popped out of beam ends. All strands were subsequently examined; they all showed some signs of corrosion. Several strands had even failed.

The central issue to the rehabilitation of the structure, besides economy, was that a new system had to be built around or added to the existing structural members while the parking garage remained essentially operational. The beams, and to a lesser degree the slabs, were the primary targets of rehabilitation.

Two major options of rehabilitation were reviewed in detail, one using steel members (trusses or channels) and the other post-tensioned tendons. The latter was adopted.

The structural design followed UBC 1982. Two tendons per beam, each consisting of six strands ∴ 13 mm (0.5") were added, one on each side of the web. At mid-spans and over columns the tendons are deviated by means of deflectors. These were made of ∴ 114 mm (4 1/2") extra-heavy pipe. In order to obtain the best possible corrosion protection, the strands were coated with epoxy. The ∴ 51 mm (2") corrugated PVC pipe was used as tendon sheathing (Fig. 65).

The work was carried out to allow continuous use of the garage by the public. Deflectors, end brackets and precast members were erected on a night shift, tendons in a day shift. The slab tendons were inspected and replaced by removing 1.22 m (4') closure strips located at approximately one-third points along the longer side of the structure. Approx. 10% of the tendons were replaced. Most stressing work was carried out from the outside of the building (Fig. 66). The repair job started in August 1986 and was substantially complete by April 1987.

It should be emphasized that shortcomings encountered in early application of unbonded strands in commercial buildings and parking structures have long been recognized and fully rectified. Today, post-tensioning of such structures is commonly the most economical and performance-healthy mode of design and construction.

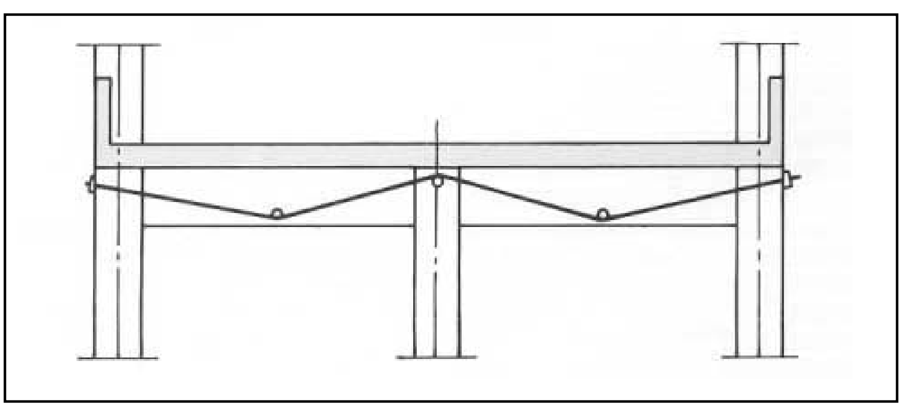


Figure 65: Scheme of added beam tendons

Figure 66: Stressing added tendons

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