Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte

Band: 53 (1986)

Artikel: Designer-contractor interaction for bridge projects

Autor: Reddi, S.A.

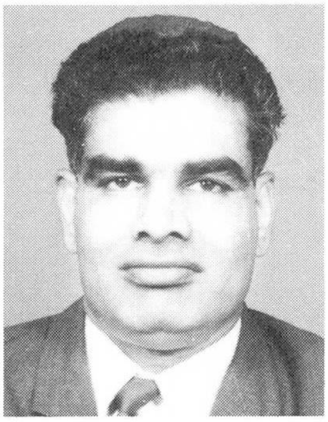
DOI: https://doi.org/10.5169/seals-41121

Designer-Contractor Interaction for Bridge Projects

Interaction projeteur-entrepreneur pour les projets de ponts

Die Wechselwirkung zwischen Konstrukteur und Bauführung für Brückenprojekte

S.A. REDDI
Chief-Engineer-Projects
Gammon India Limited
Bombay, India



S.A. Reddi, born in 1933, is a Fellow of Institution of Engineers (India). During the last 30 years, he was involved in the construction of more than 50 major Prestressed Concrete Bridges. He is actively involved in preparation of Standards for Indian Standard Institution, Indian Roads Congress, etc. He has been a member of FIP-Working Group on Hot Weather Concrete.

SUMMARY
The paper deals with the advantages of close interaction between the designer and the contractor for bridge projects, in the context of developments in design concepts and construction technology. Case histories reflecting conflicts between design and construction as well as streamlining of the system as a result of close liaison between the designer and the contractor are presented.

RÉSUMÉ
L'article traite des avantages d'une interaction étroite entre le projeteur et l'entrepreneur pour les projets de ponts, dans le cadre des développements des conceptions de projet et de technologie de la construction. L'évolution des conflits entre le projet et la réalisation, de même que la rationalisation du système résultant d'une liaison étroite entre le projeteur et l'entrepreneur sont présentés.

ZUSAMMENFASSUNG
Der Beitrag erläutert die Vorteile einer engen Zusammenarbeit zwischen Konstrukteur und Bauunternehmer bei Brückenprojekten, insbesondere unter Berücksichtigung der neuen Entwicklung in der Bemessung und der Bautechnologie. An vielen Fallbeispielen wird aufgezeigt, wo zwischen Entwurf und Ausführung Probleme entstehen und wie das System durch eine gute Zusammenarbeit zwischen Entwurfsingenieur und Bauführer verbessert werden kann.

1. INTRODUCTION

There has been spectacular developments in the design concepts as well as construction technology relating to bridge engineering during the last few decades. This has resulted in the realisation of very long bridges and large spans, involving state-of-the-art construction materials. Under the circumstances, the designer is expected to have a thorough understanding of the developments in new materials, construction plant as well as construction techniques, in order to optimise the design of bridges based on value engineering. At the same time the construction team is expected to familiarise itself with the new structural systems and concepts by liaising with the designers. Thus, a close interaction between the designer and the contractor is essential if a cost effective, quality assured bridge project is to be realised. Such interaction may result in changes in both structural configuration as well as the construction techniques to the advantage of the project. A minimum quantity-related design need not necessarily be cost effective. On the other hand such a design may even result in cost and time overruns.

2. AREAS FOR INTERACTION

The following areas readily suggest themselves :-

(i) Selection of spans and superstructure configurations in the context of difficulties in foundation construction and facilities and construction techniques available for the superstructure.

(ii) Selection of type of foundation and the type of substructure

(iii) Prestressing system and stages of prestressing

(iv) Tolerances

(v) Detailing of reinforcement and prestressing cables

3. SPAN SELECTION

In the case of contract bids based on alternative designs by the contractor, it has been possible to select the span lengths in such a way that an optimum balance is maintained between the cost of substructure and the superstructure. The contractor is in a position to do so after taking into account the type of equipment already available with him and the construction systems he is familiar with. For a bridge project in India, the original design envisaged 5 spans of 120m, based on the difficult foundation conditions involving sinking of caissons up to 50m below water level. However, the successful contractor, based on his alternative design, was able to bring down the price by nearly 20% by using short spans of 30m. Even though the alternative proposal involved more number of foundations, the overall economy was achieved due to the use of specialised type of equipment readily available with the contractor.

3.1 Type of foundations

The selection of type of foundation needs very close liaison between the designer and the contractor in order to avoid pitfalls during construction. For a bridge project in Iraq, the owner's design envisaged 1.5m diameter bored cast insitu piles. During execution, the piles had to negotiate through about 10 to 15m of hard conglomerate strata involving huge additional expenditure and time overruns. With proper interaction between the designer and the contractor, the use of shallow caissons founded at a higher level without puncturing into the conglomerate would have been the obvious cost effective solution.

In another case, for a group of four bridges in Nepal, the consultant's design stipulated the use of similar large-size piles with pile cap level at about 4m below the low water level. Here again the strata to be negotiated included hard rock, conglomerate etc. In addition, construction of cofferdams and dewatering to facilitate concreting of the pile cap involved complex time consuming operations. This had resulted in nearly doubling of the construction cost in addition to time overruns of more than 100%. In contrast, for the Euphrates Bridge at Khalidiyah in Iraq, the pile cap level was suitably decided to facilitate ease of construction (Fig. 1). Obviously in the latter case there has been closer interaction between the design and construction agencies. [1]

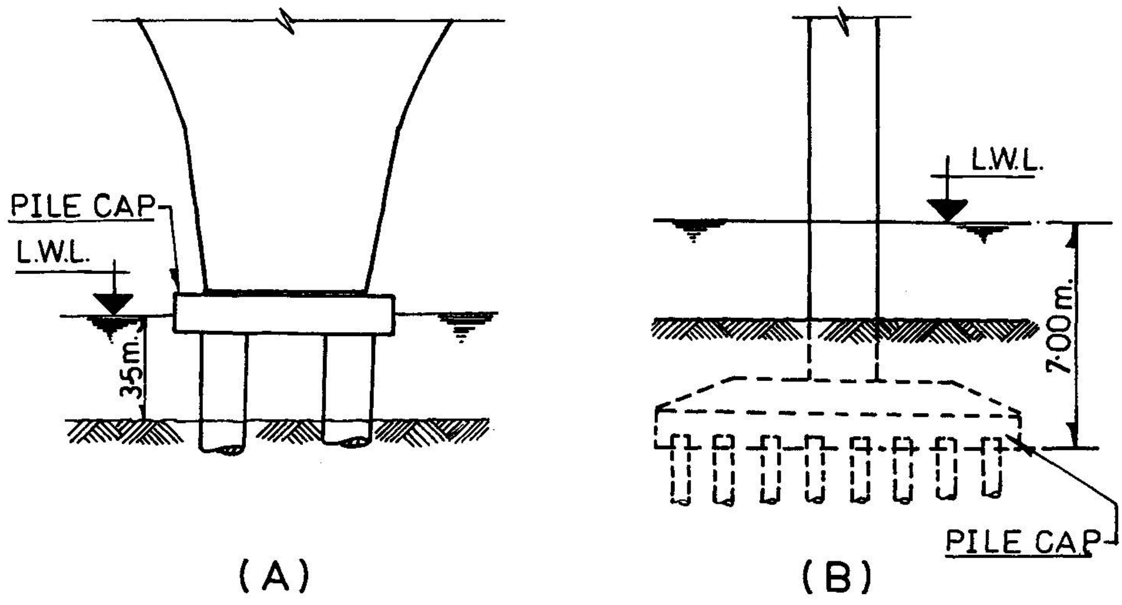


Fig. 1 Pile cap levels - two contrasting designs

3.2 Pier geometry

Details of two types of piers adopted for two different projects executed by the same construction agency are given in Fig. 2(A,B). While Fig. 2A represents the pier designed to realise minimum quantities of concrete and reinforcement, Fig. 2B is a case of design to ensure economy and quick construction. While the pier represented in Fig. 2A had to be concreted in 14 lifts, the circular pier in Fig. 2B could be concreted in only two lifts with much less quantity of form-work. For a bridge under construction across the open sea in South India, the type of pier adopted is given in Fig. 3. The structural stability is ensured, even though the pier is slender. However, if the requirements of durability under highly corrosive marine environment is taken into account, it would have been prudent to provide a more massive type of pier, which would have facilitated construction with less number of construction joints.

The dimensioning of the pier caps as well as the sizing of the bearings are preferably decided after taking into account the practical construction difficulties that are likely to arise. A case of unrealistic positioning of bearings on top of a narrow pier cap is illustrated in Fig. 4.

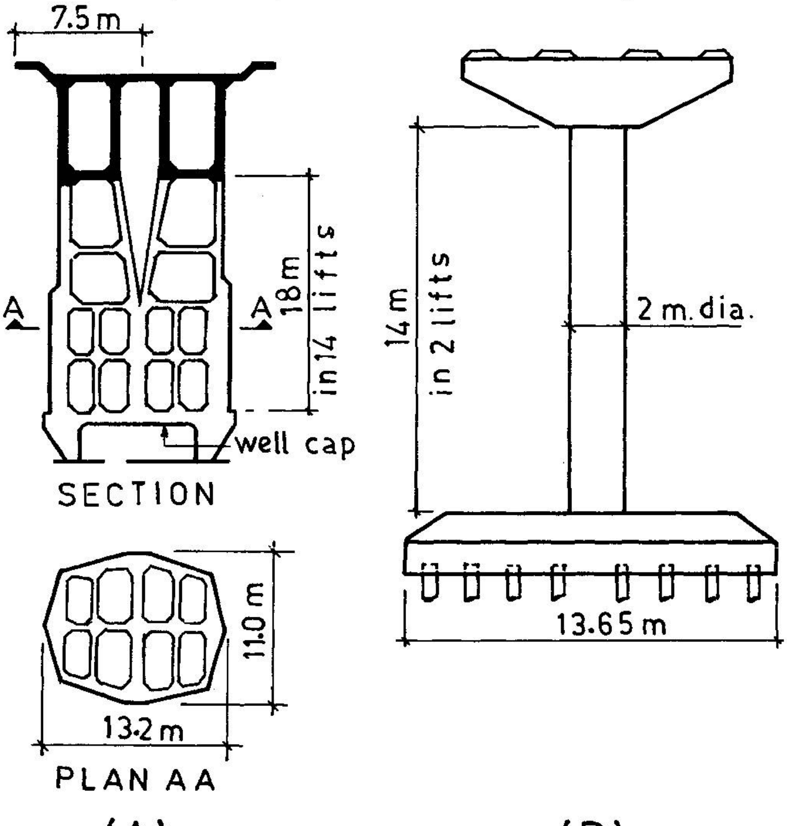


Fig. 2 Pier geometry

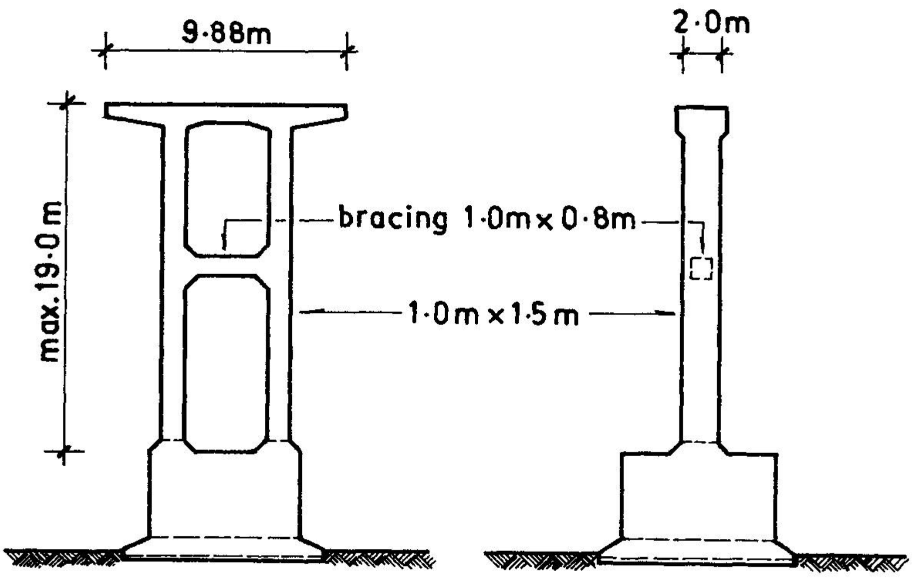


Fig. 3 Pamban Bridge - Pier

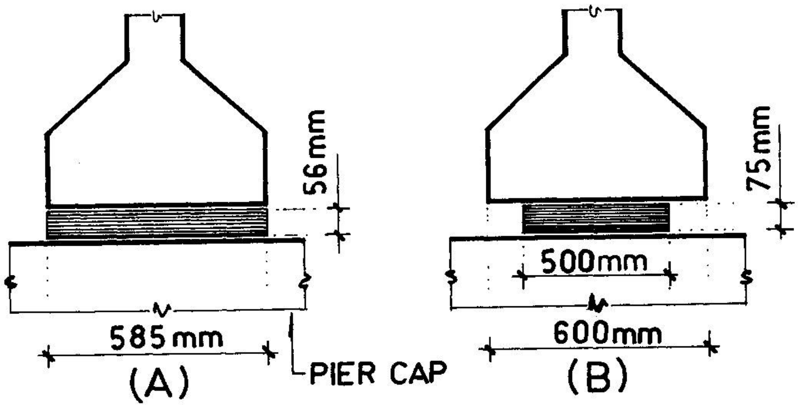


Fig. 4 Positioning of Bridge Bearings

4. SUPERSTRUCTURE

4.1 I-Beams

The decking configuration also requires close coordination between the designer and the contractor. For a 600m long bridge in India (Morhar Bridge), purely from minimum quantity design considerations, a three-beam configuration was indicated. However, from the construction point of view, an eight-beam configuration was found to be most economical taking into account immediate availability of resources.

4.2 Box girders

In the case of segmental box girder bridges, constructed either cast-in-situ or precast and erected in position, the length of the segments are determined by the equipment available with the contractor such as travelling gantries, precast beds etc. In this case a close interaction between the contractor and designer is necessary to ensure a most economical solution.

If the contractor has the option to redesign the project incorporating all the facilities available to him, a substantial reduction in cost and an improved product are generally realised. A classic example is the 25km long Saudi-Bahrain Causeway with 12.5km of bridges. Close coordination between the designer and the contractor has resulted in the most competitively priced product based on the following key factors :-

1) Large-scale prefabrication
2) A simple design using structural elements that can be assembled easily
3) A design with the use of standardized elements facilitating a large degree of repetition.
4) Use of custom-built heavy equipment

The integrated design-construction approach adopted for the project has resulted in completion of the project ahead of schedule. Cost saving of about 15% was realised through the combined coordinated efforts of the designer and the contractor.

4.3 Tolerances in weight

The design is required to take into account the tolerances to suit the construction methods. Physical verification of weights of some precast beams for a major bridge project indicated that the actual weights were about 10% more than the theoretical weights, possibly due to the increased specific gravity of the aggregates as well as the formwork tolerances. Such aspects can be catered for in the design only through close interaction between the design-construction teams.

4.4 Cross diaphragms

For deckings with I-Section beams, intermediate cross diaphragms are often provided, in order to limit the overall quantities in the superstructure. While there is definitely a saving in cost of materials, such a design complicates the construction aspects and in consequence, increase in cost and time more often outweighs the saving in the cost of materials. Moreover, because of the complicated nature of formwork for beams with cross girders, the quality of the finished project also suffers. Cost effective construction techniques such as long joint-free panels, use of prefabricated reinforcement cages etc. are far more appropriate to I-beams without intermediate cross girders.

An outstanding example of such a design being determined by the method of construction is the recently completed 6km long viaduct in Baghdad, where 36m of the deck had to be completed every week for each travelling formwork gantry. The deck is designed as a slab with two reinforced concrete beams but without any cross beams to facilitate the movement of the travelling formwork gantry. The cross beams at each expansion joint (180m apart) are concreted subsequently. By and large short spans (below 20m) are chosen to facilitate use of reinforced concrete. Incidentally the design was based on early removal of formwork, facilitated by the use of high early strength cement for the concrete proposed by the contractor. [2]

5. PRESTRESSING

5.1 Stage prestressing

Prestressing of cables in several stages may result in marginal savings, taking advantage of stress losses between stages. However, there are cases where the cost of resultant increased construction time has more than offset the advantages of stage stressing. Fig. 5 illustrates a case of multi-stage prestressing being carried to extreme refinement by the designer for a four-lane bridge deck with 6 nos. prestressed concrete I-beams supporting a reinforced concrete deck slab. However, the project was delayed substantially in consequence.

5.2 Configuration of cables

Deflecting the prestressing cables to take care of shear stresses may have to be compared with alternative provisions of conventional reinforcement for shear. When such deflected cables are curtailed progressively and anchored in the deck slab they also affect the quality of construction of the deck slab. In order to accommodate prestressing jacks, pockets are required to be left in the deck slab, and these pockets could be concreted only after the completion of prestressing. The deck slab reinforcement gets totally disturbed by the need for cutting off the reinforcement near the pockets and rewelding the same after completion of prestressing (Fig. 6).

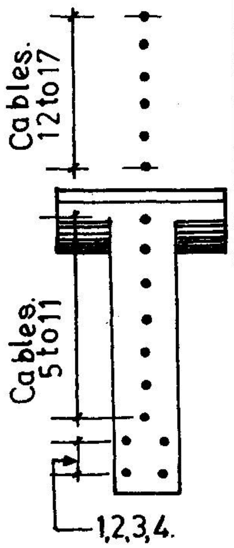


Figure	Stage of prestressing	Cables no's.	Time of prestressing		Remarks.
			age in days.	Concrete strength MPa	
	I	6 & 5	3	—	To counter shrinkage.
	II(a)	6 & 5	7	26	Cables to be Destressed.
	II (b)	1,2,7.	7	26	—
	III	3,4,8,9	50	41·5	Before casting deckslab
	IV	10 11 & 12	65	—	—
	V	13 to 17	130	—	After completing wearing coat, footpath and railing.

Note 1) Specified concrete strength 40 MPa.
2) Cable no's. 12 to 17 anchored in the deck.

Fig. 5 Multistage prestressing for bridge decking

5.3 Detailing of cables

Detailing of prestressing tendons is also required to be done with care, keeping in view the construction difficulties. Nowadays the strand tendons are generally installed into the deck on site by pushing, strand by strand, direct from the coils delivered from the factory. While this eliminates double handling and also ensures clean and rust-free tendons, there are limits for the length of tendons beyond which such pushing may not be practicable. Normally, it may not be possible to push the strand for more than about 100m. This will again depend upon the profile adopted for the tendons. A dialogue with the contractor will certainly help in detailing prestressing tendons, keeping in view the above difficulties.

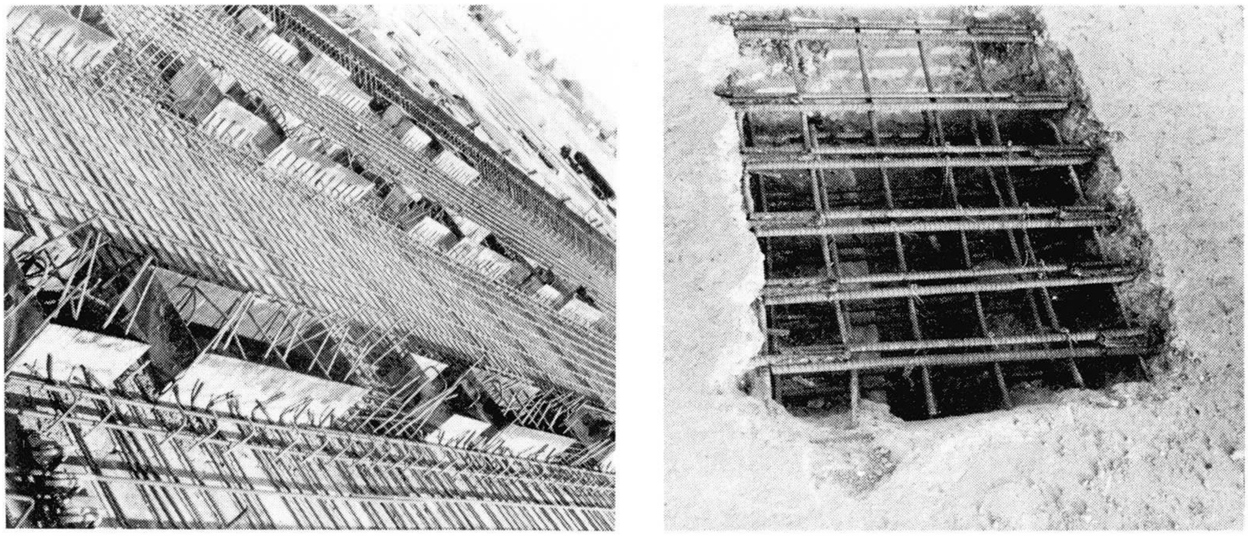


Fig. 6 Reinforcement affected by anchoring of cable in the deck
(a) Reinforcement dislocation
(b) Correction by welding

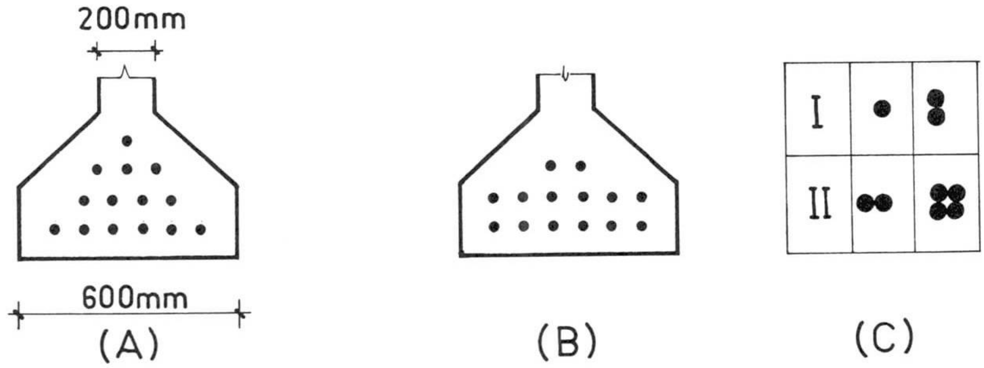


Fig. 7 Arrangement of cables for a bridge beam

5.4 Arrangement of cables

While detailing the arrangement of cables the designer should bear in mind the construction requirements to facilitate correct placement and vibration of concrete. The arrangement should not interfere with the effective placement of concrete. Fig. 7(A,B) provide two alternative examples in this context. While the arrangement in Fig. 7B facilitates effective placement and vibration of concrete, arrangement in Fig. 7A is far from satisfactory in this respect. In many cases the designer resorts to grouping of cables (up to four) in order to reduce the cross section of the member.

A typical grouping of cables, permitted by CEP FIP Model Code is given in Fig. 7C.3 While the arrangement shown in Part I does not interfere with effective placement of concrete, the same cannot be said of the arrangement in Part II. In particular, the group of four has been known to create difficulties in effective placement of concrete. In consequences, the space enclosed by the four cables is generally voided or at best filled with mortar only. Such a situation also results in difficulties during grouting. The possibility of grout being injected into one cable passing out through the other cable is not ruled out. Realisation of high grade concrete for prestressed members involves concrete mix of low workability and as such vibration of such concrete effectively around and between the cables of a group has been found to be extremely difficult.

5.5 Prestress levels

Undue refinement during the design of prestressed concrete members in the context of uncertainties relating to the actual prestressing loss, creep and shrinkage, friction and wobble coefficient, values of modulus of elasticity etc., are of questionable advantage. In this context, the designer is required to take into account the quality of materials as well as the workmanship that is envisaged during construction and provide for suitable margins. The level of initial prestress (the maximum tensile force in the prestressed tendon after releasing the jacks) permitted is of the order of 85% of the proof strength. While such values are in order in areas of availability of material with uniform characteristics as well as a very high level of workmanship, the percentage has to be suitably modified where the quality of materials and workmanship may be less than ideal.

5.6 Friction and wobble coefficients

However, there are many project sites where works are executed with much greater control than what is normally expected. Cable ducts are also being manufactured using galvanised or bright metal coated cold rolled strips. These factors have led to a steady reduction in friction losses during stressing, as observed at a number of project sites in India and elsewhere. As against the friction coefficient of around = 0.3 stipulated by the owner's design, works have been successfully carried out based on the contractor's alternative design using the value of 0.2. Field observations have confirmed the lower values. As per the U.K. Report on Noteworthy Research and Development presented at the Tenth FIP Congress, friction coefficients of around 0.20 and k = 0.0001/m are now readily attainable. The designs can certainly be streamlined in such cases based on the feedback from the contractor. [4]

5.7 Measurement of prestressing force

The calculation of the distribution of prestressing forces and the elongation along the cables is based on a number of factors whose values are known to vary in practice. Some of the variations are due to manufacturing tolerances while others are due to onsite construction tolerances/deficiencies. The following variations in values have been observed in practice :-

1. Area of cross section of the tendon -5%/+10%
2. Modulus of Elasticity ±5%
3. Measurement of prestressing forces ±5%
4. Coefficient of friction μ between tendon and duct ±100%
5. Wobble coefficient (k) -100%/+200%

While items 1 and 3 are related to the manufacture of High Tensile Steel and Pressure Gauges, items 4 and 5 are basically design and construction-related variations. [5]

Considering the above possibilities, the designer should make a judicious choice of values for the various parameters. The contractor shall interact with the designer and realise the design assumptions by careful execution of the prestressing operations. The coefficient of friction (μ) can be reduced by derusting the prestressing steel and duct, using improved type of duct, removal of local obstructions before prestressing etc. In case the designed prestressing force/elongation is not realised in spite of various preventive measures, it is necessary for the contractor to interact with the designer before taking up the remedial measures.

All the variations detailed above are not cumulative. An overall variation of about $\pm 10\%$ should preferable be assumed and the designer should ensure that such a variation does not become critical for the performance of the structure.

The tolerances inherent in the measurement of prestressing force should be taken into account by the designer. The pressure gauges used for the purpose have a minimum count of about 2 MPa. Under the circumstances, calculation of cable elongations to 2 decimals of a mm during the design rarely helps. Based on construction requirements, one end prestressing may be indicated in a number of cases of short and medium spans (Fig. 8) even if the same sometimes involves additional prestressing cables.

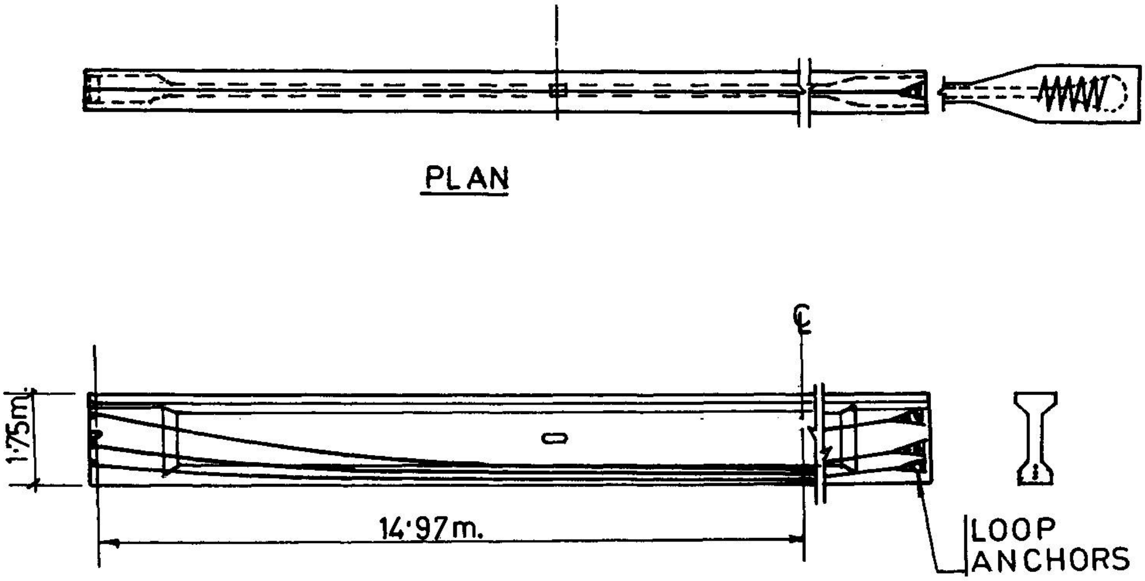


Fig. 8 One end stressing

5.8 Deck slab

The arrangement of composite deck slab for the 30m spans originally envisaged for the Pamban Bridge in South India is shown in Fig. 9, Detail 'X'. While this arrangement has resulted in a theoretically cost effective design, realisation of the same in practice was beset with difficulties. The beams were precast and launched into position at various times in varying climatic conditions. On prestressing, the initial hogging of the beams of varying magnitude has resulted in differences in levels of the deck surface. In addition, the dry mortar packing for the 25mm gaps between the precast flange ends of the beams is of questionable quality to resist the heavy corrosive atmosphere. When the contractor stepped in with his alternative design, this problem was resolved as indicated in Detail 'Y'.

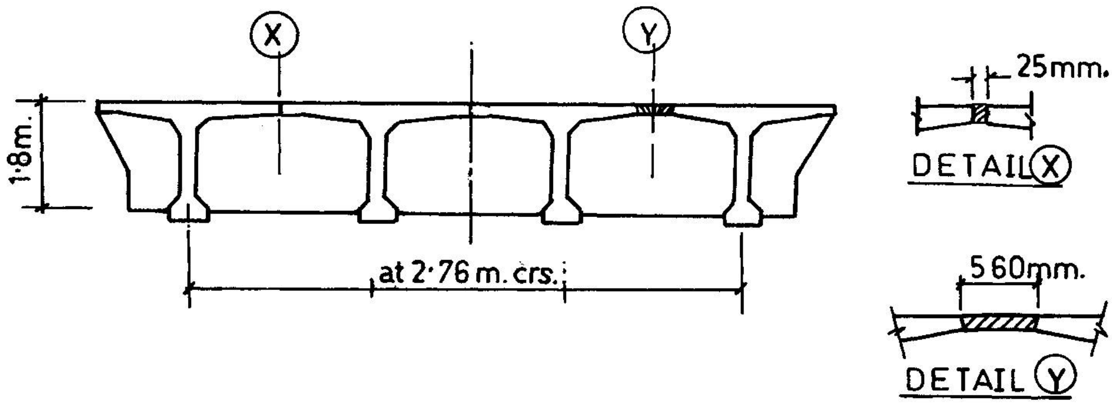


Fig. 9 Pamban Bridge - Deck Slab

6. DETAILING OF REINFORCEMENT

6.1 Waste reduction

The designer rarely takes into account the question of detailing to facilitate optimisation of cutting lengths, using commercially available standard reinforcement bars and consequently wastage of reinforcement could be quite substantial. With a little additional effort, it should be possible for the designer to ensure bar bending schedules taking into account availability of commercial lengths and thus reduce wastage by way of cut lengths. Wherever possible, detailing of reinforcement should be based on the use of sub-multiples of commercial lengths of reinforcement.

The contractor can also contribute towards the reduction in the amount of scrap generated at the project site based on the design and detailing of reinforcement already made available. During the construction of the Bahrain Causeway which has just been completed, a computer programme is reported to have been used for optimising the cutting lengths and minimising the wastage. This aspect is particularly useful. [6]

6.2 Rationalization

Rationalization of the shapes of reinforcement at the design stage will go a long way towards smooth execution. Cost is linked to accuracy of bending. The effort required is proportional to the complexity of shape. Possible use of straight bars in preference to bent up reinforcement should be preferred. Even for shear reinforcement, cost considerations may advocate the use of vertical stirrups in preference to inclined bent up bars or prestressing cables, even though the quantity of reinforcement may be increased.

While detailing stirrups, it is preferable to use minimum number of shapes. Use of same diameter of bars with constant spacings is also indicated from construction considerations.

The designer should also consider the possibility of reduction in the number of diameters while detailing. The impact of extra consumption of reinforcement vis-a-vis decrease in cutting scrap due to interchangability, decrease in risk of substitution of diameters by mistake, faster identification and better management of stores are factors to be considered at the design stage. In this context, the use of diameters of 8-10-12-16-20-25-32-40mm advocated by the CEB is highly recommended. [7]

7. CONCLUSIONS

Due to widespread use of prestressed concrete design for bridge projects, a number of alternative methods of construction have been evolved. In this context the earlier practice of designs being prepared based on the specific method of construction needs to be reviewed. In order to take advantage of the ingenuity of the designer and the contractor in developing a cost effective design, a number of countries are already adopting the concept of alternative bridge designs to be offered by the contractor where only final design requirements are frozen at the tender stage and method of constructing the bridge is the responsibility of the contractor. This procedure demands close interaction between the designer and the contractor in preparing and checking for designs and drawings.

Almost every component of a bridge structure is affected by the above process. A minimum quantity design is not necessarily the economical design; nor does it necessarily recommend itself towards completion of the project at the shortest possible time. Many of the prevailing specifications and codes of practice are based on the expertise and degree of control available in the respective countries. With more and more bridges being realised as export projects in Third World Countries, a fresh look is called for in respect of the specifications and also cooperation between the designer and the contractor.

ACKNOWLEDGEMENT

Many of the case histories cited in the Paper are based on the Author's experience on jobs carried out by Gammon India Limited. The Author is indebted to the Company for affording the experience and to Mr. T.N. Subba Rao, Managing Director, Gammon India Limited, for his enlightened guidance over the years.

REFERENCES

[1] REDDI S.A., Construction of Khalidiyah Bridge Across Euphrates in Iraq, Indian Roads Congress Journal 1986.

[2] HEIMERDINGER M., Salal-Al-Deen Al-Ayubi Expressway, Baghdad, Spannbetonbau in der Bundesrepublik Deutschland 1983-1986, Deutscher Beton-Verein.E.V. 1986.

[3] CEB-FIP Model Code for Concrete Structures, FIP 1978

[4] LONG J.E., Post-Tensioning Practice, U.K. Report on Noteworthy Research and Development, Cement and Concrete Association, U.K. 1986.

[5] WOELFEL E., Tensioning of Tendons-Force-Elongation Relationship, Proceedings, Tenth FIP Congress 1986.

[6] NIJE F. and SWENSSON K., Saudi Arabia-Bahrain Causeway Management Aspects, IABSE 12th Congress, Vancouver, Final Report.

[7] CEP Manual on Concrete Reinforcement Technology, Georgi Publishing Company, Switzerland 1983.