VSL

DETAILING FOR POST-TENSIONED

General Principales

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Preface

The purpose of this report is to provide information related to details for post-tensioned structures It should assist engineers in making decisions regarding both design and construction. This document does not represent a collection of details for various situations. Instead, VSL has chosen to present the basic information and principles which an engineer may use to solve any detailing problem. Examples taken from practice are used to illustrate the concepts.

The authors hope that the report will help stimulate new and creative ideas. VSL would be pleased to assist and advise you on questions related to detailing for posttensioned structures. 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 Objective and Scope

"Detailing for Post-tensioning" addresses the important, but often misunderstood details associated with post-tensioned structures. It has been written for engineers with a modern education who must interpret and use modern design codes. It is hoped that this report will be of interest to practising engineers and aspiring students who want to "get it right the first time"!

The objectives of this document are:

- to assist engineers in producing better designs which are easier and more economical to build;
- to provide previously unavailable back ground design information regarding the more important VSL anchorages;
- to be frank and open about what is actually being done and to disseminate this knowlege; and
- to present a balanced perspective on design and.correct the growing trend of over
- analysis.
  The emphasis is on design rather than analysis!

The scope of this report includes all of the forces produced by post-tensioning, especially those in anchorage zones and regions of tendon curvature (see Figs. 1.1 and 1.2). The emphasis is on standard buildings and bridges utilizing either bonded or unbonded tendons, but the basic principles are also applicable to external tendons, stay cable anchorages and large rock or soil anchors.

The scope of this report does not include such items as special corrosion protection, restressable/removable anchors, or detailed deviator design, as these are dealt with in other VSL publications [1, 2, 3]. In addition, conceptual design and overall structural design is not addressed as these topics are covered in many texts. We wish to restrict ourselves to the "mere" and often neglected details!

We freely admit that one of VSL's objectives in preparing this document is to increase profits by helping to avoid costly errors (where everyone involved in a project looses money), and by encouraging and assisting engineers to design more post-tensioned structures. We therefore apologize for the odd lapse into commercialism.

Figure 1.1: Anchorages provide for the safe introduction of post-tensioning forces into the concrete.

1.2 Background

When Eugene Freyssinet "invented" prestressed concrete it was considered to be an entirely new material - a material which did not crack. Thus, during the active development of prestressed

concrete in the 1940's and 1950's the emphasis was on elastic methods of analysis and design. The elastically based procedures developed by Guyon [4] and others [5, 6] worked. In fact, the previous VSL report [7] which addressed anchorage zone design was based on

elastic methods. Designers were guided by a few general solutions which would be modified with judgement to suit the specific situations. With the development of computers in the 1960's and 1970's, analysis became overly, perhaps even absurdly detailed. There was little if any improvement in the actual structures inspite of the substantially increased analytical effort. Blunders occasionally occurred because, as the analysis became more complex, it was easier to make mistakes and harder to find them. More recently there was a realization that prestressed concrete was just one part of the continuous spectrum of structural concrete which goes from unreinforced concrete, to reinforced concrete, to partially prestressed concrete to fully

prestressed concrete. It was realized that even prestressed concrete cracks. If it did not crack, there certainly would be no need for other reinforcement. Codes moved ahead, but designers lacked guidance. Fortunately the principles of strut-and-tie analysis and design were "rediscovered" in the 1980's. Rather than being a mere analyst, with these methods, the designer can, within limits, tell the structure what to do. We as designers should be guided by elasticity (as in the past), but we need not be bound to it.

It is from this historical setting that we are attempting to provide designers with guidance on the detailing of posttensioned structures.

1.3 Organization of the Report

Chapter 2 of this report presents the general engineering principles used throughout the rest of the document. This is followed by a chapter on several specific VSL anchorages. Chapter 4 deals with general anchorage zone design and items related to tendon curvature. This is followed by real world design examples to illustrate the concepts in detail.

The report is basically code independent. Through an understanding of the basic engineering principles the reader should be able to readily interpret them within the context of any specific design code. S.I. units are used throughout. All figures are drawn to scale so that even when dimensions are omitted the reader will still have a feeling for correct proportions. When forces are given on strut-and-tie diagrams they are expressed as a fraction of P, the anchorage force.

While symbols are defined at their first occurrence, a few special symbols are worth mentioning here:

$f'c$ = the 28 day specified (characteristic) concrete cylinder strength.
To convert to cube strengths one may assume that for a given concrete the characteristic cube strength will be 25 % greater than the cylinder strength.

$f'ci$ = the concrete cylinder strength at the time of prestressing. With early stressing, this will be less than $f'c$.

GUTS = the specified guaranteed ultimate tensile strength of the tendon (i.e. the nominal breaking load).

It should be noted that this document refers specifically to the VSL "International" system hardware and anchorage devices. The VSL system as used in your country may be somewhat different since it is VSL policy to adapt to the needs of the local users. Your local VSL representative should be contacted for specific details.

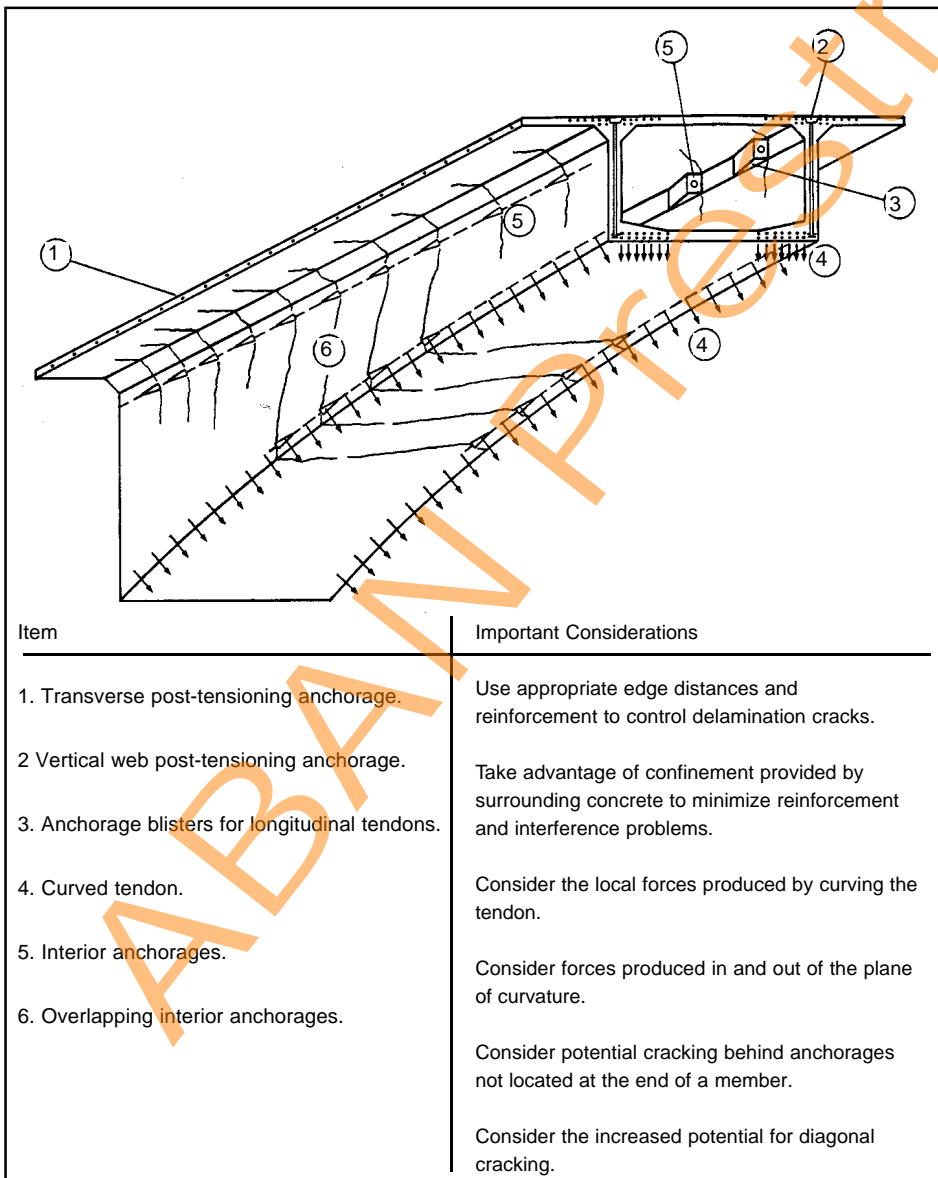| Item	Important Considerations
1. Transverse post-tensioning anchorage.	Use appropriate edge distances and reinforcement to control delamination cracks.
2 Vertical web post-tensioning anchorage.	Take advantage of confinement provided by surrounding concrete to minimize reinforcement and interference problems.
3. Anchorage blisters for longitudinal tendons.	Consider the local forces produced by curving the tendon.
4. Curved tendon.	Consider forces produced in and out of the plane of curvature.
5. Interior anchorages.	
6. Overlapping interior anchorages.	Consider potential cracking behind anchorages not located at the end of a member.
	Consider the increased potential for diagonal cracking.

Figure 1.2 Special stress situations must be recognized and provided whith appropriate detailing

2. General Principles

2.1 Post-tensioning in a Nut Shell

While it is assumed that the reader has a basic understanding of post-tensioning, some general discussion is warranted to introduce terms as these are not always internationally consistent. There are many helpful text books on the subject of prestressed concrete. American readers may wish to reference Collins and Mitchell [8], Lin and Burns [9] or Nilson [10]. International readers may wish to reference Warner and Foulkes [11], Collins and Mitchell [12] and Menn [13] in English; or Leonhardt [14] and Menn [15] in German.

Post-tensioning is a special form of prestressed concrete in which the prestressing tendons are stressed after the concrete is cast. Post-tensioning utilizes high quality high strength steel such that 1 kg of post-tensioning strand may replace 3 or 4 kg of ordinary non-prestressed reinforcement. This can reduce congestion in members. Post-tensioning tendons are usually anchored with some form of mechanical anchorage device which provides a clearly defined anchorage for the tendon. With bonded systems the tendons are positioned inside of ducts which are filled with grout after stressing. This introduces a compatibility between the prestressing steel and concrete which means that after bonding any strain experienced by the concrete is experienced by the prestressing steel

and vice versa. With unbonded systems, the tendon is only anchored at the ends and bond is deliberately prevented along the length of the tendon. Thus, concrete strains are not translated directly into similar strains in the prestressing steel. With post-tensioning a variety of tendon profiles and stressing stages/sequences are possible. The post-tensioning tendon introduces anchor forces, friction forces and deviation forces (where tendons curve) into the concrete. These forces can generally be used to advantage to balance other loads and thus control deflections and reduce cracking.

2.2 Design Models

Without elaborating on the details, a few general comments on design models are warranted.

Strut-and-tie models are a suitable basis for detailed design. Schlaich et al. [16], Marti [171 and Cook and Mitchell [181 provide details on the general use of these models. It is essential that the model is consistent. A detailed elastic analysis is not necessary provided that one is cognizant of the general elastic behaviour when developing the strut-and-tie model.

One should never rely solely on concrete tensile strength to resist a primary tensile force. With judgement and adchloral safety margins, one can relax this rule.

Confinement of concrete in two orthogonal directions will enhance its bearing capacity in the third orthogonal direction. For every 1 MPa of confinement stress, about 4 M Pa of extra capacity is produced. This is in addition to the unconfined compressive strength.

Reinforcement used to confine concrete should have strains limited to about 0.1 % (i.e. 200 MPa stress) under ultimate loads. Reinforcement used to resist primary tie (tension) forces should have stresses limited to about 250 MPa under service loads.

With the above approach, it is useful to consider the post-tensioning as a force on the concrete. As shown in Fig. 2.1, after bonding, that portion of the stress-strain curve of the prestressing steel not used during stressing is available to contribute to the resistance of the member, similar to non-prestressed reinforcement. Hence, with due recognition of the bond properties of strand and duct, it can be treated like ordinary non-prestressed bonded reinforcement with the yield stress measured from point A in Fig. 2.1. Unbonded tendons are treated differently.

The above design models have proven to be suitable for standard applications with concrete strengths of 15 MPa to 45 MPa. Caution should be used in unusual applications and with concrete strengths significantly different than those noted.

a) Stress-strain response of basic materials.

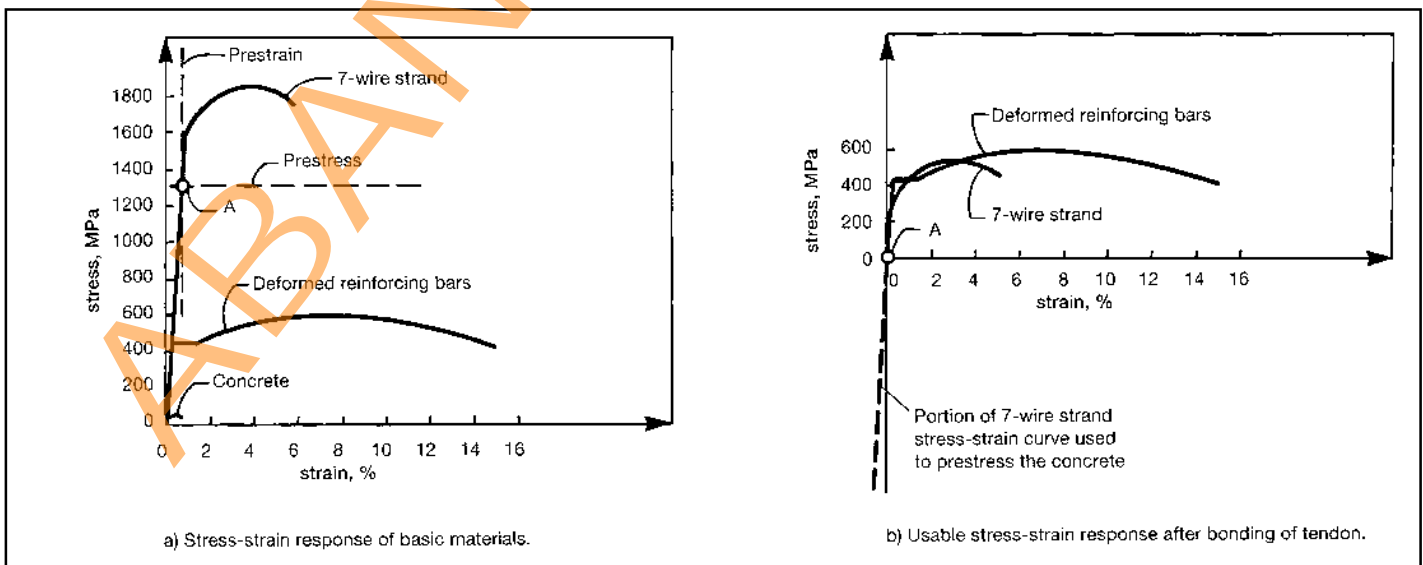b) Usable stress-strain response after bonding of tendon.

Figure 2.1: After bonding, prestressed reinforcement can be treated like non-prestressed reinforcement.

As a final comment, sound engineering judgement is still the most important ingredient in a good design. Throughout this report you may find what might appear to be inconsistencies in design values for specific cases. This has been done deliberately to reinforce the point that the information presented is not a set of rigid rules, but rather a guide which must be applied with judgement.

2.3 Performance Criteria

Before one can adequately detail a post-tensioned structure, one must understand what the performance requirements are. The general objective is obviously to provide a safe and serviceable structure. The question is "What are reasonable ultimate and service design loads for strength and serviceability checks?"

Modern safety theory could be used to determine design loads by considering all of the relevant parameters as statistical variables and examining the combined effect of these variations. The net result would be load and resistance factors selected to provide some desired probability of failure. For example, if one took a load factor of 1.3 on the maximum jacking force, and a resistance factor of 0.75 for the anchorage zone, one would get a factored design load greater than realizable strength of the tendons - a physical impossibility! More significantly, the corresponding resistance factors result in unrealistically low predicted design strengths for the concrete. Using such proposed load and resistance factors would render most current anchorage designs unacceptable. Since the current designs have evolved from many years of satisfactory experience, one must conclude that it is the proposed load and resistance factors which are not satisfactory!

Fortunately, by reviewing the construction and load history of a post-tensioning system, one can arrive at reasonable design values in a rational and practical manner. In typical applications the history is as follows: 1. Post-tensioning is stressed to a maximum temporary jack force of 80 % GUTS when the concrete has a verified compressive strength of 80 f'c, the specified 28 day strength.

2. By design, immediately after lock-off, the maximum force at the anchorage is at most 70 % GUTS.

3. For bonded systems, the tendon is grouted shortly after stressing. For cast-in-place members, the shoring is removed. In the case of precast members, they are erected.

4. The structure is put into service only after the concrete has reached the full specified strength.

5. Time dependent losses will reduce the effective prestressing force and hence, the anchorage force will decrease with time to about 62 % GUTS.

6. (a) Bonded systems - Actions (loads and imposed deformations) applied to the structure which produce tension strains in the concrete and bonded non-prestressed reinforcement produce similar strain increases in the bonded prestressed reinforcement. In zones of uncracked concrete, these strains produce negligible increases in force at the anchorage. Bond demands (requirements) in uncracked zones are small. Once cracks develop, the force in the bonded reinforcement (prestressed and no n-prestressed) increase via bond. When the maximum bond resistance is reached, local slip occurs. If the anchorage is located further away from the crack than the development length of the tendon, again only insignificant increases in the force of the anchorage result. However, for anchorages close to the crack, increases in the anchorage force up to the maximum realizable capacity of the tendon assembly may be reached. The realizable capacity is the anchorage efficiency times the nominal capacity. The maximum tendon capacity realized is usually about 95 % GUTS. At this point, the wedges may start to slip, but usually individual wires in the strand begin to break. The strain experienced by the structure is the strain capacity of the strand (at least 2 %) less the strain introduced to the strand prior to bonding (about 0.6 %) and is thus about six times the yield strain for the non-prestressed bonded reinforcement. Note that should an anchorage fail, the tendon force often can be transferred by bond in a manner similar to ordinary pre-tensioned members. The bond provides an alternative

load path for the introduction of the tendon force into the concrete thereby improving safety through redundancy. Locating anchorages away from sections of maximum stress, as is normally done, therefore provides improved safety. (b) Unbonded systems - Due to the absence of bond, the prestressed reinforcement does not normally experience the same strain as the nonprestressed bonded reinforcement when actions are applied to the structure. With large structural deformations, the changes in tendon geometry produce increases in the tendon force, but these are not necessarily sufficient to cause tendon yielding. The deformations required to produce the changes in tendon geometry necessary to develop the realizable capacity of the tendon are enormous and usually can not be sustained by the concrete.

A satisfactory design is possible if one examines what can go wrong during the construction and use of a structure, along with the resulting consequences. By looking at such fundamentals, one can readily deal with unusual construction and loading histories. For anchorages with the typical construction and load histories, one can conclude that the anchorage typically receives its maximum force during stressing when the concrete strength is 80 % f'c. In service, the anchorage forces will be smaller and the concrete strengths will be larger. It is possible to exceed the usual temporary jacking force of 80 % GUTS during stressing but not by very much and certainly not by a factor of 1.3. First, the operator controls the stressing jack to prevent excessive overstressing. Unless an oversized jack is used to stress a tendon, the jack capacity of about 85 % GUTS will automatically govern the maximum jacking force. Finally, if an oversized jack is used and the operator blunders (or the pressure gage is defective), the anchorage efficiency at the wedges will limit the realizable tendon force to about 95 % GUTS. This is accompanied by tendon elongations of at least 2 % (about 3 times greater than normal) which cannot go unnoticed. For anchorages in service it is possible but not usually probable that the anchorage force increases as discussed in point 6 above. In any event, the maximum realizable force is governed by the anchorage efficiency.

With extremely good anchorage efficiency and overstrength strand, one may reach 100 % GUTS under ultimate conditions, but there would be ample warning before failure since the structure would have to experience large strains and deformations. It is not necessary to design for a force larger than the realizable force in the tendon assembly based on the minimum acceptable anchorage efficiency. In summary, **the probable maximum load on an anchorage for strength design checks is about 95 % GUTS.**

It is possible to have lower than nominal resistance (calculated with nominal material properties), but not much lower. First, in-place concrete strengths are verified prior to stressing. The stressing operation provides further confirmation of the concrete strength which is rarely a problem. On the other hand, improper concrete compaction around the anchorages is occasionally revealed during stressing. Such honeycombing manifests itself by usually cracking and spalling of the concrete during the stressing operations. This "failure" mechanism is benign in that it is preceeded by warning signs, and occurs while the member is temporarily supported. When it occurs, the stressing is stopped, the defective concrete is replaced, and the anchorage is restressed. The most serious consequence of an anchorage zone failure during stressing is usually a delay in the construction schedule. Since early stressing to 80 % GUTS with 80 % f'c provides a "load test" of each and every anchorage, deficiencies in the resistance of the anchorage zone are revealed during construction when they do little harm. Successfully stressing a tendon removes most of the uncertainty about the resistance of the anchorage zone. Failures of anchorage zones in service due to understrength materials are unheard of. In summary, **it is reasonable to use 95 % of nominal material properties in strength calculations when the ultimate load is taken as 95 % GUTS. While designing for 95 % GUTS** with 95 % f'ci is proposed, other proportional values could be used. For example designing for GUTS with f'ci would be equivalent. Note that for unreinforced anchorage zones with f'ci = 0.8 f'c, these proposals would be equivalent to designing for 125 % GUTS with f'c. For the permanent load case, the overall

factor of safety would be not less than 125 % / 70 % = 1.79 which is quite substantial and in line with typical requirements for safety factors used in structural concrete design.

Adequate crack control is the usual serviceability criterion of interest for anchorage zones. Extreme accuracy in the calculation of crack widths is neither possible nor desirable since it implies undue importance on crack width. The presence of adequate high quality (low permeability) concrete cover is more important for good durability. Most crack width calculation formulas predict larger crack widths for increased concrete cover. If a designer chooses to increase concrete cover to improve durability, he is "punished" by the crack width calculation which predicts larger cracks. The explicit calculation of crack widths is of dubious value.

From the typical construction and load history described, it is apparent that the anchorage force under service load will be between 62 % and 70 % GUTS. For serviceability checks, one may conveniently use an anchorage force of 70 GUTS. In an unusual application where the anchorage force increases significantly due to applied actions, the anchorage force resulting from such actions at service load should be used for serviceability checks.

The maximum permissible crack width depends upon the exposure conditions and the presence of other corrosion control measures. For moderately aggressive environments (e.g. moist environment where deicing agents and sea salts may be present in mists, but where direct contact of the corrosive agents is prevented), a crack width of 0.2 mm is generally considered acceptable. This limit is usually applied to "working" flexural cracks in a structure. It is possible that larger crack widths may be acceptable in anchorage zones where the cracks are "non-working", that is, the crack width is relatively constant under variable loading. There are no known research studies specifically aimed at determining permissible cracks in anchorage zones, but it is clear that one may conservatively use the permissible crack widths given in most codes. This may be too conservative since inspections of existing structures with anchorage zones containing cracks larger than 0.2 mm rarely reveal service ability prob

lems. One must appreciate that many successful structures were built in the "old days" before crack width calculations came into vogue. The secret of success was to use common sense in detailing.

For design, adequate crack control can be achieved by limiting the stress in the non-prestressed reinforcement to 200 to 240 MPa under typical service load. CEB-FIP Model Code 1990 (first draft) [20] would support the use of these specific stresses provided that bar spacings are less than 150 mm to 100 mm, respectively, or bar sizes are les than 16 mm to 12 mm diameter, respectively. As a practical matter, in the local anchorage zone where reinforcement is used to confine the concrete to increase the bearing resistance, the strain in the reinforcement is limited to 0.1 % to 0.12% under ultimate load. As a result, under service loads, the local zone reinforcement stresses will alwmays be low enough to provide adequate crack control. Further, if the general anchorage zone reinforcement used to disperse the anchorage force over the member cross section is proportioned on the basis of permissible stress at service load (say 250 MPa), ultimate strength requirements for the general zone will always be satisfied.

Unless special conditions exist, it is sufficient to deal with serviceability considerations under an anchorage force of 70 % GUTS. For moderately aggressive exposures, serviceability will be acceptable if service load stresses in the nonprestressed reinforcement are limited to 200 to 250 MPa.

In summary:

- For ultimate strength checks, the ultimate anchorage force may be taken as 95 % GUTS, and 95 % of the nominal material properties for the concrete and non-prestressed reinforcement (with strain limit for local zones) may be taken when calculating the ultimate resistance.

- For serviceability checks, the service anchorage force may be taken as 70 GUTS. Serviceability will be satisfied if the stress in the non-prestressed reinforcement is limited to acceptable values of about 200 to 250 MPa which are independent of steel grade.

2.4 General and Local Anchorage Zones

Anchorage zones for post-tensioning tendons are regions of dual responsibility which is shared between the engineer of record and the supplier of the posttensioning system. To prevent errors as a result of simple oversight, the division of responsibility must be clearly defined in the project plans and specifications.

The supplier of the post-tensioning system is usually responsible for the design of the anchorage device and the local zone immediately surrounding the device. The supplementary reinforcement requirements (spirals, etc ...) relate to the design of the anchorage device itself which in turn involves proprietary technology.

The engineer of record is responsible for the design of the general zone which surrounds the local zone. While the design of the local zone is usually standardized for standard anchor spacings and side clearances, the design of the general zone is different for each applica

tion as it depends on the position of the tendon and the overall member geometry.

AASHTO [19] have proposed the following definitions:

General Zone - The region in front of the anchor which extends along the tendon axis for a distance equal to the overall depth of the member. The height of the general zone is taken as the overall depth of the member. In the case of intermediate anchorages which are not at the end of a member, the general zone shall be considered to also extend along the projection of the tendon axis for about the same distance before the anchor. See Fig. 2.2.

Local Zone - The region immediately surrounding each anchorage device. It may be taken as a cylinder or prism with transverse dimensions approximately equal to the sum of the projected size of the bearing plate plus the manufacturer's specified minimum side or edge cover. The length of the local zone extends the length of the anchorage device plus an additional distance in front of the anchor
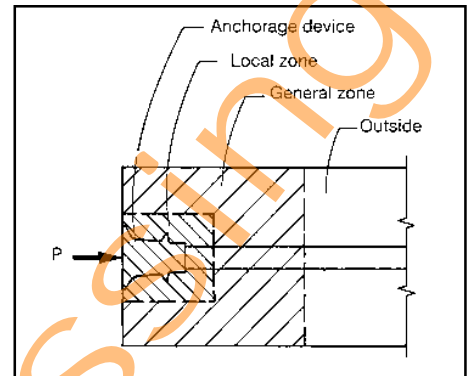

Figure 2.2: Design of supplementary reinforcement in the local zone is the responsibility of anchorage supplier.

equal to at least the maximum lateral dimension of the anchor. See Fig. 2.2.

It must be emphasized that this is an artificial boundary for legal purposes and that other definitions are possible. The essential point is that there must be consistency between the local anchorage zone and the general anchorage zone design.

Key Principles

1. Post-tensioning tendons introduce anchor forces, friction forces and (in zones of tendon curvature) deviation forces into the concrete.
2. Strut-and-tie models which appropriately identify the primary flow of forces are sufficient for design.
3. Primary tension tie forces should normally be resisted by reinforcement.
4. Primary compression strut and node forces should normally be resisted by concrete.
5. The construction and load history should be reviewed to identify governing situations for strength and serviceability.
6. The design of anchorage zones is an area of dual responsibility between the engineer of record and the supplier of the post-tensioning system.

Practical Consequences and Considerations

These forces must be accounted for in the design. Failures are bound to occur if these forces are ignored.

The reinforcement detailing must be consistent with the design model.

Under ultimate load conditions, reinforcement stresses may approach yield. Under service load conditions steel stresses should be limited to about 200 to 250 MPa for crack control. In normal applications don't rely on concrete tensile capacity to resist a primary tension force.
Often confinement of the concrete is used to enhance its compressive strength. For every 1 MPa of confinement stress about 4 MPa of additional compressive strength is produced. (Strain in the confinement reinforcement should be limited to about 0.1 % under ultimate loads).
For the typical applications described, the maximum realizable capacity of the tendon (about 95 % GUTS) will be the limiting ultimate anchor force, while the force immediately after lock-off will be the limiting service anchor force. Strength considerations during stressing will generally govern local zone designs with early stressing at f'ci <f'c Serviceability will usually govern general zone design. The designs for the local zone and the general zone must be compatible. Understanding, cooperation and communication between the engineer and the supplier of the post-tensioning system is essential.

3. Local Zone Design

3.1 General

Four of the more important VSL anchorages will be treated in this chapter. There are of course many other anchorages and couplers available from VSL. Simplified models suitable for design are given. The models produce local zone designs which are similar to those in standard VSL data sheets, and brochures. The standard VSL local zone designs may differ slightly because they are based on experimental results and more complex analysis. The intention is to present sufficient information so that the engineer of record can understand what goes on inside the local zone of various types of anchorages. Through this understanding, it is hoped that better and more efficient designs can be developed and that serious errors can be avoided.

3.2 VSL Anchorage Type E

The E anchorage is a simple and versatile plate anchorage (see Fig. 3.1). Since the bearing plate is cut from mild steel plate, the dimensions can easily be adjusted to suit a wide range of concrete strengths. Anchorages for special conditions can be readily produced. While it can be used as a fixed anchorage, it is more often used as a stressing anchorage. Variants of the basic E anchorages are usually used for soil and rock anchors since the plates can be installed on the surface of the concrete (with suitable grout bedding) after the concrete is cast.

Bearing plates for VSL type E anchorages can be sized in accordance with the following design equations:

$$A^2 - \frac{\pi E^2}{4} = \frac{0.95\, P_N}{2.2\, f'_{ci}} \qquad (3.1)$$

$$B = 6.4\,(A-E)\sqrt[3]{\frac{f'_{ci}}{E_S}} \qquad (3.2)$$

where P_N = specified tensile strength (GUTS) of anchored cable, N.

f'_{ci} = minimum required concrete cylinder strength at stressing, MPa.

E_S = 200,000 MPa = modulus of elasticity of steel bearing plate.
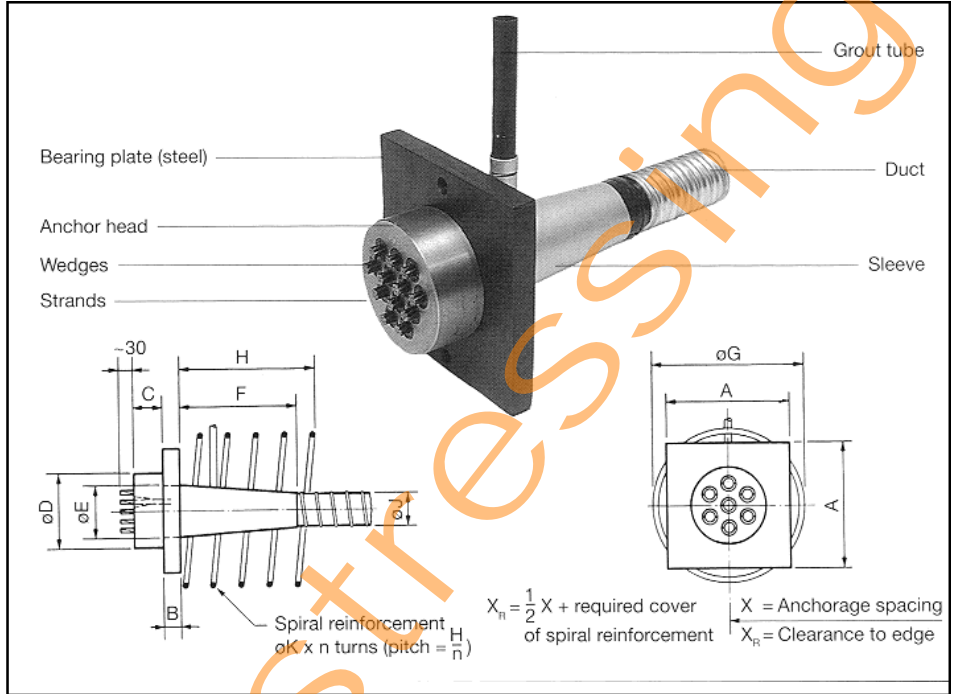
Other parameters as per Fig. 3.1

Figure 3.1: The VSL Type E anchorage is a versatile anchorage.

a) Forces acting on cylinder of concrete under an anchorage bearing plate.

$T = f_l \cdot pitch \cdot r_i$
= Spiral Tension

b) Forces acting on confinement reinforcement.
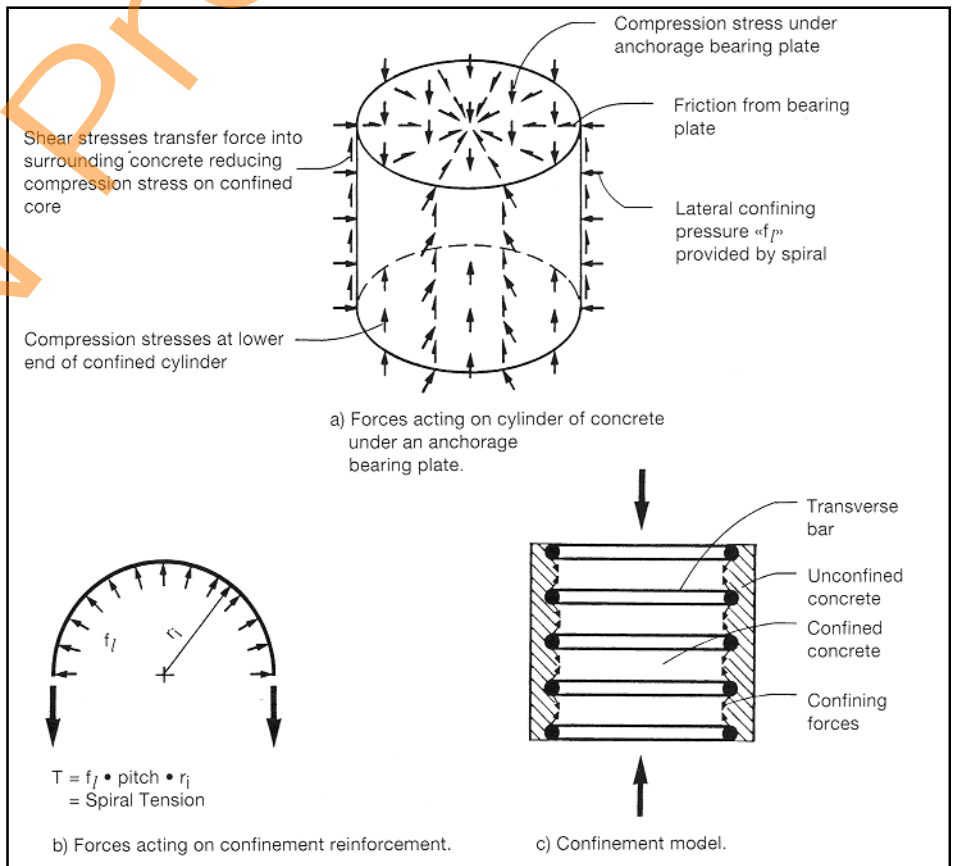
c) Confinement model.

Figure 3.2: Spiral reinforcement confines the concrete and enhances its bearing capacity.

Through experience and testing, these equations have been found to be satisfactory. Eq. (3.1) results in an average net bearing pressure of 2.3 times the initial concrete cylinder strength under a force of GUTS. Eq. (2) comes from the fact that for mild steel (fy ≥ 240 MPa) stiffness rather than strength governs the plate thickness. Finite element analysis will show that this thickness is not sufficient to produce a uniform bearing pressure under the plate but tests will show that it is sufficient to produce acceptable designs. It is advantageous to have somewhat reduced bearing pressures near the perimeter of the plate as this helps to prevent spalling of edge concrete when the anchorage is used with the minimum edge distance.

It is obvious from Eq. (3.1) that the concrete receives stresses greater than its unconfined compressive strength. The local zone concrete under the anchorage must have its strength increased by some form of confinement. Most design codes permit an increase in permissible bearing stress when only a portion of the available concrete area is loaded. While such provisions are valid for post-tensioning anchorages, they are usually too restrictive since they do not account for confinement provided by reinforcement. VSL anchorages normally utilize spiral reinforcement.

Before dealing with confinement of the local zone, one must determine what zone has to be confined. In VSL anchorages the size of the confined zone is controlled by the capacity of the unconfined concrete at the end of the local zone. This for example determines X, the minimum anchor spacing. The local zone may be assumed to be a cube with side dimensions of X determined by:

$$0.8\ f'_{ci}\ (X_C^2 - \frac{\pi}{4}\ j^2\ ) = 0.95\ P_N \quad\text{———(3.3a)}$$

or

$$f'_{ci}X^2 = 0.95\ P_N \quad\text{———(3.3b)}$$

where : X_C = X + (2 * clear concrete cover to the reinforcement).

Eq. (3.3b) is a useful simplification of Eq.(3.3a) which yields similar results for practical situations. It should be noted that the standard HIP [20] load transfer test prism, which is tested with a concrete strength of 85 % to 95 % of f'$_{ci}$ has dimensions of X by X by 2X. The results support Eq. (3.3b).
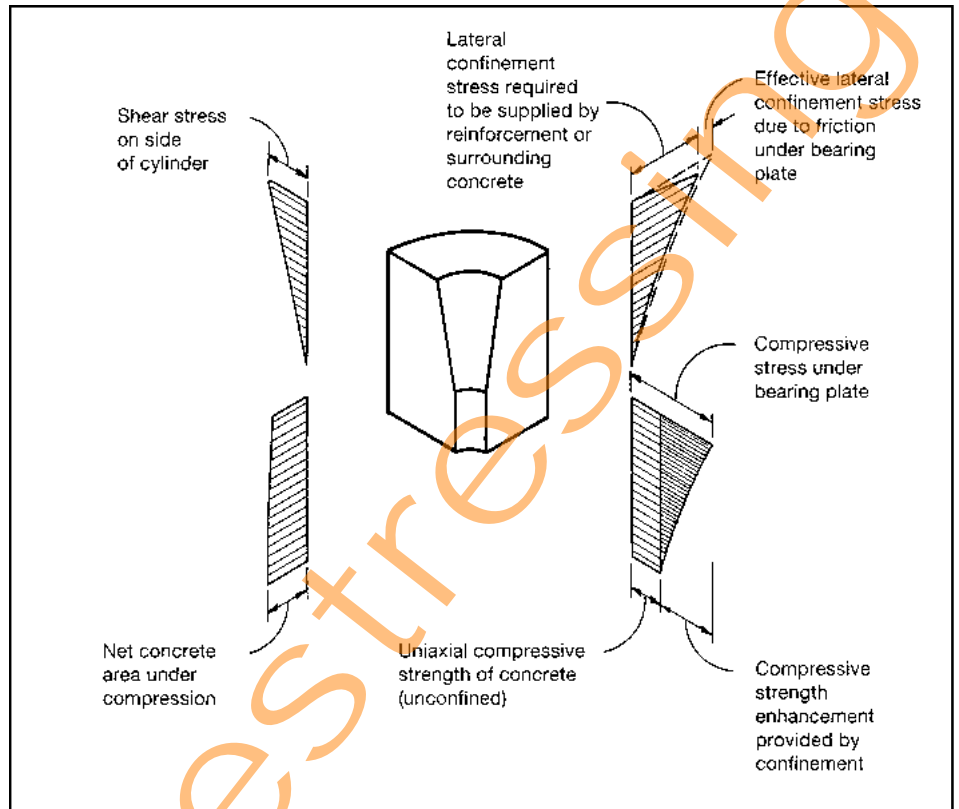

Figure 3.3: Side friction on the confined concrete reduces the axial compressive stress at the end of the local zone.

Back to the matter of confining the local zone which is a cube with side dimensions X. Usually only a portion of this zone needs to be confined. It is common practice and sufficient to proportion spiral reinforcement to confine a cylindrical core of concrete which is capable of resisting the realizable capacity of the tendon. A practical spiral would have an outside diameter of about 0.95X to allow for fabrication tolerances, and a clear space between adjacent turns of 30 to 50 mm to allow proper concrete placement. With larger spacings, one looses the benefit of confinement between adjacent turns in the spiral. Figure 3.2 illustrates the confined core concept. The unconfined concrete outside of the spiral carries a portion of the anchorage force. For standard anchorage conditions, the calculations can be simplified by ignoring the additional capacity provided by the unconfined concrete and ignoring the loss in capacity due to the reduced core area as a result of trumpet and duct. The effective confined core diameter may be conveniently taken as the clear inside diameter of the spiral. This approximately

accounts for the arching between adjacent turns when the recommended spacings are observed.

For practical design, the spiral confinement reinforcement can be sized in accordance with the following design equations:

$$(0.85\ f'_{ci} + 4\ fl)\ A_{core} = 0.95\ PN \quad (3.4)$$

$$fl = \frac{A_{spiral} * f_s}{r_i * p} \quad (3.5)$$

where

A_{spiral} = cross sectional area of the rod used to form the spiral, mm2.

f_s = stress in the spiral reinforcement corresponding to a steel strain of 0.001, i.e. 200 MPa.

A_{core} = π,=rj^2, mm2.

r_i = clear inside radius of spiral reinforcement, mm.

p = pitch of spiral reinforcement, mm.

The length of the spiral reinforced zone can be set equal to the diameter of the
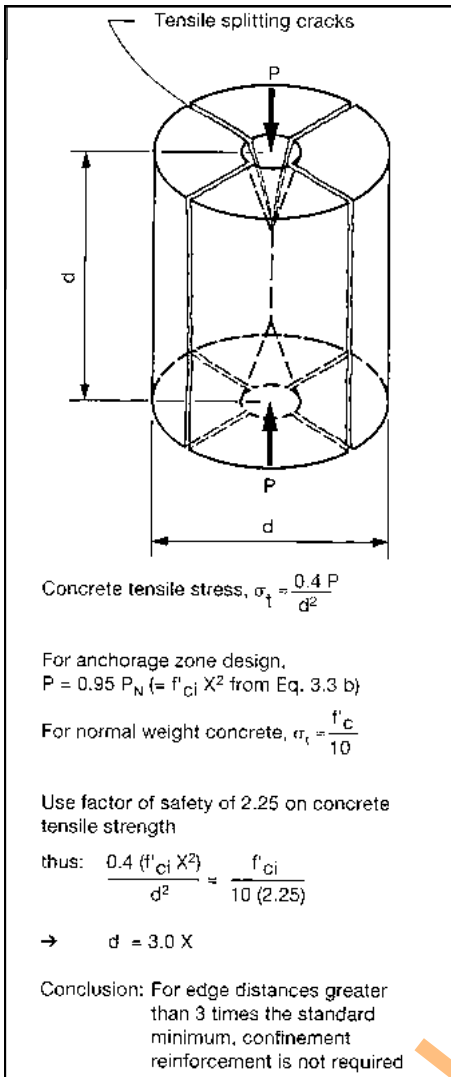
Tensile splitting cracks

Concrete tensile stress, $\sigma_t = \dfrac{0.4\,P}{d^2}$

For anchorage zone design,
$P = 0.95\,P_N\ (= f'_{ci}\,X^2$ from Eq. 3.3 b)

For normal weight concrete, $\sigma_t = \dfrac{f'_c}{10}$

Use factor of safety of 2.25 on concrete tensile strength

thus: $\dfrac{0.4\,(f'_{ci}\,X^2)}{d^2} = \dfrac{f'_{ci}}{10\,(2.25)}$

→ $d = 3.0\,X$

Conclusion: For edge distances greater than 3 times the standard minimum, confinement reinforcement is not required

Figure 3.4: Large edge distances make confinement reinforcement redundant.

spiral zone. Fig. 3.3 provides a more detailedreview of the situation in the local zone. It can be seen that the amount of confinement required varies over the length of the local zone. One could vary the confinement reinforcement to more closely match requirements and thus save reinforcement. For example, one could use: rings with varying spacing and cross sectional area; or a full length spiral with a reduced steel area combined with a short spiral or ties providing the additional necessary reinforcement in the upper zone. Often, the structure already contains reinforcement which will serve as confinement reinforcement thus reducing or eliminating the spiral reinforcement requirements.
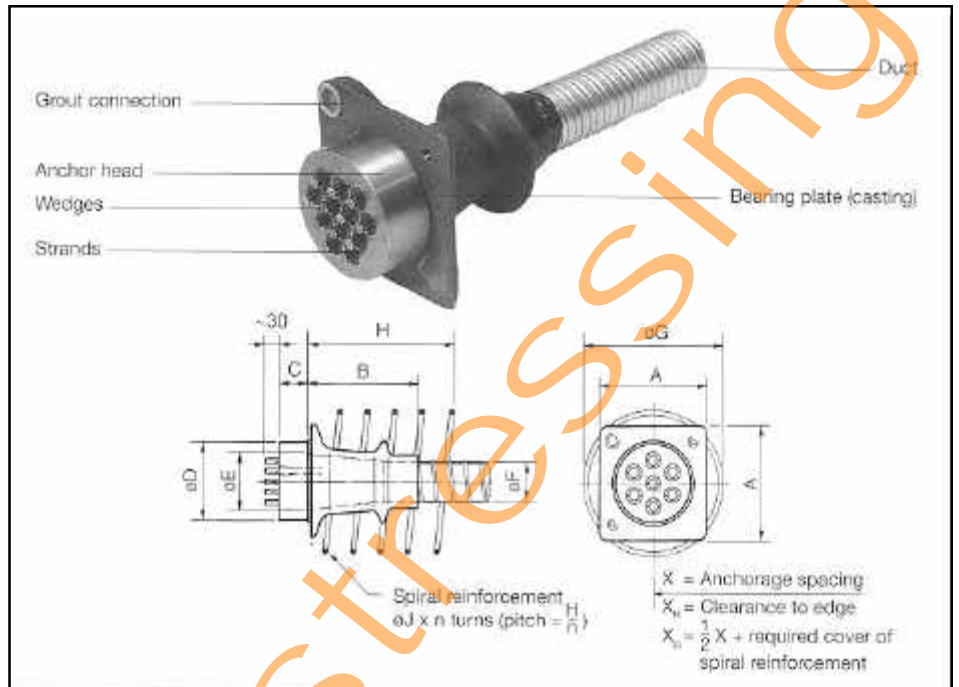

Figure 3.5: The VSL Type EC anchorage is an efficient anchorage.

Confinement of the local zone may also be provided by surrounding concrete. Here we rely on the tensile strength of the surrounding concrete so a conservative approach is warranted. Double punch tests [22] may be used as a basis for determining the minimum dimensions for concrete blocks without confinement reinforcement. This is illustrated in Fig. 3.4. The basic conclusion is that if the actual edge distance is more than 3 times the standard minimum edge distance for the anchorage, confinement reinforcement is unnecessary in the local zone (i.e. 3X/2 plus cover).

Confinement provided by surrounding concrete should not be added to confinement provided by reinforcement since the strains required to mobilize the reinforcement will be sufficient to crack the concrete.
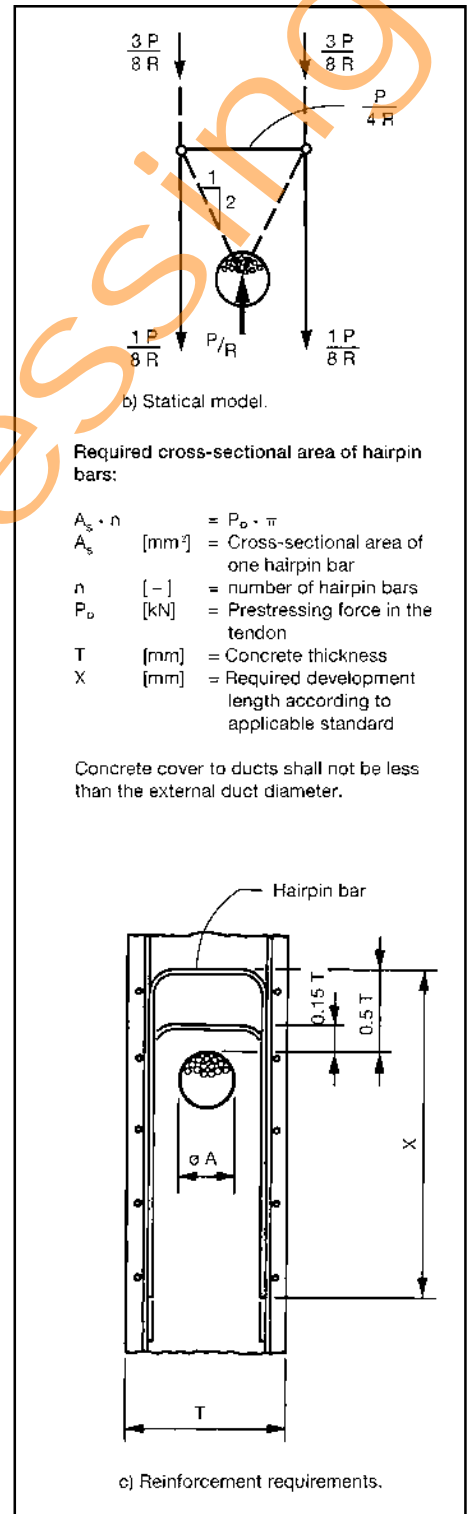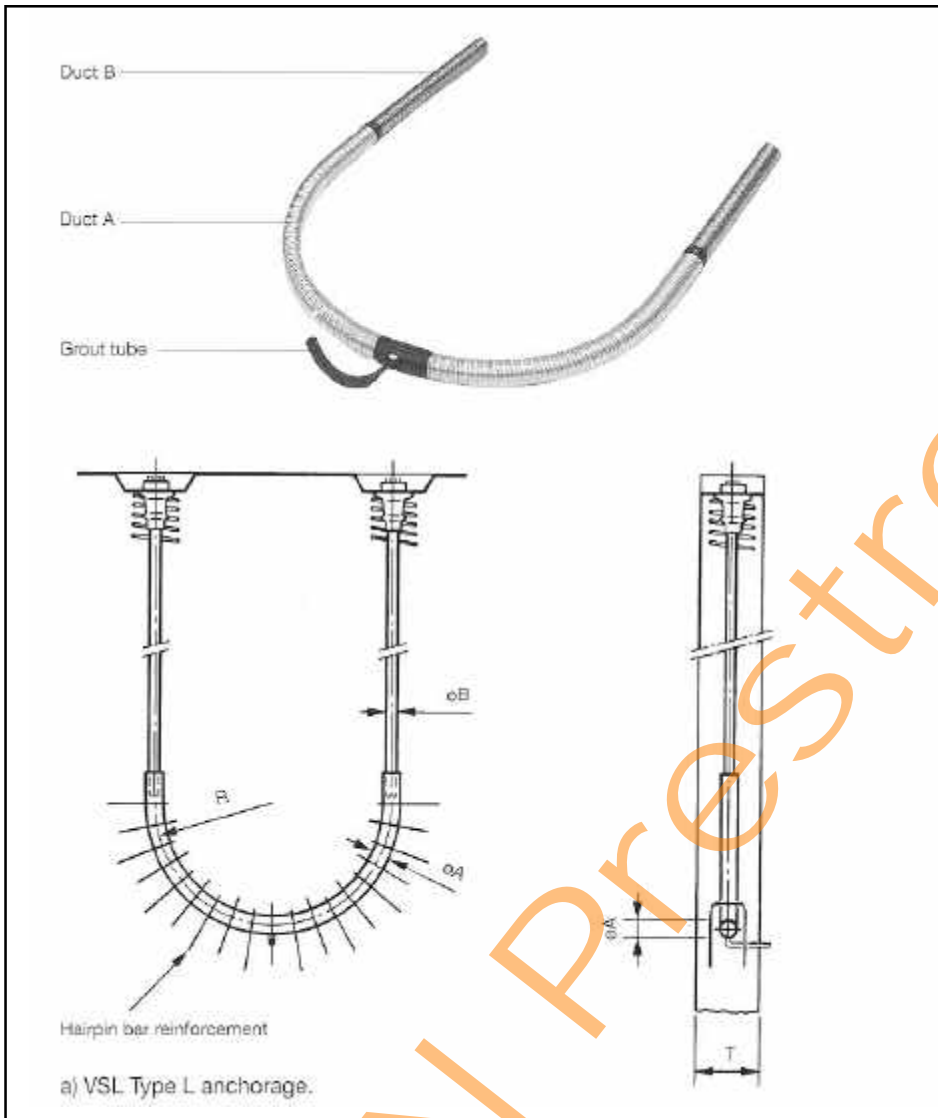
3.3 VSL Anchorage Type EC

The EC anchorage is a very efficient anchorage for concretes with a compressive strength at stressing of about 22 MPa (usually used in 20 to 30 MPa concrete). As shown in Fig. 3.5 it is a casting which has been optimized to provide a very economical stressing anchorage for

standard applications. The casting incorporates a transition so that it can be connected directly to the duct without the need of a separate sleeve/trumpet. The tapered flanges further reduce costs. The double flanges result in a forgiving anchorage when minor concrete deficiencies are present. While it can be used as a fixed anchorage, the EC is the most commonly used VSL multistrand stressing anchorage.

The design principles for the EC anchorages are similar to those for E anchorages (except for the plate of course). Since the casting dimensions are generally fixed for a given tendon regardless of concrete strength the EC confinement reinforcement will vary substantially for different concrete strengths. This variation in spiral weight helps the EC anchorage to be competitive outside of the range of concrete strengths for which it was originally optimized.

For low concrete strengths at stressing, it is occasionally possible to utilize the next larger EC anchor body (i. e. the anchor body of a larger tendon). The local VSL representative should be contacted to see what specific alternative anchor bodies will work for a given application. The standard anchor head may

a) VSL Type L anchorage.

b) Statical model.

Required cross-sectional area of hairpin bars:

$A_s \cdot n$ = $P_o \cdot \pi$
A_s [mm²] = Cross-sectional area of one hairpin bar
n [–] = number of hairpin bars
P_o [kN] = Prestressing force in the tendon
T [mm] = Concrete thickness
X [mm] = Required development length according to applicable standard

Concrete cover to ducts shall not be less than the external duct diameter.

c) Reinforcement requirements.

Figure 3.6: L anchorage reinforcement should deal with in-plane and out-of plane forces.

have to be altered to suit an oversized anchor body. The most common solutions for low strength concrete are to increase the spiral confinement or change to an E anchorage.

3.4 VSL Anchorage Type L

The L anchorage is an inexpensive loop which can be used as a fixed anchorage for pairs of tendons (See Fig. 3.6). While there are many possible applications, the L anchorage is frequently used for vertical tendons in tank walls. It is generally suitable for any surface structure (shells and plates). Overlapping loops can be used at construction joints.

The L anchorage looks like a simple 180° tendon curve, but it uses a radius of

curve which is much smaller than Rmin, the standard minimum radius of curvature. To do this safely, the L anchorage application utilizes a number of special features. The basic problem is that due to the tendon curvature, there is an inplane force (deviation force) P/R and an out-of-plane force (bundle flattening effect) of approximately P/(4R). These forces, which are associated with tendon curvature, will be discussed in detail in Section 4.4. For an understanding of the L anchorage, it is sufficient to appreciate that these forces exist.

The basic reinforcement requirements for the L anchorage are given in Fig. 3.6. The linear bearing zone is "confined" by the compression struts which react against the splitting reinforcement. In going from the statical model to the de

11

tailed reinforcement requirements a steel stress of 250 MPa has been assumed. The reinforcement is detailed so that the hair pin bars resist the bursting forces associated with the dispersion of the force across the full thickness of the wall. These bars also tie back at least onequarter of the in-plane force to prevent cracking behind the duct.

The radii for various L anchorages have been determined by test and experience, but here are some practical considerations:

1. One must be able to bend the strand to the required radius. Bending strand to a radius smaller than about 0.6 m requires special techniques.

2. One must be able to bend the duct to the required radius. The duct wall should not buckle, but more important, the duct should still be leak tight to prevent cement paste from entering the duct during concreting. Special ducts and special bending techniques are often required.

3. The bearing stress of the duct against the concrete should be limited to an acceptable value. A bearing stress of $2 * f'_{ci}$ is reasonable. In the in-plane direction, in-plane confinement is provided by the adjacent contiguous concrete. In the out-of-plane direction, the reinforcement provides out-of plane compression stresses of onequarter to one-half the bearing stress. Thus the confined bearing capacity of the concrete in this instance is at least twice as great as the unconfined compression strength.

4. The contact force of the strand on the duct should be limited to an acceptable value to prevent significant reductions in strand tensile strength.

In a multistrand tendon the strands on the inside of the curve experience pressure from over-lying strands. For example, with two layers of strand, the strand on the inside of the curve has a contact force against the duct of 2* Ps/R, where Ps is the tension force in each strand. One can define the cable factor K as the ratio of the contact force for the worst strand to the average (nominal) contact force per strand. For the case of two orderly layers of strand, K=2. With multistrand tendons in round ducts the strands usually have a more random packing arrangement making it harder to determine maximum strand contact force. For standard L anchorages which utilize ducts one size larger
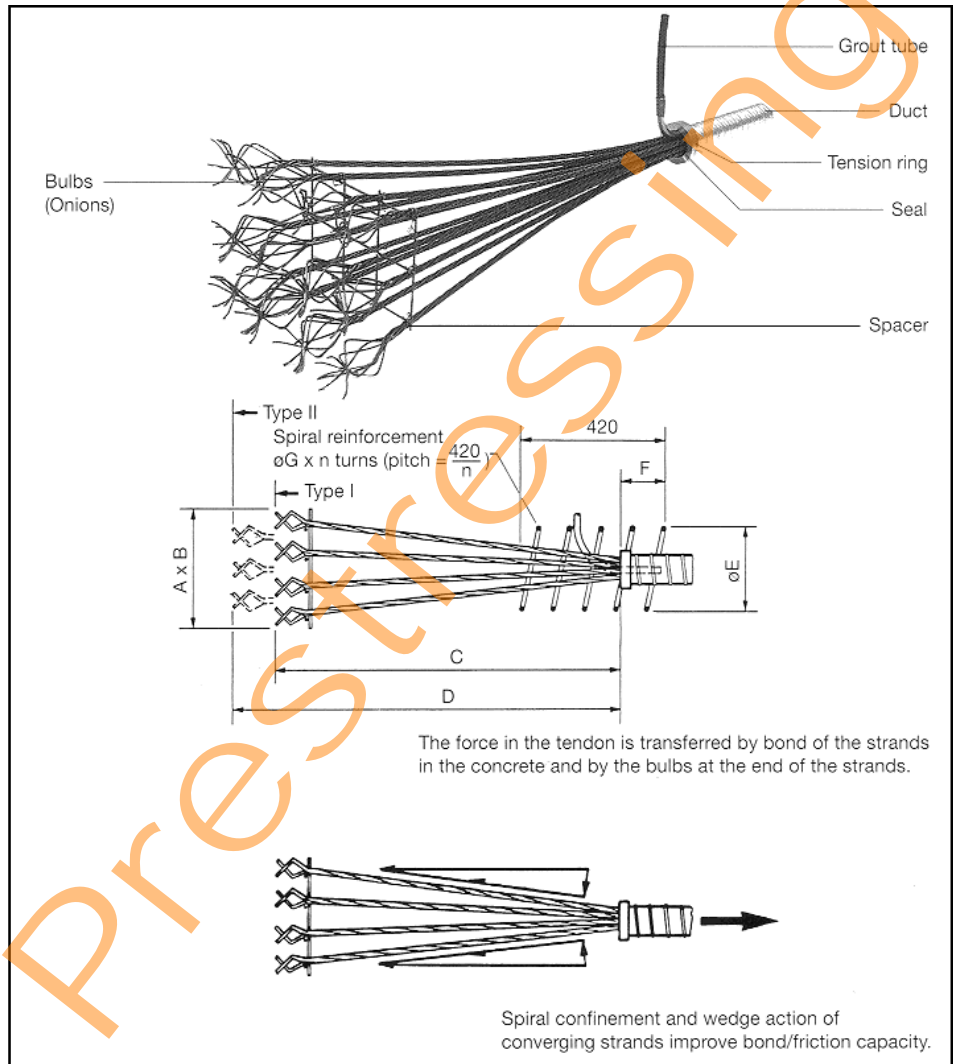

Figure 3.7: The VSL Type H anchorage is an economical fixed anchorage.

than normal, K ≈ 1 + (n/5) where n is the number of strands in the duct. This was determined by drawing different tendons with various random packed strand arrangements. The arrangements were then analyzed to determine strand contact forces. The simple equation for K was determined from a plot of K vs n. The predictions for K produced by Oertle [23] are more conservative than necessary for tendons with less than 55 strands. Oertle's analysis was based on different duct diameters than the VSL analysis.

Having addressed the method of estimating the maximum strand contact force we can return to the question of what an acceptable force might be. It depends on several factors including; strand size and grade, duct material hardness

and ductsurface profile (corrugated or smooth). For corrugated mild steel duct with "Super" strand, 700 kN/m for 0.5" Ø strand and 800 kN/m for 0.6" Ø strand are proposed. These are provisional design values with strand stresses of 95% GUTS. Higher contact stresses are likely acceptable, but this would be an extrapolation beyond the range of currently available test data.

As one final qualification, the proposed values are for situations where there is little movement of the strand relative to the duct. Tendons utilizing L anchorages generally have simultaneous stressing at both ends. With the L anchorage at the mid point of tendon, the strands in the critical zone are approximately stationary.

The standard L anchorage as presented above is just one specific solution. Other ducts and reinforcement details are possible. For example, an oval or flat duct can be used to produce a wider and flatter strand bundle. This reduces concrete bearing stresses, strand contact forces, and out-of-plane bundle flattening forces.

3.5 VSL Anchorage Type H

The H anchorage is an economical fixed end anchorage suitable for any number of strands. As seen in Fig. 3.7, it transfers the force from the tendon by a combination of bond and mechanical anchorage. The H anchorage can be used in almost any type of structure. It provides a "soft" introduction of the force into the concrete over a large zone rather than a "hard" introduction of force in a concentrated zone. The H is also a ductile anchor by virtue of the fact that when splitting is prevented, bond slip is a very plastic phenomenon. In addition, the mechanically formed onion at the end of the strand provides a "hidden reserve" strength which adds to the capacity.

The bond component of the load transfer capacity is somewhat different from the problem of force transfer in pretensioned members. The bond and friction forces are enhanced by the wedge effect of the strands as they converge into the duct. The spiral also improves the reliable capacity of the H anchorage by controlling splitting. Bond stresses of 0.14 to 0.17 f'ci can be developed in an H anchorage; the higher value being the peak stress and the lower value being the residual stress obtained after slippage. A design bond stress value of 0.15 f'ci may be used for H anchorages since they are generally rather short and have a significant portion of the bond length mobilizing the peak bond stress. See Fig. 3.8.

The onion contributes additional bond capacity (bond on the surface of seven individual wires) and mechanical resistance. The bond resistance of the onion is substantial because the seven individual wires have a surface area about 2.3 times as large as the strand surface area. The mechanical resistance as shown in Fig. 3.9, is provided by the bending and straightening of the individual wires as they pass through a curved section.
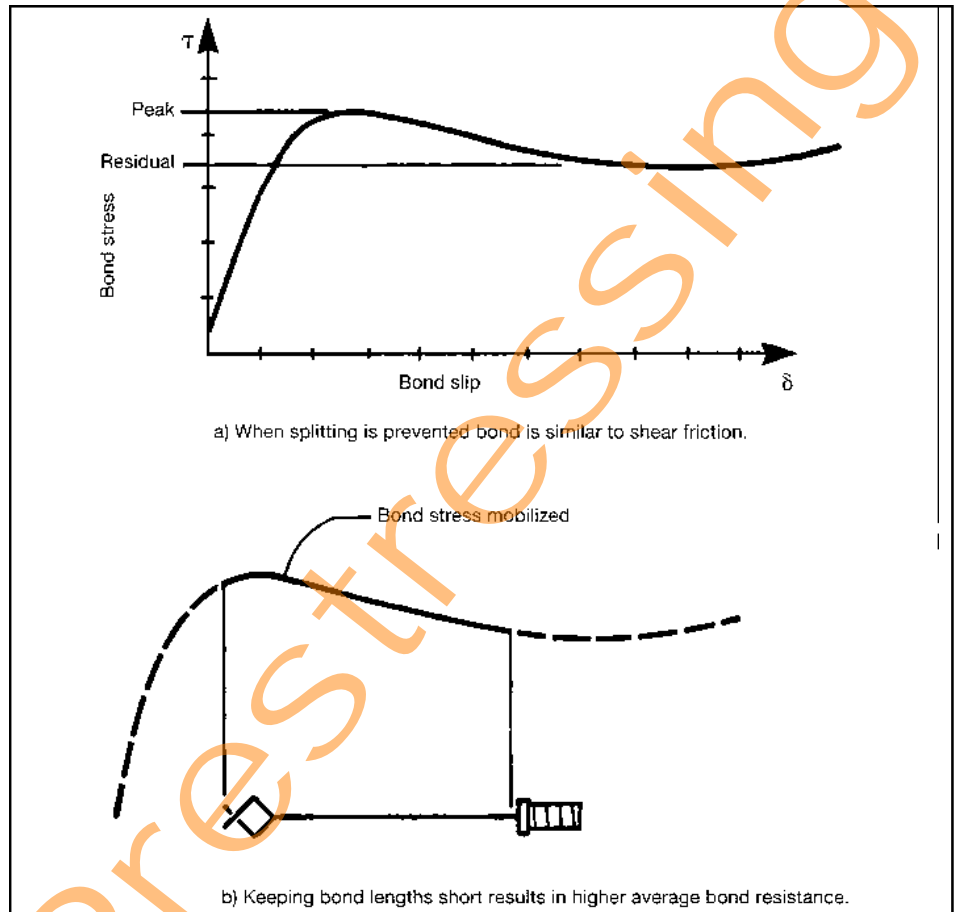

Figure 3.8: When splitting is prevented bond is a ductile phenomenon.

For typical strand, the mechanical resistance alone will develop 12 % of the tensile strength of the strand. (This considers only the single major bend in each wire as the other bends have much larger radii thereby rendering their contribution to mechanical resistance insignificant.) For concrete with f'ci = 24 MPa, the bond resistance of the individual wires in the onion will develop about 31 % of the tensile strength of the strand. Thus, for this concrete strength the onion (bond plus mechanical resistance) will develop 43 of the tensile strength of the strand. The minimum additional length of "straight" bonded strand required for full strand development can be readily determined.

The overall bonded length of an H anchorage is often greater than the minimum because of constructional considerations. First, the onions must be spread into an array, as shown in Fig.3.7, so that they do not interfere with each other. A maximum strand deviation angle of about 10° or 12° is suitable as it gives manageable deviation forces. The spiral and tension ring (at the end of the duct) are designed to resist these deviation forces. A greater maximum strand deviation angle will shorten the distance between the onion and the tension ring but will increase spiral requirements.

A great many onion array configurations are possible. For example in thin members such as slabs, a flat array can be used. In such cases the in-plane resistance of the slab is used to deal with the deviation forces thus eliminating the need for spiral reinforcement.

Strand elongation within the H anchorage should be included in the tendon elongation calculation. This elongation can be approximately accounted for by including half the length of the H anchorage in the tendon length. The actual
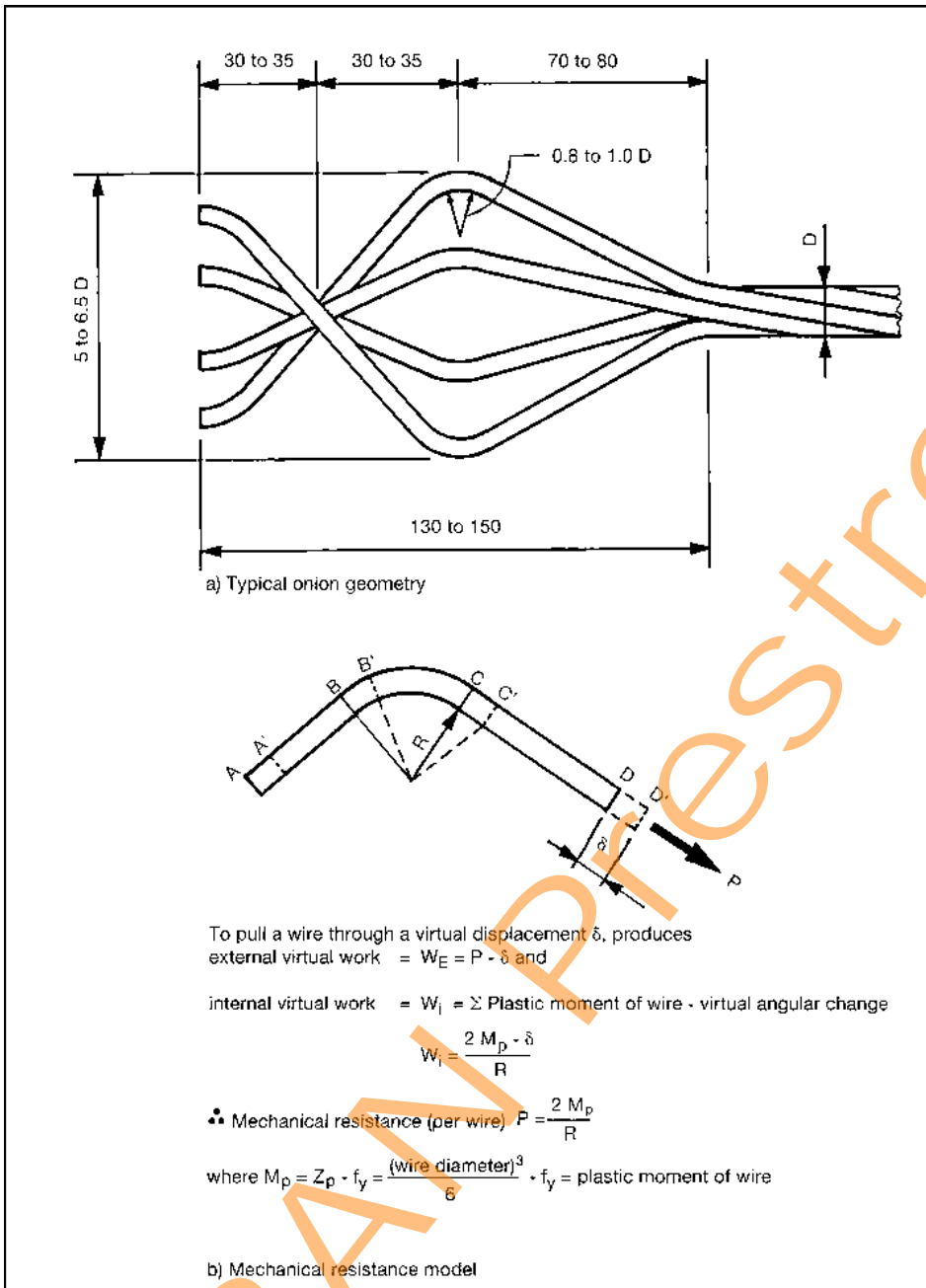
a) Typical onion geometry

To pull a wire through a virtual displacement δ, produces

external virtual work $= W_E = P \cdot \delta$ and

internal virtual work $= W_i = \Sigma$ Plastic moment of wire · virtual angular change

$$W_i = \frac{2\,M_p \cdot \delta}{R}$$

∴ Mechanical resistance (per wire) $P = \dfrac{2\,M_p}{R}$

where $M_p = Z_p \cdot f_y = \dfrac{(\text{wire diameter})^3}{6} \cdot f_y = $ plastic moment of wire

b) Mechanical resistance model

Figure 3.9: Mechanical resistance supplements bond resistance in an H anchorage.

a) Typical H anchorage (H 5 – 4 and H 6 – 4).

b) Range in force transfer for various bond stresses.

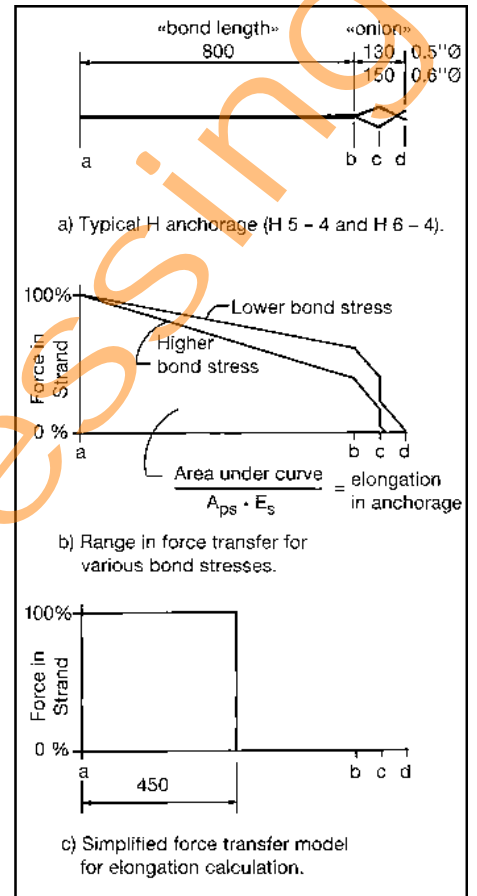c) Simplified force transfer model for elongation calculation.

Figure 3.10: About half the length of the H anchorage should be included in the tendon elongation calculation.

elongation will depend upon the quality of bond at the time of stressing as shown in Fig. 3.10 (b). With good bond, most of the force is transferred from the strand before the onion is reached. This will result in less elongation. Generally elongation variations are insignificant. For short tendons, these variations of say 5 mm can exceed the normal 5 % variation allowance. For such short tendons the force is more readily verified by jack force/pressure readings than by elongation measurements. Using the jacking pressure as a guide automatically corrects for any additional unexpected elongation due to initial bond slip in the H anchorage.

As a final comment, for purposes of general zone design, one may assume that 60 % of the tendon force acts at the mid point of the straight bond length while the other 40 % acts at the onion. This will result in a worst case estimate of bursting stresses.

a) VSL Type P dead end anchorage.

b) VSL Type U dead end anchorage.

c) VSL Type SO anchorage for use with flat duct system.

d) VSL Coupler Type SK for use with flat duct system.

e)VSL Stressing anchorage Type S-6 for use with unbonded monostrands.

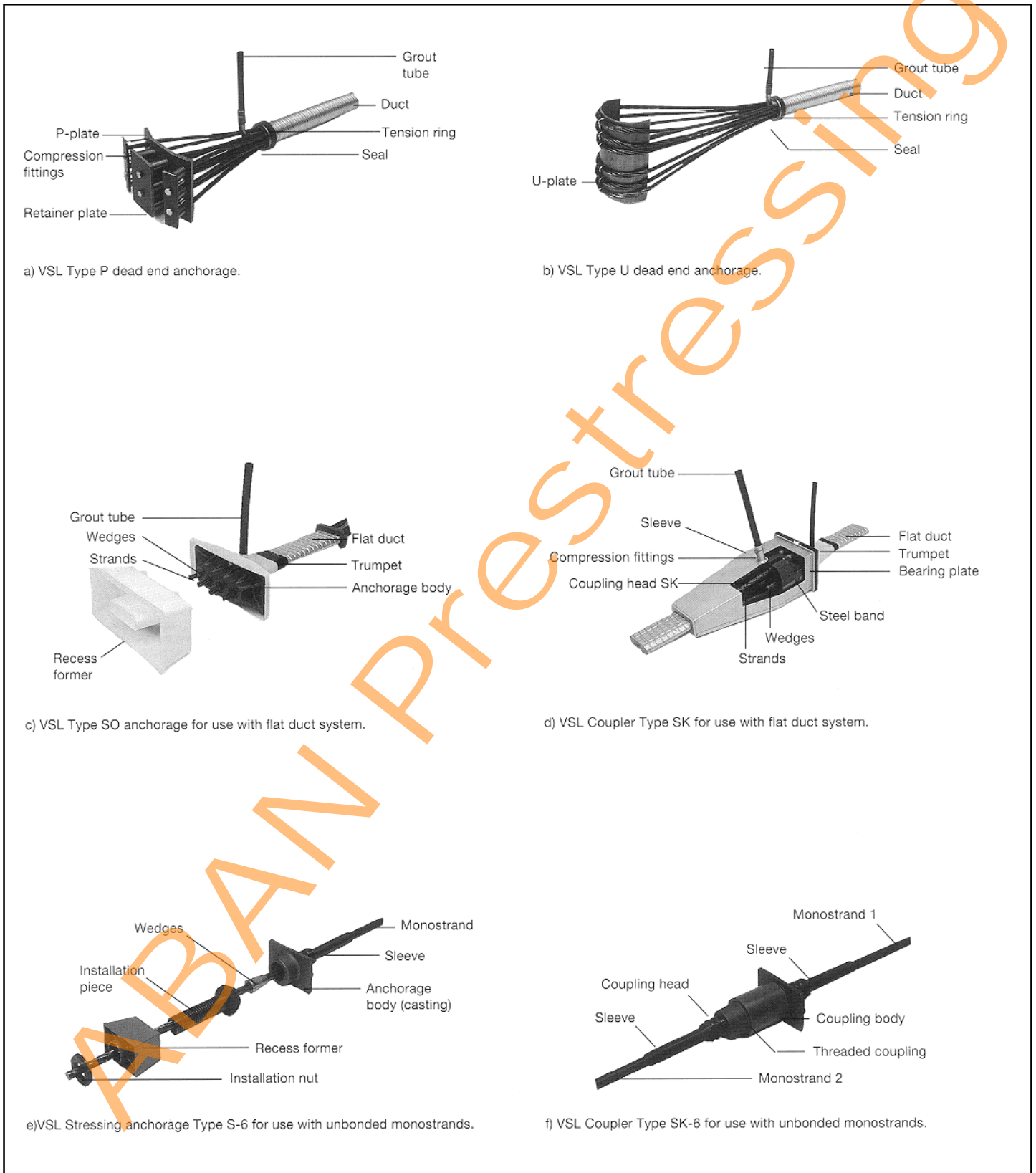f) VSL Coupler Type SK-6 for use with unbonded monostrands.

Figure 3.11: These are just a few of the many other special anchorages available from VSL. Your local VSL representative should be contacted to determine which anchors are available in your area.

4. General Zone Design

4.1 Single End Anchorages

The subject of general zone design is introduced through a discussion of single end anchorages.

The concentrated anchorage force must be dispersed or spread out over the entire cross-section of the member. In accordance with St. Venant's principle, the length of the dispersion zone or "D- region" is approximately equal to the width or depth of the member. As the anchor force fans out, a bursting force (tension) is produced perpendicular to the tendon axis. This bursting force is a primary tension force which is required for equilibrium. Spalling forces may also be produced which cause dead zones of concrete (usually corner regions not resisting primary compression forces) to crack. Spalling forces are secondary compatibility induced tensile forces. The differential strains between the "unstressed" dead zones and the highly stressed active concrete zones produce the spalling force. If compatibility is reduced by cracking the spalling forces are reduced or eliminated. The general design approach in this document is to use strut-and-tie models to deal with the primary forces, and to use other simple methods for the secondary spalling forces.

While the general use of strut-and-tie models has been covered by others [16, 17, 18], it is still worthwile highlighting some of the unique details associated with the application of these models to post-tensioned structures. There must be consistency between the local and general zone design models. For example, E, L and H anchorages introduce forces into the general zone in quite different manners. The model should include at least the entire D-region.

It is usually sufficient to replace the anchorage with two statically equivalent forces (P/2 acting at the quarter points of the anchorage). When the bearing plate is small relative to the depth of the member, only one statically equivalent force (P acting at the center of the anchorage) is necessary. The (Bernoulli) stress distribution at the member end of the Dregion should be replaced by at least two statically equivalent forces. Figure 4.1 illustrates these points. Model (a) is too simple and does not give any indication of the bursting forces. Model (d) gives correct results but is needlessly complex..
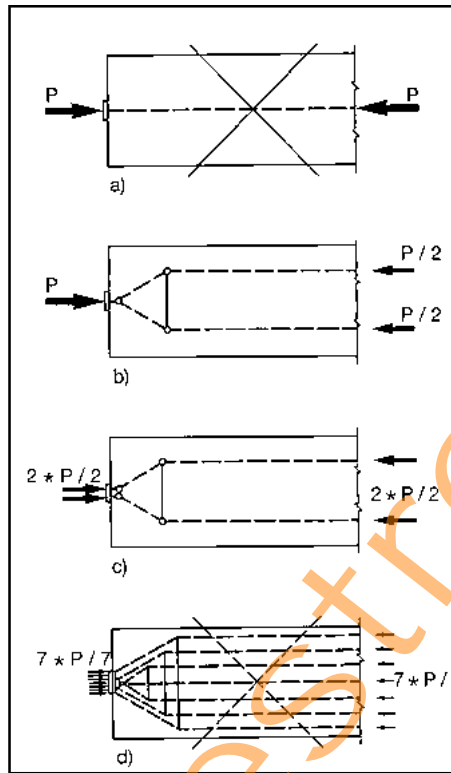

Figure 4.1: A good model is easy to use and correctly identifies the primary flow of forces in a structure.

Models (b) and (c) are both satisfactory for design. Model (b) ignores the fact that the bearing plate disperses the force over the height of the local zone and hence will result in a more conservative estimate of the primary bursting force. The appropriate choice of strut inclination O and the location of the tension tie vary. They depend upon a/d, the ratio of bearing plate height to member depth, and e, the smaller of the two edge distances when the anchorage is not at mid-depth. Figure 4.2 presents a parametric study of a simple end block and compares various strut-and-tie models with elastic theory. In part (a) of the figure, each row is a family of models with the same a/d, while each column is a family of models with the same Θ Models with the same Θ have the same primary bursting tension tie force T as shown at the top of each column. The elastic distribution of bursting tension stress is plotted in the models on the main diagonal (top left to lower right) along with Te, the total elastic bursting stress resultant force. A practical observation is that as a/d increases, the primary bursting tension force decreases

and moves further from the anchorage. Any strut-and-tie model which is "reasonably close" to those shown along the main diagonal will suffice for practical design. "Reasonably close" is a relative term, but Θ values within 20 % of those shown produce acceptable results. Figure 4.2 (b) may be used as a general guide, but it should not be used blindly without giving consideration to all relevant factors. For example "a" may be taken as the corresponding dimension of the confined local zone, and the first nodes (intersection of strut forces) may be taken at the mid-length of the local zone. Models should always be drawn to scale. If the model does not look right, it probably isn't!

The presence of a support reaction should not be overlooked. As shown in Fig. 4.3 for a variety of cases, supports can significantly alter the magnitude and location of primary forces. Three dimensional models should be used when the force must be dispersed across the width and over the height of a member. As shown in Fig. 4.4, this is particularly important for flanged sections.

Proportioning of supplementary spalling reinforcement in the dead zones may be done with strut-and-tie models as shown in the upper portion of Fig. 4.5 which is based on Schlaich's "stress whirl" [16]. The objective is to find a model which fills the dead zone and assigns appropriate forces which are self equilibrating. While it is easy to get self equilibrating forces, their magnitude is very sensitive to the model geometry. The development of a reasonable stress whirl is impractical for most applications. Alternatively, the spalling zone reinforcement may be designed for a force equal to 0.02 * P as shown in the lower portion of Fig. 4.5. This represents the upper limit for the maximum spalling forces (based on elastic analysis) reported by Leonhardt [24] for a wide variety of cases. As a practical matter the normal minimum reinforcement of 0.2 % to 0.3 of the concrete area (for buildings and bridges respectively) provided in each direction is usually more than sufficient.

Reinforcement must be statically equivalent to the tie forces in the strutand-tie model. That is, the centroids and inclination must be similar. It is possible to provide suitable orthogonal reinforcement when the tie is inclined, but this is not discussed in this document. Hence,
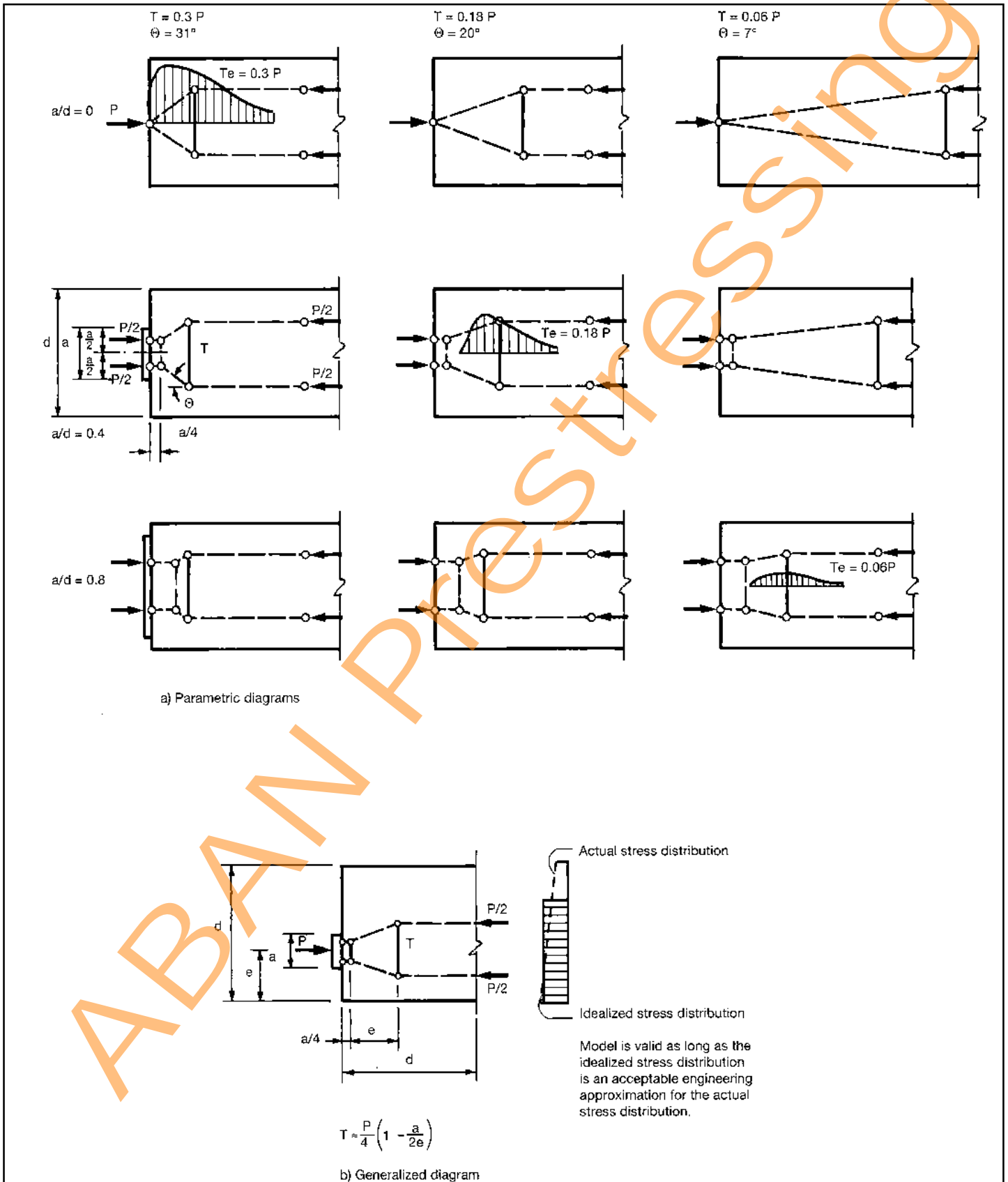
Figure 4.2: A range of acceptable Strut-and-Tie models provide flexibility in the amount and position of the renforcement.
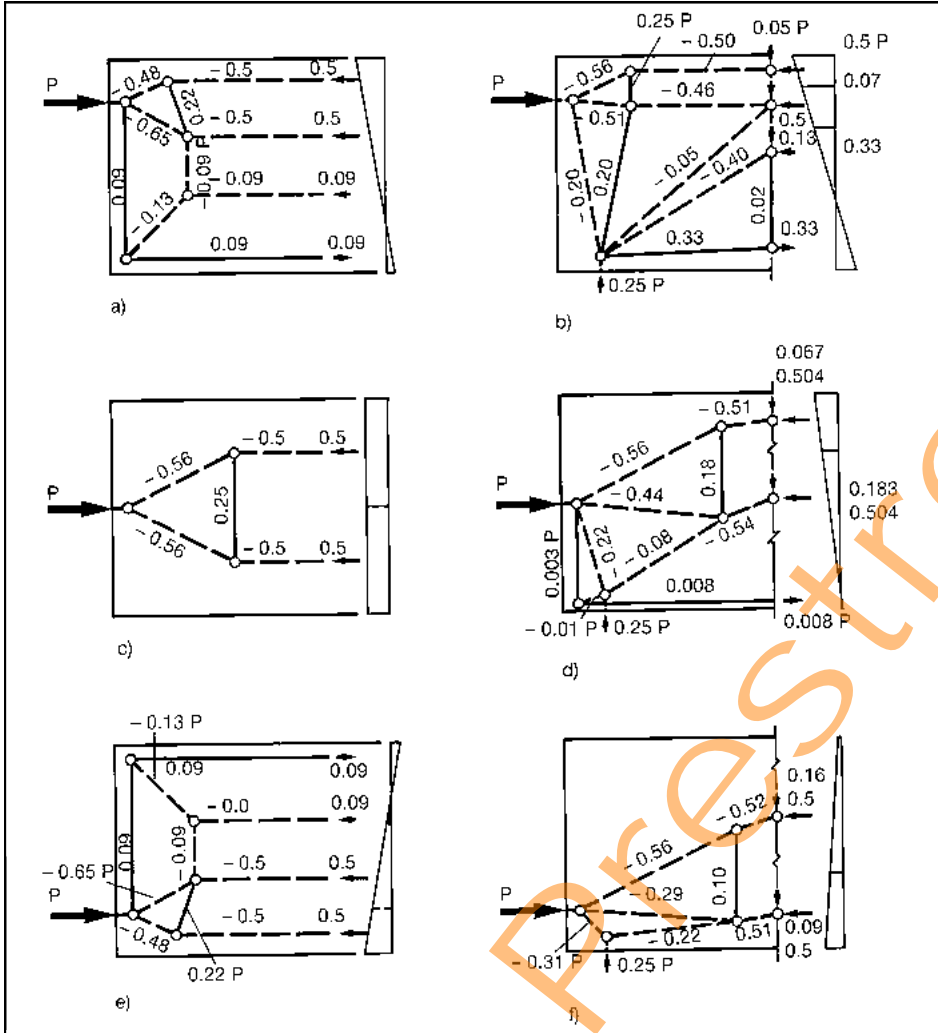
Figure 4.3: The presence of a support reaction significantly alters the stress distribution in a member.
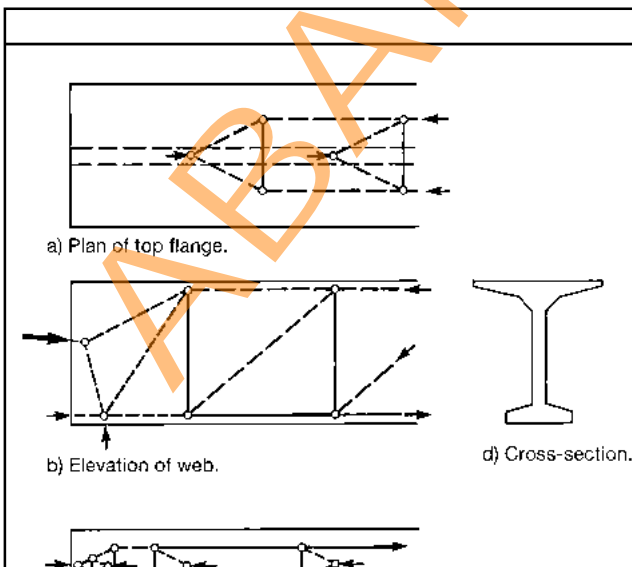

Figure 4.4: Three dimensional models should be utilized for analyzing flanged sections.
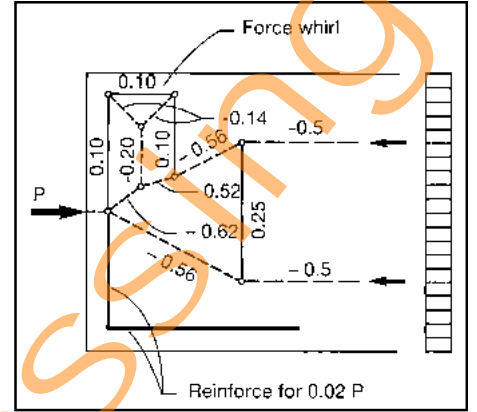

Figure 4.5: Dead zones require reinforcement to control compatibility induced cracking.

when developing strut-and-tie models, one should use ties with the same inclination as the desired reinforcement pattern. Normally several bars are used to provide reinforcement over a zone centred on the tie and extending half way to the nearest parallel edge, tension tie or compression strut. The reinforcement (stirrups) should be extended to the edges of the concrete and properly anchored.

The reinforcement must provide the required force at an appropriate steel stress. Under ultimate load conditions (95 % GUTS), the reinforcement may be taken to just below yield (say 95 % of yield as discussed in section 2.3). Under service conditions (70 % GUTS), the reinforcement stress should be limited to about 250 MPa in order to control cracking. Obviously, serviceability will govern reinforcement with yield strengths greater than about 380 M Pa.

Concrete strut stresses can be checked but this will not normally govern since the concrete stresses at the end of the local zone/beginning of the general zone are controlled by the local zone design to acceptable values.

4.2 Multiple End Anchorages

Multiple end anchorages can involve one group of closely spaced anchors or two or more groups (or anchors) widely spaced. These two cases can involve quite different reinforcement requirements.

A single group of closely spaced anchorages can be treated as one equivalent large anchorage. The discussion of Section 4.1 is thus generally applicable to this situation. As shown in Fig. 4.6 the influence zone of the group may be much larger than the sum of the individual local zones, hence concrete stresses should be checked. Using the principles of Chapter 3, if one large anchor is used instead of several smaller anchors, the entire group influence zone would be treated as a local zone which may require confinement reinforcement. For situations similar to Fig. 4.6, the force may begin to spread out from the individual local zones thus significantly reducing the concrete stresses. This effect is less pronounced for the anchors at the interior of the group hence there may be a need to extend the local zone reinforcement for the interior anchors. Alternatively, one can increase the anchor spacing, increase the concrete strength, or provide compression reinforcement. When checking ultimate strength, some judgment is required in determining the loads since it is highly unlikely that all anchorages within a group will be overloaded. For an accidental stressing overload, it would be reasonable to take one anchorage at 95 % GUTS with the remainder at 80 % GUTS.

With a group of closely spaced anchors, individual spirals can be replaced with an equivalent orthogonal grid of bars. With an array of anchorages, the interior will receive sufficient confinement from the perimeter anchorages provided that perimeter anchorages are suitably confined and tied together across the group. Rationalizing the local zone reinforcement for anchor groups can simplify construction.

When two or more groups of anchorages are widely spaced at the end of a member, the behavior is often more like that of a deep beam. The distributed stresses at the member end of the Dregion serve as "loads" while the anchorage forces serve as "support reactions".
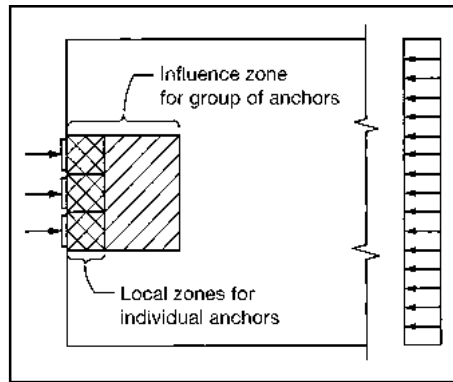

Figure 4.6: Concrete stresses should be checked when anchors are closely spaced.

Figure 4.7 shows a typical case with two anchorages. In this instance, the primary bursting force is located near the end face of the member. Other situations can be designed readily with strut-andtie models.
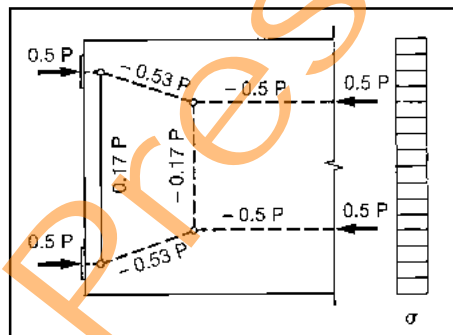

Figure 4.7: With widely spaced anchorages the member end can be designed like a deep beam.

4.3 Interior Anchorages

Interior anchorages are those located along the length of a member rather than in the end face. While the general principles used for end anchorages also apply to interior anchorages, there are some subtle differences. Podolny [25] discusses several cases where problems resulted at interior anchorages when those differences were overlooked.

The stressing pocket shown in Fig. 4.8 will be used to facilitate the discussion of compatibility cracking behind the anchorage. The primary difference between end

anchorages and interior anchorages is the cracking induced behind the anchorage as a result of local concrete deformation in front of the anchorage. See Fig. 4.9. The traditional approach to overcoming this problem is to provide ordinary non-prestressed reinforcement to "anchor back" a portion of the anchor force. Early design recommendations [24] suggested that the force anchored back should be at least P/2. An elastic analysis which assumes equal concrete stiffness in front of and behind the anchorage would support such a recommendation. Experience and some experts [26] suggest that anchoring back a force of about P/4 is sufficient. With cracking, the stiffness of the tension zone behind the anchorage becomes less than the stiffness of the compression zone in front of the anchorage, thereby reducing the force to be tied back. For typical permissible stresses in ordinary non-prestressed reinforcement and strand, anchoring back P/4 would require an area of nonprestressed reinforcement equal to the area of prestressed reinforcement being anchored. Figure 4.10 suggests a strutand-tie model for detailing the non-prestressed reinforcement. While not normally done, it is obvious that one could design the zone in front of the anchorage for a reduced compression force (reduced by the force anchored back by non-prestressed reinforcement).

An alternative pragmatic solution to the problem is to provide a certain minimum amount of well detailed distributed reinforcement which will control the cracking by ensuring that several fine distributed cracks occur rather than a single large isolated crack. A minimum reinforcement ratio of 0.6 % (ordinary non-prestressed reinforcement) will normally suffice.

Regardless of which approach one uses, one should consider where or how the anchor is used. For example, an anchorage in a bridge deck which may be exposed to salts would merit a more conservative design than the same anchor used in a girder web, bottom flange, or building where there is a less severe exposure. As a final comment, if there is compression behind the anchor (eg. prestress due to other anchorages) it would reduce the anchor force which needs to be anchored back. Conversely, if there is tension present it would increase the force which should be anchored back.
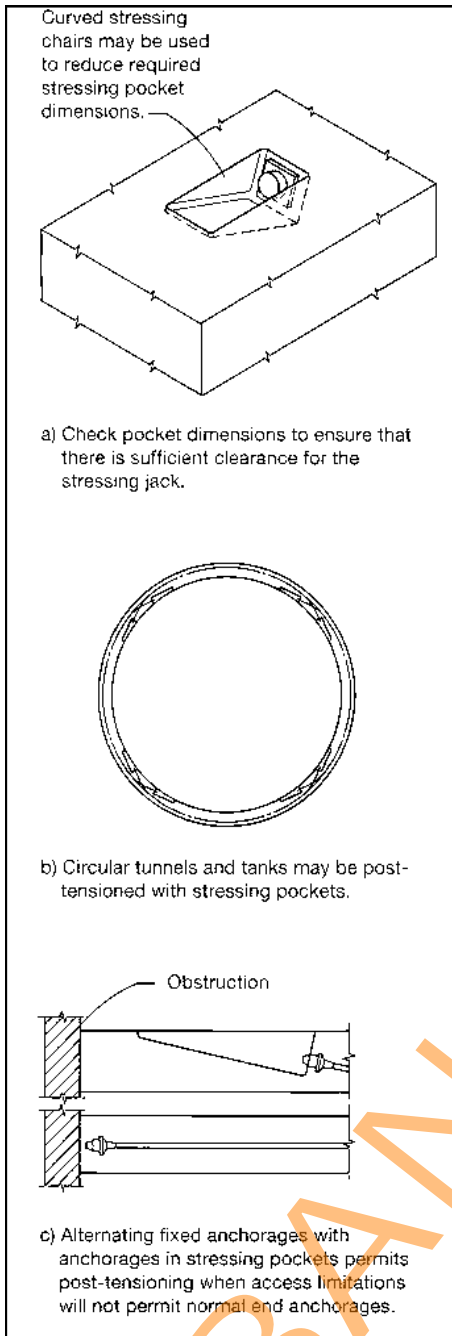
Curved stressing chairs may be used to reduce required stressing pocket dimensions.

a) Check pocket dimensions to ensure that there is sufficient clearance for the stressing jack.

b) Circular tunnels and tanks may be post-tensioned with stressing pockets.

Obstruction

c) Alternating fixed anchorages with anchorages in stressing pockets permits post-tensioning when access limitations will not permit normal end anchorages.

Figure 4.8: Stressing pockets can be used when it is undesirable or impossible to use anchorages in the end face of a member.

The difference in deformation can result in tension cracks

P

Figure 4.9: Local deformation in front of the anchorage produces tension behind the anchorage.

Figure 4.10: Simple detailing rules may be developed from more complex Strutand-Tie models.

Detailed considerations specific to stressing pockets, buttresses, blisters and other intermediate anchorages will be discussed in turn.

Stressing pocket dimensions should be selected so that there is adequate clearance for installation of the tendon and anchorage, installation of the stressing jack, post-tensioning, and removal of the jack. A curved stressing chair can often be used to reduce the necessary dimensions. The additional friction losses in the chair must be taken into account in the design. If the tendon deviates (curves) into the pocket the resulting deviation forces as discussed in Section 4.4 must be addressed. As a

P / 2
P / 8
P / 2
P / 2
P / 8
P / 2

a) Trimmer bars require proper anchorage.

Nominal trimmer bars

Total area of longitudinal bars to equal area of prestressing steel being anchored at pocket

Recessed stressing pocket or full blockout

Note: Local zone and other general zone reinforcement omitted for clarity only.

P_o

B

l_d

$\approx l_d$

$\approx B + l_d$

b) Provide top and bottom trimmer bars around
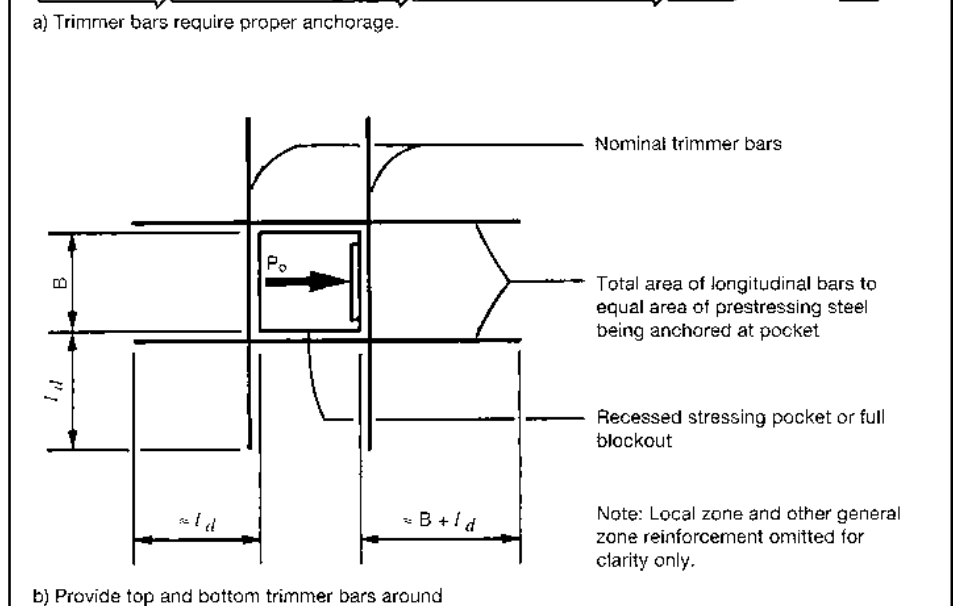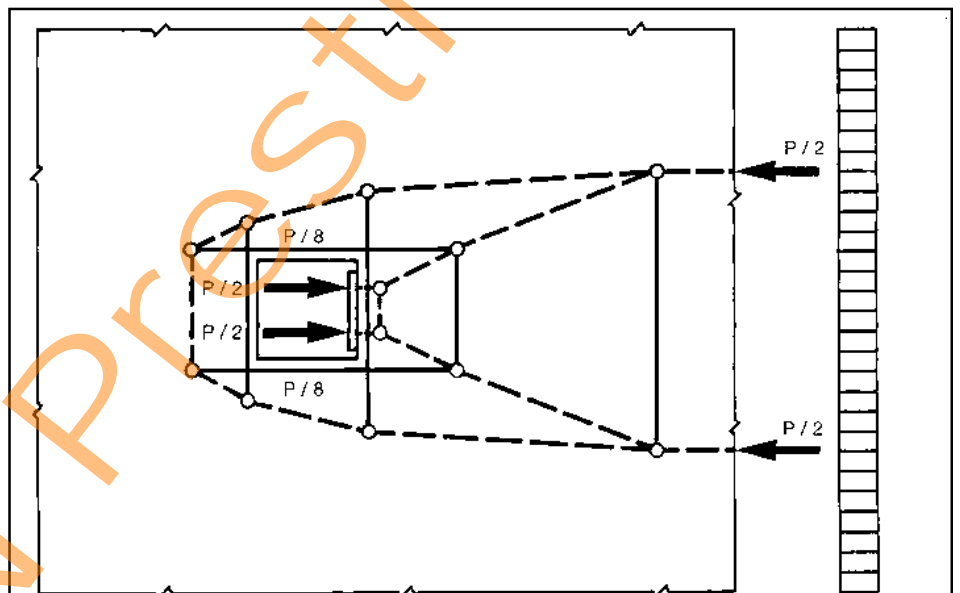
variety of stressing jacks and techniques are available, VSL representatives should be contacted for additional project specific details related to stressing pockets.

Detailing of the stressing pocket itself deserves careful consideration. Sharp corners act as stress raisers and should be avoided whenever possible. Corners should be provided with fillets or chamfers to reduce the stress concentrations and cracking associated with the geometric discontinuities. After stressing, pockets are usually filled with grout or mortar to provide corrosion protection for the anchorage. Poor mortar results at thin feathered edges, hence, pockets should be provided with shoulders at least 40 mm deep. The mortar should be anchored into the pocket. The methods may include: use of a bonding agent; providing the pocket with geometry (shear keys) which locks the mortar in place; not cutting off all of the strand tail in the pocket so that mortar may bond to and grip the strand tail; or use of reinforcement which is embedded in the concrete mass and temporarily bent out of the way until after stressing of the tendon.

Finally, it is practical to anchor only one or two multistrand tendons in a stressing pocket. (For monostrand tendons, 4 strands can readily be anchored in a single pocket.) If more than two multistrand tendons must be anchored at a specific location, a buttress or other form of interior anchorage should be considered.

Buttresses are often used in circular structures to anchor several tendons in a line along a common meridian. A typical storage tank application is shown in Fig. 4.11. With buttresses, the tendons need not and usually do not extend over the entire circumference of the structure. The tendons are lapped at a buttress and the laps of adjacent hoop tendons are staggered at adjacent buttresses. Staggering of the laps provides a more uniform stress condition around the tank. With stressing from both ends, and selection of suitable tendon lengths (buttress locations), prestressing losses due to friction are minimized and an efficient design can be achieved.

Typical buttress design considerations are given in Fig. 4.12. In (a), the tendon profile and buttress geometry are selected. If the tank radius is small, and the
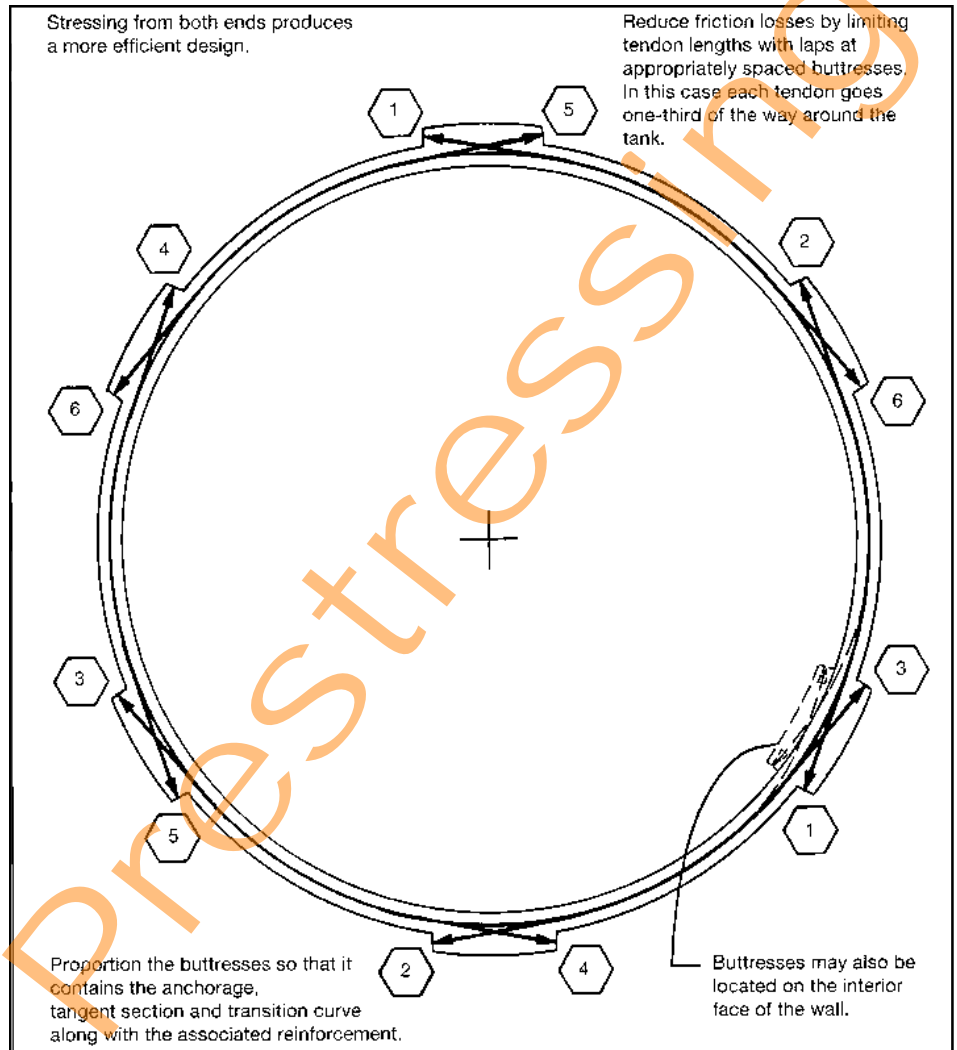

Figure 4.11: Buttresses provide flexibility which can be used to improve tendon layout and overall design efficiency

buttress is long enough, a reverse transition curve may not be required. Avoidance of the transition curve reduces the requirements for transverse tension ties but usually requires a larger buttress. This is an economic trade-off which should be considered in each project. When the wall is composed of precast segments post-tensioned together, the maximum permissible weight of the buttress panels is often restricted by the available crane capacity. In such cases it is advantageous to make the buttresses as small as possible.

The forces resulting from the posttensioning are readily determined as shown in Fig. 4.12 (b). In addition to the anchor forces, outward acting distributed forces are produced by the tendon in the transition curve zones. Tendon regions with the typical circular profile produce an inward acting distributed force. The horizontal hoop tendons are usually placed in the outer half of the wall. This prevents the majority of the force from compressing a thin inner concrete ring, and reduces the possibility of tendon tear-out or wall splitting along the plane of the tendons. This also accommodates vertical post-tensioning tendons in the center of the wall which are usually used to improve vertical flexural performance of the wall.
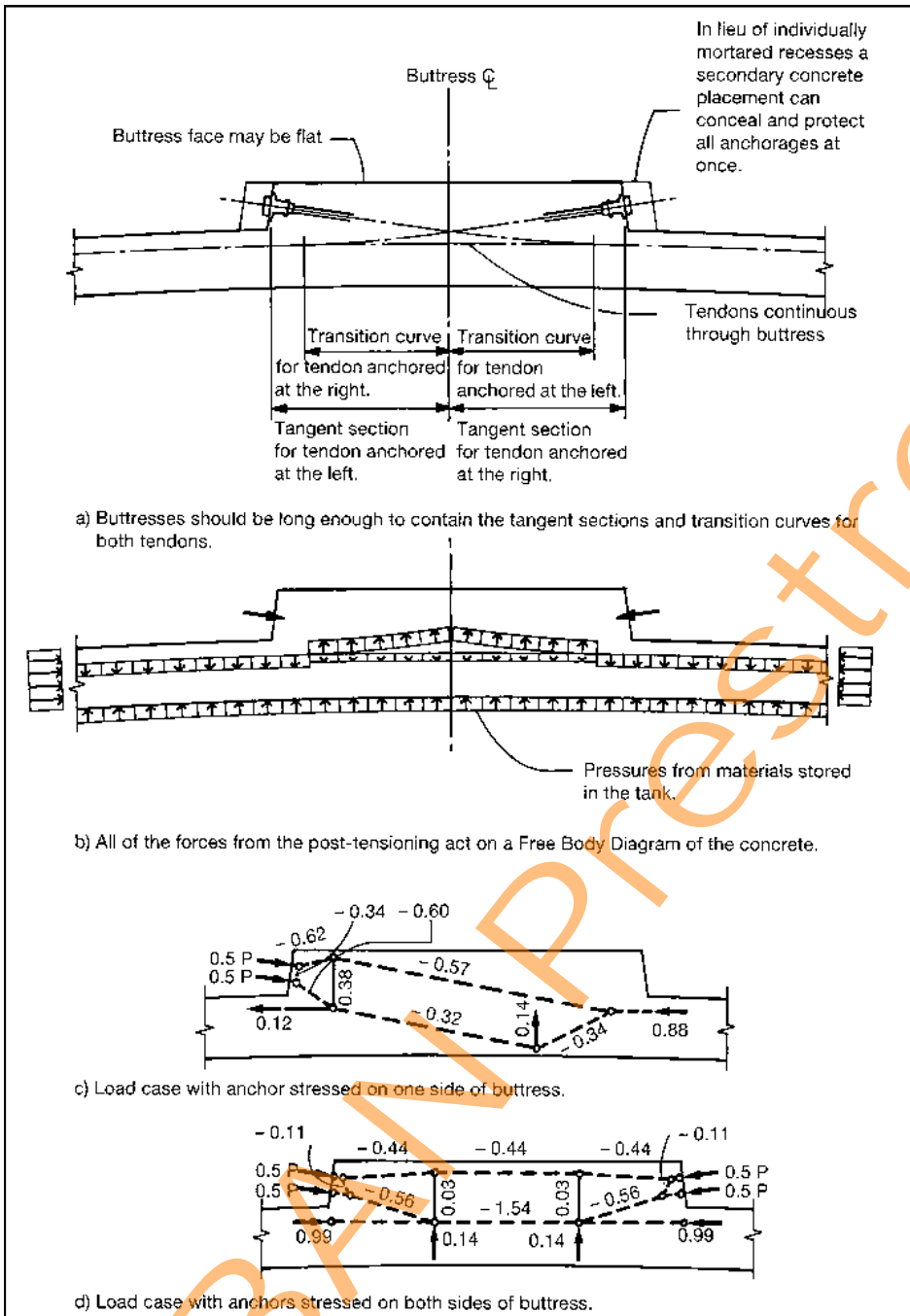
Figure 4.12: Stressing sequence should be considered in buttress design.

It is clear from Fig. 4.12 (b) that transverse ties are required in the central portion of the buttress to prevent the outward acting radial pressures from splitting the wall. What is not clear is that, depending upon the stressing sequence, transverse ties may or may not be required at the ends of the buttress. If the tendons are stressed sequentially around the tank, the load case presented in

Fig. 4.12 (c) occurs during stressing of the first of the complementary pair of anchorages at a buttress. Transverse ties are required at the anchorage which is stressed first. As in the case of stressing pockets, tension stresses are created behind the anchor. Since buttresses are usually about twice as thick as the typical wall section, the local compressive deformations in front of the anchor which

give rise to tension behind the anchor are about half as significant. Bonded nonprestressed reinforcement should be provided to anchor back a force of P/8. While bonded prestressed reinforcement may also be used for this purpose, it is not usually bonded at the time this load case occurs, hence the need for supplemental reinforcement.

If the tendons are stressed simultaneously in complementary pairs, the load case presented in Fig. 4.12 (d) occurs. This is also the load case which occurs after sequential stressing the second anchorage. In this instance, extra transverse ties are not required by analysis. Due to the geometric discontinuity created at the ends of the buttress, nominal reinforcement is recommended to control cracking.

Anchorage blisters are another method frequently used to anchor individual tendons along the length of a member. Figure 1.2 illustrates a typical example of blisters in a box girder bridge where they are often utilized. To facilitate discussion, a soffit blister is assumed. The behavior of a blister is a combination of a buttress and a stressing pocket as previously described.

The choice of the blister position in the cross section is important to the design of the blister and the member as a whole. While some designers locate the blisters away from the girder webs, this is not particularly efficient and can lead to difficulties as reported by Podolny [25]. Locating the blisters at the junction of the web and flange produces a better design as this is the stiffest part of the cross section and the local discontinuity produced by a blister is of little consequence. It is also better to introduce the force close to the web where it can be readily coupled to the compression force in the other flange. For blisters located away from the web the shear lag, which occurs between the blister and the web, increases the distance along the axis of the member to the location where the force is effectively coupled to the compression in the other flange. Locating the blister at the web to flange junction provides benefits to the blister itself. The web and the flange provide confinement so that the blister has only two unconfined faces. The shear forces between the blister and the member as a whole act on two faces rather than just one. Finally, local blister moments about

the horizontal and vertical axes are to a large extent resisted by the web and flange.

In the design of a blister, it is of some advantage if one starts with the notion of a "prestressed banana" as shown in Fig. 4.13. This notional member is a curved prestressed member which contains the anchorage, a straight tangent section of tendon, and the transition curve. The concrete section of the notional member is symmetrical about the tendon. The prestressed banana which is self equilibrating can be notionally considered as a separate element which is embedded into the overall mem ber. Once the banana itself is designed, additional reinforcement is required to disperse the post-tensioning force over the cross section, and to control compatibility associated cracking (as in the
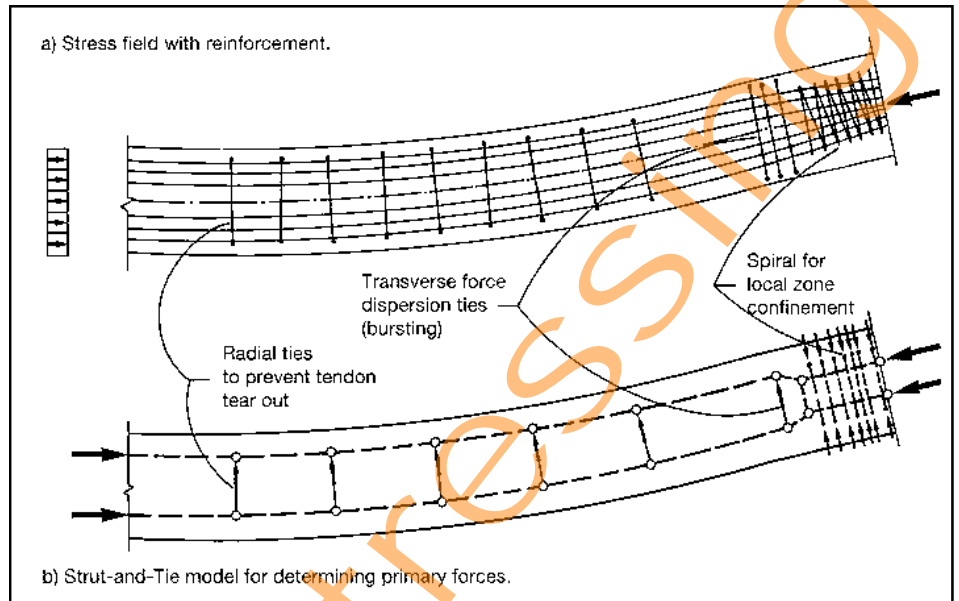
a) Stress field with reinforcement.

Transverse force dispersion ties (bursting)

Radial ties to prevent tendon tear out

Spiral for local zone confinement

b) Strut-and-Tie model for determining primary forces.

Figure 4.13: A blister is essentially a curved prestressed concrete «Banana» embedded in the main member.

Formwork simplifications can cause «pinching» of the blister

$\frac{3}{4}$ P

P

$\frac{P}{4}$

Strain compatibility results in tension behind the blister (tension is reduced by precompression from adjacent blister if they are already stressed).

Strain compatibility causes a deviation in the primary stress field resulting in a local force couple.

$T = C \approx \frac{P}{16}$

a) Strain compatibility between the blister and the main member alters the flow of forces.

$T/\sqrt{2}$ Web

Shaded portion carries axial force

$T/\sqrt{2}$

Bottom flange

Section A–A for blister at web to flange junction

Section A–A for blister away from web to flange junction

b) Blisters located away from the web to flange junction introduce local longitudinal and transverse bending moments in the flange.

c) Blisters located at the web to flange junction provide simple direct force transmission into the planes of the web and flange.

Figure 4.14: Reinforcement should be positioned to control strain compatibility induced cracking.
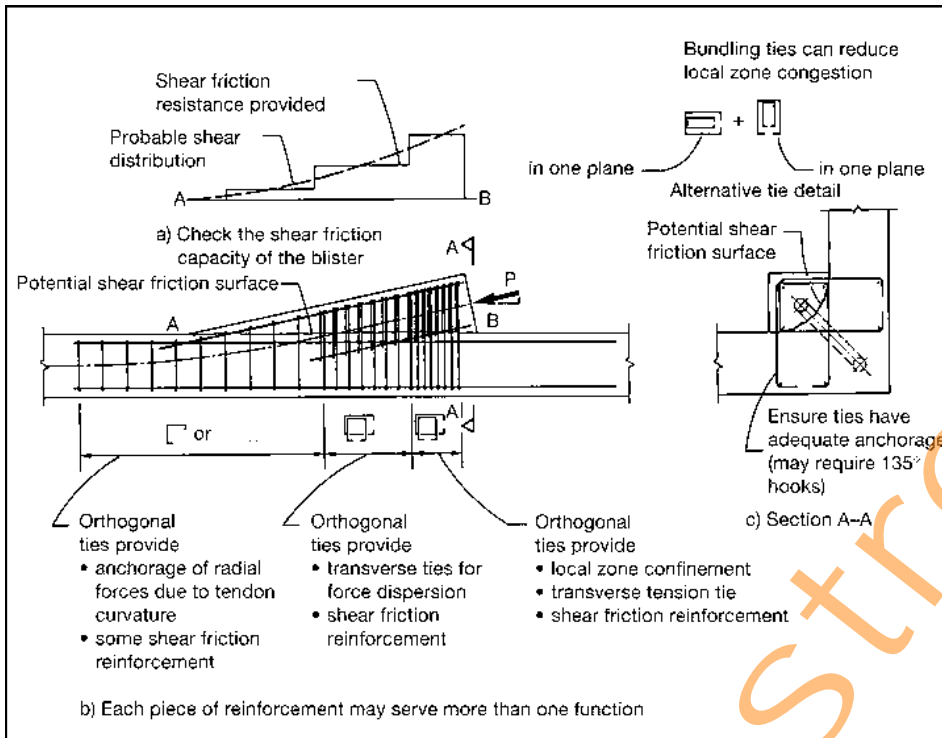
DETAILING FOR POST-TENSIONING

Figure 4.15: Rationalize the detailing to prevent needless superposition of reinforcement.

case of stressing pockets). The shear friction failure mechanism could be checked as additional insurance against the blister shearing off.

The design of the notional prestressed banana will now be discussed in some detail. The cross section of the banana should be approximately prismatic. The lateral dimensions should be at least equal to the minimum permissible anchor spacing (plus relevant concrete cover). Starting at the anchorage end of the banana, there is the usual local zone reinforcement requirement at the anchorage. This may consist of standard spirals, or rectangular ties which are anchored into the main member. At the transition curve, transverse ties are required to maximize the depth of the curved compression zone. The radial ties need not be designed to resist the entire radial force produced by the curved tendon since a portion of the radial force is resisted by direct compression of the tendon on the curved concrete strut located on the inside of the tendon curve. This portion of radial force can be deducted from the design force required by the radial ties. Finally, at the member end of the banana, (i.e. at the end of the tendon curve) one has a uniform compression

stress acting over the cross section of the banana.

When one inserts the notional banana into the main member, the force disperses from the banana into the member as shown in Fig. 4.14. When the blister is located away from the web, the force dispersion occurs along the length of the blister with less force transfer near the anchor and more force transfer where the blister completely enters the flange. While there is symmetry about a vertical plane through the tendon, there will be a local moment about a horizontal axis because the post-tensioning force which, while concentric with respect to the notional banana, is no longer concentric with respect to the cross section into which the forces are dispersed. When the blister is located at the junction of the web and the flange, forces are dispersed into both of these elements. The resulting local moments about the horizontal and vertical axes are resisted by the web and flange respectively. Regardless of the blister location, force dispersion into the main member will substantially reduce the axial stresses in the banana by the time the transition curve is reached. The effective cross section of the banana is modified as shown in Fig. 4.14 (b) and (c).

This has the effect of reducing the total compression force which acts inside the tendon curve and thus influences the radial tie force requirements. (The ties anchor back that portion of the radial tendon pressure not resisted by the curved concrete compression strut inside the curve of the tendon.) When the banana is inserted into the member the forces will tend to make use of the available concrete volume (i.e. the concrete tries to keep the strains compatible) thus producing a deviation in the compression force path which requires a transverse tension tie as shown in Fig. 4.14.

In order to make the blisters as small as possible, sometimes blister dimensions are selected so that the banana is "pinched" where the blister disappears into the member. The compression stress in the pinched zone should be checked but since the axial stresses in the banana are reduced by dispersion into the member the pinched zone is usually not critical. The shear friction failure mechanism may be checked. In principle, the prestressed banana is a curved compression member and if the reinforcement was detailed in accordance with the requirements for compression members shear would not govern. Without such reinforcement shear is resisted by the tensile capacity of the concrete, and shear may govern the design. To prevent undue reliance on the tensile strength of the concrete, the shear friction failure mechanism may be checked. A component of the post-tensioning force will provide a compression force perpendicular to the shear friction surface. The transverse and radial ties also provide force components perpendicular to the shear friction surface as shown in Fig. 4.15. When the blister is at the web to flange junction, the shear friction surface is three dimensional. From the theory of plasticity, shear friction is an upperbound failure mechanism thus one should check the shear friction capacity of the blister with the proposed reinforcement (for transverse ties, minimum reinforcement etc ...) and need only add shear friction reinforcement if it is required. Blisters will be generally on the order of 2 m in length hence the shear stresses will not be uniform along the blister. If the line of action of the post-tensioning force does not act through the centroid of the shear friction surface, one should consider the
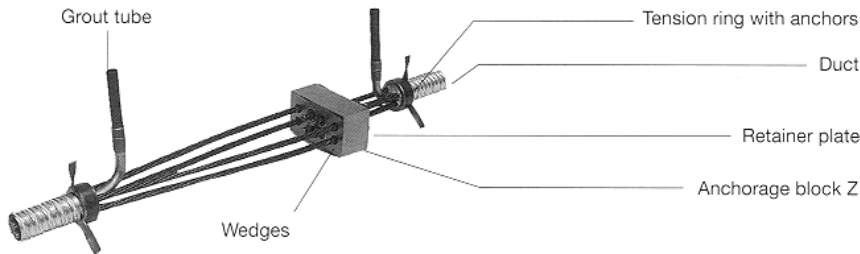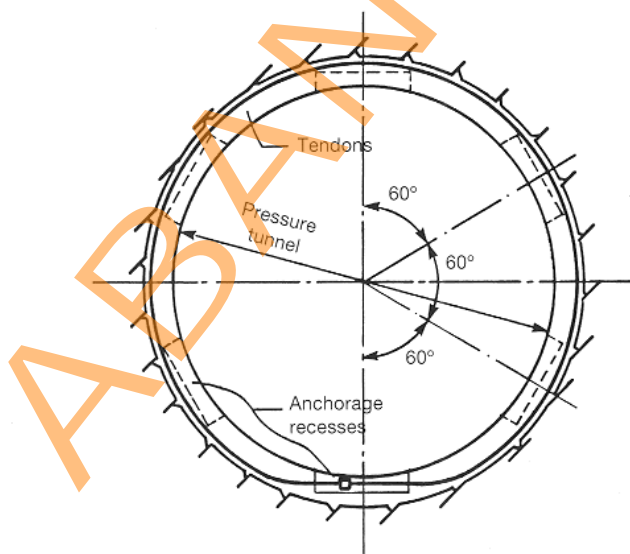
Intermediate Anchorage Type Z

Centre-stressing anchorages are used for ring tendons in circular structures, or for those tendons where the ends cannot be fitted with normal stressing anchorages.
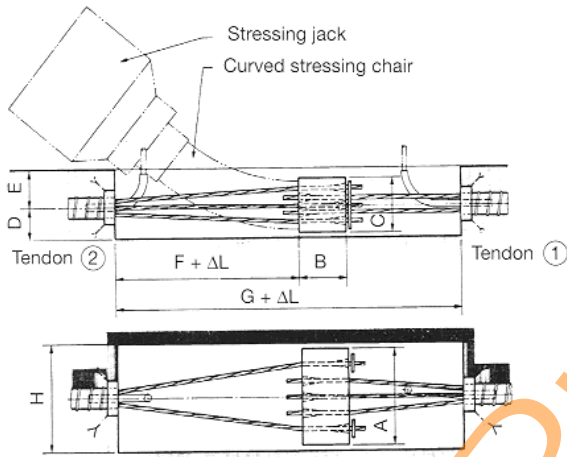
ΔL = Elongation of tendon ②

$E = \dfrac{C}{2}$ + required cover

a) The blockout should be proportioned to permit unrestrained movement of the anchorage during stressing.

b) One or more Z anchorages can be used to stress circular tendons without buttresses.

Figure 4.16: A VSL Type Z anchorage permits stressing from any point along a tendon

additional bending effects when distributing shear friction reinforcement. For design purposes, a distribution of reinforcement, as suggested in Fig. 4.15, which is concentrated toward the anchorage will suffice. One would not expect shear friction to govern.

Reinforcement congestion in a blister can be reduced by rationalizing the detailing. As suggested in Fig. 4.15, orthogonal ties provide confinement to the local zone rendering spirals redundant. The ties should be checked to ensure that sufficient confinement is provided. Often the spiral can be eliminated with little or no adjustment to the tie requirements. As stated previously, the total shear friction steel requirements will likely be satisfied by the reinforcement which is provided for other purposes, hence little if any additional shear friction reinforcement need be provided.

As a final comment concerning blisters, if there is precompression from adjacent ("upstream") blisters, the tie back force of P/4 can be reduced. The precompression will assist in controlling cracks behind the blister.

In addition to the intermediate anchorages already discussed, there are several other possibilities. A few of them are briefly introduced in this section.

The VSL Type H anchorage is often used as a dead end intermediate anchorage. The VSL Type L anchorage can also be used as an intermediate anchorage for a pair of tendons. Both the H and L anchorage are particularly economical because they can be used without additional formwork for stressing pockets, buttresses or blisters.

VSL anchorage Type Z is illustrated in Fig. 4.16. It may be used to stress a tendon at any point along it's length. The tendon is actually stressed in both directions from the anchorage. Since the anchorage does not bear against the concrete no local zone reinforcement is required. A tension ring is provided at the ends of the ducts to resist the deviation forces where the strands splay apart. There should of course be minimum reinforcement around the blockout which is filled with mortar after stressing is completed. This anchorage can be used for slabs when there is no access for stressing anchorages at the slab edges. When using such anchorages, one should account for the friction losses which occur in the curved stressing chair.

Higher than normal jacking forces are used to at least partially compensate for this friction. The higher forces occur in the strand beyond the wedges and hence have no detrimental effect on that portion of strand which remains in the completed structure.

VSL has recently developed a precast anchorage zone which contains the anchorages and related local zone reinforcement. The detailing can be made more compact through the use of higher concrete strength in the precast concrete component. It offers the advantage of permitting shop fabrication of a significant portion of the work. Figure 4.17 illustrates an application of this for posttensioned tunnel linings. This application may be thought of as a buttress turned inside out.

Your nearest VSL representative may be contacted for further details regarding these and other possibilities.
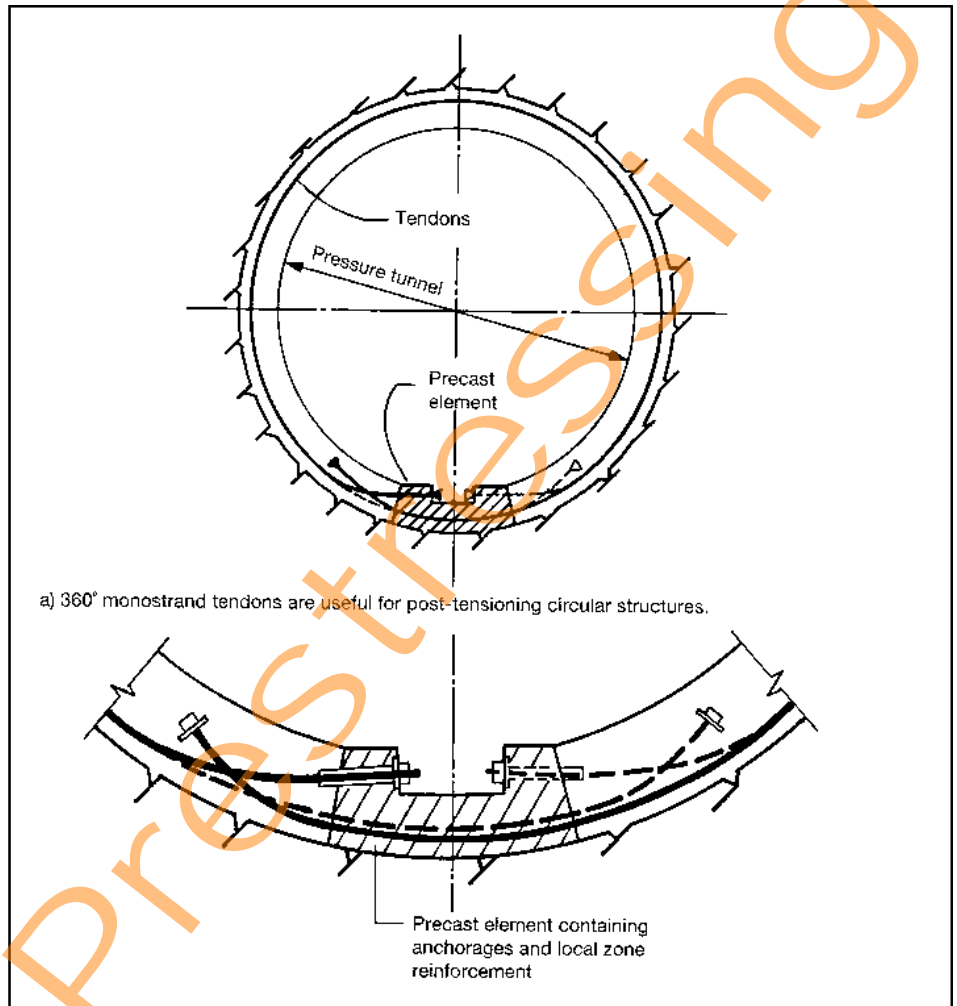

a) 360° monostrand tendons are useful for post-tensioning circular structures.

Precast element containing anchorages and local zone reinforcement

Figure 4.17: A precast anchorage zone can be used to simplify construction in the field.

4.4 Tendon Curvature Effects

This section deals with special issues associated with curved tendons including: in-plane deviation forces; out-ofplane bundle flattening forces; minimum radius requirements; and minimum tangent length requirements.

Any time a tendon changes direction it produces "radial" forces on the concrete when it is post-tensioned. The radial force acts in the plane of curvature and equals P/R, the tendon force divided by the radius of the curvature. Expressions for common tendon profiles can be found in most texts on prestressed concrete design.

Tendon curvature effects are very useful. Curvature of a tendon to almost any desired profile to introduce forces into the concrete

which counteract other loads is one of the major advantages of post-tensioning. However, when the forces are overlooked problems can result. Podolny [25] reports several examples of distress when these forces have not been recognized and designed for. Figure 4.18 illustrates tendons in a curved soffit of a box girder bridge. The vertical curvature of the tendons produces downward forces on the soffit slab which in turn produce transverse bending of the cross section. When curved soffit tendons are used, they should be placed near the webs rather than spread uniformly across the cross section. This reduces the transverse bending effects. In addition, it permits the tendons to be anchored in blisters at the junction of the web and flange.

Horizontally curved girders usually have

horizontally curved tendons which produce horizontal pressures. When tendons are located in the webs, they can produce significant lateral loads on the webs.

In addition to these global effects on the structures, one must look at the local effects of tendon curvature. Specifically, is the radial force acting on the concrete sufficient to cause the tendon to tear out of the concrete as shown in Fig. 4.19? Podolny [25] reports examples where the tendons did in fact tear out of the webs of two curved box girder bridges. The solution is to provide supplementary reinforcement to anchor the tendons to the concrete or ensure that the tendons are sufficiently far from the inside curved concrete surface that the concrete has adequate capacity to prevent local failure. When a number of tendons are placed in one
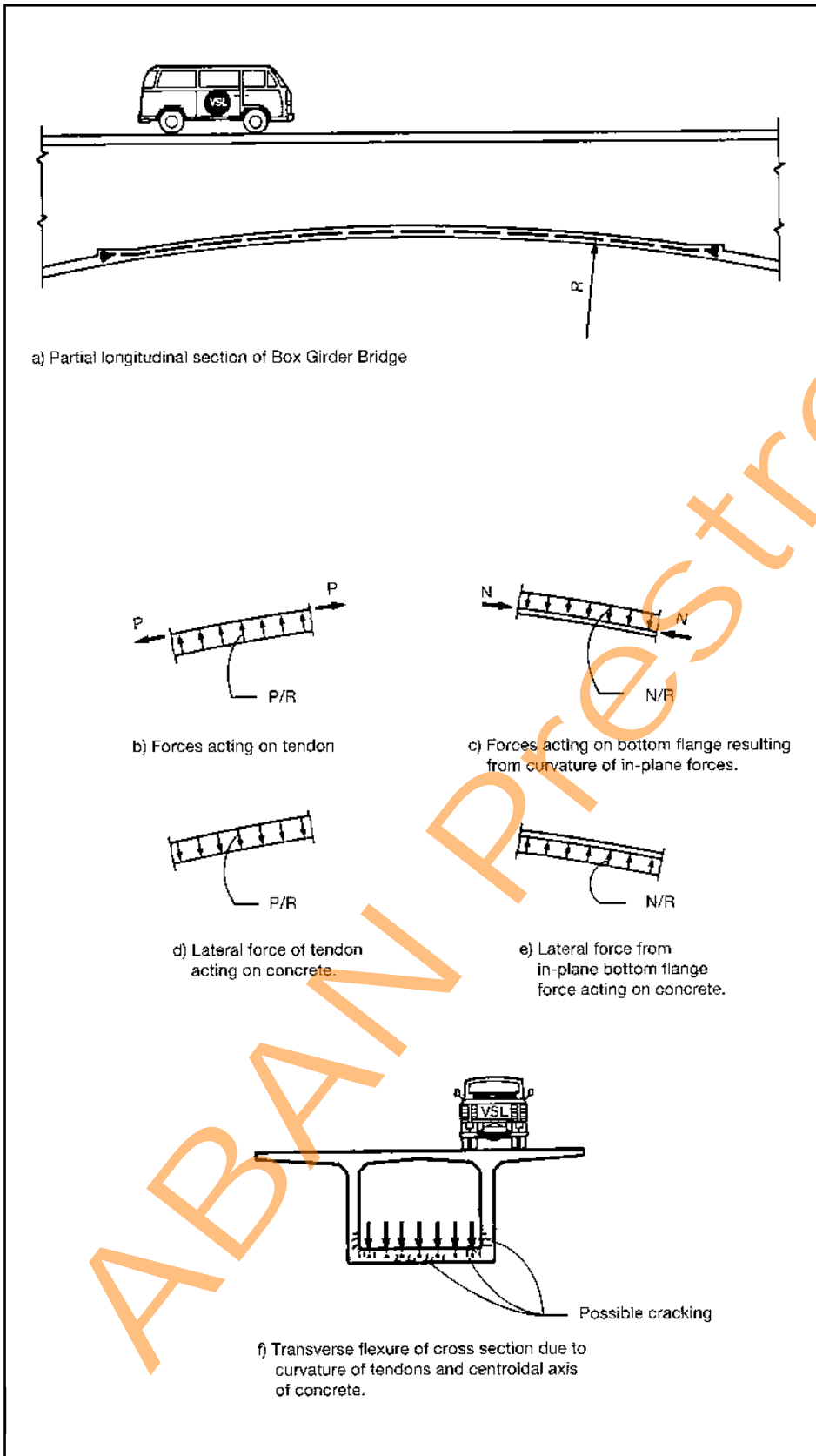
a) Partial longitudinal section of Box Girder Bridge

b) Forces acting on tendon

c) Forces acting on bottom flange resulting from curvature of in-plane forces.

d) Lateral force of tendon acting on concrete.

e) Lateral force from in-plane bottom flange force acting on concrete.

f) Transverse flexure of cross section due to curvature of tendons and centroidal axis of concrete.

Figure 4.18: Girders with curved soffits experience effects from curvature of the tendons and curvature of the bottom flange.

plane, one should check to ensure that delamination along the plane does not occur.

As in the case of tendons, when the centroidal axis of a member changes direction, transverse forces are produced. In this case, the transverse forces result from changes in direction of the compression force (compression chord forces) and are automatically accounted for when strut-and-tie models are used for design. It is worth pointing out that the curved soffit slab in Fig. 4.18 has a curved centroidal axis and a radial pressure as a result. If N is compressive, the pressure is upward. This can be significant near the supports where N is large. Near midspan, N may be tensile in which case it would exert a downward pressure on the soffit. Similar situations exist in the webs of curved girders.

Bundle flattening forces out of the plane of curvature are produced by multistrand or multiwire tendons. The problem is somewhat similar to an earth pressure problem. As discussed previously, tendon curvature produces a radial force in the plane of curvature. For the tendon orientation shown in Fig. 4.20, this is equivalent to each strand being pulled downward by a "gravity force" which is dependent upon the radius of tendon curvature. It is apparent that the vertical force produces a horizontal force in a manner similar to earth pressure generated in a granular material. Based on test data, it would appear that O, the friction angle is approximately 40 degrees. The friction angle of strand on strand is approximately 10 degrees. The remaining 30 degrees come from macrointerlocking of the strands since the contact angle of strands relative to the direction of movement is 30 degrees as shown in Fig. 4.20 (d).

The total lateral force depends primarily upon the axial tendon force, radius of curvature, and depth of strands within the duct. The first two items are selected or determined by the designer as a matter of routine. The later can be determined by drawing (to scale) a cross section of the duct with the strands "packed" toward the inside of the tendon curve. One can approximately determine the vertical pressure distribution. It is linear with depth, but the pressure at the invert must be determined such that when the vertical pressure is integrated over the gross cross sectional area of the bundle,
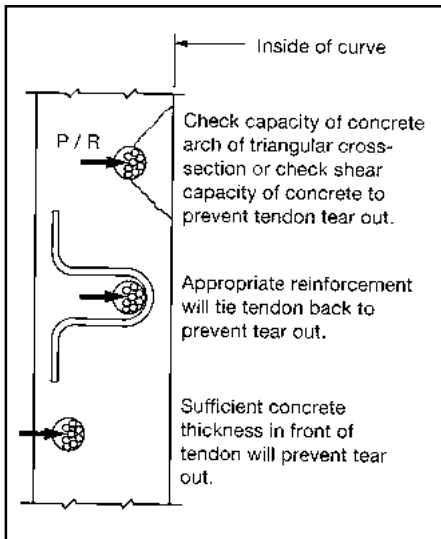
Figure 4.19: Ensure that curved tendons cannot tear out of the

the correct total resultant force is obtained. From the vertical pressure distribution, one determines the horizontal pressure distribution and total horizontal force. Stone and Breen [27] use a similar approach with some simplifying conservative assumptions with regard to the pressure distribution.

Fortunately, for most practical situations a simpler approach will suffice. For VSL tendons in normal size ducts, it turns out that the total horizontal force is approximately equal to 25 % of the vertical force. This has been verified by discrete analysis (graphic statics) of the forces within various random packed strand arrangements (as per Fig. 4.20 (b) with from 3 to 22 strands).

In smaller tendons there is little macrointerlocking (in a 4 strand tendon, all 4 strands may lie in direct contact with the duct invert). This is counteracted by the fact that with such a configuration, the tangent planes of contact between the strands and duct are fairly horizontal so the horizontal component of the contact force is small. The net result is that for smaller tendons, the horizontal force is a little more than 25 % of the vertical force but the difference is sufficiently small that a 25 % rule is adequate for design with standard VSL tendons.

For oversized ducts (also flattened or oval), the bundle flattening forces are significantly reduced. The easiest method of determining
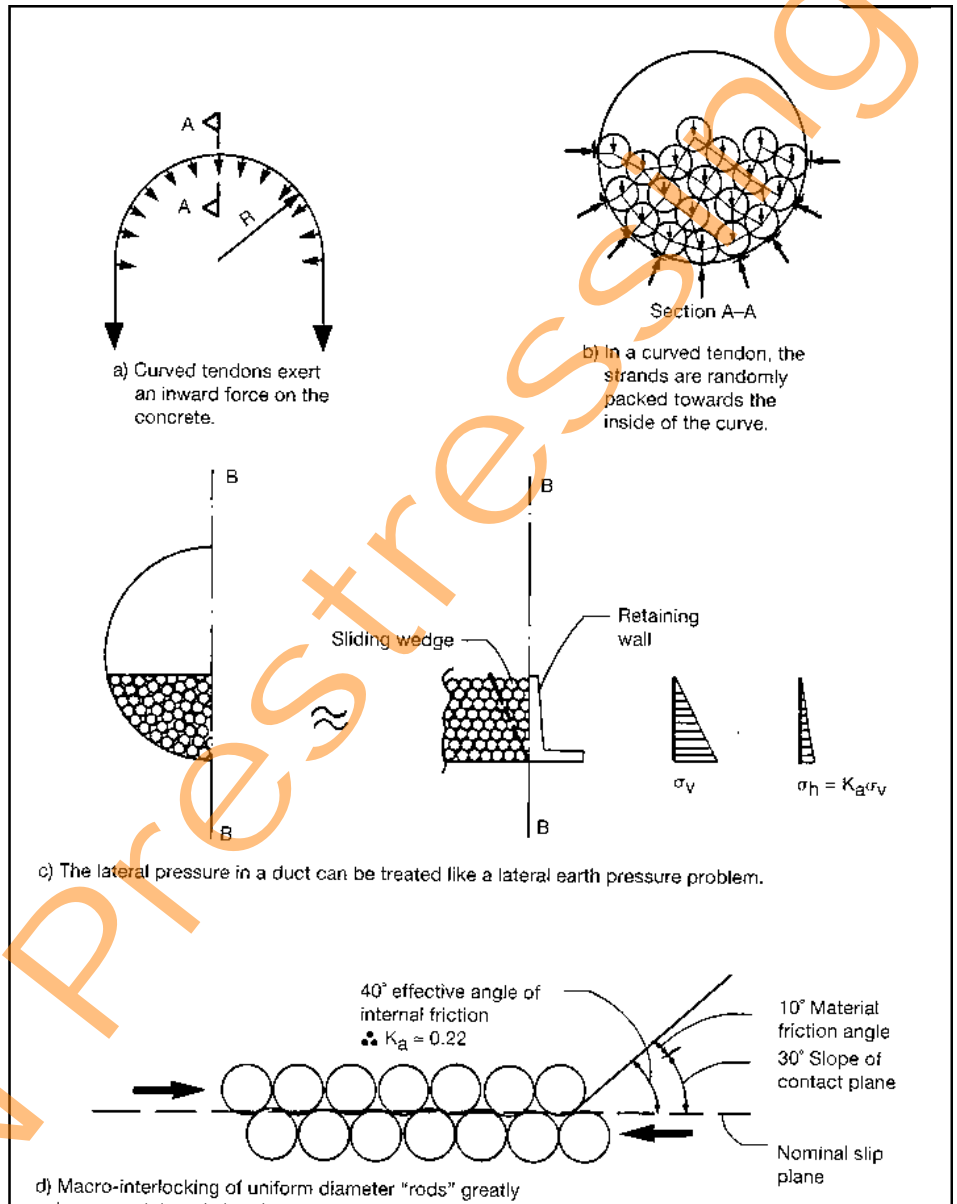

Figure 4.20: Bundle flattening can be treated as a special type of earth pressure problem.

the forces is to draw suitable randomly packed arrangements of strand in the duct and analyze the force system ignoring the benefits of friction. Even using graphical statics, a 22 strand tendon can be analyzed with this method in less than one hour.

One common method of providing supplementary reinforcement to resist bundle flattening effects is shown in Fig. 4.21. The bearing pressure from the tendon has to be dispersed across the cross section so the strut-and-tie model is similar to the one that

is used for the general zone in front of an anchorage.

In this instance, the horizontal tie force is set equal to the horizontal bundle flattening force.

This determines the slopes of the struts. The location of the tie is determined by the thickness of the member. As in the case of intermediate anchorages, reinforcement should be provided to anchor one quarter of the force behind the tendon. To ensure that there is sufficient local reinforcement to prevent a side shear failure of the concrete (similar to the tendon tear out
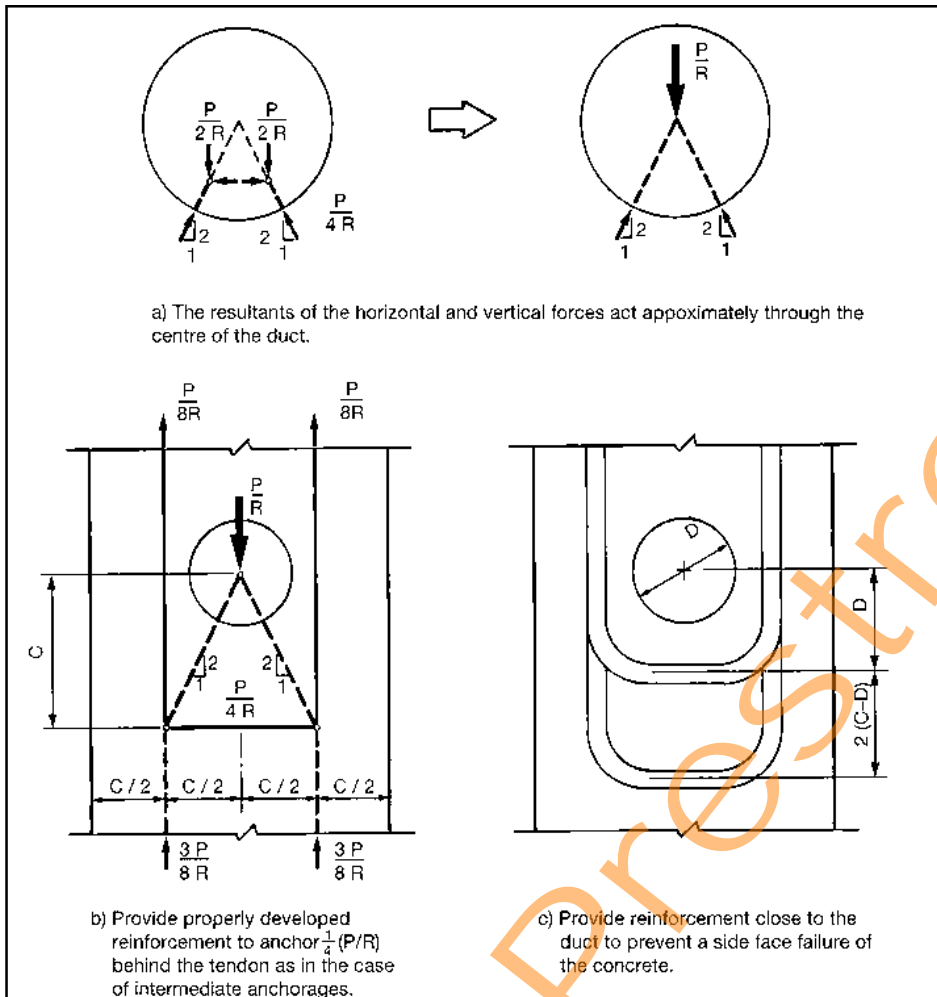
a) The resultants of the horizontal and vertical forces act appoximately through the centre of the duct.

b) Provide properly developed reinforcement to anchor $\frac{1}{4}$ (P/R) behind the tendon as in the case of intermediate anchorages.

c) Provide reinforcement close to the duct to prevent a side face failure of the concrete.

Figure 4.21; Reinforcement resists bundle flattening pressures and disperses them across the thickness of the member.

shown in Fig. 4.19), it is suggested that the reinforcement be placed in two layers. The first layer should be located one duct diameter from the center line of the duct. The second layer should be located so that the center of gravity of the two layers is located at the tie location as determined by the strut-and-tie model. Other reinforcement schemes are possible. For example, straight bars with plate anchors welded to each end can be used for the ties. Plate anchors are approximately 3 times the diameter of the bar they are anchoring, and have a thickness approximately equal to half the bar diameter. Tying back one quarter of the tendon bearing force is easily accomplished by the minimum reinforcement which is usually found in each face of a member.

When over-lapping loop anchors are used as in the case of some tank walls, thoughtful detailing will avoid excessive congestion where the tendons cross.

The minimum radius of tendon curvature is influenced by many factors but is primarily a function of the tendon force. For practical purposes the minimum radius of curvature in metres may be taken as $R_{min} = 3 (Pu)^{1/2}$ where Pu is the specified tensile strength (GUTS) of the tendon in MN. For constructive reasons, Rmin should not be less than 2.5 m. The recommended minimum radius assumes multistrand tendons in corrugated metal ducts being used in a "typical" posttensioning application. It is assumed that the curve may be located near a stressing anchorage, hence as

the tendon elongates, it rubs against the duct. It is also assumed that the strands occupy no more than 40 % of the duct cross section. These conditions are considered to be "typical". Other "non-typical" conditions such as: smooth thick walled steel duct; high density polyethylene duct; curves located where relative movement between strand and duct is small and larger ducts may permit smaller radii. For "non-typical" conditions, one should consult a VSL technical representative to determine suitable minimum radii. For example with suitable precautions and details, deviators for external tendons may use radii approximately equal to $R_{min}/2$.

The recommended minimum radius of curvature implies that the bearing stress imposed by the duct against the concrete is approximately 6.4 MPa when the tendon receives a jack force of 80 GUTS. Thus, the concrete bearing stresses are so low that they will not govern practical cases. The recommendations imply contact forces between the strand and the duct of less than 300 kN/m. Figure 4.22 provides details on the strand contact force for the worst strand in the bundle. As shown in Fig. 4.22 (a) and (b) differing strand arrangements produce different contact forces. Oertle [23] defined "K" as the ratio of the highest strand contact force to the contact (deviation) force for one strand. With computer simulation of many randomly packed strand arrangements he produced Fig. 4.22 (c), which can then be used to produce Fig. 4.22 (d). Figure 4.22 (d) indicates that as the tendon size increases, so does the contact force for the worst strand, hence for very large tendons with small radii of curvature, one should exercise more caution. The VSL PT-PLUS^{TM} system which utilizes a special polyethylene duct provides a much better local contact stress situation for the strand thus improving fatigue resistance. The wall thickness and profile have been designed to prevent wearing through the wall even with tendon elongations of 1 m.

A radius of curvature smaller than the recommended minimum may be used, but only after careful consideration of all the relevant factors. The VSL type L anchorage for example uses radii smaller than the proposed minimum. A sharp curve near a stressing anchorage may be undesirable because of the large friction losses which reduce the prestressing
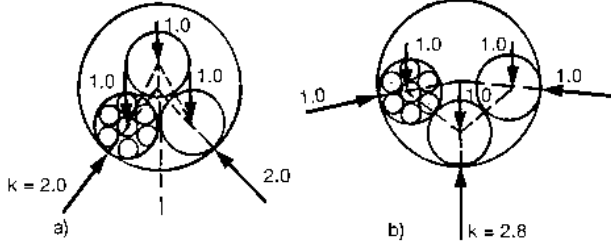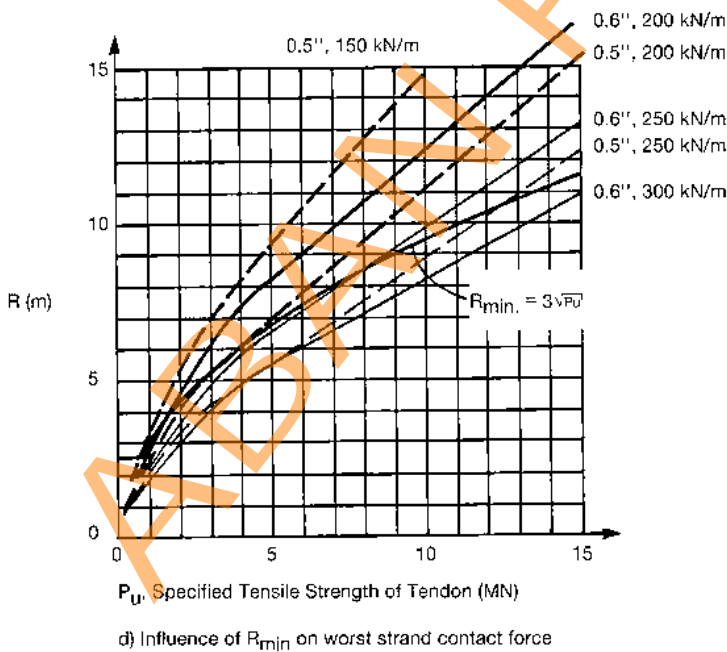
Examples of geometric position of 3 Ø 0.6'' strands inside a duct
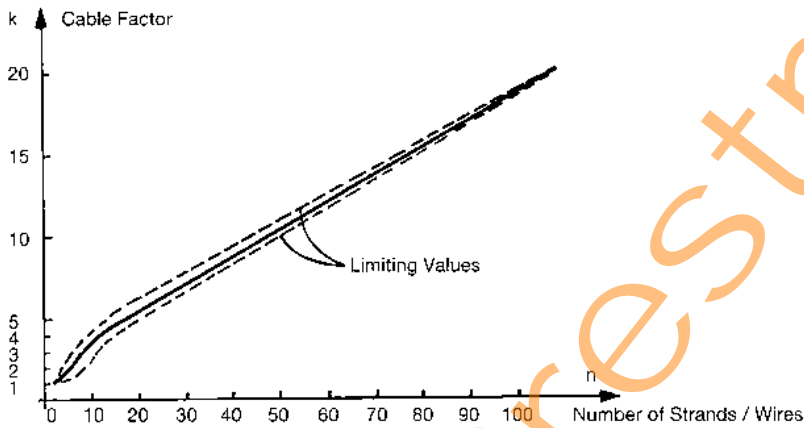
c) Cable factor k as a function of n, the number of strands/wires in cables made of strands or parallel wires

d) Influence of R_{min} on worst strand contact force

force over the rest of the tendon. In addition, if there is a lot of tendon travel (elongation), the strand can wear through the duct producing even greater friction losses as the strand rubs on the concrete. A similar sharp curve located near a dead end anchorage would be of far less consequence. Use of radii smaller than the recommended Rmin is not encouraged.

Figure 4.23 presents the recommended minimum radius of curvature along with the minimum tangent lengths recommended at stressing anchorages. The tangent length is required to ensure that the strands enter the anchorage without excessive kinking which can reduce fatigue life and anchorage efficiency (i.e. strand breakage at less than 95 % GUTS). To facilitate compact anchorages, the outer strands in most stressing anchorages are kinked within the anchorage and thus do not respond well to significant additional kinking. Dead end anchorages which do not use wedges do not require a tangent length. A benefit of using a tangent section at an anchorage is that it helps to disentangle the force introduction problem (general anchorage zone) and tendon curvature related problems (deviation forces and bundle flattening forces).

Figure 4.22: Strand contact forces increase with tendon capacity (Parts (a), (b) and (c) adapted from Oertle [23]).
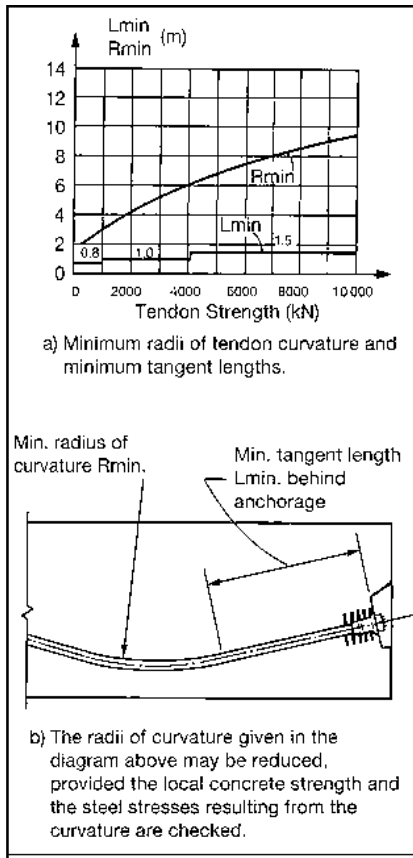
a) Minimum radii of tendon curvature and minimum tangent lengths.

Min. radius of curvature Rmin.

Min. tangent length Lmin. behind anchorage

b) The radii of curvature given in the diagram above may be reduced, provided the local concrete strength and the steel stresses resulting from the curvature are checked.

Figure 4.23: Use of less than recommended minimum tangent length or radius of curvature should only be considered in exceptional circumstances.

4.5 Additional Considerations

A variety of unrelated minor, but not unimportant considerations will now be discussed prior to discussing detailed design examples. These include minimum tendon spacing and suggestions for reducing reinforcement congestion.

Tendons must always be spaced or arranged in a manner which permits concrete to be placed and consolidated. When tendons are curved, they must have sufficient spacing to ensure that the transverse force does not cause one tendon to fail the concrete into an adjacent ungrouted duct. In the absence of a detailed check of the concrete shear strength, the minimum spacing suggested in Fig. 4.24 may be used. Note that this is not the same as the minimum anchor spacing listed in VSL product

data. It is common practice to have anchorages relatively widely spaced and at the same time have the tendons converge into a relatively tight group to maximize tendon eccentricity at locations of maximum moment.

Monostrand tendons are a special case as they do not have "empty" ducts, and do not generate forces out of the plane of curvature. They may be grouped in flat bundles of up to 4 monostrands without difficulty when the strands are not in the same plane of curvature.

When numerous curved tendons are spaced closely, as in a slab or top flange of a box girder, splitting along the plane of the tendons is possible. The tension stress in the concrete across the plane of the tendons should be checked. In this case, the portion of the force in the plane of curvature which should be anchored back by concrete tension stress is 50 %. If the concrete tensile capacity is exceeded, supplementary reinforcement capable of anchoring back 25 % of the force is sufficient. The reason for the difference in the recommended force is that the consequences of splitting along the plane of the tendons are usually severe and a greater margin of safety is required when forces are resisted by concrete tensile stresses only. As previously stated, a portion, or all of the tendon deviation force may (depending on the structure) be resisted by a concrete compression arch inside of the tendon curve. This will reduce splitting forces. Fig. 4.24 (c) and (d) illustrate the case of tendons in a slab with a modest cantilever where minimum tangent length considerations result in a concave down tendon profile. When the tendons are closely spaced slab splitting must be checked. Failures have occurred when this has been overlooked!

One frequent criticism of posttensioning is that the anchorage zone reinforcement is very congested. This need not be the case if one uses care in the design of these zones. Care means, among other things, not using extra reinforcement "just to be safe". For example, stirrup reinforcement requirements are often superimposed on general anchorage zone bursting reinforcement. If one proportions the reinforcement on the basis of a strut-and-tie model, the total reinforcement requirement is determined, hence superposition is not

necessary. Figure 4.25 illustrates the various load stages for a typical bridge girder. Such sections are usually flanged, hence, the top and bottom chord forces are concentrated in the flanges. The tendon is anchored at the centroid of the cross section (usually about 60 to 65 % of the height above the soffit), and has a 10 slope. Figure 4.25 (a) shows that for the load case of prestressing (P/S) alone, the primary tie force required to disperse the post-tensioning force over the depth of the cross section is 0.36 P. In Fig. 4.25 (b), the introduction of a support and a small vertical load actually reduces the primary tension tie requirement. With progressively greater loading in Fig. 4.25 (c) and (d), the primary tie force increases and the load path changes. For the load case of prestress plus ultimate loads, the maximum tie force is only equal to the shear force at the location. Hence, at least in this example, the stirrup reinforcement is governed only by shear and no additional reinforcement is required to deal with the general anchorage zone bursting stresses. Local zone reinforcement such as spirals will of course be required at the post-tensioning anchorage and possibly at the beam bearing.

For completeness, the detailed stress field (the strut-and-tie model is a shorthand version of the stress field) and the proposed reinforcement are also shown in Fig. 4.25. It is worth pointing out that while the bottom chord is not in tension at the support, the model only predicts a small compression force in this region. Shrinkage or temperature movements or other causes can easily produce tension in the bottom chord at the support hence at least minimum horizontal tension steel should be provided to prevent a potential shear failure. This may be provided in the form of ordinary non-prestressed reinforcement, or as is commonly done, by positioning, one of the post-tensioning tendons near the bottom of the beam with an anchorage at the beam end. See Fig. 4.4 for example.

Reinforcement congestion may be reduced by looking at all of the reinforcement which is present. It can often be rationalized or combined into other forms which simplify construction. It is good practice to draw the reinforcement details at a large scale (including bend radii and allowances for bar deformations) so
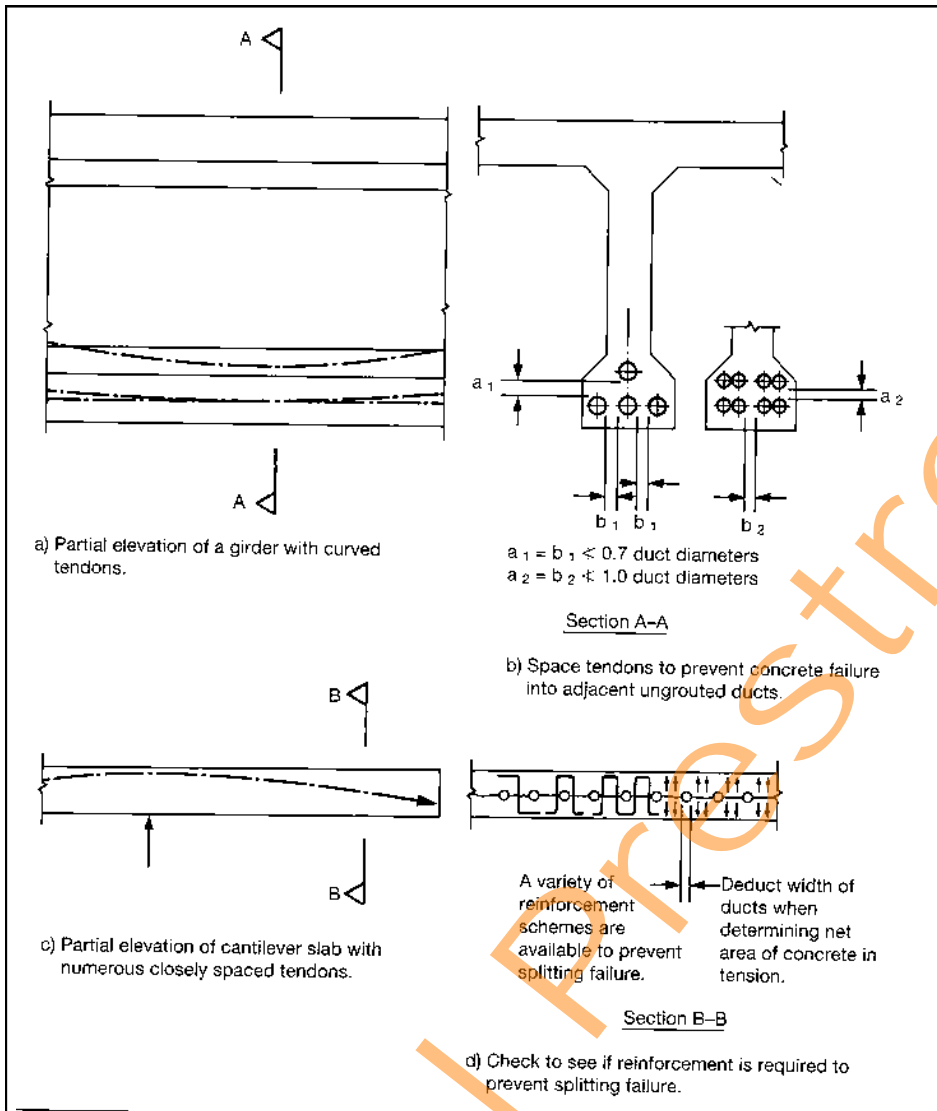
a) Partial elevation of a girder with curved tendons.

$a_1 = b_1 \lessapprox 0.7$ duct diameters
$a_2 = b_2 \lessapprox 1.0$ duct diameters

Section A–A

b) Space tendons to prevent concrete failure into adjacent ungrouted ducts.

c) Partial elevation of cantilever slab with numerous closely spaced tendons.

A variety of reinforcement schemes are available to prevent splitting failure.

Deduct width of ducts when determining net area of concrete in tension.

Section B–B

d) Check to see if reinforcement is required to prevent splitting failure.

Figure 4.24: Sufficient spacing or supplementary reinforcement is required to deal with the local effects of tendon curvature.

that conflicts can be sorted out before an angry phone call from the construction site is received. In addition all of the reinforcement (especially the distributed minimum reinforcement) should be utilized, that is, its contribution to strength should be considered. Recommended minimum values (anchor spacing, Rmin, tangent lengths etc.) are intended for occasional use, not routine use! Normally using a little more than the absolute minimum will not significantly increase costs and will result in a more buildable structure.

Finally, don't be overly conservative. Not only is it wasteful, but it also leads to more congested details and more difficult

construction which can in turn result in an inferior structure. Properly placed and consolidated concrete is far more important than extra reinforcement.

a) P/S

b) P/S + light, service loads

c) P/S + heavy, service loads

d) P/S + ultimate loads
(struct-and-tie model)

e) P/S + ultimate
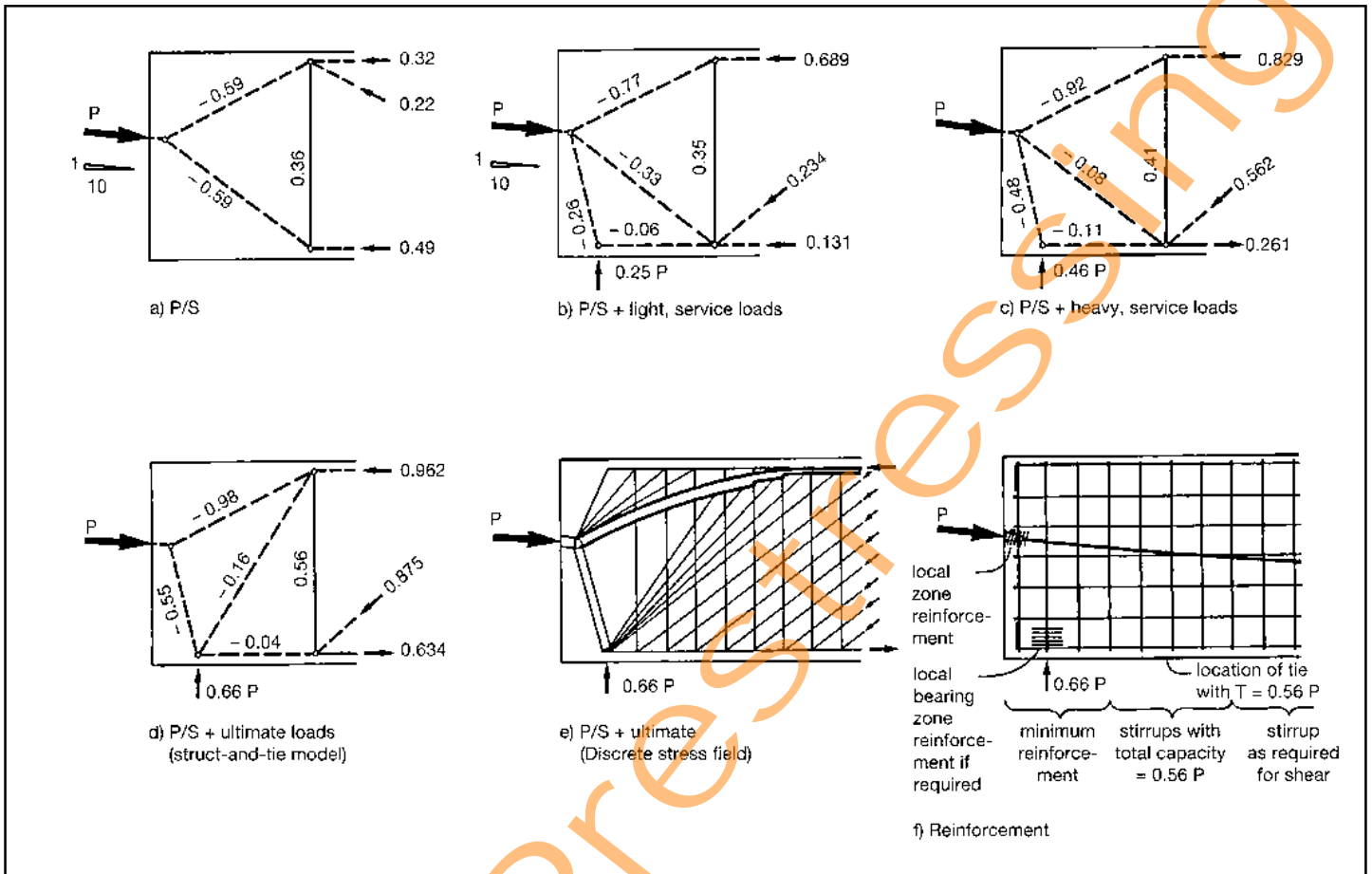(Discrete stress field)
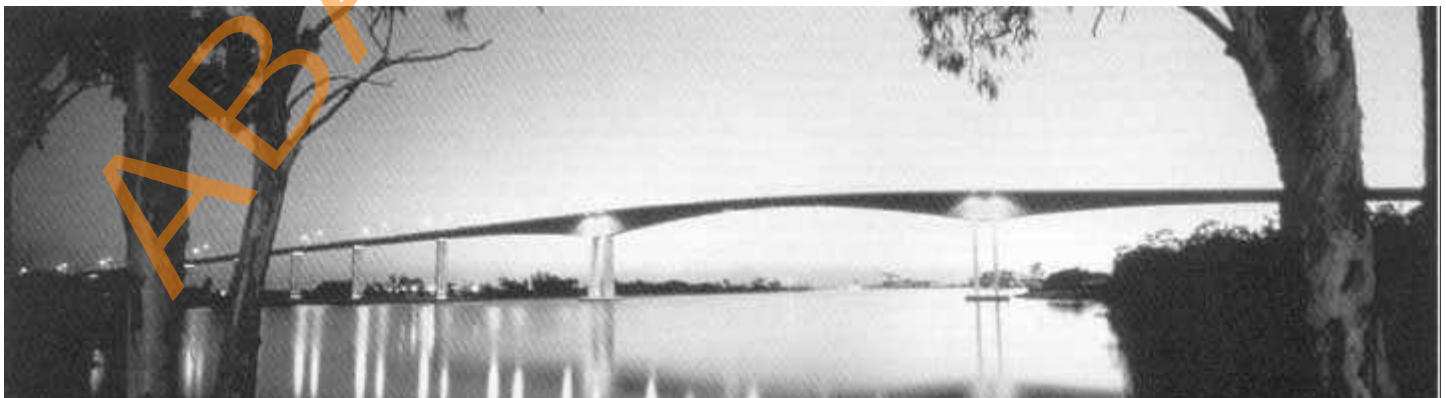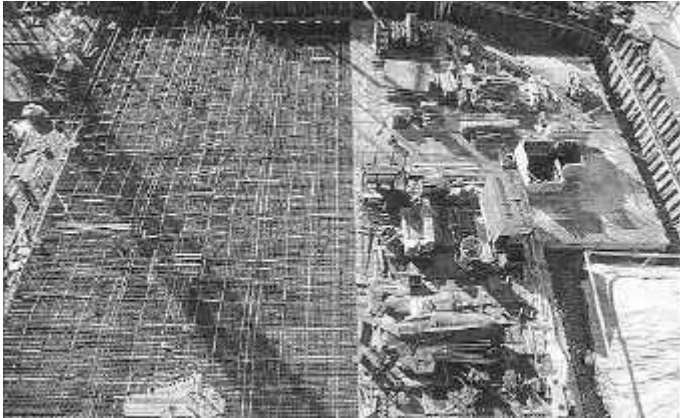
f) Reinforcement

Figure 4.25: If the design is based on a comprehensive Strut-and-Tie model, stirrup requirements do not need to be superimposed on general anchorage zone reinforcement requirements.
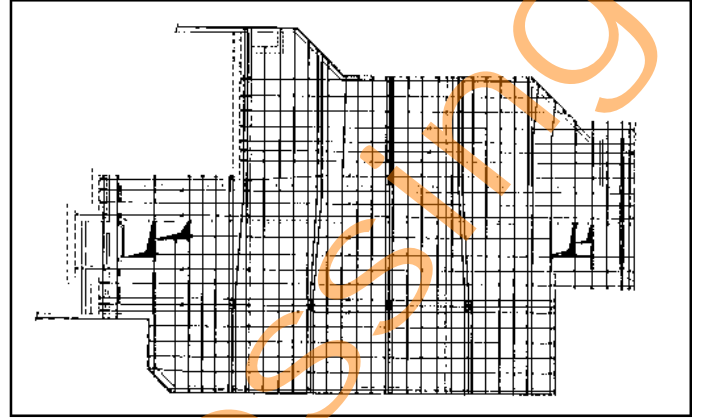

Brisbane's Gateway Bridge, was designed and detailed using many of the concepts and methods described in this report.

5. Design Examples

a) Construction (in stages) of foundation slab

b) Tendon scheme (construction stages 1-3)

Figure 5.1.1: Post-tensioned foundations usually have thick slab

Examples covering a variety of structure types are now presented. They illustrate the detailed application of the methods previously discussed. Each example is taken from a recent project and while the emphasis is on how posttensioning forces are introduced into the concrete, where appropriate, comments on general design aspects are also provided.

5.1 Multistrand Slab System

The extension of the army dispensary in Ittigen Switzerland uses a 0.8 m thick post-tensioned foundation slab. The post-tensioning system consists of VSL tendons 6-6 and 6-7 with VSL anchorage types EC, H, U, K and Z in various areas. The building has maximum dimensions of 35 m by 80 m and is somewhat irregular in plan. Figure 5.1.1 shows the construction of one of the five construction stages for the foundation slab. Half the tendons were stressed to full force after the concrete reached a cylinder strength of 22.4 MPa. The other half were stressed only after construction of three of the four upper slabs. Staged stressing was used to avoid overbalancing of the dead load and to compensate for early prestress losses.

EC anchorages were used as stressing anchorages at the slab perimeter with a typical spacing of 1500 mm. Figure 5.1.2 illustrates a typical anchorage zone configuration. The 6-7 tendon is stressed to a maximum temporary jack force of 1444 kN. In a thick slab situation with anchorages widely spaced, the concrete tensile strength may have considerable influence because it acts over a large area. While it would not be prudent to rely solely on the concrete tensile strength, one may be more liberal in the selection of a suitable strut and tie model. In Fig. 5.1.2 (a), strut inclinations flatter than used in a beam end anchorage have been selected. This reduces the primary tension tie requirements to about half of the usual beam end situation (i. e. P/8 vs P/4). In this case, the tension capacity of the concrete

a) Analysis.

Tie Requirement = 180 kN

$$A_{s \ req'd} = \frac{180 \cdot 10^3}{250} = 720 \ mm^2$$

Try 4 ties : $A_s = \frac{720}{4} = 180 \ mm^2/tie$

(Note only one leg effective)

∴ use 4 ∅ 16 per anchor @ 200 o/c

Local zone requirements
As per VSL Post-tensioning systems
use ∅ 260 x 250 Spiral
of ∅ 14 @ 50 pitch

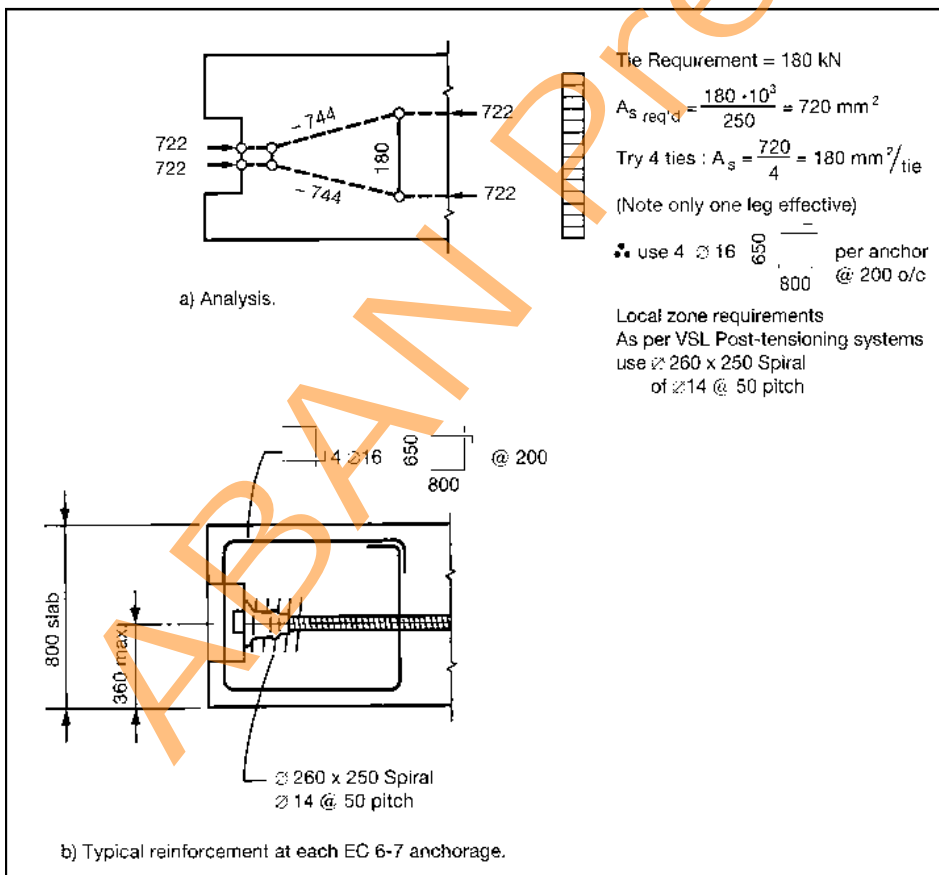b) Typical reinforcement at each EC 6-7 anchorage.

Figure 5.1.2: Anchorage zones in thick slabs benefit from the concrete tensile strength
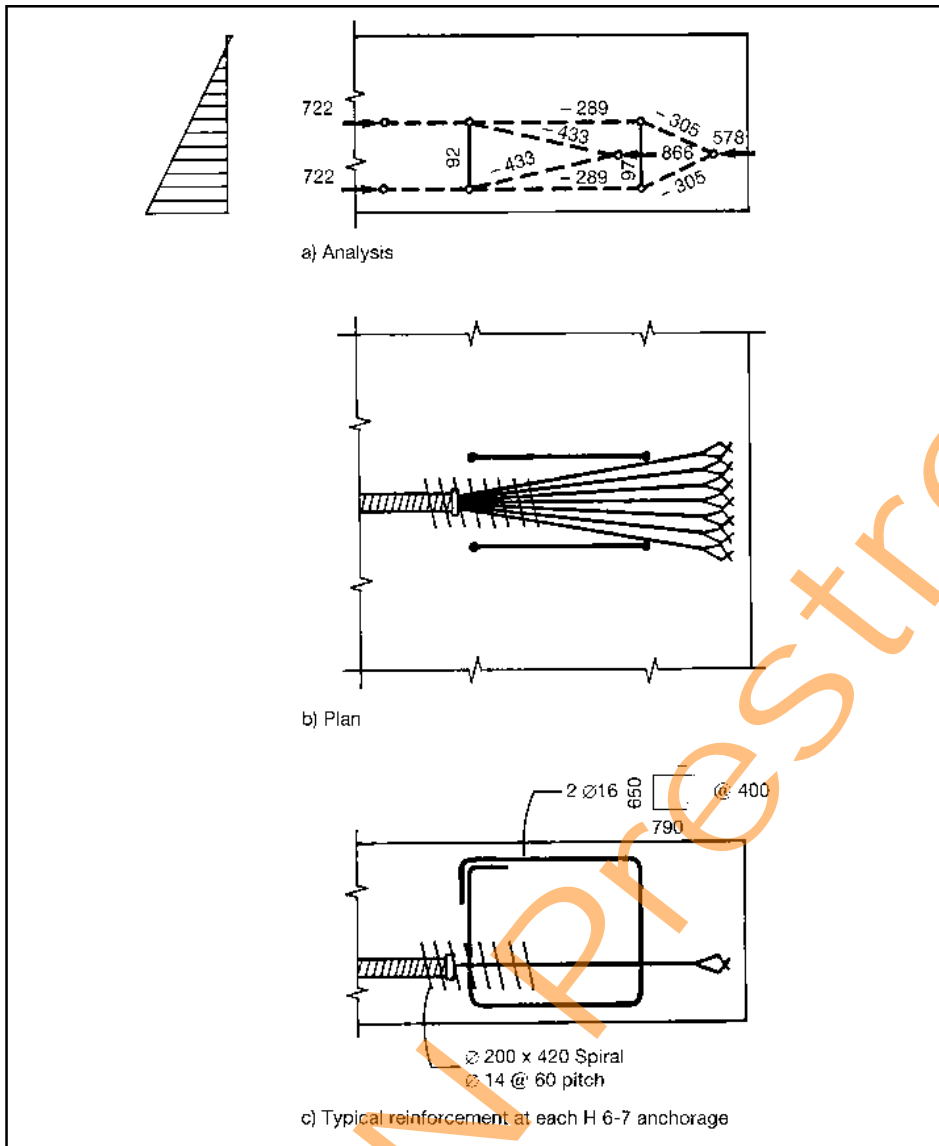
Figure 5.1.3: Force transfer from a bond anchorage requires distribution of transverse ties.

mobilized over an area of about 500 mm by 500 mm would be more than sufficient to resist the 180 kN tension force. Since the anchorage must resist a substantial force (i. e. 1444 kN), the provision of transverse reinforcement would be prudent. The modest amount of reinforcement provided is sufficient for equilibrium and will prevent an uncontrolled splitting crack. In Fig. 5.1.2 (a), the case with the largest edge distance (edge of anchor to closest top or bottom edge) governs the model. For simplicity, a subprism symmetrical about the anchor is used. Further dispersion of the

force over the full depth of the slab will occur in the zone to the right of that shown. This dispersion causes slab splitting stresses an order of magnitude smaller than those in the local anchorage zone. These can obviously be dealt with safely by the concrete tensile strength alone. Dispersion of the anchorage force in the plan view is readily achieved with the orthogonal ordinary nonprestressed reinforcement in the top and bottom of the slab.

The tie arrangement shown in Fig. 5.1.2 (b), is more practical than most other arrangements. It automatically ensures that the tension tie

reinforcement is placed at the correct distance from the slab edge, and also provides the slab edge with face reinforcement necessary to deal with the twisting moments in the slab. In the zones between anchorages the slab edge contains similar stirrups at a spacing of 400 mm. The proposed reinforcement is also easier to fix securely in position than a single plane of reinforcement at the required primary tie location.

VSL H and U anchorages were used as dead end anchorages. With both these anchorages, a large portion of the force is anchored by bond of the bare strands. A gentler or softer introduction of the anchorage force is achieved than with bearing type anchorages. The concrete tensile stresses have a lower magnitude but act over a larger area thus producing about the same total primary tension force resultant. The H anchorage situation is illustrated in Fig. 5.1.3.

In the analysis of the H anchorage 40 % of the anchored force is introduced at the onion while 60 % is introduced at the mid point of the estimated effective bond length. Two tension ties are required making it desirable to position the stirrups 500 mm from the slab edge so that both legs can act as ties. With the selected reinforcement arrangement the transverse ties are distributed over the area of the potential splitting crack which may develop at the tendon plane.

In plan view the strands at an H anchorage deviate as they exit the duct. The standard VSL H anchorage uses a tension ring at the end of the duct plus spiral reinforcement. The ring resists the large local forces where the strands deviate sharply while the spiral deals with the radial pressures resulting from the strand curvature which may occur over a somewhat larger length. For this case the standard tension ring and spiral were used. Other suitably designed arrangements would also be acceptable.

VSL Type Z anchorages were used as intermediate stressing anchorages for tendons which had inaccessible end anchorages. The Z anchorage is a centrestressing anchorage which does not bear against the concrete hence no local anchorage zone stresses are developed. The VSL Post-Tensioning Systems brochure provides details on this anchorage.

VSL Type K anchorages are couplers used for coupling to a tendon which has been placed and stressed. This was

done at construction joints between the various stages of slab construction. The K coupler is anchored, by the previously stressed tendon and does not bear against the concrete of the subsequent construction stage, hence no local anchorage zone stresses are developed. There are of course local stresses in the previous construction stage produced by the EC anchorage to which the K coupler is connected. The VSL Post-tensioning Systems brochure provides details on this anchorage.

5.2 Monostrand Slab System

In North America some 200 million square meters of typical building floor slabs have been successfully posttensioned with monostrand tendons. The standard anchorage zone reinforcement shown in Fig. 5.2.1 is adequate for "typical" applications. It is clear that the anchorage relies on the tensile strength of the concrete to resist the primary tension force required to disperse the anchorage force over the depth of the slab. Normally it would not be considered good engineering practice to rely solely on the concrete tensile strength to resist a major force. In this case, the calculations in Fig. 5.2.1 indicate that the tensile stresses are well within the capacity of concrete used in typical post-tensioned structures. This supported by the fact that some 18 million VSL monostrand anchorages have successfully used this detail leads one to conclude that this anchorage zone detail is acceptable. In typical situations the slabs are highly redundant (many anchorages widely spaced and two way continuous slab behavior) thus the failure of an anchorage is not serious for the structure. Furthermore, the greatest load on an anchorage occurs during stressing when the concrete is weakest. The stressing operation is in effect a load test of the anchorage zone. If the anchorage zone survives stressing, it will in all likelihood serve adequately throughout the life of the structure. If the anchorage zone fails during stressing, as occasionally happens when concrete is understrength or honeycombed, the failure is benign and can easily be repaired by replacing the damaged and defective concrete and restressing.
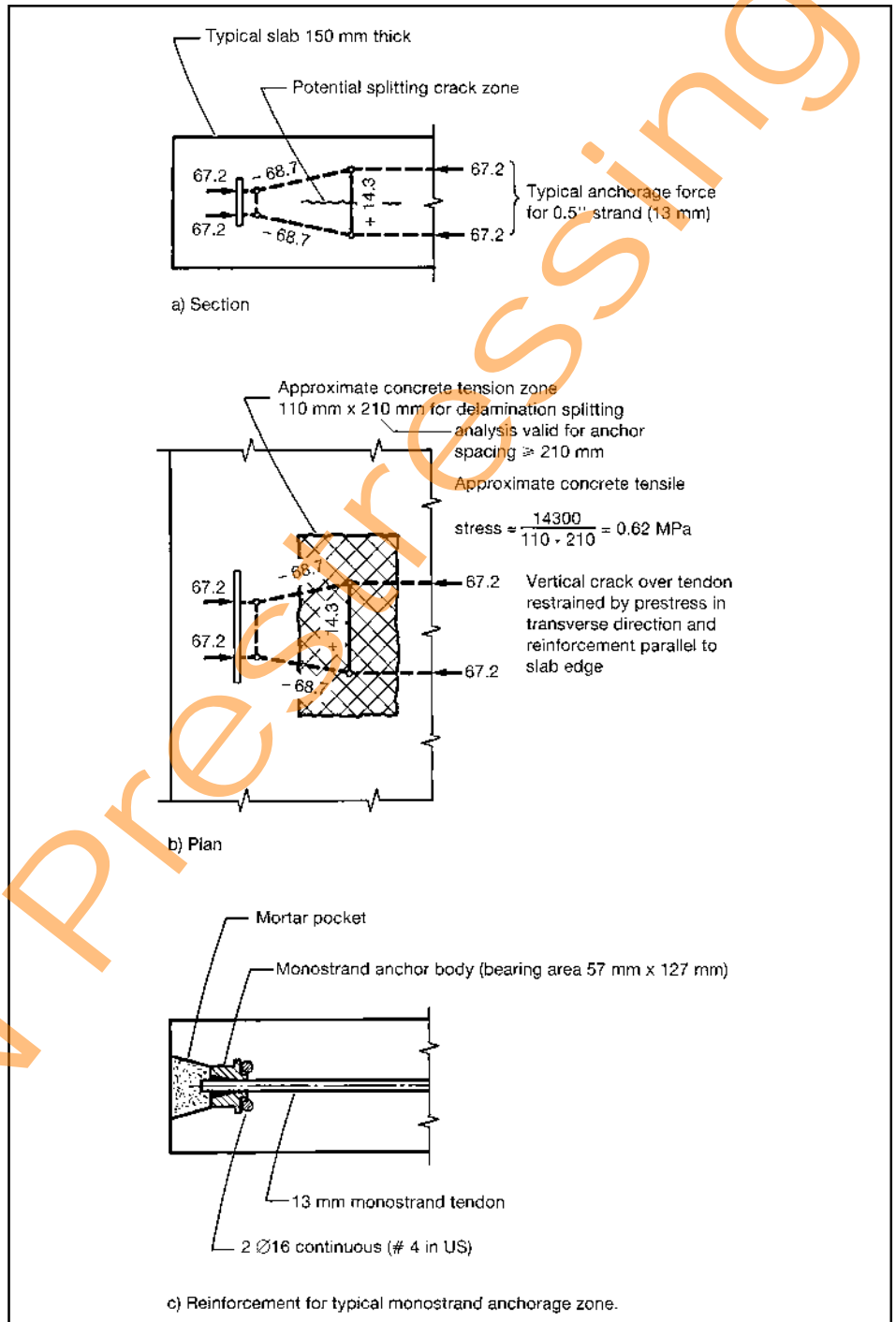
Figure 5.2.1: Typical monostrand anchorages rely on concrete tension capacity alone to resist local primary tension forces.

A "nontypical" but now common application with monostrand anchorages will now be discussed to highlight potential problems and solutions. Figure 5.2.2 illustrates a banded slab system which utilizes monostrands uniformly spaced in one direction and monostrands closely grouped or "banded" over the columns in the other direction.
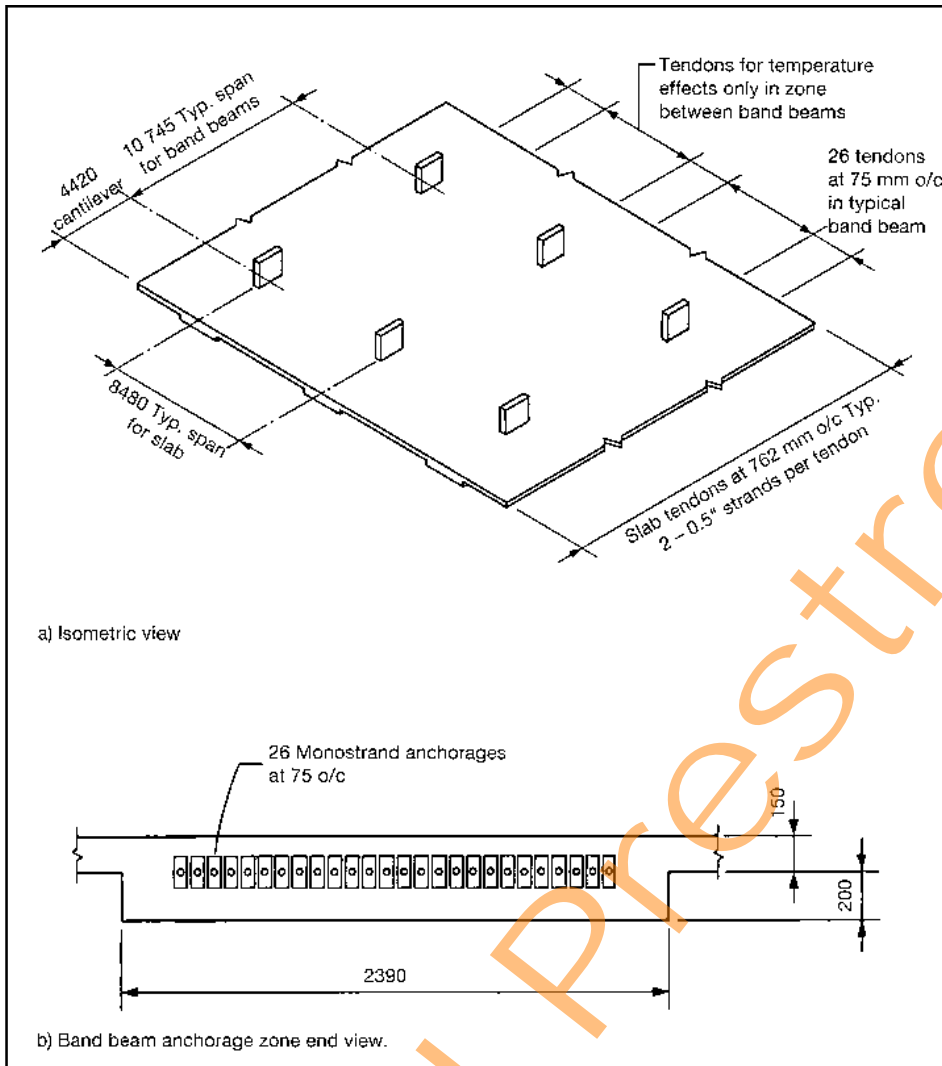
a) Isometric view

b) Band beam anchorage zone end view.

Figure 5.2.2: Banded tendon arrangements produce non-typical monostrand anchorage zones.

Often, the slab is thickened under the bands to create band beams approximately twice as thick as the typical slab areas. Such designs are usually efficient, but the "typical" monostrand anchorage detail is no longer adequate for the closely spaced anchorages at the ends of the band beams.

If one has anchors spaced at the minimum spacing, (i.e. anchorages touching with long direction vertically), as is occasionally done, the concrete tensile strength is likely to be exceeded. This is demonstrated in Fig. 5.2.3. Note that the outside diameter of the monostrand sheath reduces the width of the concrete area which provides tension resistance by 25%! The circular holes created in the concrete also tend to act as stress con

centrators raising local peak stresses well above the calculated average. In a band beam situation, the plane of the end anchorages may be very near to the re-entrant corner between the slab soffit and the beam side face, hence introducing yet another stress concentration. Beware of lightweight and semi-lightweight concretes as they tend to have lower tensile strength and toughness than normal weight concrete of similar compressive strength! A slab splitting crack behind a single anchorage will likely propagate through all the closely spaced anchorages along the plane of the tendons creating a large delamination failure. Such cracks may not be visible during stressing and may not be detected until the delaminated soffit falls off some

time later. If such a failure occurs, the damage is likely confined to the delaminated region near the anchorages because of the overall redundancy available in most slab systems. Collectively, the anchorages at the end of the band beam anchor considerable force as opposed to isolated monostrands and hence band beam anchorage zones should not rely on the concrete tensile strength for their structural intergrity.

The band beam anchorage zone design for the system illustrated in Fig. 5.2.2. is shown in Fig. 5.2.4. The actual structure, which used semi-lightweight

a) Section

Approximate concrete tension zone with reduction for monostrand sheath O.D. 195 mm assumes anchors touching and 15 mm O.D. strand sheath x 42 mm

Approximate concrete tensile stress

$$= \frac{14300}{195 \times 42} = 1.75 \text{ MPa}$$

Stress is too high to safely rely on the concrete tensile strength.

See Fig. 5.2.1 for comparison with a typical monostrand anchorage situation

b) Plan
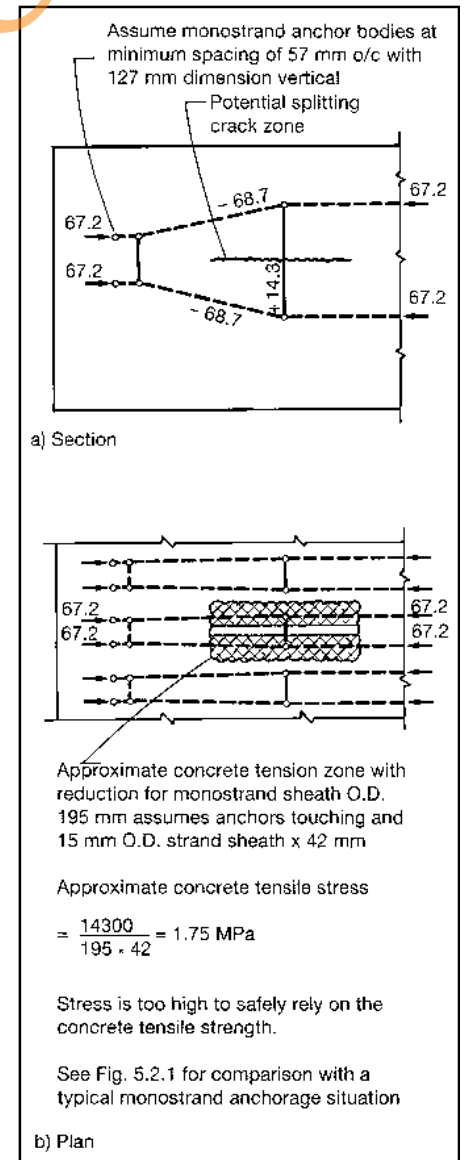
Figure 5.2.3: Closely spaced banded anchorages produce high concrete tensile stresses.

Analysis is for one strand anchorage from the banded beam anchorage zone shown in Fig. 5.2.2

Required vertical tie reinforcement per strand

$$= \frac{29200}{250} = 117 mm^2$$

say 1 Ø 12 leg per strand (A_s within 3% of A_s required)

Place Ø 12 between every second strand with middle of at 130 mm from anchor body

a) Analysis.

13 Ø 12 at 150 (placed between every second strand)

Ø 12 support bars

2 Ø 16 continuous

Min. Ø Bend ≈ 75 mm

b) reinforcement for banded tendon anchorage zone of structure shown in Fig.5.2.2
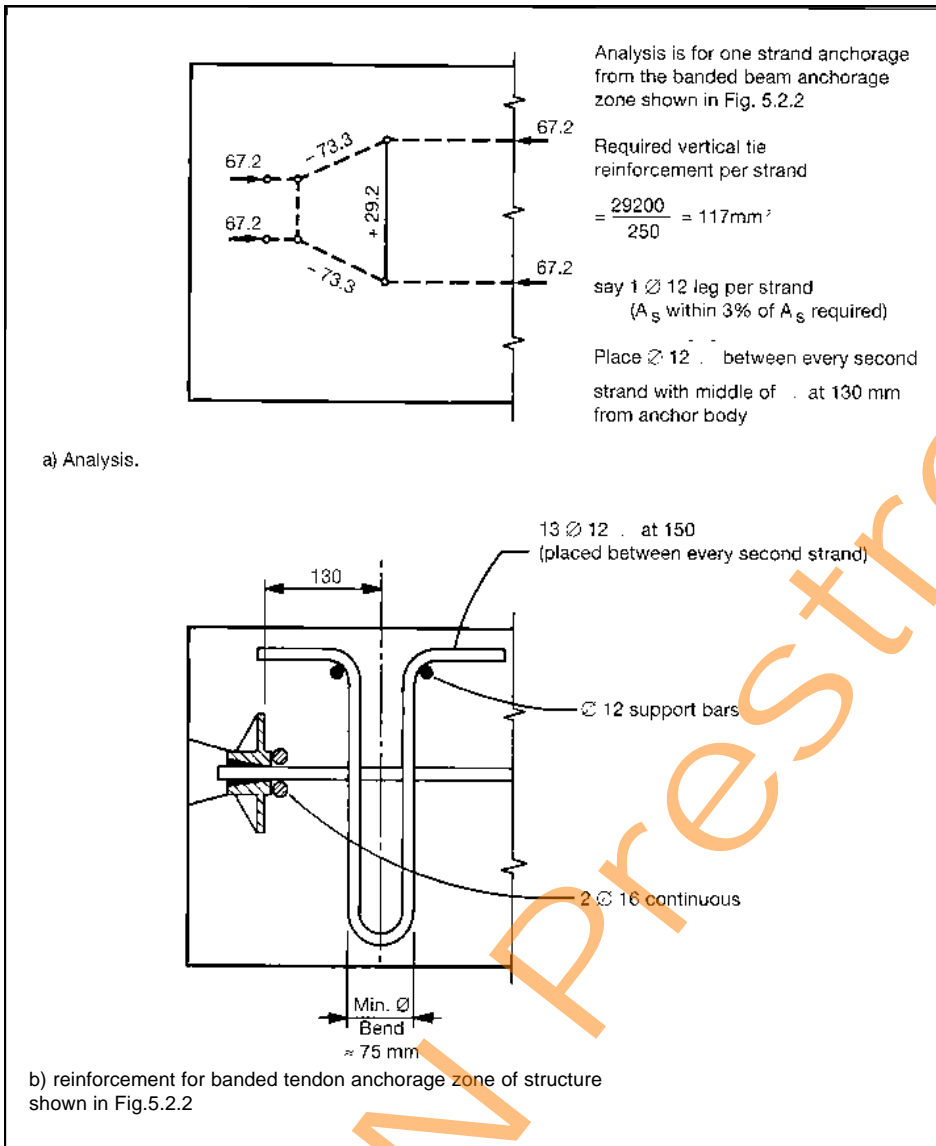
Figure 5.2.4: Reinforcement should be provided to resist local primary tension forces at banded tendon anchorages.

concrete with a specified strength of 20.6 MPa at stressing, did not have supplemental reinforcement in the anchorage zone other than 2-16 mm diameter bars horizontally behind the anchorages and parallel to the slab edge. Failures of band beam anchorage zones did in fact occur. Fig. 5.2.4 shows one of many possible solutions. As seen, it does not require very much supplementary reinforcement to produce an acceptable design.

5.3 Bridge Girder

Figures 5.3.1 and 5.3.2 illustrate a major bridge project in Malaysia which utilized a VSL alternative design for the 520 girders. The basis for the alternative design was to utilize precast girders with top flanges which were wide enough to serve as the formwork for the cast-inplace deck slab. A gap of 30 mm between flanges was provided to accommodate construction tolerances. Elimination of formwork for the deck slabs speeded construction, and reduced overall costs. The end block design for a typical girder will be investigated in this example.

Typical spans were 40m, with a cross section consisting of six precast girders spaced at 2.1 m, and a composite concrete deck slab. Diaphragms were provided at the girder ends only. The bridge was designed in accordance with British Standard BS 5400 using specified concrete strengths of 45 MPa and 30 MPa for the girders and slabs, respectively. Non-prestressed reinforcement had a specified yield strength of 410 MPa

Figure 5.3.1: Large bridges are an excellent opportunity for optimization with post-tensioning systems.

Figure 5.3.2: Modified T-beams replaced the originally designed I-beams and eliminated the deck slab formwork.
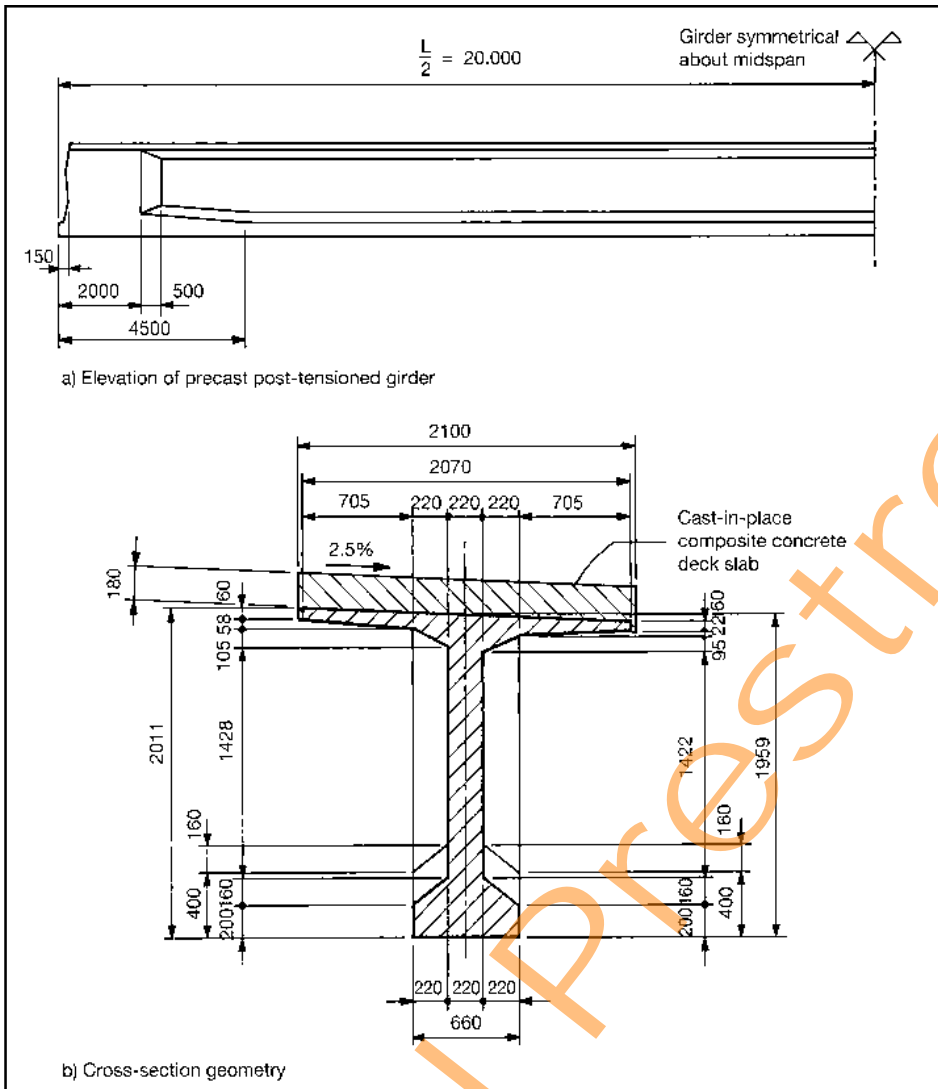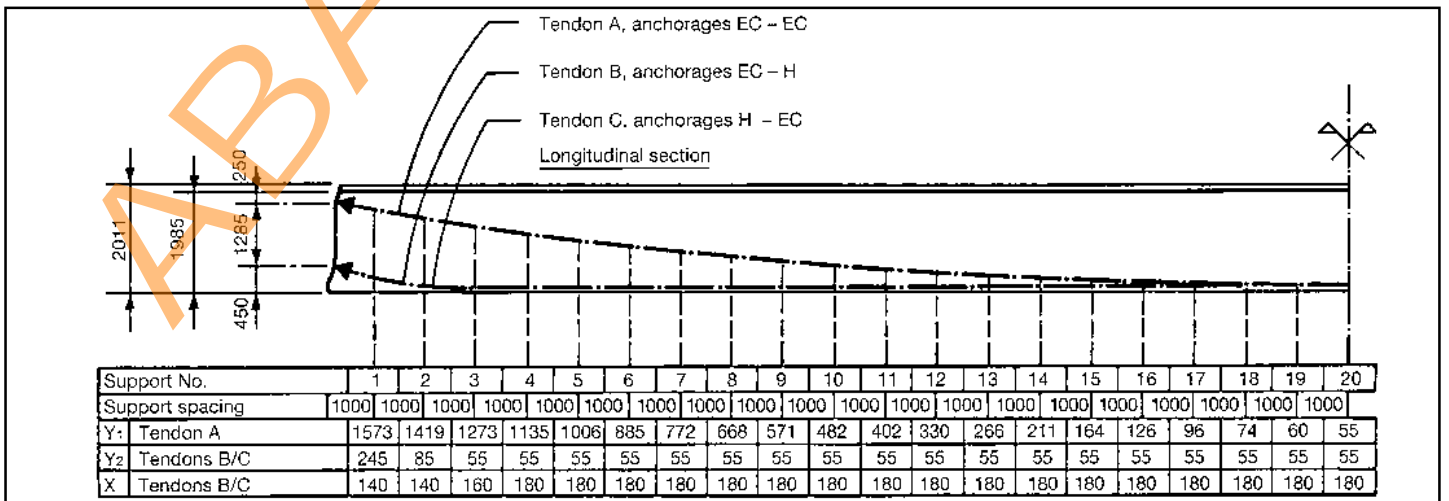
while the prestressed reinforcement consisted of seven-wire 12.9 mm diameter low relaxation Superstrand.

Figure 5.3.3 illustrates the geometry the girder. To form a simple end block, the web width is increased to match tlbottom flange width over the last 2 m each end of the girder. The irregular en face of the girder accommodates the stressing anchorages and the transver cast-in-place diaphragm.

The overall tendon profile is present in Fig. 5.3.4. Details of the tendon geo metry in the end block are provided in Fig. 5.3.5. The two bottom tendons we kept horizontal until the end block whe they were deviated upward using a minimum radius curve. This results in a concentrated upward deviation force from the lower tendons about 2 m fron the end of the girder. Under full design loads, a substantial portion of the she in the end block is resisted by the inclined tendons. For lesser load case: such as the bare girder during erection the tendon deviation force produces a "reverse" shear stress which is confine to the end block. A very efficient overa design was achieved. This would not have been the case if simple parabolic profiles had been used.

It is worth noting that to reduce congestion in the lower anchorages, a Type H anchorage is used for the dear end anchorage.

Figure 5.3.3: Determination of member form must take into account the method of construction.

Figure 5.3.4: Post-tensioning permits a infinite variety of tendon profiles which can be optimized to suit project requirements.

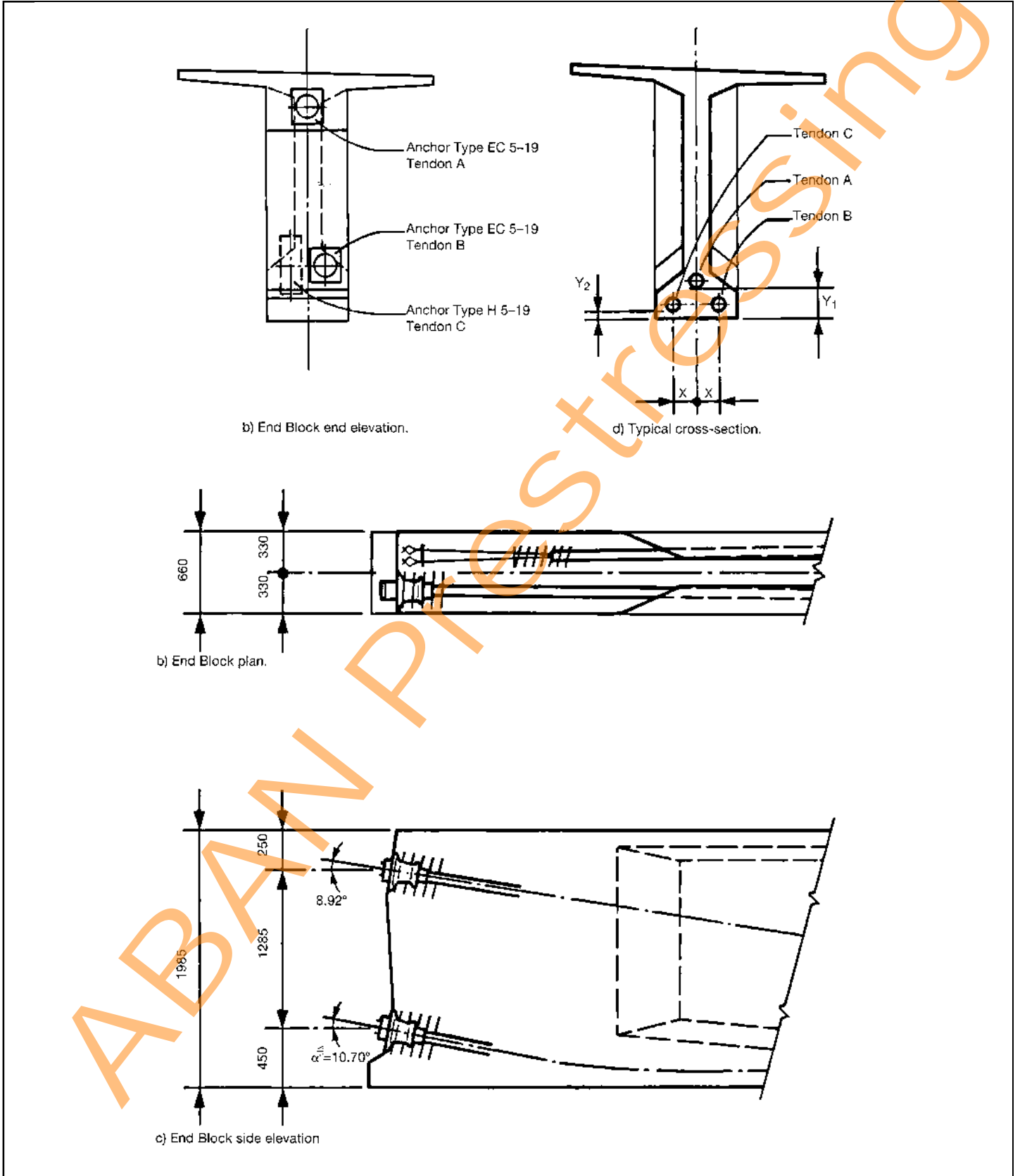| Support No.	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Support spacing	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000	1000
Y: Tendon A	1573	1419	1273	1135	1006	885	772	668	571	482	402	330	266	211	164	126	96	74	60	55
Y2 Tendons B/C	245	85	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55	55
X Tendons B/C	140	140	160	180	180	180	180	180	180	180	180	180	180	180	180	180	180	180	180	180

Figure 5.3.5: Careful detailing during the design produces easier construction and fewer problems in the field.
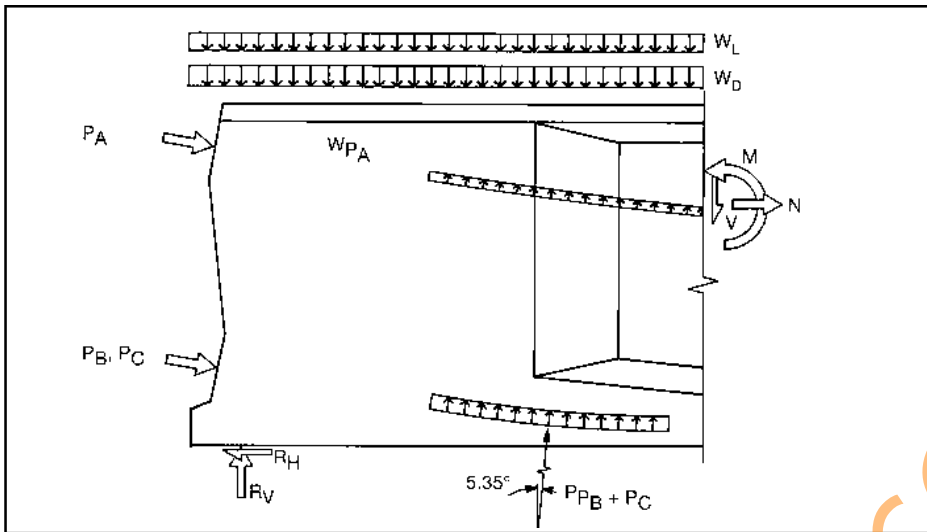
Figure 5.3.6 All forces acting on the concrete must be considered when detailing the End Block

Table 5.3.1 Load cases/Construction stages

Load case	P_A [kN]	P_B [kN]	P_C [kN]	R_V [kN]	R_H [kN]	W_D [kN/m]	W_L [kN/m]		Remarks
1	1325	0	0	0	0	0	0		
2	1214	1325	0	0	0	0	0		
3	1214	1175	1195	350	40	17.5	0		
4	1214	1175	1195	0	0	0	0		Suspended during move to storage/curing area.
5	2650	1175	1195	350	0	17.5	0	Non-composite Section	
6	2427	2650	1195	350	20	17.5	0		
7	2427	2350	2390	350	40	17.5	0		
8	2427	2350	2390	350	0	17.5	0		Suspended during transportation and erection (reaction supported 700 mm from soffit on large Ø pin)
9	2427	2350	2390	540	20	17.5	9.5		During casting of deck, wet concrete is a live load
10	2427	2350	2390	1480	115	27	47	Composite Section	Full dead plus live service loads (loads placed for max. shear).
11	2427	2350	2390	2085	170	33.75	70.5		Full ultimate factored loads (1.25 D + 1.5 L)

Note:

$$W_{P_A} = \frac{8yP_A}{L^2} = 8.37 * 10^{-3} P_A \ [kN/m] \qquad P_{P_B + P_C} = 2 \sin \frac{\alpha}{2} \ (P_B + P_C) = 0.186 \ (P_B + P_C) \ [kN]$$

Figure 5.3.6 illustrates all of the forces acting on the end block. For the sake of clarity, the forces perpendicular to the plane of the web have been omitted from the figure. Note that friction forces along the tendon exist, but these are very small and have been taken as zero. The height of the centroidal axis of the cross section varies from the end block to the typical cross section producing an additional deviation force. This additional deviation force is very small and occurs in the transition zone beyond the end block hence it has been taken as zero. The tendons introduce point loads into the concrete at their respective anchorages. In addition, due to changes in the tendon direction with respect to the centroidal axis of the concrete section, deviation forces are imposed on the concrete. For tendon A which is parabolic a uniformly distributed upward load is produced. For tendons B and C, a minimum radius vertical circular curve is used. This produces a uniformly distributed load along the curved portion of the tendon which, for practical purposes, can be treated as an equivalent point load.

Table 5.3.1 outlines the magnitude of each force for a variety of load cases. While the table is not exhaustive, it covers the load cases of interest for the design of the end block. Vehicle live loads have been converted to uniformly distributed loads which produce equivalent maximum shear forces. The stress resultants N, M and V on any section can be determined from equilibrium. The construction strategy was to provide just enough first stage post-tensioning to the girders to permit early form stripping when the concrete reached a compressive strength of 22.5 MPa. This permitted rapid form reuse. The girders were placed into storage until the concrete reached its full strength of 45 MPa. The final stage of post-tensioning was applied shortly before erection. Upward camber growth which can occur due to too much prestress too early was thus avoided.

The general approach to the analysis is to use a truss (strut-and-tie) model with a panel length of 2 m throughout the length of the girder. The end block is thus modelled with a special end panel which takes into account the various forces acting on the end block. A freebody diagram of the first 3 m of the girder

a) Load case 1 – After initial tensioning of first tendon

b) Load case 3 – After first stage post-tensioning

c) Load case 8 – During erection

d) Load case 9 – During casting of deck slab

e) Load case 11 – Ultimate loads acting on composite cross section
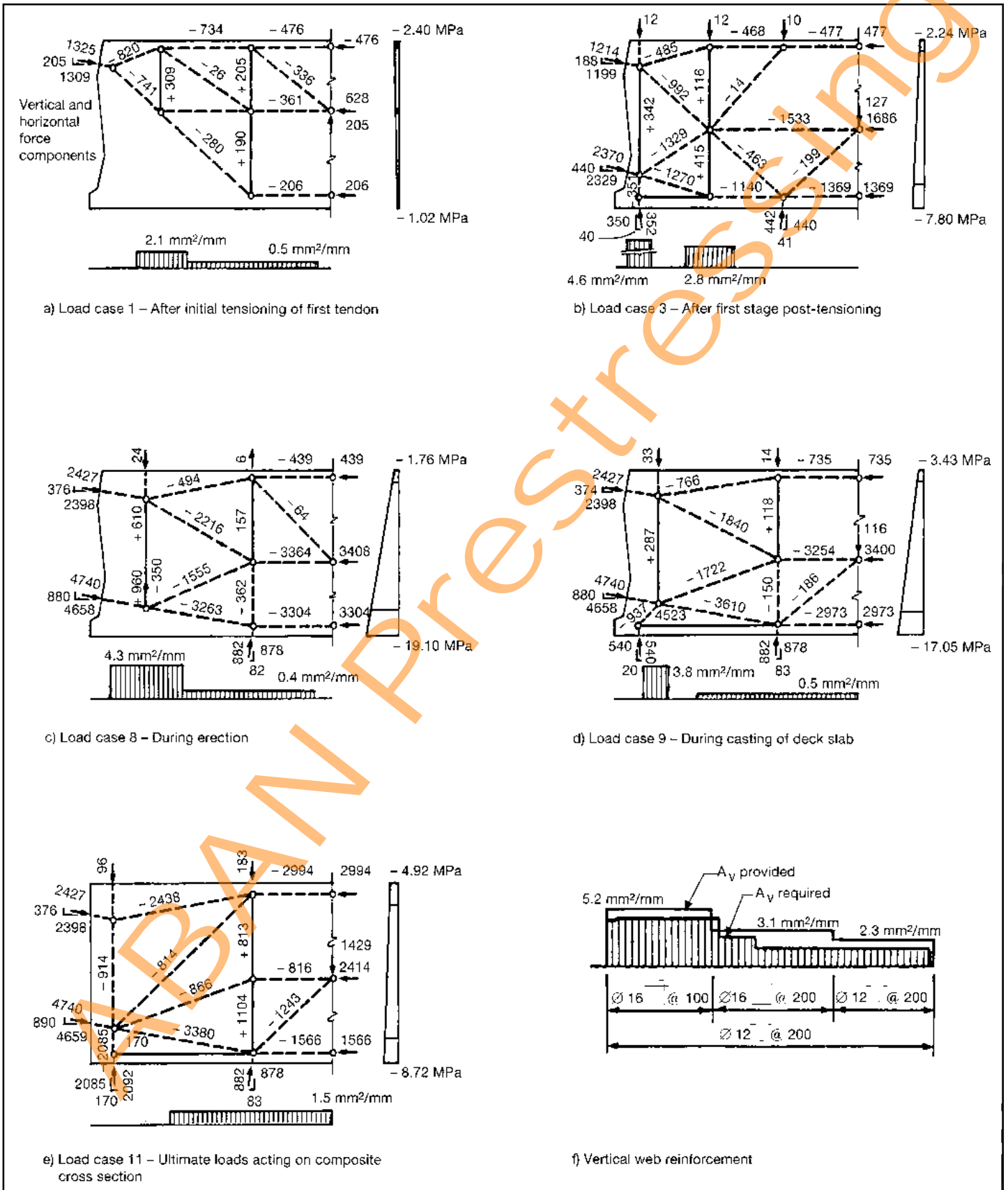
f) Vertical web reinforcement

Figure 5.3.7: Reinforcement in different regions of a member will be governed by various load cases and construction stages.
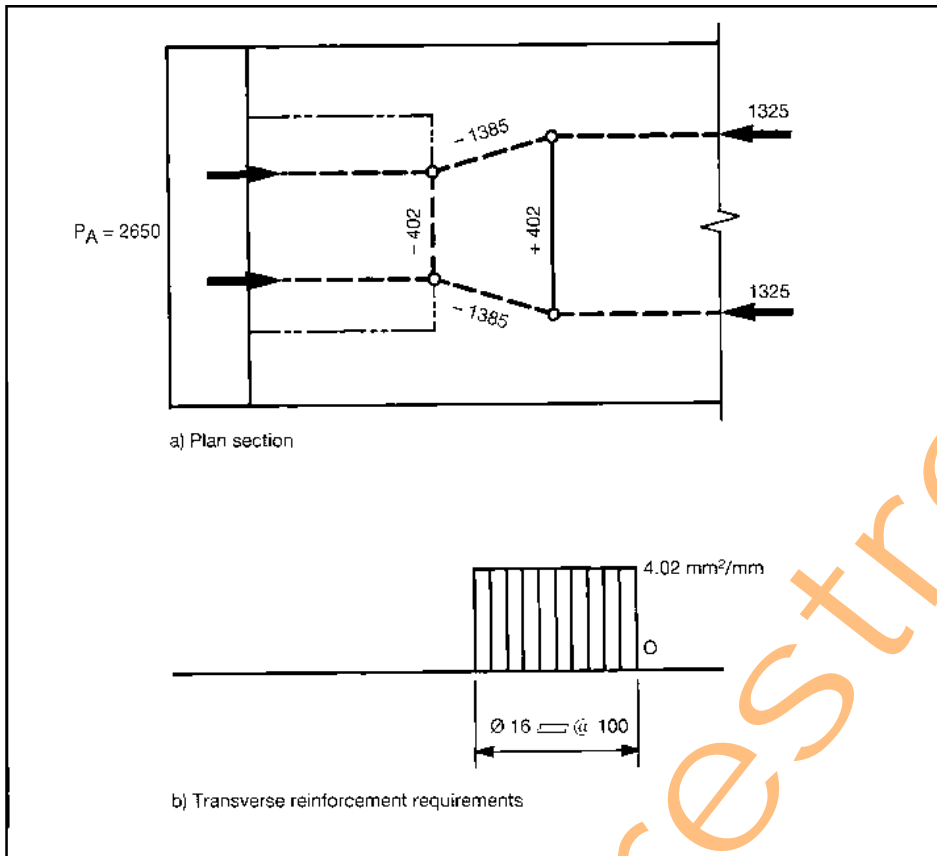
Figure 5.3.8: Reinforcement perpendicular to the plane of the web is governed by the anchorage with the largest edge distance.

represents one and a half panels of the overall strut-and-tie model. This is sufficient for the design of the end block. The stress distribution on the right face of the freebody diagram in Fig. 5.3.6 can readily be determined. Statically equivalent strut forces in the top flange, web and bottom flange are determined. Note that in general, due to the stress gradient, the forces act through the centroids of the respective stress blocks rather than at mid-depth of the flanges and web.

For the local zone reinforcement the standard VSL spiral was used. The outside spiral diameter was reduced slightly to accommodate a minimum edge distance and still maintain adequate concrete cover. By inspection, the amount of reinforcement is more than adequate. A standard VSL spiral is proportioned to anchor a force equal to 75 % of GUTS for the tendon with a concrete strength at the time of stressing equal to 80 % of the specified 28 day strength. The staged stressing sequence adopted did not

introduce the maximum force until the concrete reached the specified 28 day strength. The first stage stressing to 50 % of 75 % of GUTS with 50 % of specified concrete compressive strength is less critical than the final stage of stressing with the full specified concrete strength.

The analysis and design of the vertical web reinforcement is summarized in Fig. 5.3.7. The vertical stirrups are generally governed by temporary load cases 3 and 8 which occur during construction. A variety of different strutand-tie models are used since one model alone would deviate too far from the natural primary flow of forces for several of the load cases. The stirrup requirements are plotted together in Fig. 5.3.7 (f) along with the reinforcement actually used in the project. For working load cases (1 through 10) a working stress of 250 MPa is used to resist the tie forces which are spread over an appropriate length of girder. It is worth noting that in load case 11, the

ultimate load case, the post-tensioning forces have been used at their initial value before time dependent losses. While the losses do occur, the force in the posttensioning steel increases as the member strains under load. As a conservative design simplification, it is assumed that these two effects cancel each other under ultimate loads.

The post-tensioning introduces tension perpendicular to the plane of the web due to bundle flattening effects, and the transverse dispersion of force across the width of the beam.

The tension forces produced by bundle flattening effects at the curve in tendons B and C are small and can be resisted by the tensile strength of the concrete provided that the curve meets VSL standard minimum radius requirements and that the side cover to the duct is at least one duct diameter. In this design the radius of curvature is 7600 mm, and the side cover is 146 mm hence, no supplementary reinforcement is required.

The dispersion of force in the web is shown in Fig. 5.3.8 along with the required reinforcement. Tendon A has the largest edge distance and produces the largest tensions since the force must be deviated through the largest angle. Since reinforcement required for tendon A is fairly nominal, similar reinforcement is used for tendons B and C which can be enclosed within the same set of ties.

The dispersion of force in the top flange is dealt with in a similar manner. In this case, the position and magnitude of the force applied to the top flange is taken from Fig. 5.3.7. The resulting transverse tensions must be considered in conjunction with the transverse bending of the flange.

The reinforcement actually used for the project is detailed in Fig. 5.3.9.

5.4 Anchorage Blister

This design example is modelled after the anchorage blister used on the Western Bridge of the Storebaelt Project in Denmark. The project uses large precast box girders not unlike the one illustrated in Fig. 1.2. A portion of the longitudinal tendons are anchored in blisters located inside the girder at the web to flange junctions. The blisters used for the EC 6-22 anchorages at the web to bottom flange junction will now be discussed.
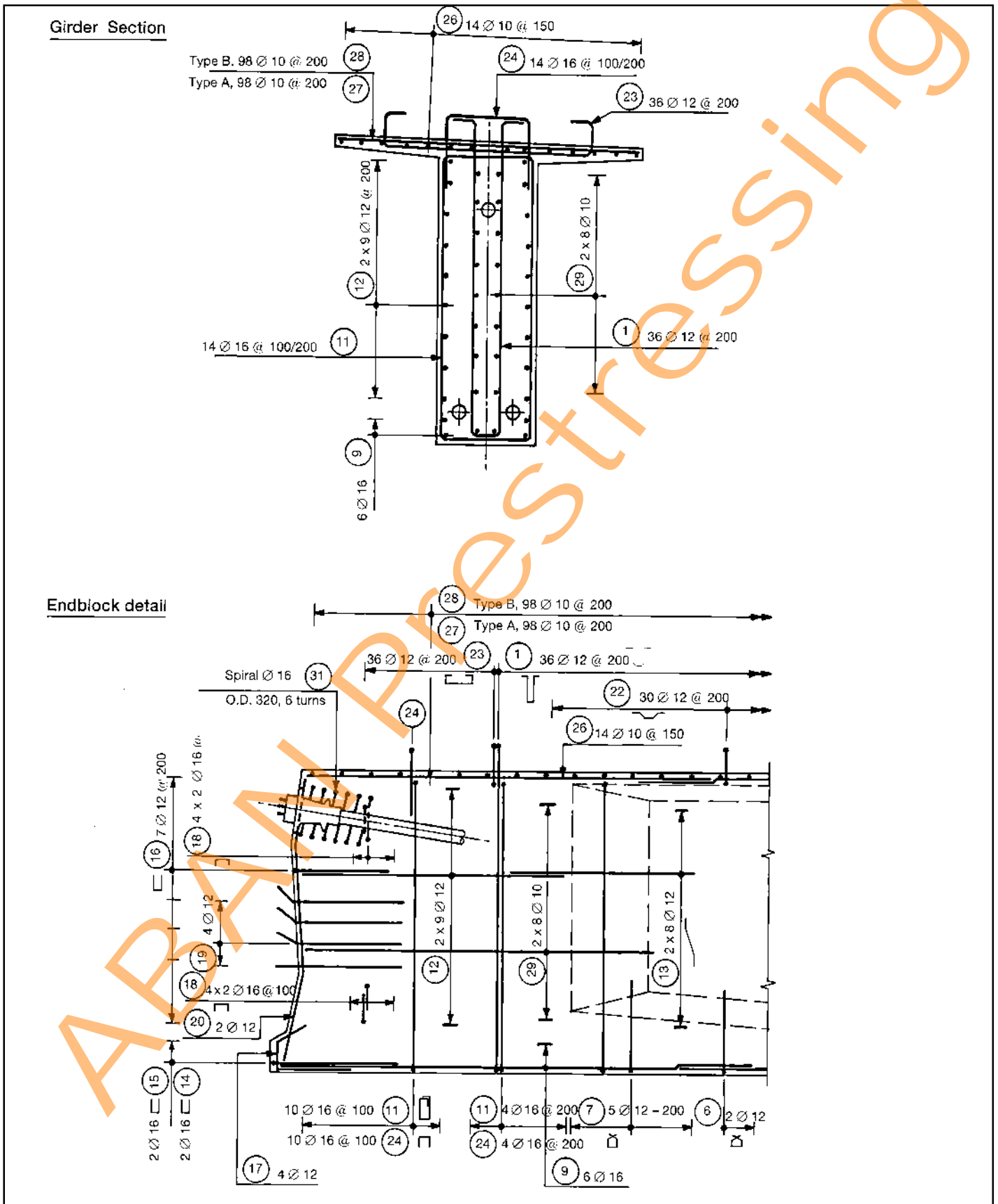
Figure 5.3.9: Clear detailing simplifies construction

Geometric Considerations

The geometry of the blister is based largely on constructional considerations. Minimum edge distances for the anchor body, minimum radius of tendon curvature and minimum tangent length should be respected. Finally, the geometry should provide sufficient clearance for the stressing jack.

Typical but simplified geometry for the blister in question is shown in Fig. 5.4.1. The actual blisters are provided with small draft angles on appropriate faces to facilitate form removal. A minimum tangent length of 1.5 m at the anchorage is used. To simplify the presentation, assume that the horizontal and vertical tendon curve zones coincide. (In general this need not be the case.) The 10.5° vertical deviation and 4° horizontal deviation result in a true deviation angle of 11.2°. Using a circular curve with a minimum radius of curvature of 6.8 m results in a curve length of approximately 1.34 m. Due to foreshortening in plan and elevation views, the horizontal and vertical curves will be slightly flatter than a circular curve and will have a

radius larger than the minimum radius. For design, the vertical and horizontal curves may be treated as circular curves with a length of 1.34 m and the relevant deviation angle. The concrete dimensions and tendon geometry are thus defined.

Applied Forces

One must determine all of the forces acting on the blister prior to conducting the detailed analysis. The maximum temporary jack force of 4810 kN (4081 kN after lock off) with a concrete cylinder strength of 30 MPa at the time of stressing (45 MPa at 28 days) governs the design. With respect to the longitudinal axis of the girder, the true longitudinal, vertical and transverse maximum jack force components are 4718 kN, 874 kN and 330 kN, respectively. Thus, in plan view the jack force is 4730 kN with the horizontal component of the deviation force due to tendon curvature being 248 kN/m (R = 19.06 m). In elevation view, the jack force is 4798 kN with the associated tendon curvature force being 660 kN/m (R = 7.27 m).

The longitudinal reinforcement required to prevent cracking behind the anchorage deserves specific discussion as this results in a "force" acting on the blister. Normally providing reinforcement to anchor back one quarter of the longitudinal jack force is sufficient to control cracking. The desired tie back force of 1180 kN would require some 4700 mm2 of longitudinal reinforcement in the web and flange adjacent to the blister. The presence of longitudinal compression from behind the blister (due to other stressed blisters) reduces the longitudinal reinforcement requirements. A simplified calculation model based on a 45° spread of force from the previous blister is given in Fig. 5.4.2. The net tie back force of 320 kN would require some 1280 mm2 of reinforcement (p = 0.0016). This is less than minimum distributed shrinkage and temperature reinforcement hence no additional reinforcement is required. For the statical calculations, tie back forces of 160 kN in the web, and 160 kN in the flange may be used. The forces act in the plane of the first layer of the reinforcement and in line with the point of application of the jack force.
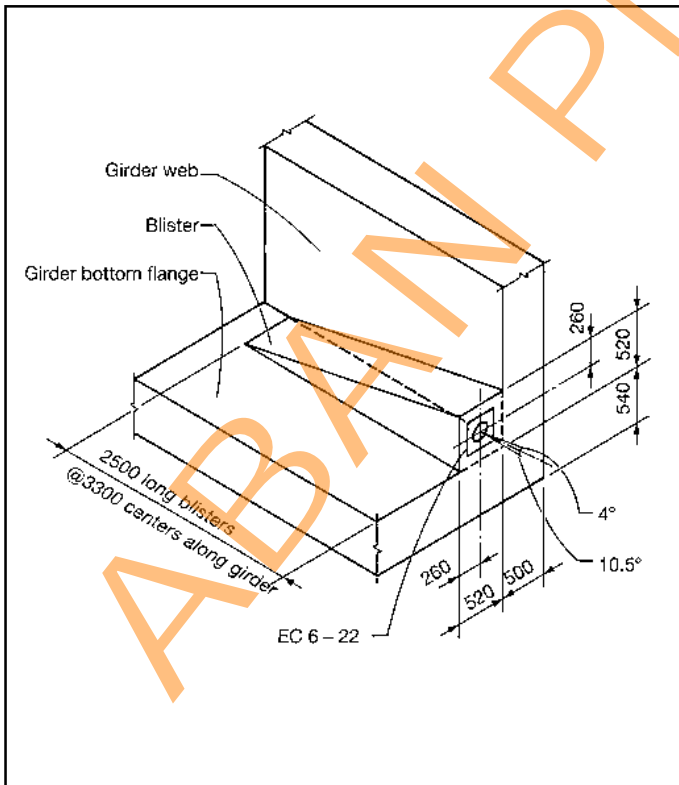

Figure 5.4.1: A series of blisters can be used to anchor longitudinal tendons in a box girder.
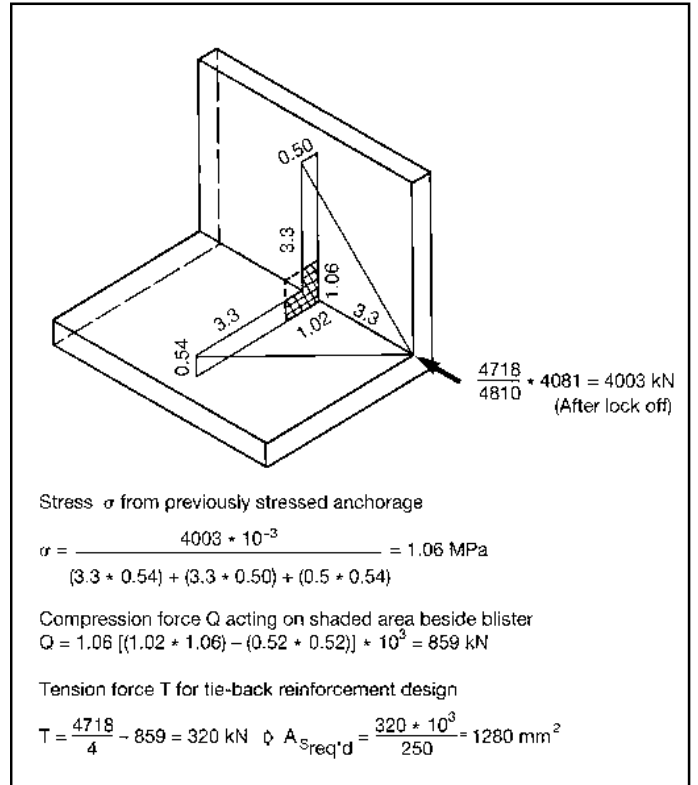

Stress σ from previously stressed anchorage

$$\sigma = \frac{4003 * 10^{-3}}{(3.3 * 0.54) + (3.3 * 0.50) + (0.5 * 0.54)} = 1.06 \text{ MPa}$$

Compression force Q acting on shaded area beside blister
$$Q = 1.06 [(1.02 * 1.06) - (0.52 * 0.52)] * 10^3 = 859 \text{ kN}$$

Tension force T for tie-back reinforcement design

$$T = \frac{4718}{4} - 859 = 320 \text{ kN} \quad \Diamond \quad A_{s_{req'd}} = \frac{320 * 10^3}{250} = 1280 \text{ mm}^2$$

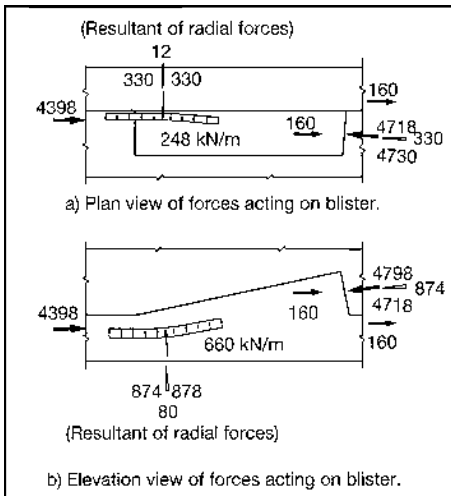Figure 5.4.2: Precompression behind blister helps to reduce tie-back reinforcement.

Figure 5.4.3: All forces acting on a blister must be considered for a consistent analysis and design.

Figure 5.4.4: A simple graphical analysis is sufficient for dimensioning and detailing reinforcement

Figure 5.4.3 shows all of the forces acting on the blister. The magnitude and position of the longitudinal force at the left end of the blister are determined from equilibrium of the force system.

Analysis

The blister may be conveniently analyzed graphically as shown in Fig. 5.4.4. A few comments on the analysis are warranted. Since it was desired to use ties perpendicular to the tendon immediately in front of the anchorage, the tension ties in the statical model (members 1-2) were given this inclination. Fixing the position of point A then fully defines the geometry of the strut-and-tie model.

A few iterations were required to arrive at a satisfactory solution. The subtleties of the problem can best be appreciated by attempting to duplicate the analysis. For example, the tendon curvature deviation forces cause the compression struts to deviate gradually thus forming compression arches.

Reinforcement

Reinforcement for the blister is detailed in Fig. 5.4.5. The local zone is reinforced with orthogonal ties which extend far enough that the last sets provide the necessary tension tie between points A and B in Fig. 5.4.4.
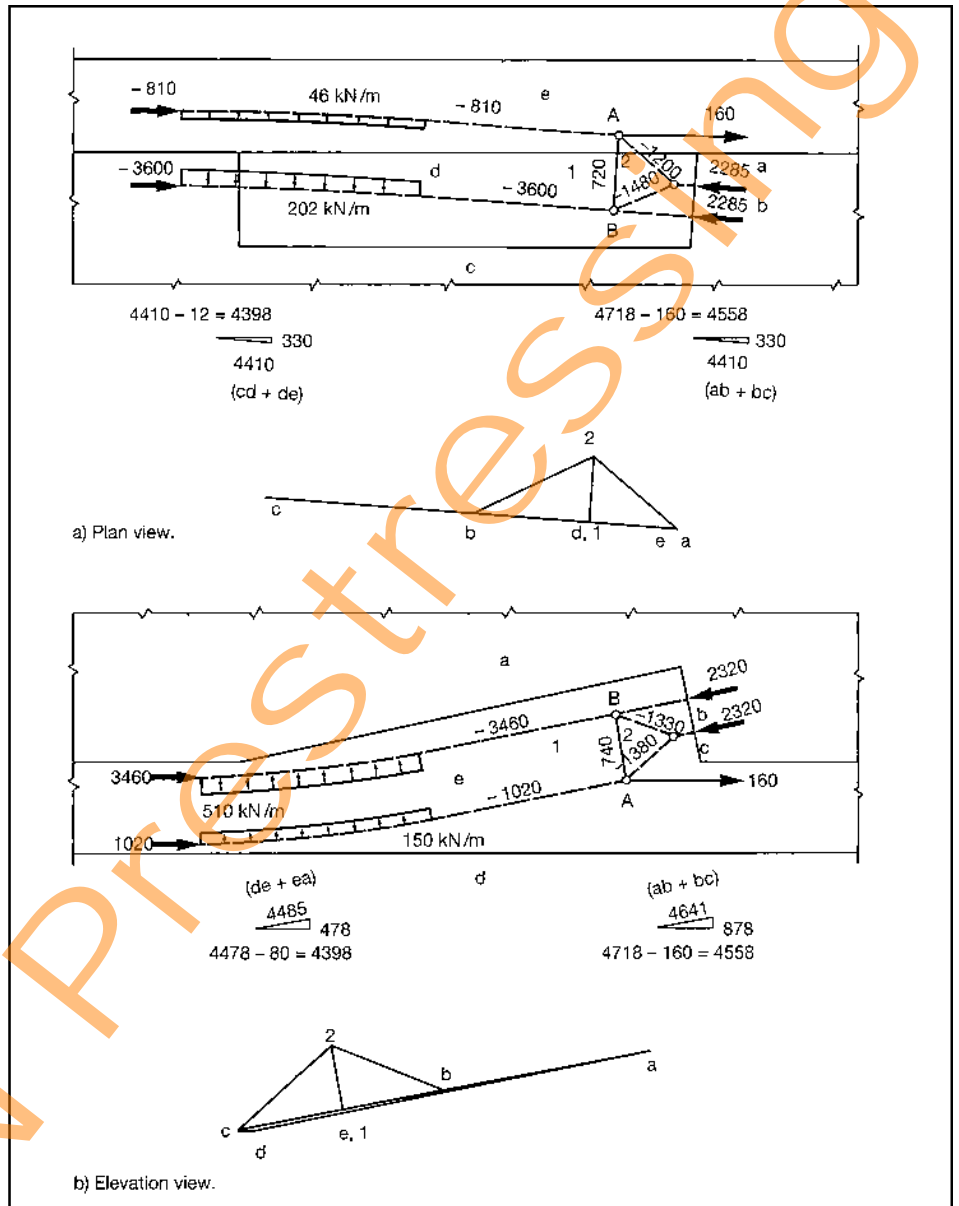
To reduce congestion in the local zone the spiral has been omitted and replaced with orthogonal ties which provide sufficient confinement. Each tie set consists of four hair pin bars: a smaller inner hair pin with a larger outer hair pin in one plane with legs vertical, and a similar pair in a second plane with legs horizontal (see Fig. 5.4.6). This bundling arrangement provides maximum clearance for concreting and reduces the reinforcement diameter to a size where tight bending

tolerances are achievable. The tangent zone between the local zone and the tendon curve requires only minimum reinforcement. The tendon curve zone requires ties to prevent the tendon from tearing out of the concrete. The tie design forces must be at least as large as those required to mobilize the outer arched struts shown in Fig. 5.4.4 (i.e. 46 kN/m and 150 kN/m). In theory the inner arched struts can receive the necessary radial forces by direct

compression from the inside of the tendon curve. In the elevation view, the relative position of the inner arched strut and the tendon does not permit the complete force transmission by compression to all portions of the strut cross-section. For simplicity, the tie design is based on 100% of the tendon curvature force. For similar reasons the vertical legs of some of the $\varnothing$ 12 hair pin bars are moved closer to the tendon.

Selection of a hair pin arrangement with straight legs facilitates the insertion of these bars after the basic web and flange reinforcement is in place. Bar development lengths must be checked. In this example the selection of small bar diameters assists in achieving adequate development without hooks. For clarity, longitudinal reinforcement has been omitted from Fig. 5.4.5. There would be longitudinal bars within the blister in the corners of the ties to facilitate placing and securing the reinforcement. The necessary longitudinal tie back reinforcement would be part of the typical web and flange longitudinal reinforcement.

Concrete Strut Check

Concrete stresses should be investigated. Except for the local zone, the strut to the left of point B in Fig. 5.4.4 is the worst case. From the edge distances, the maximum strut dimensions available are approximately 340 mm x 400 mm. The resulting stress is approximately 21.3 MPa or 71 % of the concrete cylinder strength. This is considered acceptable because of the following reasons:

1. Actual concrete strengths are confirmed by testing prior to stressing. Stressing is not allowed to proceed if the concrete is under strength. A safety margin to allow for under strength concrete is not required.

2. The maximum jack force is a very temporary load case lasting for at most a few minutes. The anchorage force after lock off is 15 % (4081 kN vs 4810 kN) lower than the design force considered in this analysis. A safety margin to allow for long duration of loading at the maximum stress is not required.

3. The maximum jack force is limited by the jack capacity, hence an accidental overload is not possible.

4. The blister receives confinement and support from the web and flange. The stress/strain gradient effect increases the concrete strength and improves safety.

5. If a failure did occur during stressing, the failure would be benign. It would be preceded by warning cracks, and the girder would return to the temporary supports.

6. After the tendon is grouted, a different load path is possible via the bonded prestressing steel so that the blister is redundant. 7. For construction load cases, factors of safety lower than those for in service load cases are accepted practice. (Safe working practices are of primary importance for construction safety.) 8. The 28 day specified concrete strength is 45 MPa. The concrete stress in service will be less than 40 % of the specified concrete strength.

Concrete Local Zone Check

The check of the local zone confined by orthogonal reinforcement will now be discussed. Figure 5.4.6 provides an end view of the blister showing how the concrete is confined. The girder web and flange with their substantial reinforcement provide confinement to two faces of the blister. Take the upper left hand quadrant as the worst case and assume that the other three quadrants are similar. This approach is conservative but is sufficient to demonstrate that the reinforcement is acceptable. The simplified gross confined core area is thus (216 * 2) * (216 * 2) = 186624 mm2. Deducting the 115 mm diameter duct area plus the unconfined parabolic areas (assume rise to span ratio of 8), one gets a net confined core area

$$= 186624 - \pi (115)\,2/4 - 4 * \frac{2}{3} * \frac{(216 * 2)^2}{8}$$

$$= 114029 \text{ mm}^2$$

The tie sets are spaced at 100 mm centres resulting in further arching and a reduction in net confined area. The clear space between tie sets is 72 mm and with an 8 to 1 parabola, the boundaries of the confined core move inward 9 mm from those at the ties. The final net confined core is thus approximately

114029 - 4(216 * 2) (9) = 98477 mm^2. The average axial compression stress in the

net confined core for a maximum realizable tendon force of 95% GUTS is 5538 kN * 10^3 / 98477 = 56.2 MPa. The unconfined concrete strength is 0.85 * 30=25.5 MPa leaving 56.2-25.5 = 30.7 MPa to be provided by confinement reinforcement. Since 1 MPa of lateral confinement stress produces about 4 MPa of axial capacity, 30.7/4.0 = 7.67 MPa of lateral confinement stress is required. Returning to the upper left hand quadrant there are 4 $\varnothing$ 14 bar legs providing lateral stress to an area 100 mm by approximately 200 mm. For steel stresses of 250 MPa the resulting lateral confinement stress provided to the concrete is (4 * 154 * 250)/(100 * 200) =7.7 MPa. Since this is greater than the required confinement of 7.67 MPa, the local zone is acceptable. For reasons discussed when checking compression strut stresses large safety margins for this load case are not warranted. In any event, the calculations presented are quite conservative. More refined calculations would predict more capacity.

Shear Check

As a final calculation, the shear friction capacity of the reinforcement in the blister may be checked. On principle, because a strut and tie model (lower bound solution) was used, there is no need to do a shear friction calculation (upper bound solution). It is however a simple check on the overall design and is worth doing.

The shear friction check postulates a failure whereby the blister slides forward on the face of the web and top of the bottom flange (i. e. on straight sides of shaded zone in Fig. 5.4.6). The driving force for the failure is 4718 kN, the longitudinal component of the jack force. The other components of the jack force, 874 kN and 330 kN act as clamping forces on the sliding surfaces. These forces taken together with the 10124 mm2 of reinforcement which cross the failure (sliding) surfaces provide the normal force for the shear friction resistance. For failure to develop, the reinforcement must yield hence a stress of 400 MPa is used in the reinforcement. The coefficient of friction for cracks in monolithic concrete is generally taken as about 1.4, thus the failure capacity is

[874 + 330 + (10124 * 400 * 10^{-3})] * 1.4
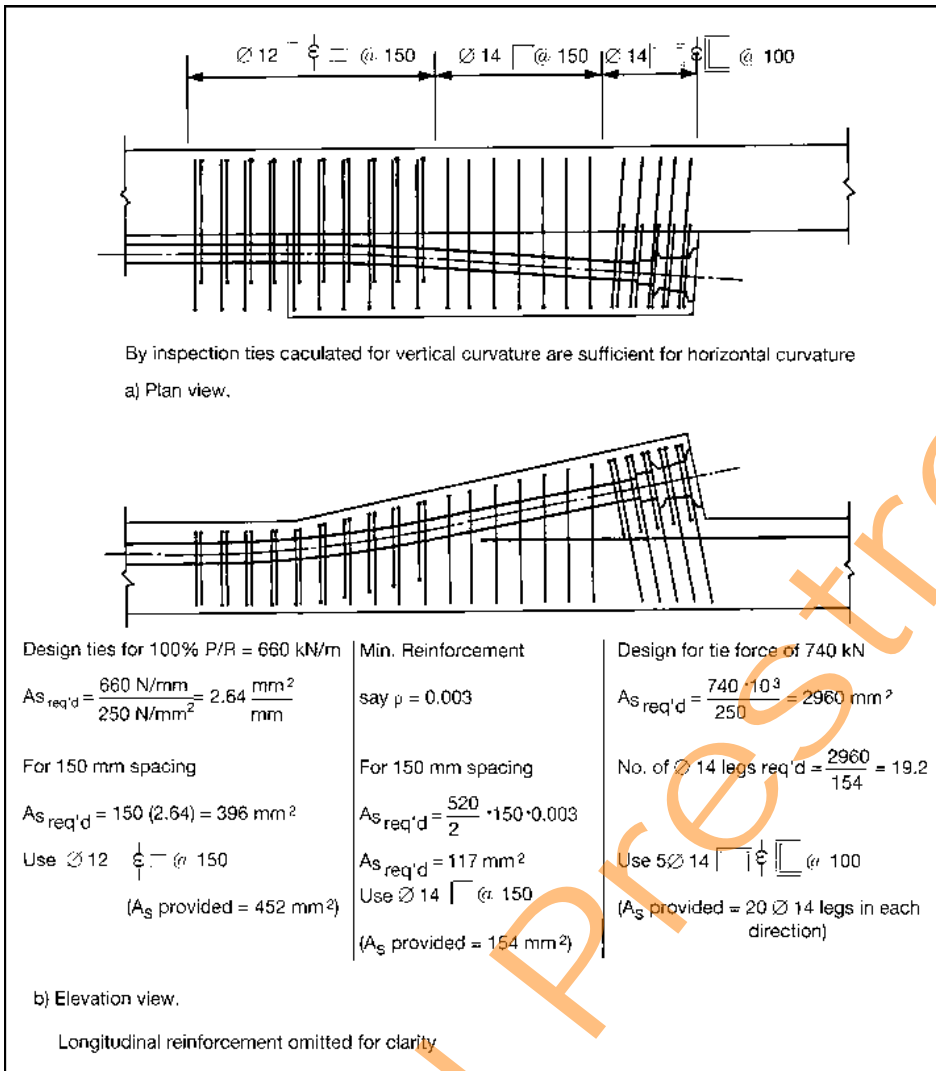
$$= 7355 \text{ kN}$$

By inspection ties caculated for vertical curvature are sufficient for horizontal curvature

a) Plan view.

Design ties for 100% P/R = 660 kN/m	Min. Reinforcement	Design for tie force of 740 kN
$As_{req'd} = \dfrac{660 \text{ N/mm}}{250 \text{ N/mm}^2} = 2.64 \dfrac{\text{mm}^2}{\text{mm}}$	say $\rho = 0.003$	$As_{req'd} = \dfrac{740 \cdot 10^3}{250} = 2960 \text{ mm}^2$
For 150 mm spacing	For 150 mm spacing	No. of $\varnothing$ 14 legs req'd $= \dfrac{2960}{154} = 19.2$
$As_{req'd} = 150\,(2.64) = 396 \text{ mm}^2$	$As_{req'd} = \dfrac{520}{2} \cdot 150 \cdot 0.003$	
Use $\varnothing$12 $\,\square\,$ @ 150	$As_{req'd} = 117 \text{ mm}^2$	Use 5$\varnothing$14 $\,\square\,$ @ 100
(A_s provided = 452 mm²)	Use $\varnothing$14 $\,\square\,$ @ 150	(A_s provided = 20 $\varnothing$ 14 legs in each direction)
	(A_s provided = 154 mm²)	

b) Elevation view.

Longitudinal reinforcement omitted for clarity

Figure 5.4.5: Blister reinforcement should be detailed for ease of steel and concrete placement.

Figure 5.4.6: Sifficient orthogonal ties can lead to the deletion of spiral reinfocement.

This is substantially greater than 4718 kN, the force which must be resisted. Note that other more "correct" three dimensional curved shear friction surfaces are possible. When analyzed rigorously with the theory of plasticity, taking into account three dimensional effects, the resistance will be higher than predicted by the simple shear friction calculations done in this example.

Other Considerations

The calculations presented are sufficient for the blister itself. One should not overlook the general zone problem of dispersing the force from the blister into the entire girder cross section. In par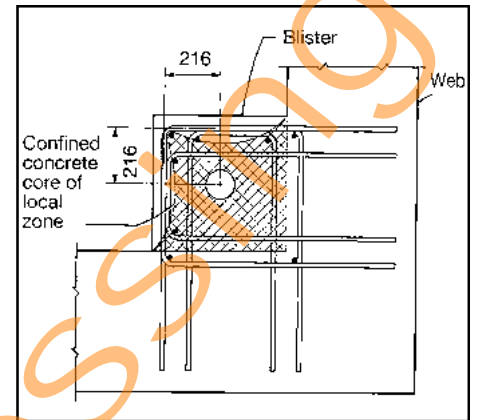ticular, in zones where blisters occur in both the top and bottom flanges the webs are subjected to stresses which increase diagonal cracking. Podolny [25] discusses this problem. Menn [26] has proposed increasing the design shear force in these zones by 20 % of the jacking force as a practical means of overcoming this problem.

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