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Contents

CASES OF DAMAGE TO PRESTRESSED CONCRETE

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Preface

Cases of damage which occurred in prestressed concrete structures are reviewed in this publication. Besides discussing the actual damage, it describes the correct method of construction or reports the information yielded by investigation into the problems concerned. In most cases the damage was attributable to incorrect structural detailing, whether or not in combination with other causes. Some readers will perhaps be astonished at the "mistakes" which have been made and declare that such things could never have happened to them. The examples of damage described in this report however, have been taken from a large number of structures built by different contractors for different clients. Most of the "mistakes" were due to the fact that those responsible for design or construction, while acting in complete good faith, failed to recognise the problem involved. If successful results alone are reported and failures kept secret, there will be a considerable risk that others will commit the same errors. This would certainly not be to the advantage of the development of prestressed concrete technology. It is therefore a gratifying fact that a large number of concrete construction engineers have co-operated in supplying the information for the present publication.

In view of what has been said above, it would be helpful if designers could inspect their structures after several years, so that they, and others, could learn even more from their findings. It would thus become possible further to improve the quality of prestressed concrete structures.

Acknowledgment

This study has been carried out at the Institute TNO for Building Materials and Building Structures – IBBC, at Delft, Holland, and it was supported by the Netherlands Committee for Concrete Research CUR. Especially the CUR-working-Group C 13 was involved in the investigation and the members of this group were of great assistance in collecting the several cases of damage discussed in this publication.

Ir. A. C. van Amerongen translated the original Dutch CUR-report no. 38: „Schadegevallen bij voorgespannen beton" (1969).

SCHADEGEVALLEN BIJ VOORGESPANNEN BETON

Samenvatting

In dit rapport worden een aantal schadegevallen besproken, die bij het bouwen in voorgespannen beton zijn opgetreden. Het blijkt, dat vooral de constructiedetails – al of niet in combinatie met andere factoren – vaak aanleiding tot schade hebben gegeven. In veel gevallen had dit eenvoudig voorkomen kunnen worden. Velen hebben zich echter niet altijd gerealiseerd, dat de goede eigenschappen van voorgespannen beton alleen gelden in de voorspanrichting. Hierdoor werd onvoldoende rekening gehouden met die trekspanningen, die niet door de voorspanning worden opgeheven.

Alle schadegevallen zijn, na de inleiding en probleemstelling, zo goed mogelijk in 5 hoofdstukken ondergebracht, waar achtereenvolgens ter sprake komen:

Hoofdstuk 2. Balken belast via consoles
Hoofdstuk 3. Opleggingen
Hoofdstuk 4. Spanningen bij eindveranderingen
Hoofdstuk 5. Dwarskrachtproblemen
Hoofdstuk 6. Diverse oorzaken van schade (o.a. slechte uitvoering, vorstschade, ongevallen bij de montage en corrosie).

Voor zover dit mogelijk is wordt steeds, naast de bespreking van een schadegeval, een betere constructie aangegeven. Wanneer van de betreffende punten researchresultaten bekend zijn, worden deze ook vermeld.

In het laatste hoofdstuk zijn een vrij groot aantal betrekkelijk los van elkaar staande gevallen besproken. De bestrijding van corrosie is in het algemeen genomen uiteraard een veel omvattend onderwerp. In dit rapport worden hieromtrent een aantal waarschuwingen gegeven.

CASES OF DAMAGE TO PRESTRESSED CONCRETE

Summary

In this report a number of cases of damage which occurred in prestressed concrete construction are discussed. It appears that especially the structural details – whether or not in combination with other factors – have often caused damage. In many cases it would have been a simple matter to avoid this. Many engineers have, however, failed to realise that the good properties of prestressed concrete exist only in the prestressing direction. Because of this, not enough account was taken of those tensile stresses which are not cancelled by the prestress.

After the introduction and the statement of the problem, all cases of damage have as far as possible been assigned to five chapters dealing with the following subjects:

Chapter 2. Beams loaded through corbels
Chapter 3. Bearings
Chapter 4. Stresses at end anchorages
Chapter 5. Shear problems
Chapter 6. Various causes of damage (including poor workmanship, frost damage, mishaps during erection, and corrosion)

Where possible, every discussion of an instance of damage is accompanied by an indication as to how a better type of construction could have been obtained. Whenever the results of research relating to the points in question are known, they are likewise mentioned.

In the last chapter a fairly large number of relatively unconnected cases are discussed. Combating corrosion is, in general, a very comprehensive subject. Some warnings relating to this are given in the present report.

Cases of damage to prestressed concrete

1 Introduction

Our ancestors must have had few problems in building their wattle and daub huts, and any difficulties of construction that may have arisen were left to their womenfolk to solve anyway. Building was largely a matter of "acquiring the knack". Indeed, no very great importance was attached to achieving sound construction. If the hut collapsed, due to whatever cause, it could be rebuilt in a day.

The art of building developed gradually, largely as a result of trial and error. The early master builders did not, it is true, have to work under such pressure, but because of the limited possibilities open to them they were nevertheless at a considerable disadvantage in comparison with their modern counterparts. This is one reason why many of the great structures of the past are still very much admired from the viewpoint of technical achievement as well.

The appreciation that earlier generations felt for good construction was evident from the fact that they were only too pleased to copy the successful work of their forbears. The principle of an eye for an eye and a tooth for a tooth provided the guarantee that this was done with due care.

By degrees the knowledge of construction materials and structural mechanics increased to such an extent that the designers dared to build structures of a less traditional kind. Whereas the economics of building construction had formerly in many cases been a more or less subordinate factor, it became increasingly important in later periods.

Building thus evolved into an art which required that a structure should be so designed that it would fulfil certain – minimum – requirements as to soundness and that, in addition, it would be possible to build it as cheaply and often also as quickly as possible.

In this process of evolution the designer will evidently be on the lookout for new materials and possibilities. Of course, the "novelty" that is put directly into practice involves the risk of turning out to be a failure. Such failure may, on the one hand, be due to the fact that too much is demanded of the new materials without accurate knowledge of how they will behave in the long run. On the other hand, it may be that certain problems have not been recognised for what they are. A very common snag, for example, is that the importance of careful structural detailing is underrated. In most cases the damage must be attributed to incorrect structural detailing.

As is generally known, concrete has a high compressive strength and a relatively low tensile strength. Because of this low tensile strength, flexurally loaded structural members are affected by cracking and thus by brittle fracture at relatively low values of the loading. At that stage the concrete in the compressive zone is as yet loaded to only a fraction of its available compressive strength and is therefore very incompletely utilised. For this reason plain (unreinforced) concrete is not a good construction material for members subjected to flexural loading, the more so as the tensile strength cannot be determined accurately for various reasons including the occurrence of shrinkage.

A much more favourable condition is achieved by the provision of reinforcing bars in the tensile zone. When the concrete has cracked, these bars undertake the function that this zone performed before cracking occurred. The combination of steel and concrete – reinforced concrete – offers great possibilities, but cracking remains a problem. Prestressed concrete constitutes a further development. Prestressing defers the occurrence of cracking, and for this reason the concrete continues to behave longer as a more or less homogeneous material.

It has, however, often not been properly realised that this homogeneity actually exists only in the direction of prestressing. Because of this, in a number of cases trouble has resulted from a failure to take account of those tensile stresses which are not cancelled by the prestress. This may, for example, occur whit loads acting in a very localised manner or at discontinuities in structural shape. For a time, indeed, the prevalent view was that – except for end-block reinforcement at the tendon anchorages – all untensioned reinforcement in prestressed concrete beams was superfluous. Many instances of structural damage have arisen particularly from this neglect or failure to appreciate these tensile stresses.

Quite often it is relatively difficult to calculate these stresses at all reliably. For this reason it has not proved possible, in this report, to present an appropriate theory to fit every problem encountered. However, in most cases it is possible, on the basis of some general insight into the nature of the problem, to decide what is a good form of construction even though the magnitude of the tensile stresses in question is not accurately known.

This report reviews cases of damage which occurred in prestressed concrete construction. Frequently damage arises from a situation where errors of structural design occur in conjunction with poor workmanship. In cases where a particular aspect gives rise to problems whose causes lie both in the design and in the execution of the structure these causes will be discussed together in this report. This is quite logical, since the designer, too, must take account of the practicability of the design in terms of actual execution. In addition to an examination of damage, examples of sound construction are also presented in the report, or insight yielded by investigations is reported.

2 Beams loaded through corbels

In the preceding chapter it has been noted that the majority of cases where damage occurred have been directly attributable to the absence of reinforcement in regions where tensile stresses were acting. As will appear from the following treatment of the subject, a factor of great importance is how the loads are applied to the structure. In the case of a normal prestressed simply-supported beam the loading can in general be applied in any of these ways:

1. uniformly distributed loading – over the entire length – applied to the top of the beam;
2. loading by concentrated loads likewise applied to the top of the beam;
3. loading applied through corbels (brackets);
4. loading applied by means of holes formed in the beam.

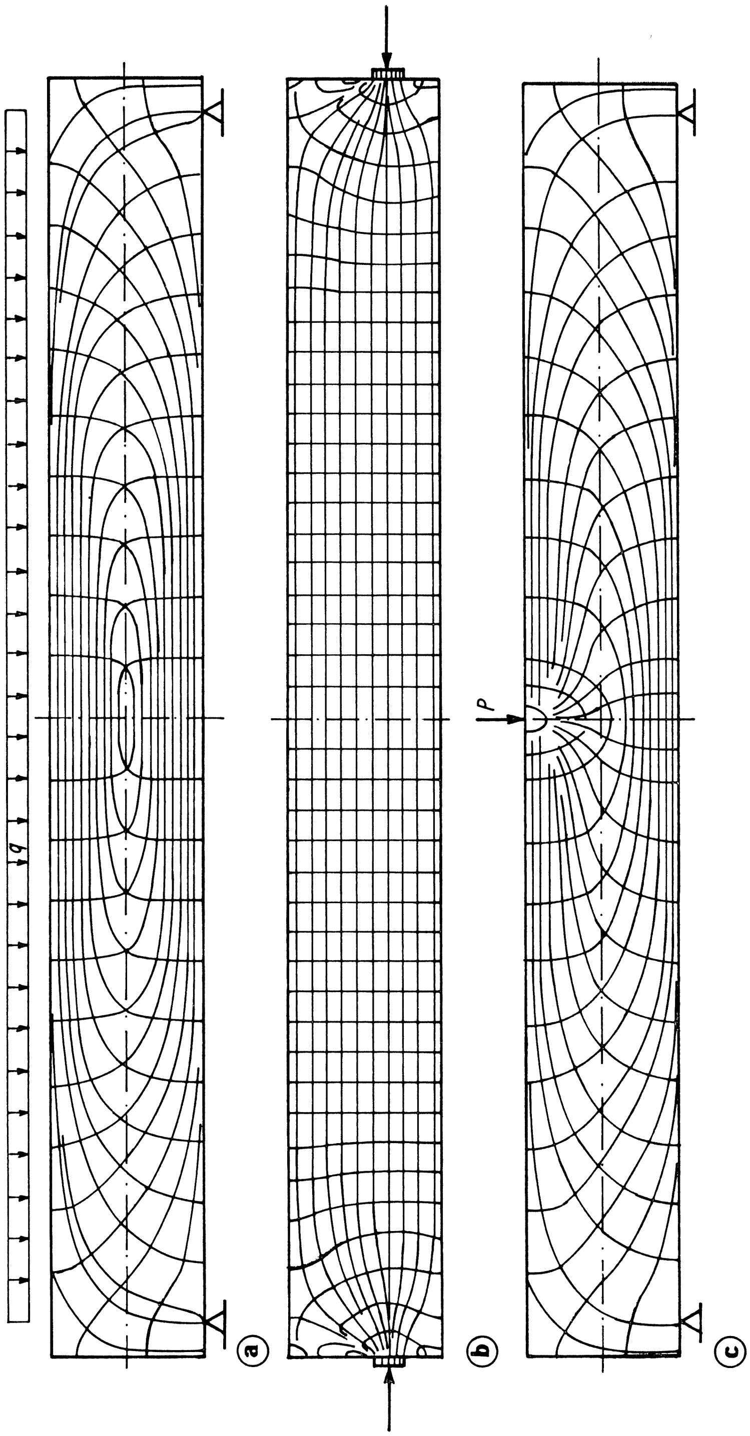
To convey a clear understanding of the stress conditions associated with the load application, the stress trajectories for the four above-mentioned cases are shown in the accompanying diagrams.

In the case of a uniformly distributed loading on the top of the beam (Fig. 1a) a regular pattern of the trajectories is found to exist. In the central part the principal stresses act in the longitudinal direction of the beam. It is possible, without much difficulty, to cancel nearly all the tensile stresses by means of a prestress in the longitudinal direction (Fig. 1b). A disturbance occurs only in the vicinity of the bearing reaction and at the end anchorage of the tendons which will be further considered later on in this report.

In the case of concentrated loading (Fig. 1c) the pattern presented by the trajectories is already rather less regular. The disturbances that manifest themselves in this case occur mainly in the compressive zone. In general they do not give rise to difficulties, however. But if the loading is applied through corbels (Fig. 1d) or by means of holes (Fig. 1e), then a considerable disturbing influence is exercised in the vicinity of the point of load application. Tensile stresses are produced which are not eliminated by a normal prestress. From the stress trajectories pattern in Fig. 1d it appears that a large proportion of the force exerted by the corbel is, as it were, "suspended" from the compressive zone, so that tensile stresses occur in horizontal sections above the corbel.

Essentially the same phenomenon occurs in beams to which the loading is applied by means of holes formed in them. However, since the loading must in this case be directed "around the hole", an even more unfavourable state of stress develops here.

Clearly, in these two last-mentioned cases it is necessary to take precautions to transmit the loading to the compressive zone. These will take the form of reinforcement or a prestress. That the omission of such precautions may have disastrous consequences is illustrated by Fig. 2. The beams that had been used in this structure had contained prestressing tendons only. This beam failed as a result of "overloading" by snow. The bending moment at the instant when this happened was substantially lower than the anticipated failure moment. This case of structural damage led to a laboratory investigation into the behaviour of such beams. It was found that beams loaded through corbels behave in the manner of beams to which the loading is applied on top, provided that arrangements are made to ensure that the corbel loading can always be transmitted to the compressive zone. Later on, the correct detailing of the beam in the vicinity of the corbels will be discussed. It is of interest now first to consider what consequences the omission of a "suspension reinforcement" at corbels is liable to have. From the laboratory investigation, which comprised full-size beams as well as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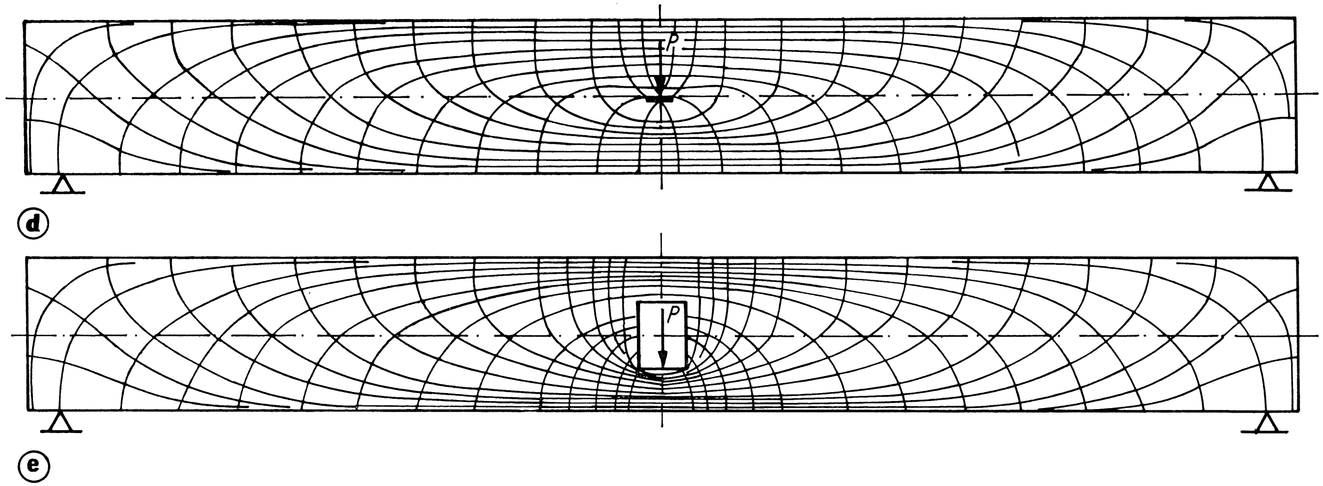

Fig. 1. Stress trajectories
(by definition the trajectories indicate only the direction of the principal stresses)
a. beam with uniformly distributed loading
b. beam subjected to prestress only
c. beam with a point load at mid-span
d. beam loaded through a corbel at mid-span
e. beam loaded by means of a hole.

Fig. 2. Damage caused by absence of stirrup reinforcement in beams loaded through corbels (1963).

scale models of beams, it was possible to study the phenomena in a comparatively leisurely manner.* Two cases are to be distinguished, namely:

1. the beams are provided only with prestressing tendons; there is no suspension reinforcement and no normal stirrup reinforcement;
2. in addition to the prestressing tendons the beams contain normal stirrup reinforcement, but no suspension reinforcement.

In the first case the consequences, for both cross-sectional shapes investigated, were

*) CUR-report no. 39: "The behaviour of prestressed concrete beams, more particularly of beams loaded through corbels and beams provided with holes". "Betonvereniging", Postbox 61, Zoetermeer, The Netherlands.

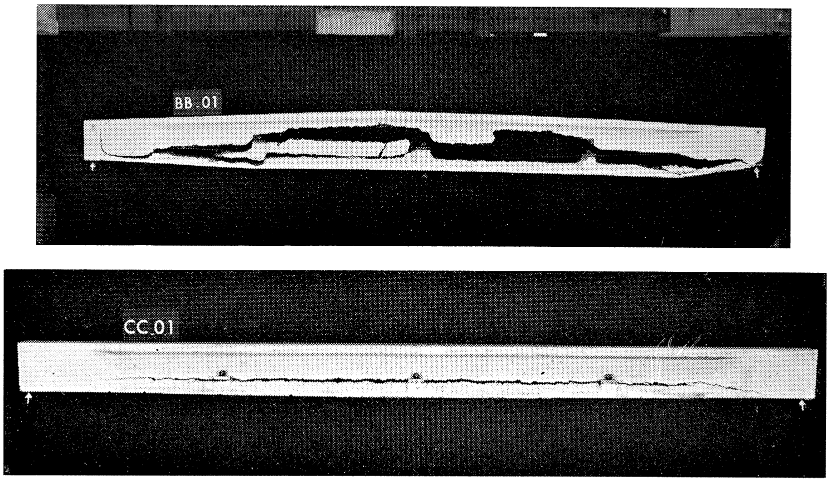

Fig. 3. Two beams which were loaded through corbels and failed in consequence of the complete absence of stirrup reinforcement.

disastrous. This clearly emerges from Figs. 3 and 4. Just as in the actual instance of damage illustrated in Fig. 2, at a particular instant the bottom flange becomes completely detached from the rest of the beam (see Fig. 5). The bending moment at which this happens is, however, relatively far below the anticipated failure moment of the beam section and generally corresponds to the cracking moment. Detachment of the bottom flange takes place very abruptly, without any previous warning. From the investigation of the beams with corbels and from the strain measurements performed in this context it became clearly apparent what actually happens in such circumstances.

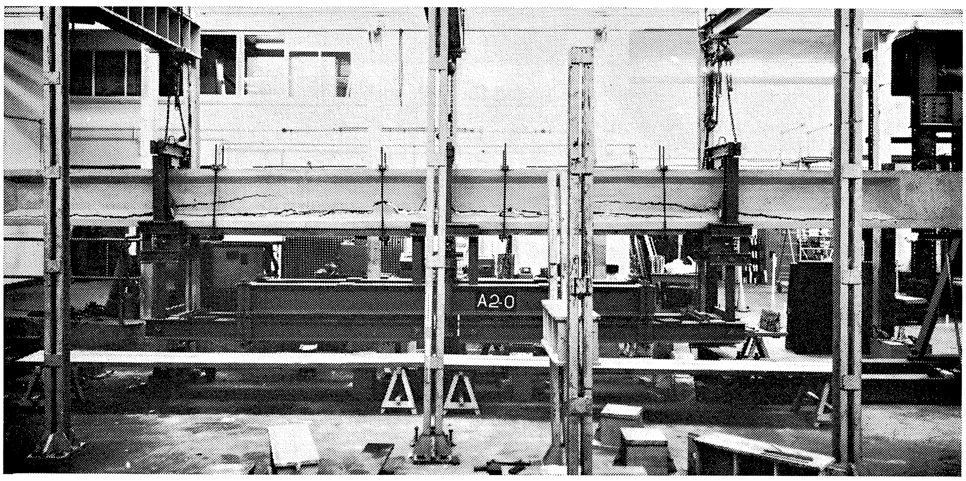

Fig. 4. Beam loaded through corbels and containing no stirrups. The photograph shows the condition on reaching the cracking moment.

Fig. 5. Detached bottom flange of a beam loaded through corbels and containing no stirrups.

In the uncracked concrete a tensile stress occurs in the part of the web above the corbel, as indicated in Fig. 6a. As a result of the vertical flexural cracks which develop on each side of the corbel (Fig. 6b), the region through which the tensile force has to be transmitted is much reduced. At a certain instant the corbel thus has so little horizontal concrete section available to transmit the load acting on the corbel that the tensile strength of the concrete is exceeded. At the level of the top of the corbel a crack develops in the web (Fig. 6c) and spreads with "explosive" rapidity. The beams investigated, in which the depth of the corbels was approximately 0.3 times the beam depth, failed in this way at 65–70% of the failure moment. In certain cases these phenomena can cause failure even before the cracking moment of the beam is attained.

In general, the weakening envisaged here should not be regarded solely as a function of the bending moment. The magnitude of the corbel load itself also plays a major part. The percentages 65–70% have been mentioned merely to give some idea of the extent of the weakening.

If the beams loaded through corbels have no suspension reinforcement but do contain normal stirrup reinforcement in a quantity corresponding to 0.2–0.3% of the horizontal web sectional area, then the situation is less dangerous than described above. In that case the spreading of the horizontal crack is restricted by the reinforcement and complete detachment of the bottom flange from the rest of the beam is thus prevented. The quantity of reinforcement encountered by the crack bears a direct relation to the seriousness of the damage that can occur and to the rate at which the width of the horizontal crack increases. In such cases, too, the beams did in fact become unserviceable long before the failure moment was reached (Fig. 7). In a few instances the failure moment was indeed reached by the beam, but by then the crack had widened to more than 3 cm. Basing oneself on the principle that a beam must be considered to have failed for practical purposes when the horizontal crack has reached a certain width (e.g., 2 mm), it is found that in these cases, too, there occurs a serious decline in "strength".

It is therefore necessary to provide corbel beams with suspension reinforcement in order to ensure that the corbel loading can always be transmitted upwards. This reinforcement can be designed on the assumption that the yield stress can permissibly be attained in it when the loading applied to the corbels corresponds to the failure loading of the beam. From what has been said it will be evident that the suspension

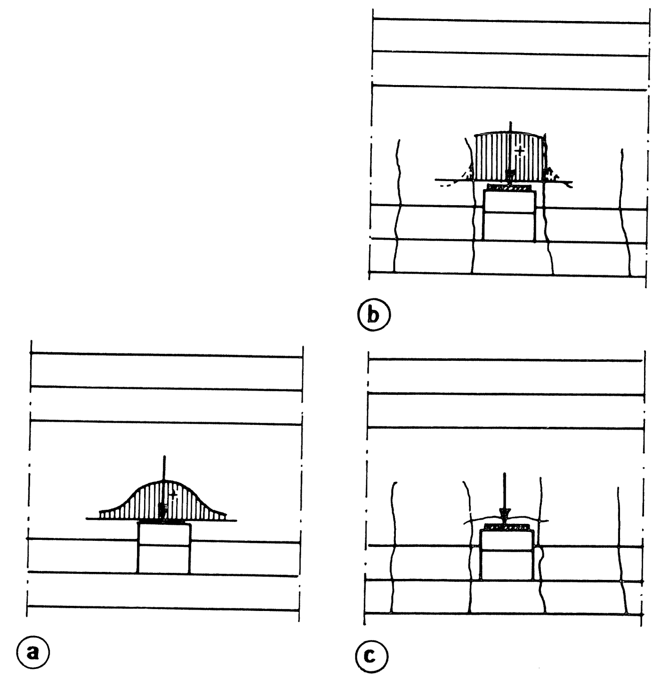

Fig. 6.
a. tensile stresses in vertical -direction at level of top of corbel
b. tensile stresses in vertical -direction at level of top of corbel after development of flexural cracks
c. the "detachment" of the corbel after development of flexural cracks and a horizontal crack above the corbel.

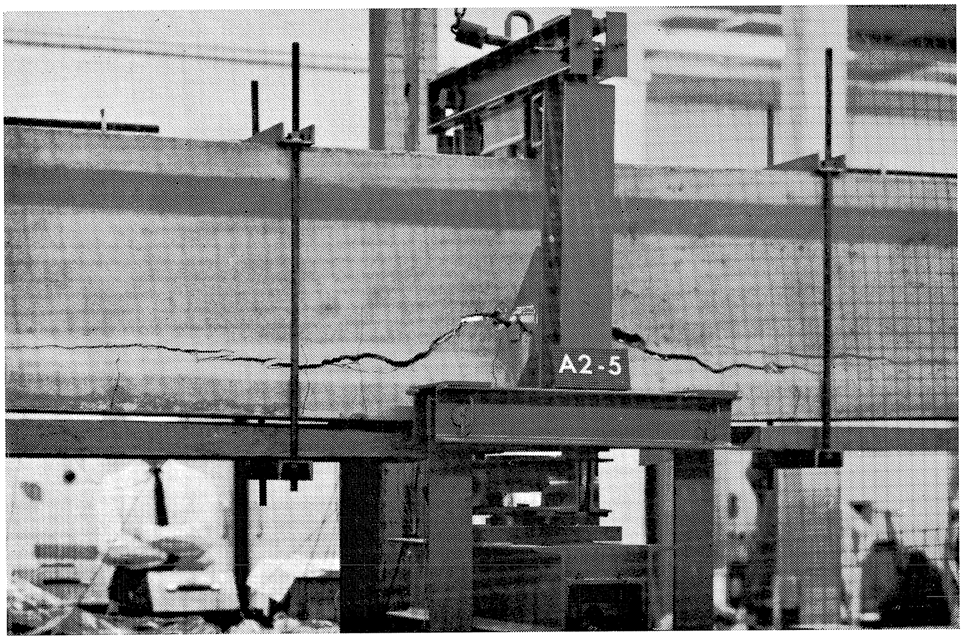

Fig. 7. Beam loaded through corbels and containing no suspension reinforcement, but provided with normal stirrup reinforcement. The photograph shows the condition just after the cracking moment was reached.

reinforcement should preferably be installed in the web between the corbels concreted on to each side of the beam (see Fig. 8). In actual practice this is often difficult to do if a relatively large quantity of such reinforcement is needed. A solution which has been found satisfactory consists in disposing the suspension reinforcement as close as possible beside the corbels and to interconnect the bars securely (see Fig. 9). In Appendix 1 (page 29) the procedure for calculating the requisite suspension reinforcement is indicated.

Although, for various reasons, it cannot be regarded as a commendable solution, it is possible to use external stirrups, placed around the beam (see Fig. 10), in lieu of internal suspension reinforcement. With this makeshift arrangement the beam can so be strengthened that it will attain its failure moment.

What has been said concerning corbel beams is essentially also applicable to beams which are loaded by means of holes formed in them. In such beams the requisite suspension reinforcement should be disposed as close beside the holes as possible and its bars should be securely interconnected.

In corbel beams the corbels themselves may also give rise to difficulties. An example of structural damage of a type that occurred quite frequently is illustrated in Fig. 11. The primary cause of this cracking is the absence of a loaddistributing layer. Besides, in many cases the reinforcement is unable to prevent the outer corner of the corbel from spalling off. This kind of failure is of general occurrence at bearings and at ends of beams.

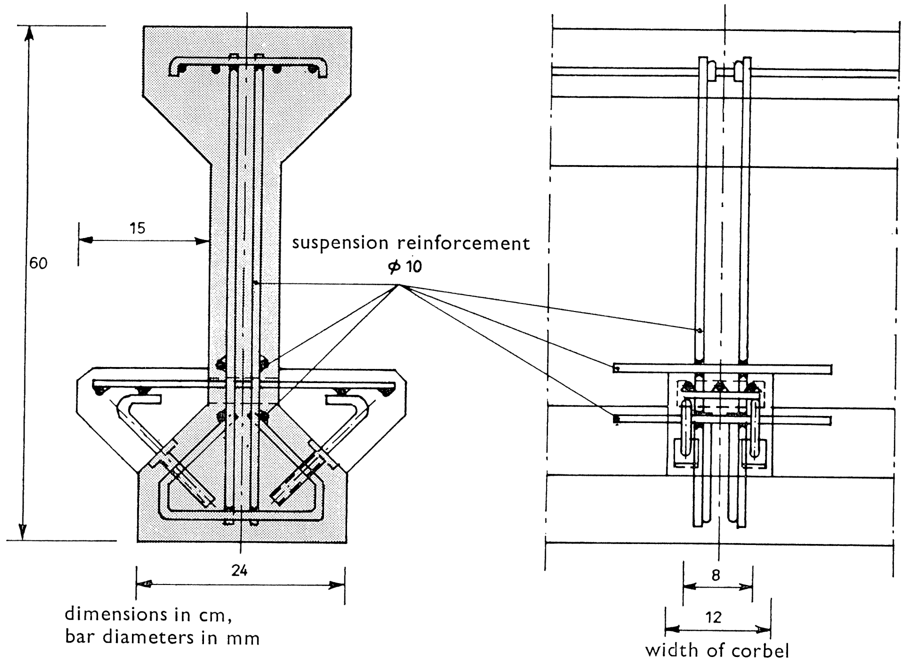

Fig. 8. Corbels subsequently cast on to the beam. The corbel reinforcement passes through holes preformed in the web. The suspension reinforcement is preferably installed between the two corbels.

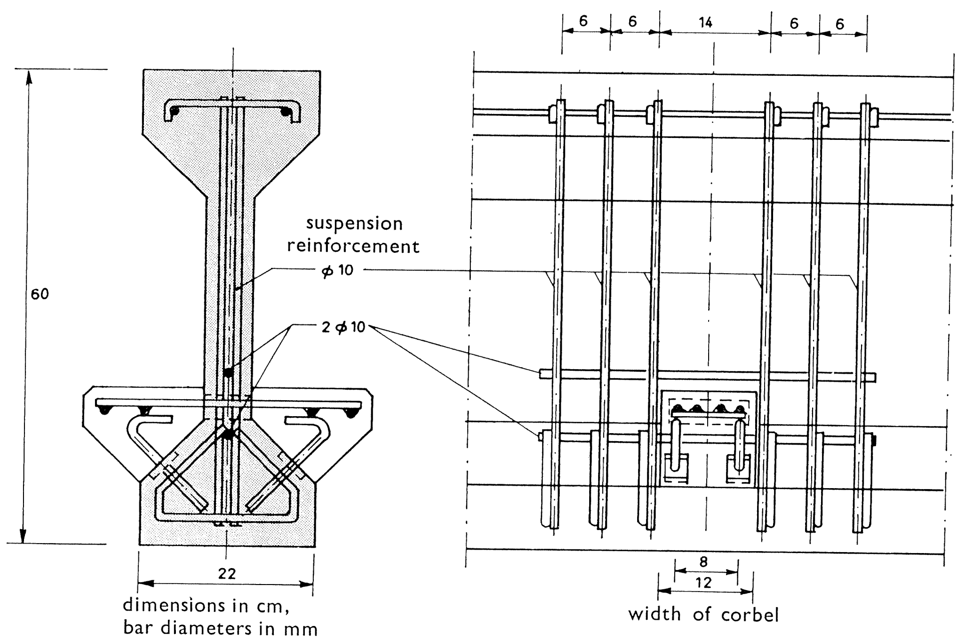

Fig. 9. Corbels subsequently cast on to the beam. The corbel reinforcement passes through holes preformed in the web. If there is not sufficient space between the two corbels, as in this case, the suspension reinforcement is installed on each side of them and the bars are connected together.

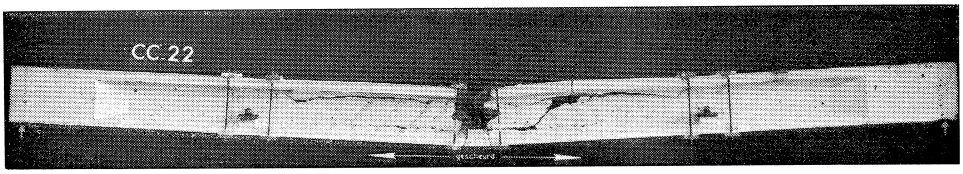

Fig. 10. View of a beam after failure. The loading was transmitted to the compressive zone by stirrups placed around the beam.

The specific properties of a corbel will first be dealt with.

The function of the corbel is directly evident. In addition to shear force it must also resist a bending moment. According to the latest views on the subject of corbel design, a reinforcement installed at the top of the corbel will, in the main, be sufficient.

It should be noted, however, that these "latest views" relate only to corbels on columns.

According to various recent publications *) the pattern of forces acting in these corbels can best be approximated by a single "lattice" system, as illustrated in Fig. 12, for example. In this system the load R is transmitted by an inclined compressive force D and by a horizontal tensile force T acting at the top of the corbel. These tensile and compressive forces can be determined by drawing a triangle of forces or from considerations of equilibrium.

The lattice theory should be applied to corbels which approximately satisfy the

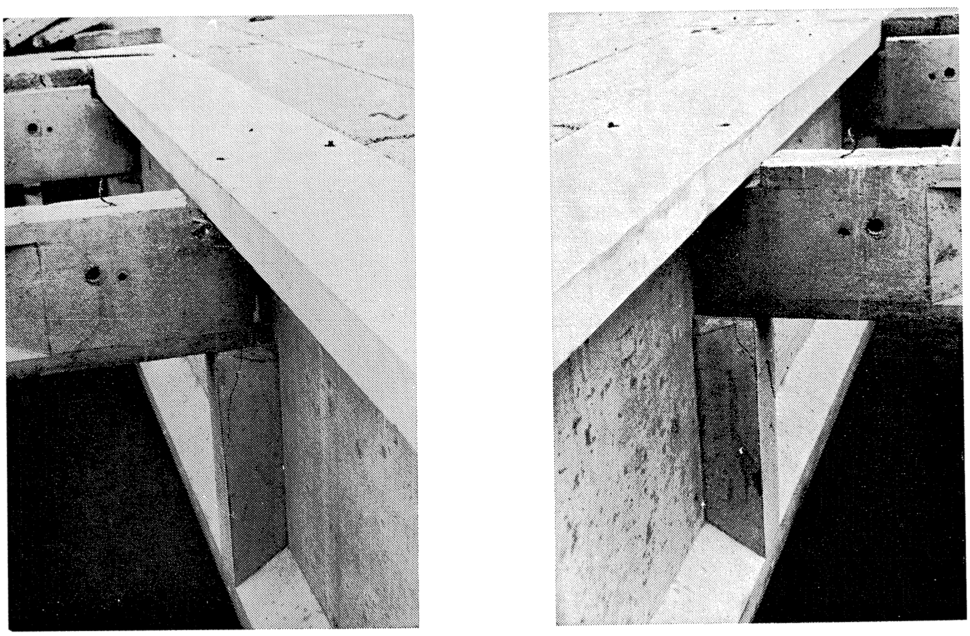

Fig. 11. Example of direct concrete-to-concrete contact; the beams are connected by means of pins engaging with sockets.

*) See footnote on page 21.

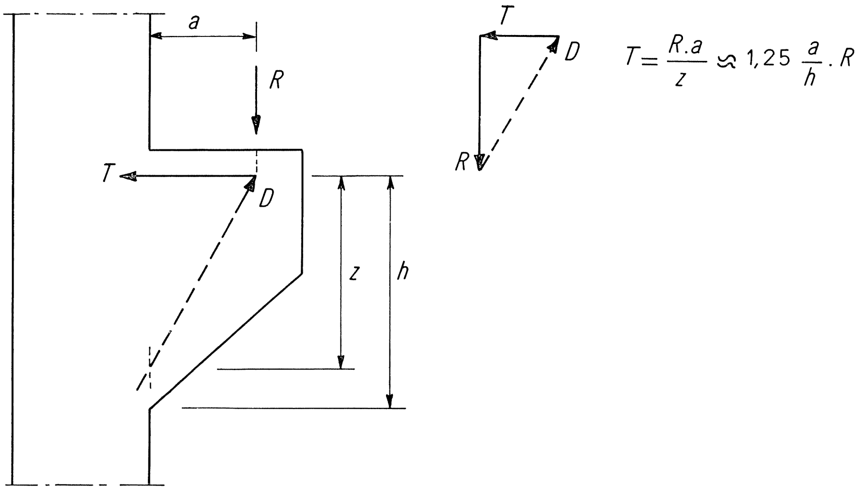

Fig. 12. Forces acting in a corbel which is monolithically connected to a column.

condition $a/h \leqslant 1$. If the internal lever arm is taken as equal to $0.8h$, the tensile force is expressed by $T = R\,(a/0.8h)$. To this tensile force must be added the effect of possible horizontal loads. Let R_u denote the vertical failure load and H_u the horizontal failure load, then the requisite quantity of reinforcement is:

$$A \approx 1.25 \frac{R_u}{\sigma_e} \cdot \frac{a}{h} + \frac{H_u}{\sigma_e} \; {}^{**)}$$

In this formula σ_e denotes the yield stress of the reinforcement employed. From the above considerations it appears that the tensile force T acts over the entire distance a and does not diminish to zero towards the load. The force T can develop only if the bars are securely anchored at both ends. The hypothesis on which the analysis is based is that the corbel will fail as a result of yielding of the reinforcement. This will be the case only so long as this reinforcement does not exceed a certain maximum quantity. It emerges, however, that the maximum reinforcement percentage in corbels is considerably lower than in normal beams loaded in bending. For corbel design

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*) This subject has been studied by the CUR Committee A 11 "Shear force". The final report, CUR-report no. 47: "Gedrongen balken en korte consoles" (in dutch) includes a comprehensive analysis of the literature. For this reason only two references will be mentioned here:
(a) Franz, G. and H. Niedenhoff: "Die Bewehrung von Konsolen und gedrungenen Balken". Beton- und Stahlbetonbau 1963, No. 5.
(b) Kriz, L. B. and C. H. Raths: "Connections in precast concrete structures – strength of corbels". Journal of the Prestressed Concrete Institute, Vol. 10, No. 1, 1965.

**) Having regard to the considerable influence that horizontal forces can exercise upon the strength of corbels, it is important to know the magnitude of the horizontal forces as accurately as possible. The ACI-ASCE Committee 512 accordingly recommends that the horizontal loading should be taken as not less than half the vertical loading, unless a realistic calculation proves otherwise or unless special precautions to obviate horizontal forces are taken.

calculations it is therefore recommended that a substantial margin be allowed in relation to the currently accepted value for the maximum percentage of reinforcement.

It is further to be noted that the compressive force also gives rise to tensile stresses (splitting stresses) at right angles to the direction of that force. In general, it is advisable to provide horizontal light stirrup reinforcement to cater for these stresses.

In deep corbels the compressive force will be in a steeply inclined direction approaching the vertical. In such cases a reinforcement such as is used in columns, with corner bars and binders, is appropriate.

A significant difference between corbels on columns and those on beams exists in the mode of support. Whereas the compressive force D in the corbel shown in Fig. 12 can be transmitted directly into the column, in the case of a beam corbel the compressive force D has to be transmitted by reinforcement to the top of the beam. If this condition is satisfied, then the above considerations relating to a column corbel will in principle also be valid with regard to a beam corbel.

The following relates more particularly to corbels which are not cast as integral features with the beam but are subsequently cast on to it, because such corbels are frequently employed in practice and have also been tested in the laboratory investigation (see, for example, Fig. 9).

The effective depth h of the corbel may be taken as equal to the distance from the corbel reinforcement to the underside of the beam (see Fig. 13).

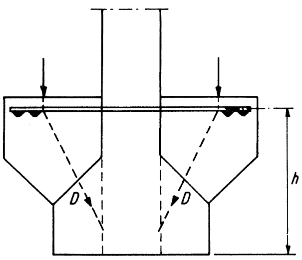


Fig. 13.

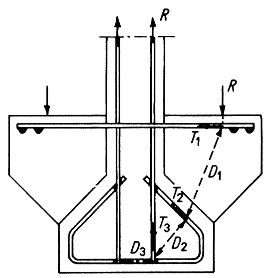


Fig. 14.

Fig. 13.
Effective depth of corbel to be adopted in calculations.

Fig. 14.
Force in a vertical section – provided with closed stirrups – through a corbel subsequently concreted on; all welds to be executed as structural welds (capable of transmitting the full force).

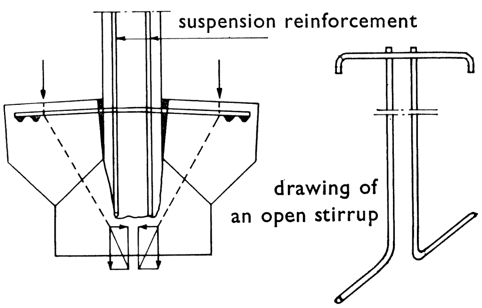


Fig. 15.

Fig. 15.
Vertical section through the subsequently concreted corbel with open stirrups. Because of the virtual absence of reinforcement the corbel, together with the bottom flange of the beam, may become completely detached.

For absorbing the vertical component of the compressive force D and transmitting it to the compressive zone of the beam a good solution is provided by closed stirrups in the case of subsequently cast-on corbels, as illustrated in Fig. 14. In the specimens tested in the laboratory investigation already referred to, open stirrups were also employed in a number of cases. Figs. 15 and 16 show that this is essentially not a good solution. For corbels of low depth it is indeed essential to use closed stirrups as suspension reinforcement in order to obtain a reliable construction. It is important to form all the connections in the stirrup reinforcement by means of welds capable of transmitting the forces concerned.

A second important point that determines the strength of the corbel is the provision of reliable anchorage of the horizontal reinforcement which has to resist the force T. This reinforcement is most effective and also simplest to install if the bars are continued through the web of the beam. Good anchorage at both ends can most suitably be obtained by means of welded-on transverse bars (Fig. 17b). In this way, too, the weak corner where the concrete is liable to spall off when bent bars are used (Fig. 17a), is obviated.

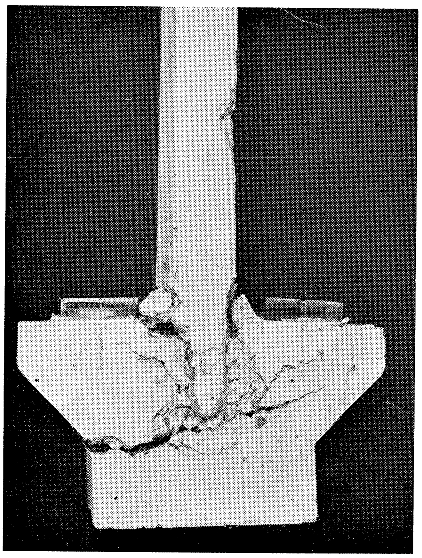


Fig. 16.
Fracture pattern of a corbel provided with the stirrups shown in Fig. 15 as suspension reinforcement. The corbel was concreted monolithically with the beam section.

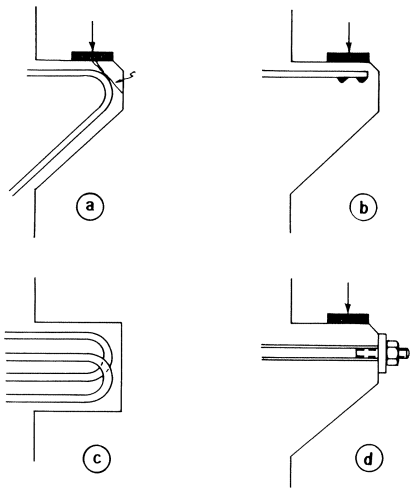


Fig. 17.

a. As a result of bending down the main reinforcing bars an unreinforced corner is formed, which is liable to crack off, especially if the load is applied close to the edge of the corbel.
b. It is therefore preferable to ensure anchorage of the bars by means of welded on cross-bars. The main reinforcing bars should, however, extend as far as possible to the outer edge of the corbel, and the welds should conform to stringent requirements.
c. Unreinforced corners of substantial size occur also when horizontal looped reinforcing bars are used.
d. Structurally this method of anchoring the corbel reinforcement constitutes a good solution, but it is liable to be aesthetically objectionable.

Fig. 18.
Initial stage in the failure of a corbel.

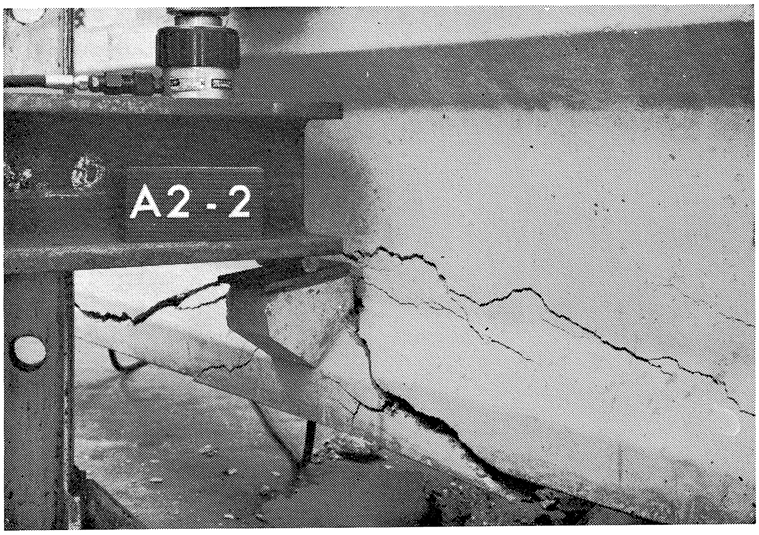


Fig. 19.
The corbel reinforcement has become detached.

Another form of good anchorage is shown in Fig. 17d, but there are aesthetic objections to this solution, while it will also entail a certain amount of maintenance in course of time.

It must be emphatically pointed out that the welds connecting the main and the transverse reinforcement in Fig. 17b must be very carefully executed. In the laboratory investigation it was found, in the failure tests performed on the corbels, that failure was almost invariably initiated by failure of the welded connections.*) Because of this the failure load was only 60–65% of the theoretical failure load of the corbel itself. In all cases the yield stress was indeed attained in the suspension reinforcement, however.

*) Separate tests performed on bars (steel grade QR 24, i.e., hot-rolled with minimum specified yield strength of 24 kg/mm^2) welded crosswise on to each other showed that the stress in the main reinforcing bar could reach something approaching the tensile strength only if the welds had been very carefully executed. It was, however, found possible to reach the tensile strength in those cases where the transverse bar had been welded against the end of the main reinforcing bar.

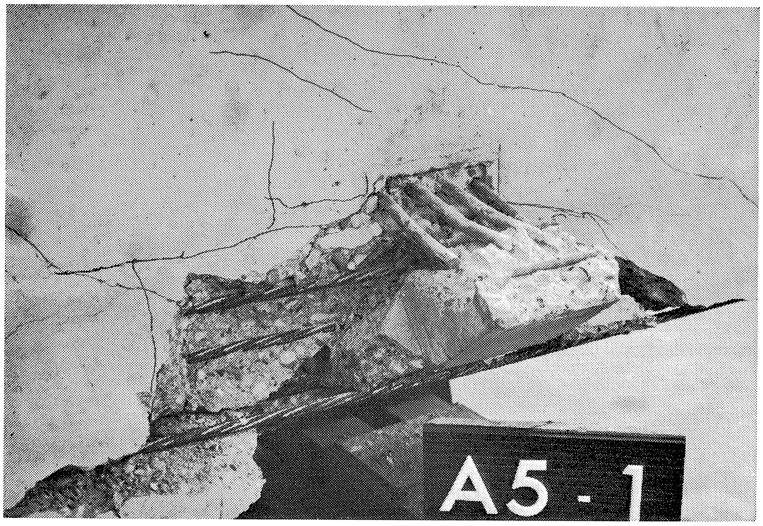


Fig. 20.
The cross-bar has been pulled off the corbel reinforcement.

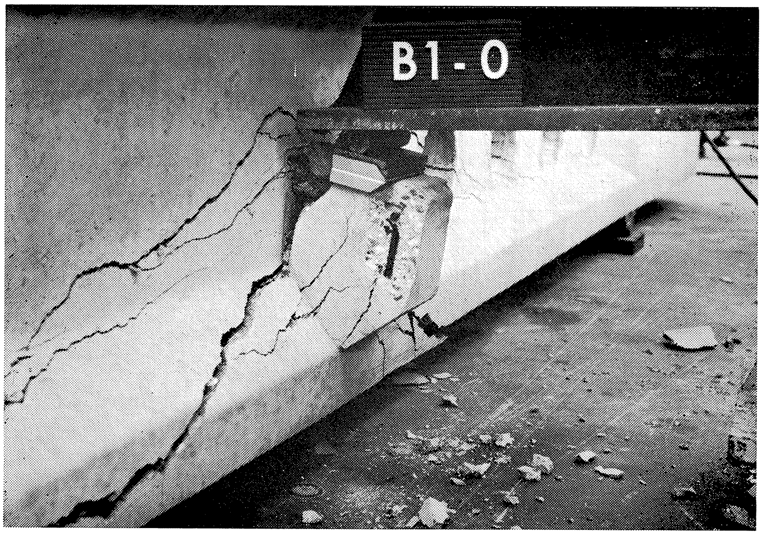


Fig. 21.
The corbel reinforcement has become detached.

Fig. 22.
A portion of the bottom flange drops off when open stirrups are used as suspension reinforcement.

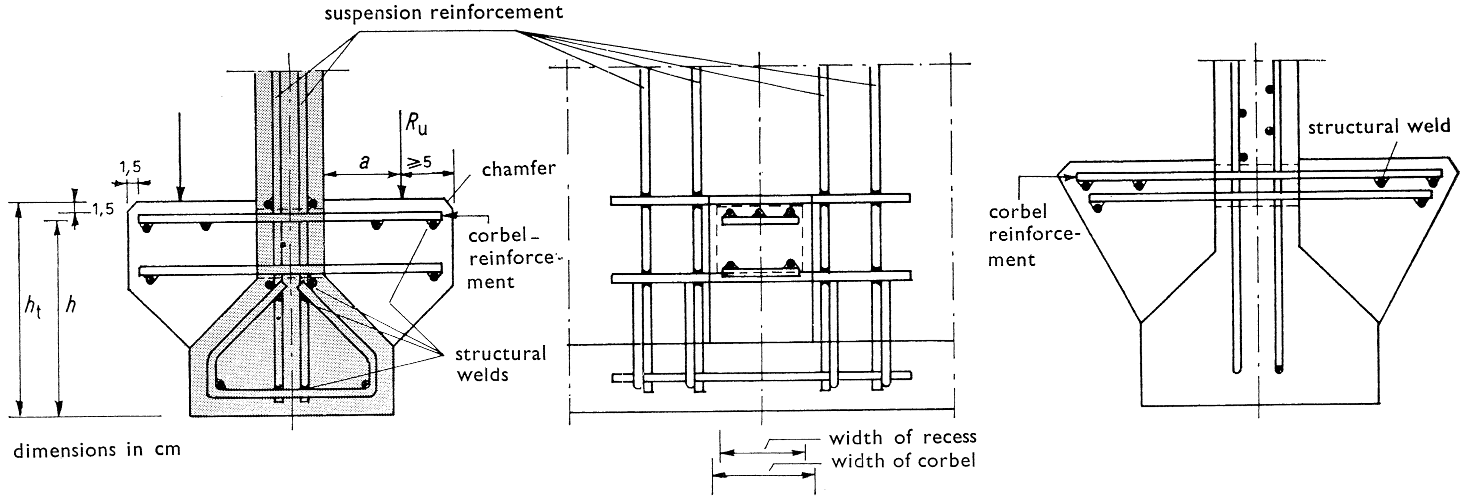


Fig. 23. Good construction of a corbel of low depth subsequently cast on to the beam

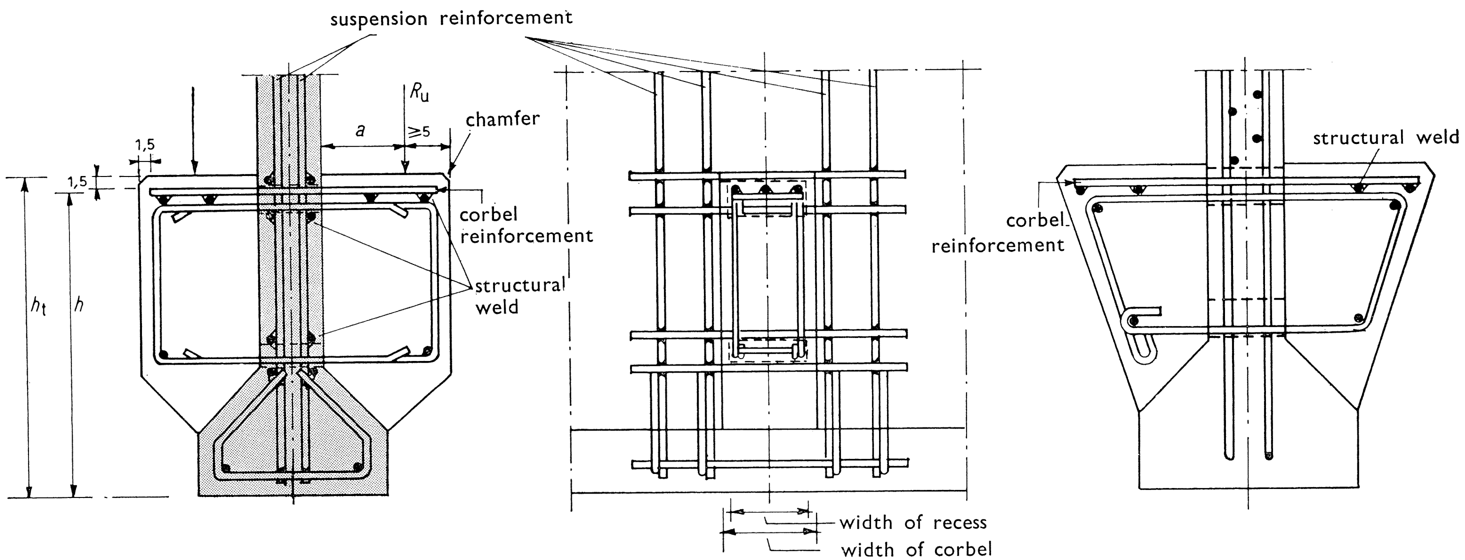


Fig. 24. Good construction of a deep corbel subsequently cast on to the beam.

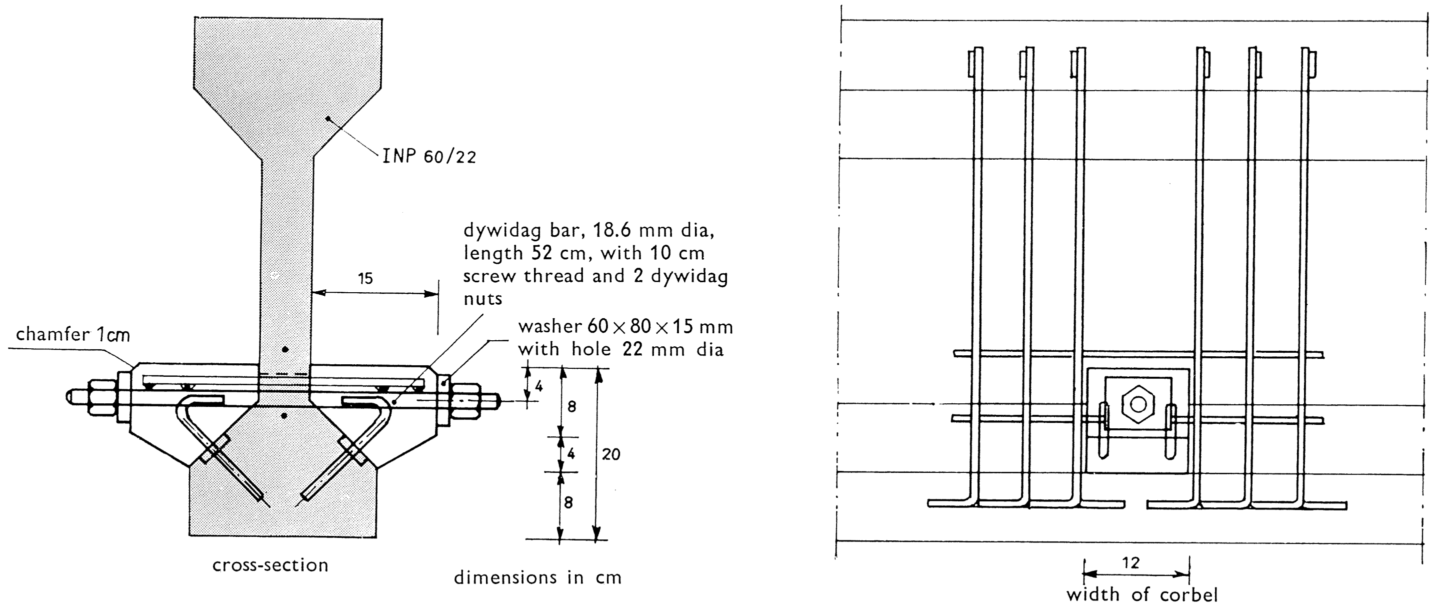


Fig. 25. Corbel subsequently cast on to the beam and provided with a Dywidag prestressing bar.

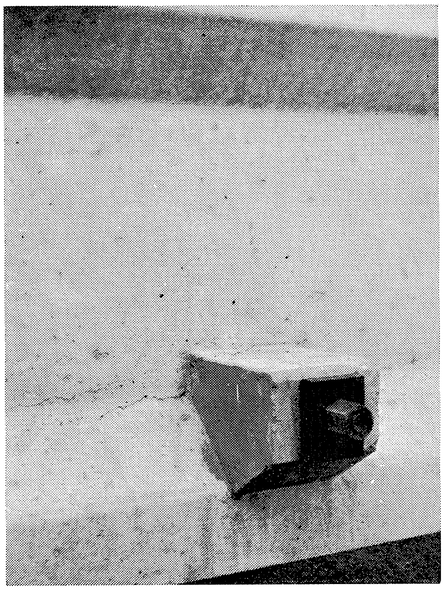


Fig. 26. Prestressed corbel.

In Figs. 18–22 some characteristic forms of failure of corbels are shown. The presence of horizontal strands in the bottom flange of the beam prevented the corbel from being completely detached.

Figs. 23 and 24 show good forms of construction for a corbel of low depth and of large depth respectively, while the design calculation is summarised in the Appendix to this chapter. In view of the uncertainties in the calculation and the fact that the requi-

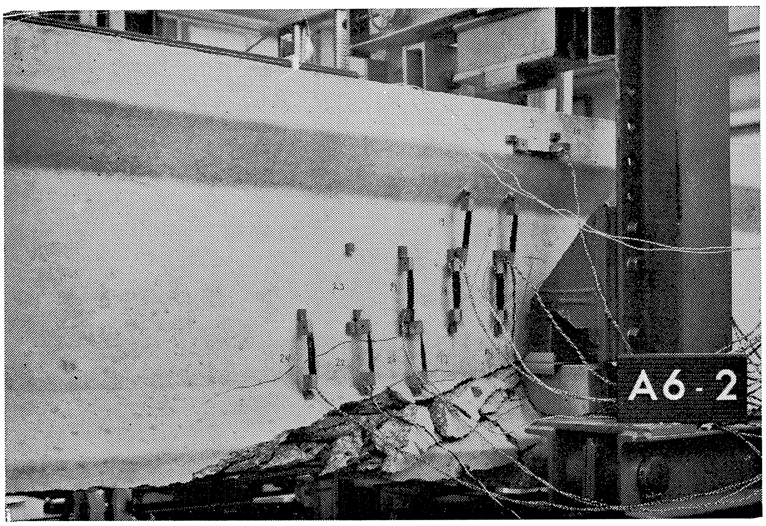


Fig. 27. Beam without suspension reinforcement. The bottom flange has been pulled off the beam by the prestressed corbel.

site quantity of reinforcement is relatively small anyway, it is considered desirable to adopt a somewhat higher factor of safety in corbel design.

Initially it used to be thought that the prestressing of corbels in the manner indicated in Figs. 25 and 26 would always constitute a better form of construction. This was investigated on two beams, each provided with prestressed corbels and with corbels in which the nut of the Dywidag prestressing bar was merely hand-tightened. Furthermore, one beam was provided with proper stirrups and suspension reinforcement, whereas the other contained no suspension reinforcement.

The first, properly reinforced, beam clearly revealed that both the prestressed and the non-prestressed corbels are, on account of the external anchorages, indeed stronger than the corbels containing only conventional reinforcement. In the beam in which the suspension reinforcement had been omitted the prestressed corbel, however, gave a much more unfavourable result. The ultimate load attained by these inadequately reinforced beams was further reduced by 25% (see Fig. 27). This is because the prestressing of the corbels is attended by splitting stresses in the web of the beam. In consequence of these splitting stresses – together with the vertical tensile stresses caused by the corbel loading – the tensile strength of the concrete was reached more quickly. In addition, on account of the very rigid connection, the corbels obtained less co-operating length in the web.

Appendix 1

a. Requisite "suspension reinforcement" at corbels:

The quantity of "suspension reinforcement" needed for the transmission of the loading on a corbel to the compressive zone of the beam can be calculated from:

$$A = f \cdot \frac{P_u}{\sigma_e}$$

Where:

A = requisite quantity (cross-sectional area) of reinforcement (in cm^2);

σ_e = yield stress of the steel employed (in kg/cm^2); in order to limit the elongation it is preferable to use steel grade QR 24 or to substitute a maximum value of 2400 kg/mm^2 for σ_e in the formula;

P_u = γ times the sum of the working loads on the two corbels, i.e., on both sides of the beam (in kg); the value for γ is obtained from the Netherlands Code of Practice RVB-67, clause 10.2.2 and clause 10.2.3, where the values $\gamma = 2.25$ and $\gamma = 2.75$ are respectively laid down for the structures concerned;

f = a factor 1.15 which somewhat increases the safety of the suspension reinforcement in relation to that of the beam; this ensures that the corbel, a secondary structural feature, will fail after the main structure, which is always a good structural principle.

The suspension reinforcement should as a general principle consist of closed stirrups. The various components of this reinforcement should be so interconnected by welding that the welds are able to transmit the forces involved. The stirrups should preferably be installed in the web of the beam, between the two corbels. If this is not possible, these stirrups should be placed as close beside the corbels as possible. In the latter case they should be interconnected by a number of horizontal bars.

If the depth of the corbel is more than, say, one-third of the beam depth, open stirrups may permissibly be employed as suspension reinforcement. The designer should then satisfy himself in each particular case that there is no danger of the corbel being "torn off" along unreinforced concrete (see Fig. 16, for example).

b. Corbel reinforcement (see Figs. 23 and 24):

The horizontal main reinforcement in the corbel can be calculated from:

$$A = f\left\{1.25\,\frac{R_u}{\sigma_e}\cdot\frac{a}{h}+\frac{H_u}{\sigma_e}\right\}$$

Where:

A = requisite quantity (cross-sectional area) of reinforcement (in cm^2);

σ_e = yield stress in the steel employed (in kg/cm^2); in order to limit the elongation it is preferable to use steel grade QR 24 or to substitute a value not exceeding 2400 kg/mm^2 for σ_e in the formula;

R_u = γ times the vertical loading on one corbel under working load conditions of the beam (in kg); the value for γ is obtained from RVB-67, clause 10.2.2 and clause 10.2.3, where the values $\gamma = 2.25$ and $\gamma = 2.75$ are respectively laid down for the structures concerned;

H_u = γ times the horizontal loading on the corbels under working load conditions of the beam (in kg); if this loading cannot be ascertained, it is recommended to adopt $H_u = 0.5\,R_u$;

a = eccentricity of the corbel loading (in cm);

h = effective depth of corbel (in cm);

f = a factor 1.15 which somewhat increases the safety of the corbel reinforcement in relation to that of the beam.

The maximum quantity of reinforcement that can permissibly be taken into account is:

$$\omega_{0_{\max}} = 15\cdot\frac{\sigma_u'}{\sigma_e}$$

Where:

ω_0 = tensile reinforcement as a percentage of the effective cross-section $b \times h$;

σ_u' = maximum compressive stress in the concrete at failure, equal to 0.6 times the average 28-day cube strength σ_w';

σ_e = the same value for the steel yield stress as has been used for calculating A.

The minimum quantity of reinforcement to be provided is:

$$\omega_{0_{\min}} = 4\cdot\frac{\sigma_u'}{\sigma_e} > 0.2$$

The bearing pressure on the corbels under working load must not exceed:

$$\sigma_p = \frac{0.4\,\sigma_w'}{\gamma}$$

In this expression σ_w' is the 28-day cube strength and γ is the above-mentioned safety factor obtained from RVB-67.

3 Bearings

In the foregoing discussion of corbels some incidental reference was made to the difficulties which may arise in connection with the functioning of the corbel as a bearing (see Fig. 17a). Similar problems may also occur at the ends of beams. Some characteristic examples of this are given in Figs. 28 and 29 and also in Fig. 11. Damage of this kind is caused by the following causes acting singly or in combination:

1. There is direct concrete-to-concrete contact at the bearing.
2. The part of the beam directly over the bearing is unreinforced.
3. The effect of the prestress is lacking in the part of the beam directly over the bearing.
4. The beam is fixed to the bearings without any latitude for deformation.
5. The bearing length (or depth) is too small.

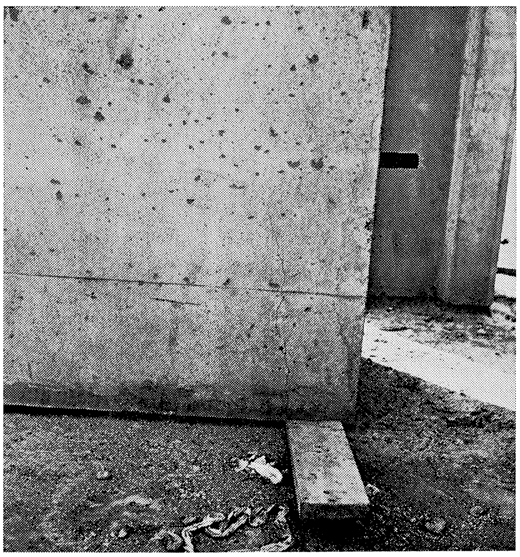


Fig. 28
Typical cracking at the end of a beam.

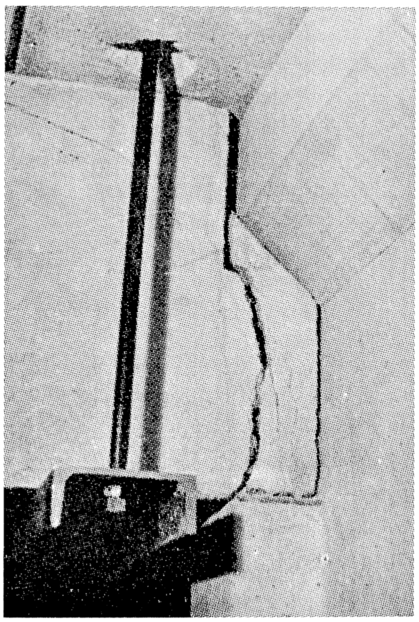


Fig. 29.
Bearing nib at end of beam severed by cracking.

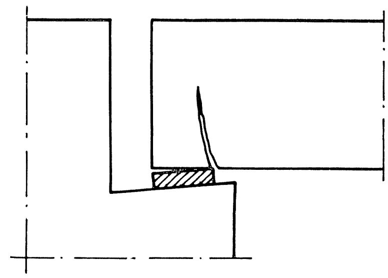


Fig. 31.
Bearing with tilted steel plate.

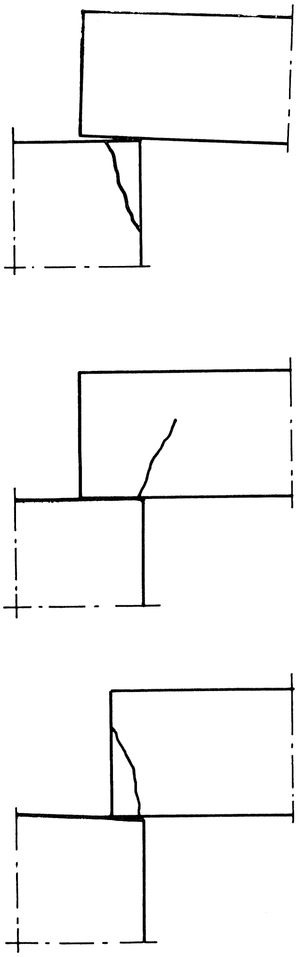


Fig. 30.
Cracking associated with concrete-to-concrete contact.

These points will be further examined. Although the considerations will relate mainly to the bearing part of a beam, they are also applicable to the bearing on which the beam is supported.

In general, when two concrete surfaces are placed in direct contact with each other at a bearing, difficulties are bound to arise. Even if the bearing surface is well finished so that it is horizontal and adequately flat, the beam which it has to support will nevertheless, when it deflects, tend to bear only at the edge of the bearing surface. In consequence of this the corner of the support or of the beam (or possibly both) may shear off (Fig. 30). This phenomenon is aggravated when, as is often the case, horizontal forces also act upon the bearings. Such forces may be caused by shrinkage, creep or temperature effects. Because of the high frictional resistance, there will in reality be no question of the two concrete contact surfaces sliding in relation to each other.

Approximately similar conditions arise when the beam is supported on a bearing layer which is inadequate to provide a uniform distribution of pressure. Steel plates, for instance, are often not quite flat or do not rest horizontally on the bearing. In the latter case the plate may cut like a knife into the end of the beam, so that high splitting stresses are produced, with the result illustrated in Fig. 31.

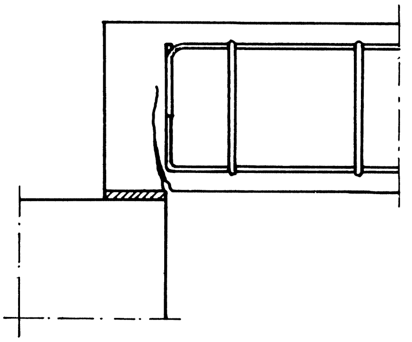


Fig. 32.
Cracking due to reinforcement stopping short of the bearing.

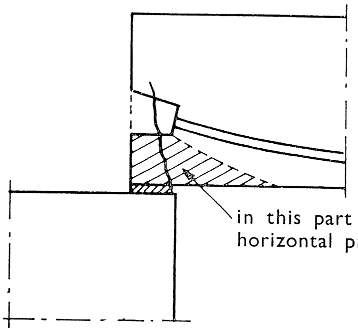


Fig. 33.
Cracking due to absence of prestress over the bearing.

In itself such cracking need not – under normal conditions – constitute a direct danger to the structure, provided that there is sufficient reinforcement at the cracks. Not infrequently, however, this requirement is not fulfilled. In that case the bearing part of the beam consists essentially of unreinforced concrete. For example, in the particular instance of a reinforced concrete beam it was found that the reinforcement had shifted in the formwork, with the result that at one end of the beam there was no concrete cover to the steel, while at the other end the reinforcement did not extend even to over the bearing. This is shown in Fig. 32. In this instance the trouble was largely attributable to inadequate dimensional control, resulting in such displacement of the reinforcement cage. In addition, there was the effect of the unreinforced corner due to the bending-up of the reinforcement; this aspect has already been referred to (see page 23).

It also often occurs that in the part of the beam over the bearing the prestress has not, or has not fully, developed. One reason for this is that, according to St. Venant's principle, the prestress will not attain a distribution over the cross-section in accordance with the elementary theory of bending before a certain distance from the anchorage, where the prestressing force is applied. Particularly if this point of application is located relatively high up, there will be practically no prestress acting in the part over the bearing.

In beams with post-tensioned tendons the end anchorage is moreover usually recessed into the end of the beam (Fig. 33), which arrangement may in itself already constitute a significant weakening.

Damage to beams in which the end bearings had been formed as nibs or heels has also been of fairly frequent occurrence. Such cases are illustrated in Fig. 34.

Fig. 34.
a. Cracking at a re-entrant angle
b. The nibs on the secondary beams are too small, so that it is practically impossible to install properly anchored reinforcement in them.

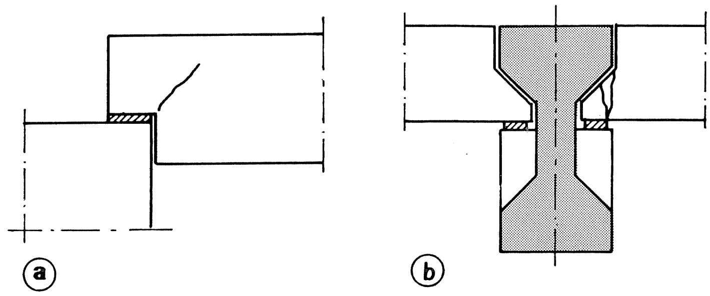
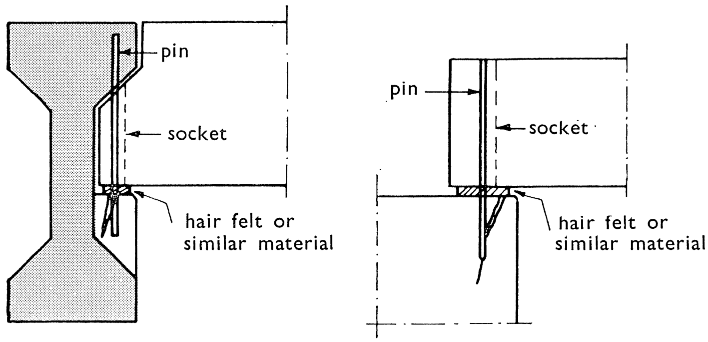


Fig. 35.
If the secondary beam has no freedom of movement, the cracking illustrated here is liable to develop.



A good deal of trouble has been caused by the use of steel pins inserted into vertical holes in the beam for securing the latter to its bearings (see Fig. 35). It often happens that, in consequence of dimensional errors or inadequate clearance, the pin presses against the wall of the hole. When horizontal movements occur as a result of

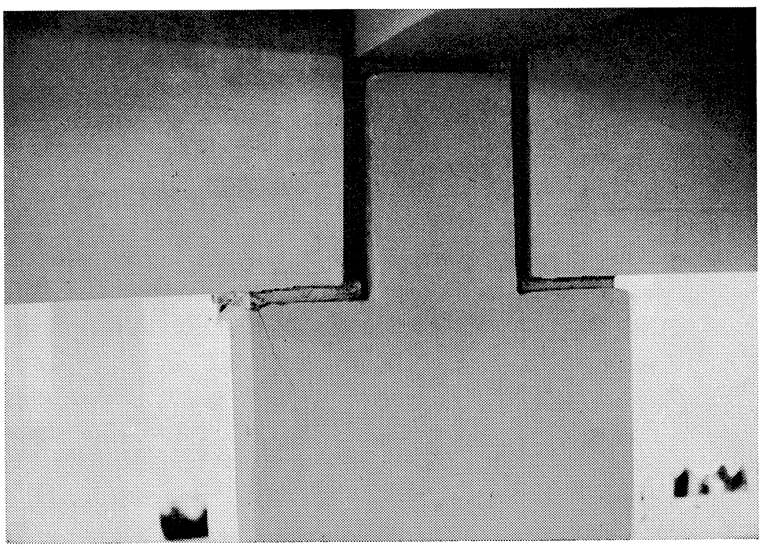


Fig. 36. Damage at a pin-and-socket connection.

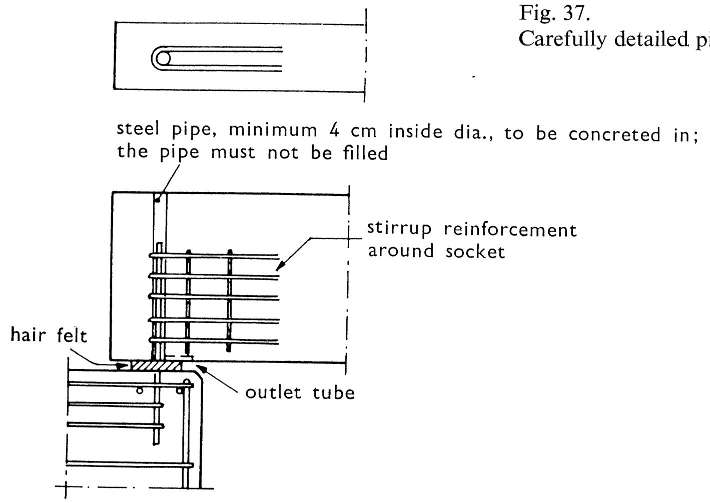


Fig. 37.
Carefully detailed pin-and-socket connection.

temperature variations, shrinkage, etc., they either cause splitting of the end of the beam or they cause the pin to be forced out of the column or corbel (see Fig. 36). In such cases the horizontal binder reinforcement that could greatly limit the damage is moreover often lacking.

Besides, jamming of the pin in the hole is not the only source of trouble. During construction, or possibly at a later stage, water may collect in the vertical hole that receives the pin. In a number of cases this has given rise to frost damage. An example of this will be discussed on page 60. In principle, this kind of damage can be prevented by providing an outlet for water, but since this is liable to become blocked, the solution remains a somewhat unreliable one. On the basis of these considerations the hole should preferably be formed by means of a concreted-in length of steel tube. Clearly, if the hole is filled up with mortar or grout – as has indeed sometimes been done – this will often cause cracking, particularly when infilling walls are constructed between the

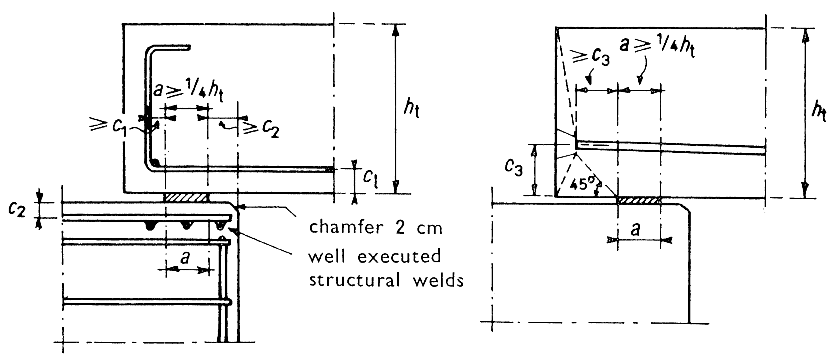


Fig. 38. Detailing of the bearing provided with conventional reinforcement (left) or utilising the prestress (right).

columns, in which case there is of course no "give" at all in the support. Cracking is also very likely to occur if the hole is filled with bitumen.

An acceptable pin-and-socket connection is illustrated in Fig. 37. This solution is rather elaborate, however, and it may well be asked whether it is worth the cost, especially when it is considered that these solutions are in fact adopted largely for psychological reasons only. An equally good arrangement to prevent the beam from being dislodged from its bearing is provided by a simple bearing seat or a connection comprising a convex and a concave component mating with each other. Neither of these solutions is affected by the above-mentioned trouble.

Damage to bearings aristing from any of the causes referred to is liable to become much more serious if the bearing length (or depth) is made too small. Besides, designers often tend to support a beam at its extreme ends.

If the designer duly considers that the above-mentioned factors are liable to cause damage, then in many cases the correct solution to adopt will be self-evident.

The interposition of a good pressure-distributing layer, e.g., hair-felt or some such material, between the beam and its bearing will already reduce the damage hazard quite considerably. In order to avoid an excessive concentration of the bearing pressure at the edge (where the concrete is unreinforced, being merely the cover to the steel), the intermediate layer should always be set back a little, as shown in Fig. 38. In addition, the concrete should always have chamfered edges, the two equal sides of the cut-off triangle being at least 1.5 cm in length. The bearing length (or depth) should preferably not be less than $\frac{1}{4}$ of the depth of the beam, with 8 cm as the absolute minimum. The beam will have to project beyond the bearing a sufficient

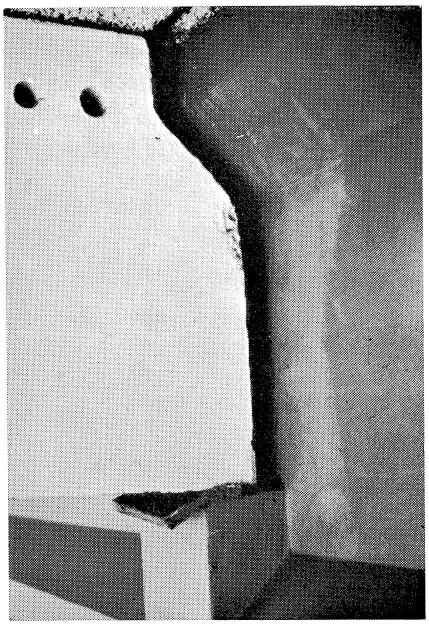


Fig. 39.
Hair felt has crept out from under the beam.

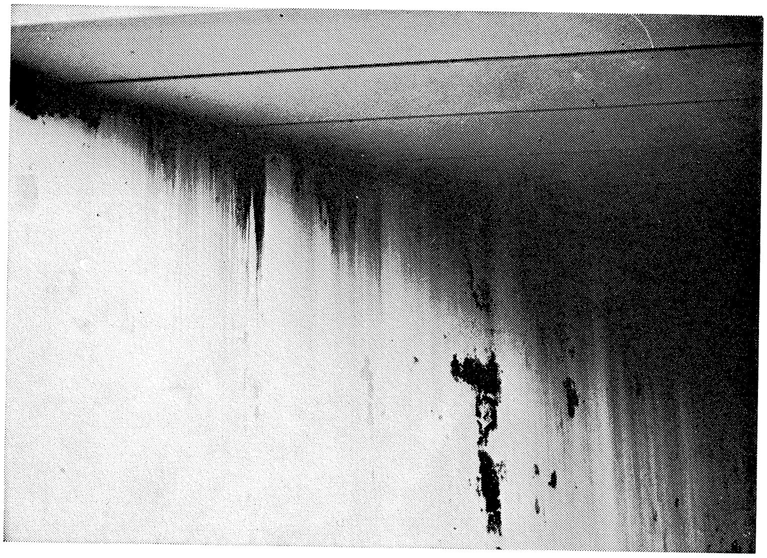


Fig. 40.
This unsightly result was caused by filling material oozing out of the joint.

distance to ensure that there is reinforcement or prestress over the bearing itself. To ensure that prestress will be available to resist tensile stresses over the bearing, it can be assumed that the effect of the prestressing force spreads from the anchorage at an angle of about 45°. These dimensions and figures relate only to normal cases encountered in actual practice. They are shown in Fig. 38. Appendix 2 on page 44 gives an empirical formula for calculating the extreme bearing pressure and the requisite quantity of reinforcement.

On account of various causes, often difficult to ascertain, it sometimes happens that the pressure-distributing layer creeps out from between the beam and its bearing (see Fig. 39). In general, therefore, it is advisable to take precautions to prevent this. If adhesives are used, it is important not to use a substance which may in fact have the very opposite of the desired effect by acting as a "lubricant" or which oozes out and thus looks unsightly. An example of this latter blemish is shown in Fig. 40. The aesthetic aspect also calls for arrangements to ensure proper discharge of rainwater. These were evidently lacking in the case illustrated in Fig. 41.

Special bearing blocks or pads composed of alternating layers of rubber and steel plates, bonded together by vulcanising, are employed in structures whose bearings have to transmit large reactions. In such cases it must be borne in mind that the concrete surfaces are still essentially in contact with steel plates. Local concentration of the bearing pressure at the "high spots" on the concrete surfaces can here, too, be prevented by the interposition of a suitable pressure-distributing layer between concrete and steel.

A word of caution is necessary in connection with these large bearing reactions. If the pressure-distributing intermediate layer is too soft or very plastic, splitting stresses may be generated. For example, Fig. 42 shows a slab of rubber which was squeezed out of the joint in consequence of the pressure between the two faces under

Fig. 41. Inadequate provision for rainwater discharge produced this unsightly appearance.

compression. The concrete structure must be able to resist the splitting stresses occurring as a result of this, otherwise its structural load capacity will diminish. In order to obtain some idea of the importance of this effect, tests were performed in which a number of cubes with different intermediate layers placed between them were tested to failure. The test set-up is shown schematically in Fig. 43, and the results are summa-

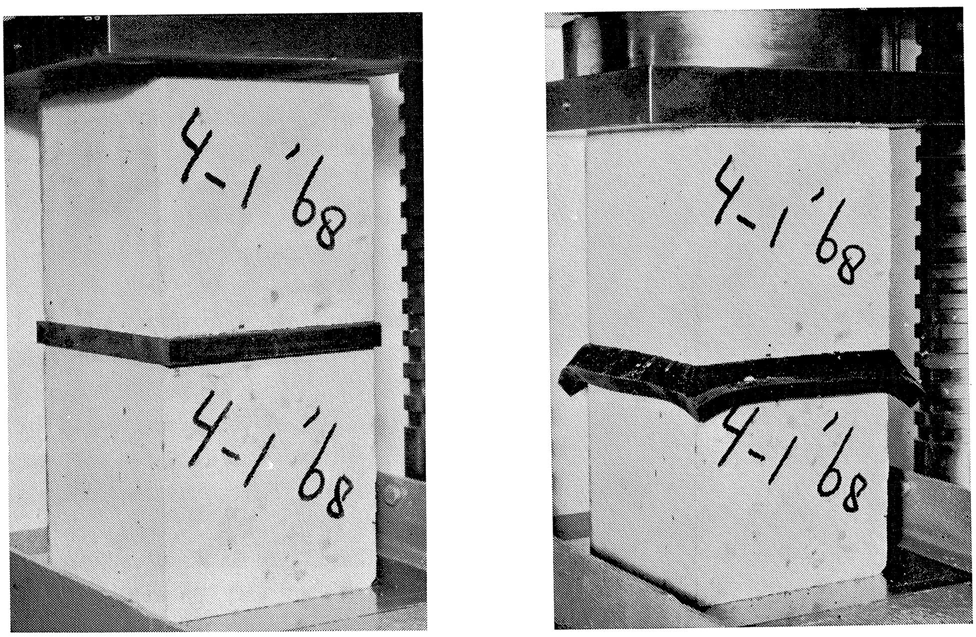
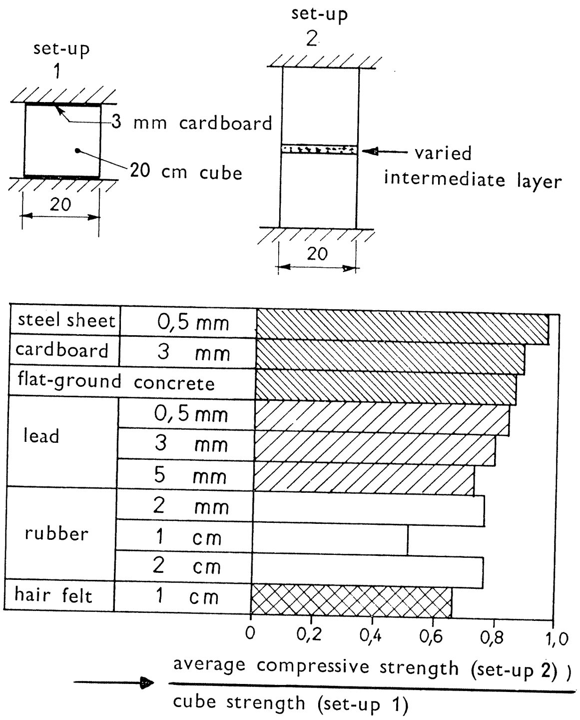


Fig. 42. Rubber slab squeezed out from between the bearing faces.

Fig. 43.
Test results which give some idea of the effect of soft and plastic intermediate layers upon the strength of concrete.



rised. It clearly emerges that in consequence of employing pressure-distributing layers the strength of unreinforced cubes is substantially reduced. The results will of course depend to a great extent on the quality of the layer thus interposed. Those indicated in Fig. 43 were determined mainly with the object of revealing a particular trend.

Fig. 44 is a photograph of a damaged rubber bearing pad. The cause of the damage is difficult to determine with certainty; the pad was found to be in this condition two years after the structure in question had been built. This pad had not been encased in

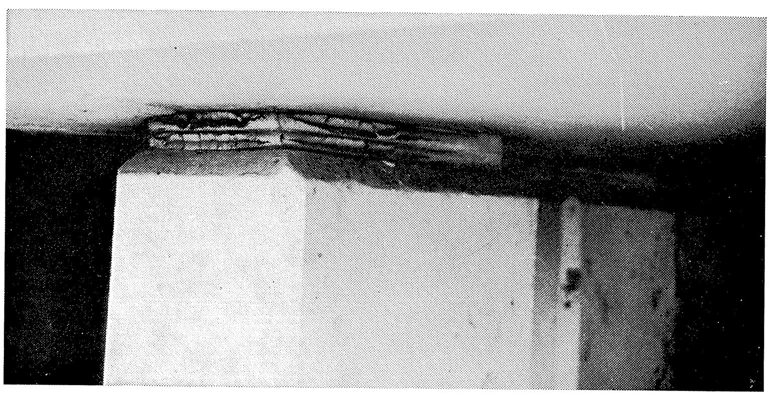


Fig. 44. Cracked rubber bearing.

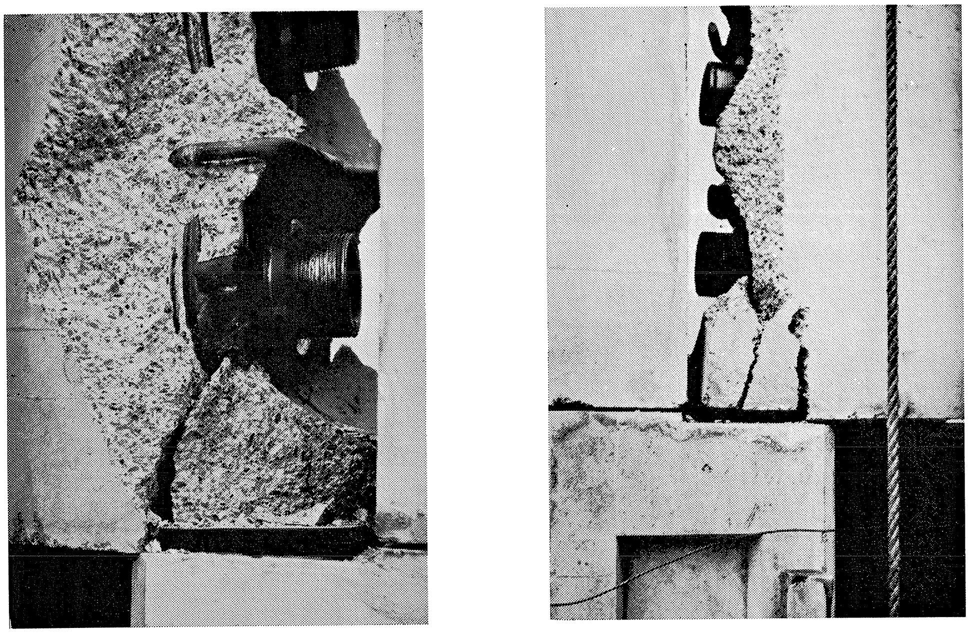


Fig. 45. Example of a bearing that failed.

neoprene, however. In view of the aggressive nature of the environment, chemical attack cannot be ruled out. It is advisable always to use neoprene-encased bearing blocks or pads under outdoor conditions. For the design of these bearings reference should be made to the relevant literature *) and to the directives issued by the suppliers.

An example of damage due to a combination of causes is shown in Fig. 45. In this structure none of the points referred to in the foregoing had received attention from the designer, namely:

- the beam bears on a steel plate;
- the bearing length is too small;
- the concrete over the bearing contains no reinforcement and is not prestressed;

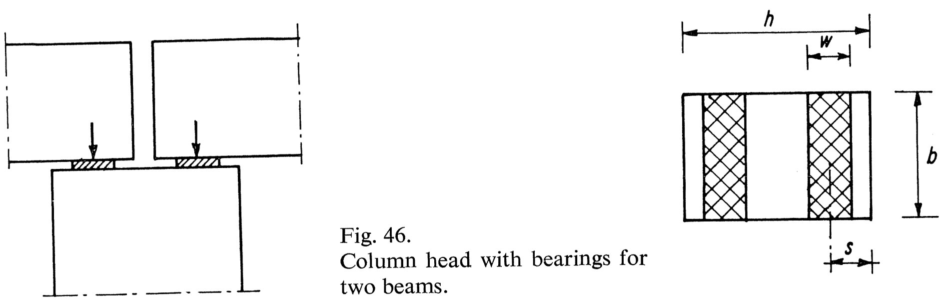


Fig. 46.
Column head with bearings for two beams.

*) Dr. Ing. B. Topaloff: "Gummilager für Brücken, Berechnung und Anwendung". Der Bauingenieur, 1964, No. 2, pp. 50–64.

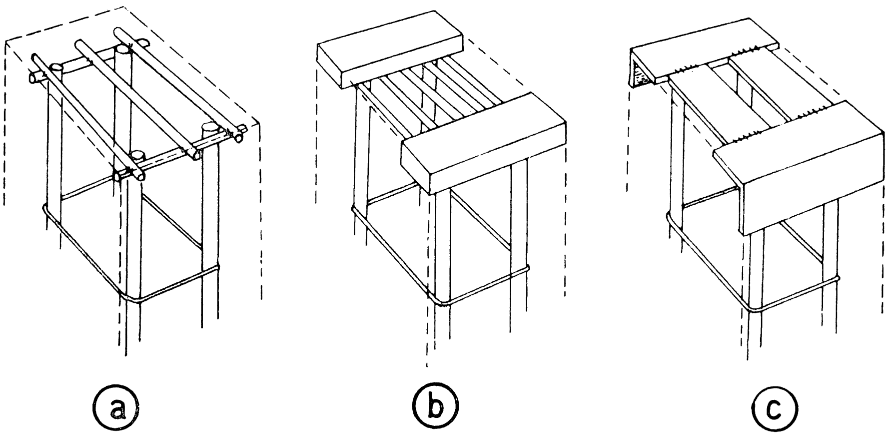


Fig. 47.

- the anchorage element causes considerable weakening at the critical section; in addition, on account of its shape, the end anchorage promotes the spalling of the unreinforced concrete.

Besides, in this case a large horizontal force was developed because there was no clearance in the pin-and-socket connection, used also in this case, even before the beam had been finally prestressed.

It is evident that the combination of these causes was bound to result in damage.

The above comments and considerations relate more particularly to the bearing parts of the beams themselves. In the main, they are valid also for the supports on which the beams are laid. Thus it is important to make arrangements to ensure that the heads of columns will not cause trouble in consequence of the bearing reactions to which they are subjected. L. B. Kriz and C. H. Raths *) have conducted elaborate experimental investigations into the bearing strength of column heads and obtained the following results:

- The bearing strength of a column head is determined chiefly by the tensile strength of the concrete.
- The dimensions of the steel bearing plates employed directly affect the bearing strength, as also does the location of the plates in relation to the edge.
- The manner in which the column head will fail if no extra reinforcement is provided depends on the location of the bearing plates.
- If the distance *s* (see Fig. 46) is more than about 5 cm, then splitting will occur under the plate. If this distance is less, then a shell of concrete under the plate is displaced outwards by shearing action.
- By providing reinforcement parallel to the top surface of the column and perpendicular to the bearing plates it is possible to increase the bearing strength of an

*) Kriz, L. B. and C. H. Raths: "Connections in precast concrete structures: bearing strength of column heads". Journal of the Prestressed Concrete Institute, No. 6, Dec. 1963.

unreinforced column head by about 100%. However, this is so only if (as in the case of the 20 × 30 cm column heads investigated) $s \geqslant 5$ cm, because the reinforcement alone prevents the splitting of the column head. The shearing of the shell of concrete if $s < 5$ cm will occur under the same loading in a reinforced as it does in an unreinforced column head.

- Here too, just as in corbels, the reinforcement must be securely anchored. The solutions illustrated in Fig. 47 were found to be very satisfactory.
- The quantity of vertical reinforcement in the column has no appreciable effect on the bearing strength of the head.
- If the column head is subjected to loading by horizontal forces as well, the bearing strength is greatly reduced.

From the experimental investigations an empirical equation has been derived whereby the failure load on a column head can be calculated. This expression can also suitably be used for determining the ultimate bearing pressure for the bearing part of a beam. This calculation is given in Appendix 2 on page 44.

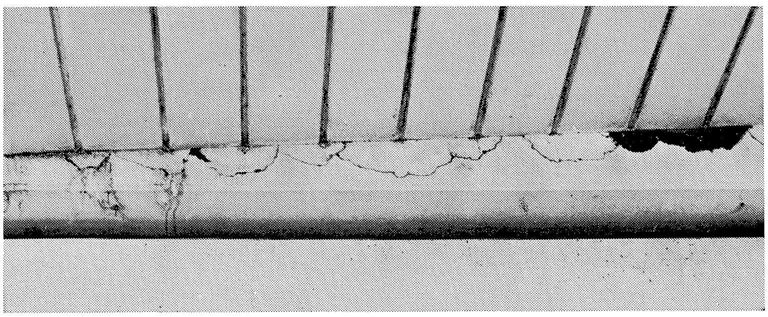


Fig. 48. Damage at the bearing on an intermediate support.

Fig. 48 shows a case of damage associated directly with the bearing. The structure is illustrated schematically in Fig. 49a. When the top of this structure is heated by sunshine, the beams will tend to "hog", i.e., bend upwards, with the result that the situation shown in Fig. 49b develops. It should be noted that the creep of the prestressed concrete has a similar effect and may have contributed to the damage. Owing to the absence of a coupling reinforcement at the bottom of the beams, horizontal forces were exerted on the bearings and caused the damage.

The CUR Committee D 3, in concert with the Rijkswaterstaat (Netherlands Government Waterways and Highways Administration) and the Provincial Waterways and Highways Administration of Overijsel, conducted a limited investigation into the magnitude of the temperature differences which can occur within the thickness of the deck slab of a bridge. It was found that for slabs 50 and 60 cm thick the difference in temperature between the top and the bottom was not more than 10° C. These values were measured on exceptionally fine summer days. The highest temperature measured 1 cm below the surface of the bare concrete was 31° C. At the same time and

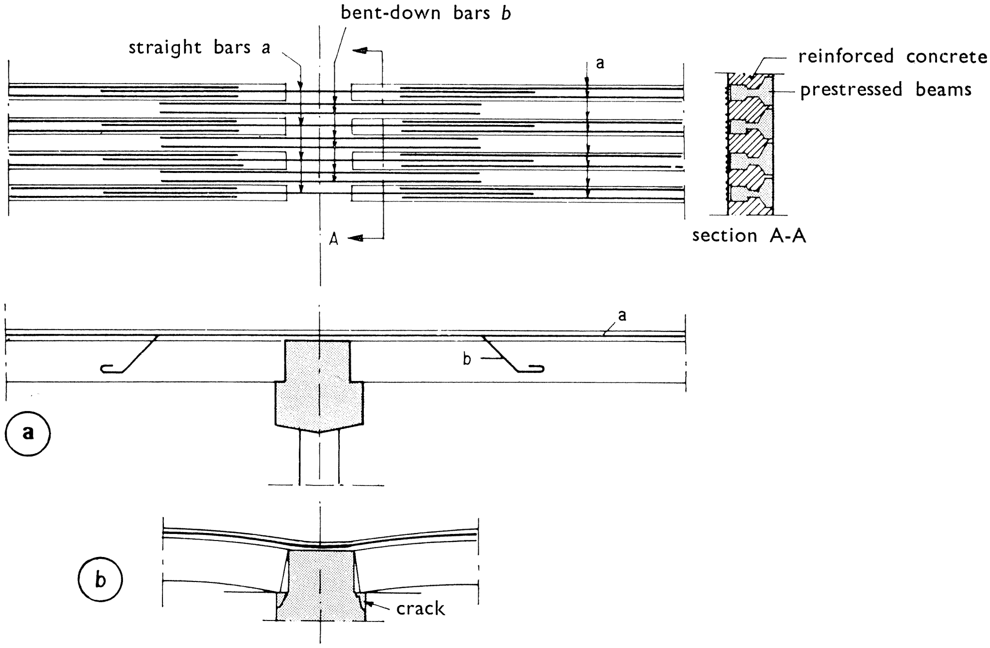


Fig. 49. Bridge constructed from precast prestressed beams made structurally continuous over several spans; location of reinforcement over an intermediate support.

in the same slab, but a few metres farther on, a temperature of 33° C was measured at the centre of a 5 cm thick asphalt surfacing. Under this surfacing the temperature difference over the thickness of the slab was found to be slightly less than in the bare part of the slab, i.e., where there was no surfacing over the concrete.

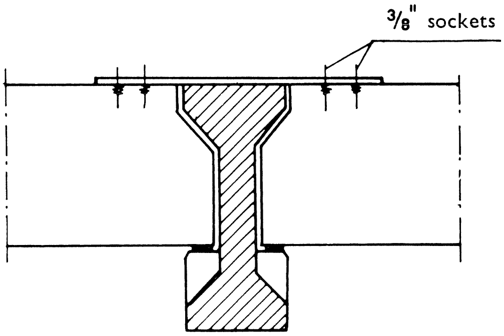


Fig. 50.
Secondary beams connected by means of a steel strip and $\frac{3}{8}''$ screw sockets.

Fig. 51.
As a result of the rigid connection by means of a steel strip a portion of the concrete has become detached.

The difficulties which are encountered in practice in forming the connections between main and secondary beams have in the main already been referred to in discussing the pin-and-socket connection. An alternative solution is presented in Fig. 50. Here, too, substantially similar difficulties are liable to arise if the tensile stresses due to shortening effects are not absorbed by reinforcement. In the example illustrated in Fig. 51, a portion of the top part of the beam was pulled off along the apertures preformed to allow the passage of services (pipes, wiring, etc.).

A better solution consists in providing the strip (see Fig. 50) with slots which allow some movement. In the long run, however, their functioning is unreliable. If it is indeed essential to couple the secondary beams across the main beam, the solution shown in Fig. 52 is preferable. Here the longitudinal reinforcement of the secondary beams, which must of course be able to resist the horizontal forces, is passed through the main beam and is spliced by welding.

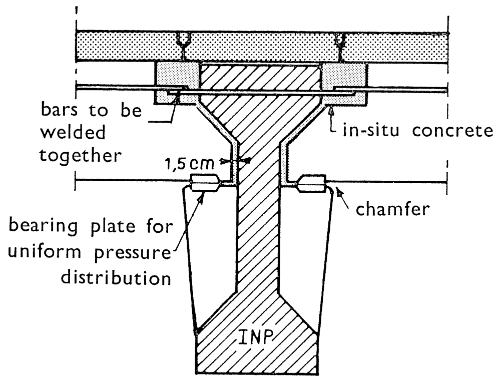


Fig. 52
Secondary beams connected by means of reinforcement.

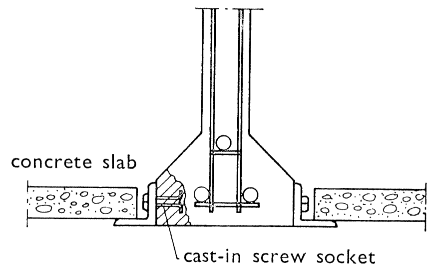


Fig. 53.
The force exerted on the screw socket is not transmitted by the reinforcement.

Some instances of serious damage have been caused by the incorrect use of cast-in screw-threaded sockets. Fig. 53 illustrates one such case. The forces exerted on the sockets are not absorbed by the reinforcement. Here, too, the designer has unwittingly relied too much on the tensile strength of the concrete. The tendency to adopt this kind of solution is promoted by the fact that prestressed concrete structures usually contain only little stirrup reinforcement, so that the possibilities for securing the screw-threaded sockets are limited. The only good solution is to fix these sockets to closed stirrups.

Appendix 2

Bearing pressures

The extreme bearing pressure can be calculated from the empirical equation: *)

$$\sigma_b' = 15\sqrt{\sigma_w'} \cdot \sqrt[3]{\frac{s}{w}} \cdot \left\{1 + C_1 \sqrt{\frac{A}{b}}\right\} \left\{\frac{C_2}{500}\right\}^{\frac{H}{V}}$$

*) See footnote on page 41.

where:

σ_b' = ultimate bearing pressure under the plate V/bw (in kg/cm^2; see Fig. 46);
σ_w' = cube strength (in kg/cm^2);
s = distance from centre of bearing plate to edge of column (in cm);
w = width of bearing plate (in cm);
b = length of bearing plate (in cm);
A = total cross-sectional area of reinforcement (in cm^2) (in the equation the yield point for steel grade Q 24 has been introduced; because of the greater amounts of elongation, higher-strength steels provide no increase in the strength of the bearing);
the maximum quantity of steel that may be taken into account is 0.4 cm^2 per cm length of bearing plate; therefore: $A_{max} = b \times 0.4$ cm^2;
C_1 = 0 if $s < 5$ cm, and
= 1.5 if $s \geqslant 5$ cm;
C_2 = $s \times w$, with a maximum of 60 cm^2;
V = vertical bearing reaction (in kg);
H = horizontal bearing reaction (in kg).

If the horizontal bearing reaction cannot be determined with certainty, it is recommended to adopt a value equal to half the vertical reaction in the calculation.

For determining the permissible bearing pressure under working load conditions it is recommended to introduce a safety factor of 2.5.

Of course, the above equation can alternatively be used for calculating one of the other quantities if σ_b' is given.

The reinforcement A should be installed with a small depth of concrete cover under the bearing plate and be very well anchored at the ends. Good forms of construction are obtained by installing the reinforcement generally in the manner illustrated for column heads in Fig. 47.

4 Stresses at end anchorages

Another problem associated with the ends of beams is constituted by the concentrated application of the prestressing force at the tendon anchorages. It is a well known fact that any concentrated force gives rise to splitting stresses. These occur as tension acting in a direction approximately perpendicular to the prestressing force. The region where the tensile stresses are developed begins at some distance behind the point of application of the load. In Fig. 54 the stress distribution along a prestressing cable is shown schematically. A large number of investigations have been carried out for determining the magnitude and location of the splitting force. In most cases the problem has been treated as a two-dimensional one, which is of course not an entirely realistic approach. Rather widely varying results have been obtained, so that it is difficult to decide which of them provide the closest approximation to reality. The results of Zielinski and Rowe, published in the Cement and Concrete Association's

Research Report No. 9, inspire most confidence. The reason for this is that they base their conclusions on model investigations performed on the actual ends of beams. The main conclusions are:

- The stress distribution, as indicated in Fig. 54, is independent of whether the anchorage element is embedded in the concrete or bears externally against the end of the beam.
- If the anchorage is embedded, the stress distribution is the same, but the magnitude of the tensile stresses is increased by about 10%.
- The most important factor with regard to the magnitude of the stresses and the failure load is the "load concentration" $\beta = a/h_t$, i.e., the quotient of the depth "a" of the anchorage element and the depth "h_t" of the end of the beam (circular anchorage bearing plates are transformed to square ones having the same area).
- The load concentration has virtually no effect on the location of the point where the stress is zero (0.1 h_t behind the anchorage element) and the point where the splitting stress attains its maximum (0.25 h_t behind the anchorage element). The region where reinforcement should be installed can thus be determined fairly precisely, namely, from 0.1 h_t to 0.5 h_t behind the anchorage element. A helical binding is found to be more effective than a reinforcing cage of straight bars.
- The quantity of reinforcement has a considerable effect on the load capacity of the anchorage zone, up to the value at which the compressive stress under the anchorage element ($\sigma_1 = V/F$, i.e., the quotient of the prestressing force and the area of the anchorage bearing plate) is about twice the cube strength. For higher values of the compressive stress the ultimate load V_u does not further increase with the quantity of reinforcement provided.
- For $0.3 < \beta < 0.7$ the maximum splitting stress $\sigma_{y_{max}}$ and the resultant splitting force S can be calculated from:

$$\sigma_{y_{max}} = \{0.46\beta^2 - 1.3\beta + 1.1\} \frac{V}{bh_t}$$

and: $S = \{-0.4\beta^3 + 1.53\beta^2 - 1.57\beta + 0.71\} V$

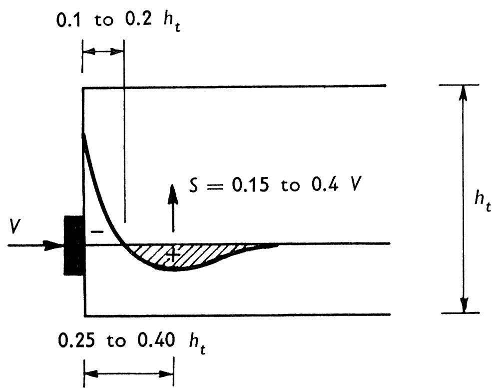


Fig. 54.
Splitting stresses due to prestressing force.

They both always act in a direction approximately perpendicular to the line of action of the prestressing force.

The cracking load is reached when $\sigma_0 = V/bh_t$ has the value 0.16 and 0.28 times the cube strength respectively, corresponding to $\beta = 0.3$ and $\beta = 0.7$ respectively.

- The presence of the cable duct has no effect on the stress distribution; it has only a very slight effect on the magnitude of the splitting stresses.

Although these investigations have not solved all the problems relating to the tendon anchorage zone, the above conclusions do indeed greatly help to clarify the matter. It is normal practice to resist the splitting force by means of reinforcement, and the equations presented above can be used for determining the quantity of such reinforcement.

Not so well known is the fact that, as a result of the simultaneous application of a number of prestressing forces, other tensile stresses are also produced. These occur between the points of application of the forces; they are indicated schematically in Fig. 55. In practice, because of these stresses, cracks are still frequently encountered in precast prestressed concrete beams. Examples of this are shown in Figs. 56 and 57.

Investigations *) have shown that the tensile stresses in these cases are smaller in

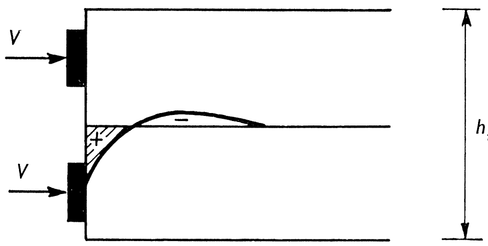


Fig. 55.
Stress distribution in a section between two prestressing forces.

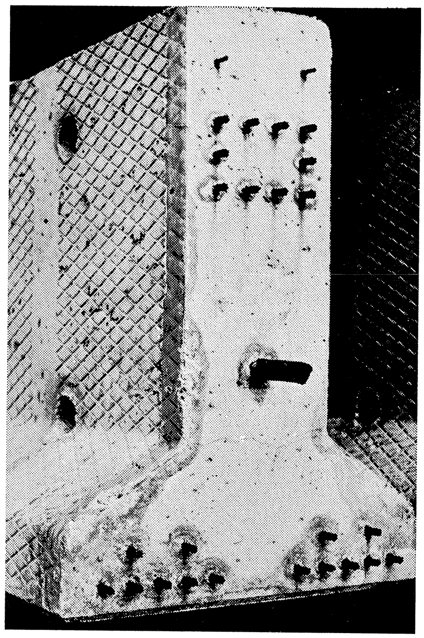


Fig. 56.
Crack between concentrated prestressing forces at the end of a beam (prestressing wires arranged in groups).

*) J. Zielinkski and R. E. Rowe: "The stress distribution associated with groups of anchorages in post-tensioned concrete members". Research Report No. 13, Cement and Concrete Association.

M. Sargious: "Beitrag zur Ermittlung der Hauptzugspannungen am Endauflager vorgespannter Betonbalken". Doctoral thesis, Techn. Univ. Stuttgart, 1960.

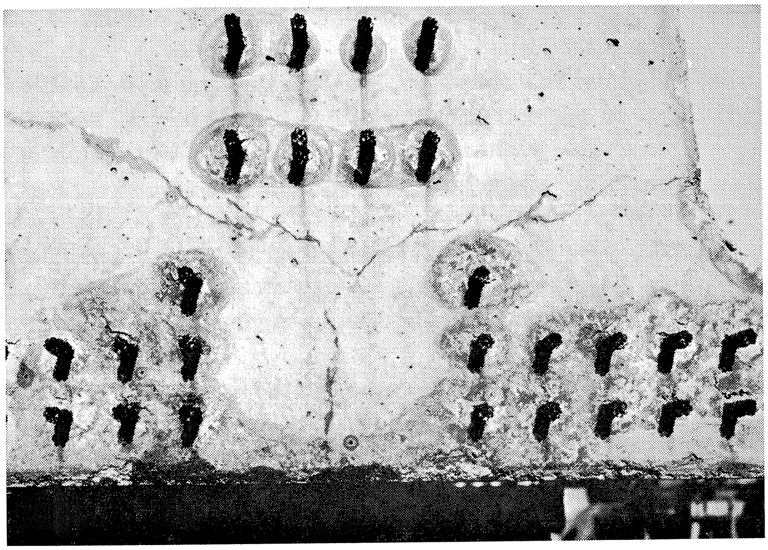


Fig. 57. Cracks between concentrated prestressing forces at the end of a beam (prestressing wires arranged in groups).

magnitude than the splitting stresses that occur along the prestressing cables. Yet in actual practice the former cause more visible damage, probably because the maximum tensile stress is developed just at the concrete surface. Clearly, this kind of cracking cannot be prevented by reinforcement; but it is possible to ensure that the cracks do not become excessively wide and that the end of the beam will not split over too great a length. In general, it is advisable to investigate whether a different arrangement of the wires over the end cross-section of the beam is practicable, so as to obtain a more uniform distribution of the application of the forces. In that case the above-mentioned tensile stresses will be limited as much as possible.

In general, prestressing cables are anchored with the aid of commercially available anchorage elements. Before these systems are applied in actual practice they are, as a rule, subjected to thorough investigation in the laboratory. Any teething troubles that may still occur can generally be put right in a short time. Yet such phenomena can give rise to very serious problems, because the strength of the structure, up to the time when the grout in the tendon ducts has hardened, depends entirely on the anchorages.

One type of anchorage which not infrequently causes trouble is the "dead-end" anchorage. In some cases it is so designed that its complete embedment in concrete can be ensured only by very careful workmanship. In actual practice this is often difficult to achieve.

5 Shear problems

Problems associated with shear force are one of the greatest sources of trouble in reinforced concrete and prestressed concrete construction. Although a considerable amount of research work has been done in this field particularly in recent years, the final solution is still a long way off.

Tests show that the bending moment at which a shear crack develops may be of smaller magnitude than the failure moment. In a beam without stirrup reinforcement the formation of a shear crack usually brings about failure of the beam. It has been found *) that this manner of failure can take place at a loading which is only 84% of the loading that the beam would be able to support in the case of normal flexural failure. The absence of stirrups therefore considerably reduces the actual safety. Since these results were obtained from short-term loading tests, they are moreover somewhat on the optimistic side: under loading of long duration the tensile strength will decrease.

It appears, however, that a relatively small quantity of stirrup reinforcement is already sufficient to enable the beam to attain its normal failure moment.

Some examples of beams which failed in shear are shown in Figs. 58, 59 and 60.

Although no instances of damage are known where premature failure was due to shear force alone, the shear force problem can nevertheless be said to have played a part in particular cases of structural damage. As appears from Figs. 58 and 59, the inclined shear crack in the web passes into a long horizontal crack at the junction between web and flange. Now if a cable duct is located at this junction, then the loading at which shear cracks will develop in the beam will clearly be unfavourably affected by the presence of this duct. This is all the more important because of the current trend to employ larger and larger tendons and therefore larger ducts.

Fig. 61 gives test results for a number of beams formed with cavities at the web-to-flange junction. The weakening caused by the large ducts clearly emerges from this diagram. If no stirrups are installed, then the failure load will be low. As already stated, however, a relatively small quantity of stirrup reinforcement will – even if large cable ducts are present – enable the normal failure moment to be attained. But this reinforcement will not appreciably delay the instant when cracks develop. Although the tests for which the results are presented in Fig. 61 relate only to one particular beam cross-section, it can nevertheless be inferred from them that holes exceeding, say, 30% of the web thickness produce a considerable reduction of the cracking load.

A very unfavourable situation arises if sheaths or ductubes are installed at the web-to-flange junction and deviate from their correct position in consequence of inadequate support. Fig. 62 shows cracking due to this, while in Fig. 63 it is clearly seen that the cable duct, originally straight, has been displaced as a result of insecure fixing.

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*) H. Rüsch and G. Vigerust: "Schubsicherung bei Spannbeton ohne Schubbewehrung". Deutscher Ausschuss für Stahlbeton, Heft 137.

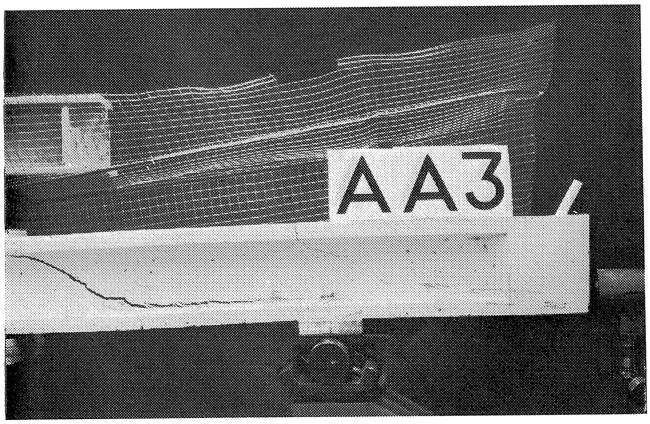


Fig. 58.
Characteristic failure due to loading in which relatively large shear forces were applied.

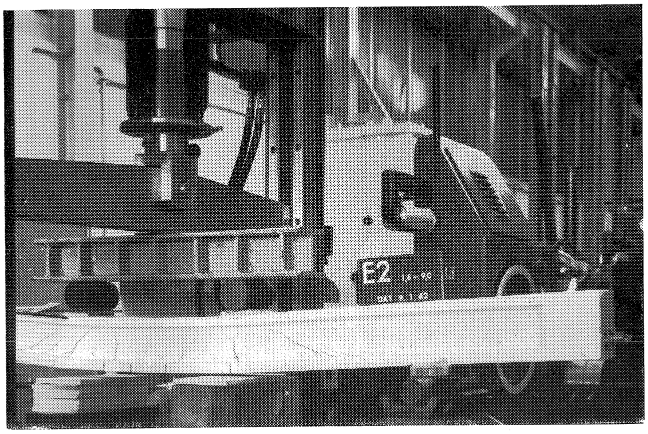


Fig. 59.
Beam under load: relatively large shear force applied.

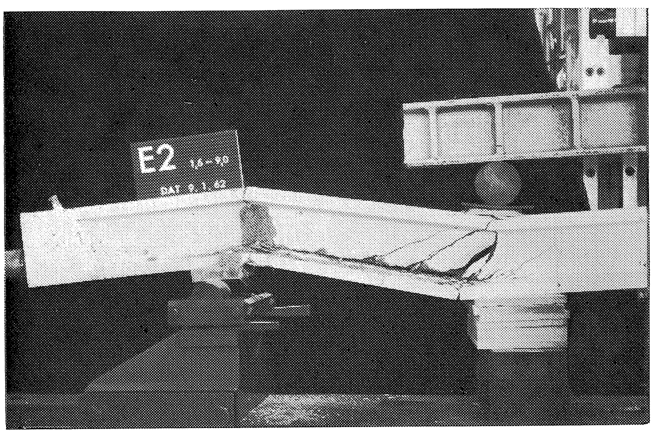


Fig. 60.
Beam after failure in shear loading test.

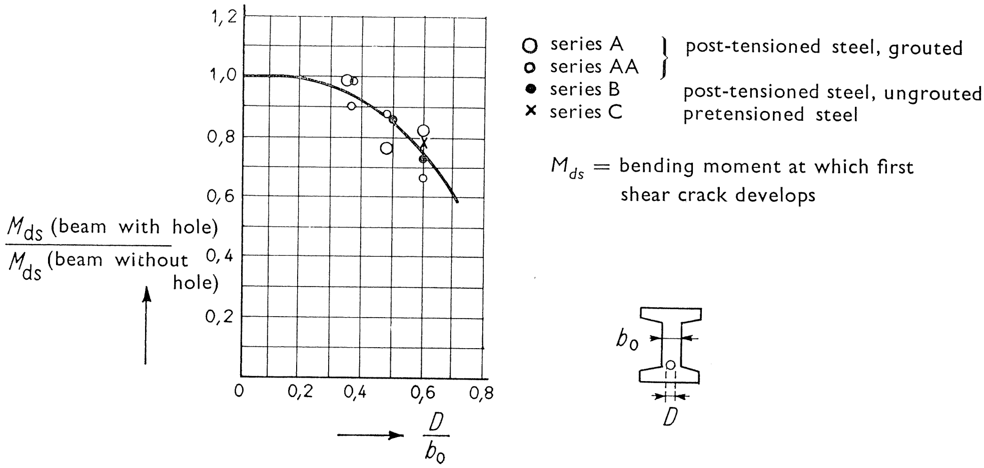


Fig. 61. Cracking moment due to shear force as a function of the size of the cable duct.

Fig. 62.
Cracking due to secondary tensile stresses caused by a displaced cable duct.

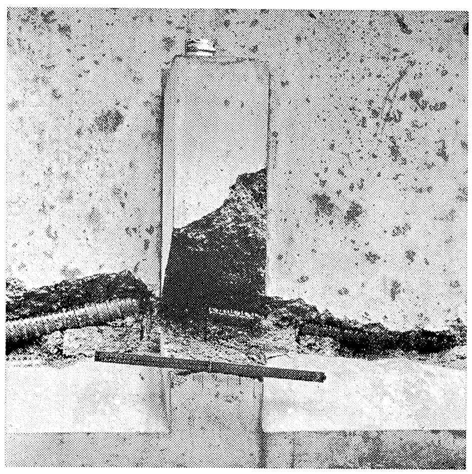


Fig. 63.
The cable duct (sheath) was displaced by a significant amount during concreting.

With the aid of a simple approximate calculation it can be shown that a "wavy" cable duct may cause very large tensile stresses indeed. From Fig. 64 it follows that:

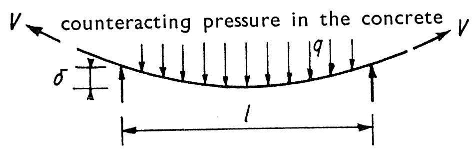
$$V \cdot \delta = \tfrac{1}{8}\, q l^2 \qquad (1)$$

where:

l = length along which the deflection occurs;
δ = deflection of the cable duct;
V = prestressing force;
q = counteracting pressure exerted by the concrete, per unit length.

The counteracting pressure q produces tensile stresses σ_b in the web:

$$q = \sigma_b\,(b_0 - D) \qquad (2)$$



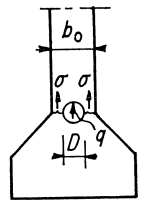


Fig. 64.
Diagram illustrating the pressure exerted on the concrete by a curved tendon.

On substitution of (2) into (1) we obtain:

$$\sigma_b = \frac{8 V \delta}{(b_0 - D)\, l^2}$$

For $V = 120\ t$, $b_0 = 15$ cm, $D = 7$ cm and $l = 100$ cm, for example, we have: $\sigma_b = 12.0\ \delta$ kg/cm^2.

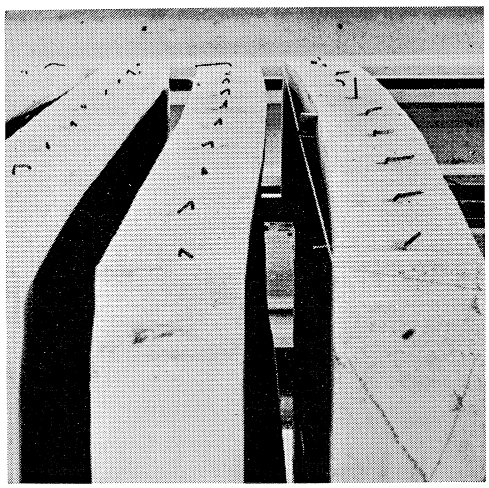


Fig. 65.
Horizontal deflection of beams in which the prestressing force is acting eccentrically in consequence of dimensional inaccuracies.

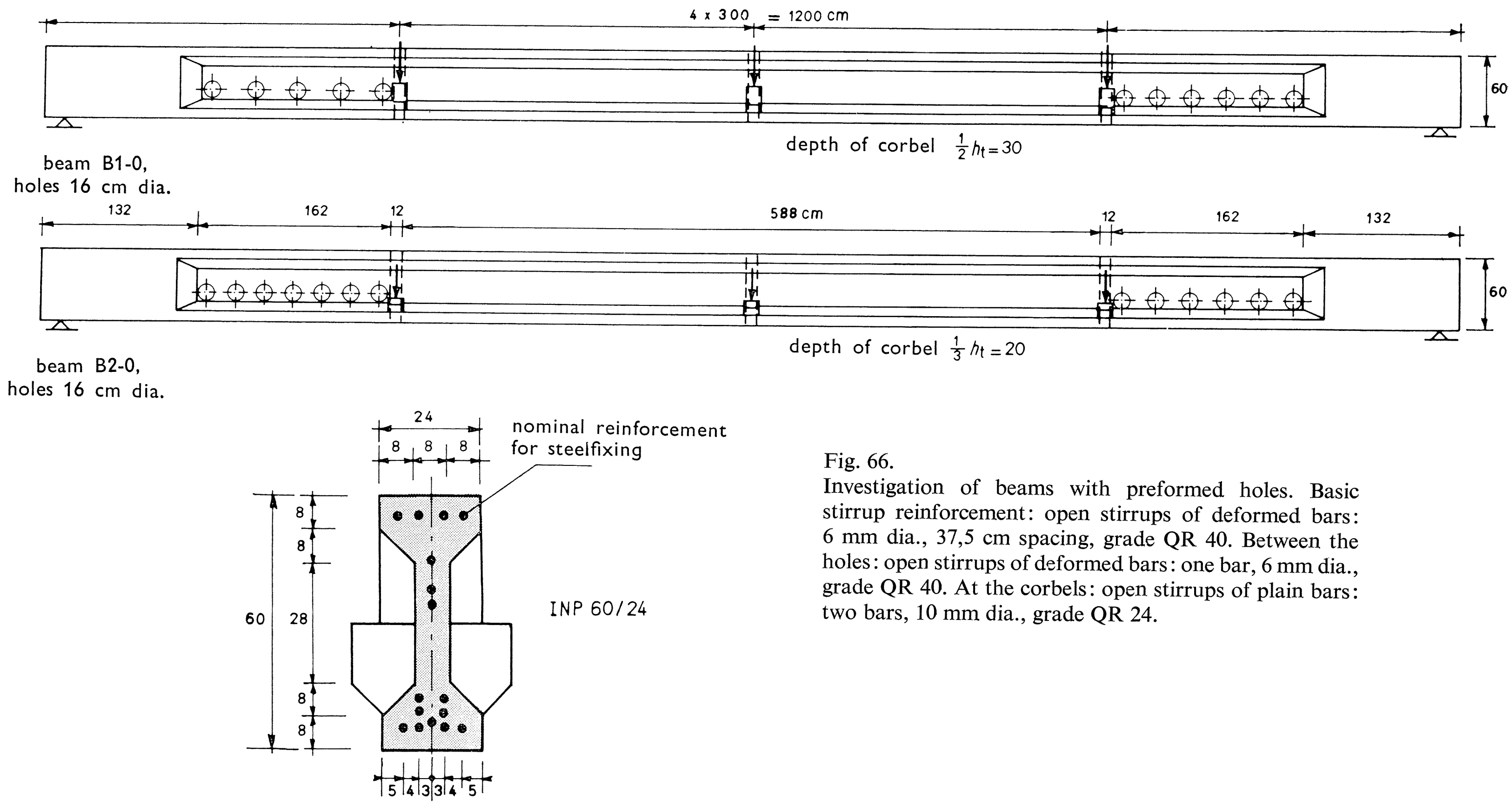


Fig. 66.
Investigation of beams with preformed holes. Basic stirrup reinforcement: open stirrups of deformed bars: 6 mm dia., 37,5 cm spacing, grade QR 40. Between the holes: open stirrups of deformed bars: one bar, 6 mm dia., grade QR 40. At the corbels: open stirrups of plain bars: two bars, 10 mm dia., grade QR 24.

Depending on the quality of the concrete, deflections (deviations from the tendon profile) of the order of magnitude of 2 to 5 cm will, in this example, cause the tensile strength to be reached. Although here, too, stirrups can prevent the bottom flange from becoming detached, they will not defer the instant at which cracking takes place. It should be noted that the cracks develop in the very place where the prestressing steel is located and may constitute a corrosion hazard.

Inaccurate positioning of the prestressing steel will of course also affect the normal cracking and failure moments.

In prestressed concrete beams with pretensioned steel the latter is also often found to be inaccurately positioned. This may be due to the fact that the side formwork panels are not straight or not properly adjusted. Instances have been observed where the prestressing wires crossed one another because they had not been inserted in the corresponding holes in the end abutments. Fig. 65 shows some beams in which the prestressing tendons is not correctly positioned. As this photograph clearly reveals, this may give rise to considerable horizontal deflection. In principle, it is possible to straighten these beams by the application of fairly small lateral forces.

When a crooked beam such as these is loaded, the loading will produce a horizontal as well as a vertical deflection. In a structure such beams, when functioning as main

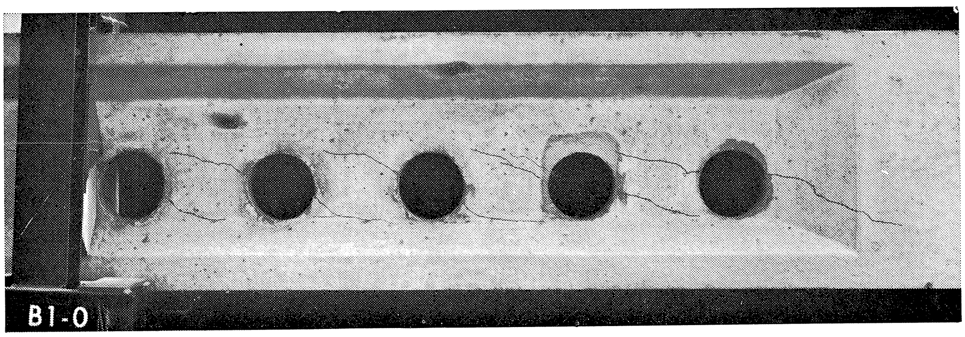


Fig. 67. End region of a prestressed beam (see Fig. 66) provided with five holes, here shown after being loaded to failure.

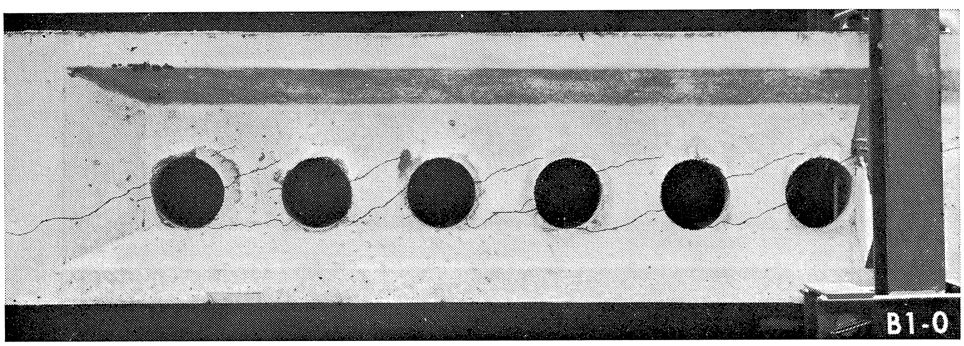


Fig. 68. End region of a prestressed beam (see Fig. 66) provided with six holes, here shown after being loaded to failure.

beams supporting secondary beams, will therefore exert horizontal forces on the latter. Also, during erection a large degree of curvature may cause the beam to tilt. It may well be asked what becomes of dimensional accuracy if such beams are actually used. In this context one need only think of the pin-and-socket connections already referred to.

In recent years it has become increasingly common practice to form holes or recesses in beams. If these are located in the central part of a beam, then the structural safety will remain essentially unaffected, provided that the compressive zone necessary for the failure moment to develop is not reduced by the apertures. Holes or recesses may, however, give rise to secondary stresses in consequence of the local shifting of the position of the centroid of the section. The external loading as well as the prestress may produce such stresses. In such cases it must be investigated how the serviceability of the beam, i.e., with regard to the attainment of the cracking moment, is affected by the holes or recesses. When the altered section properties have been determined, the calculation for determining this moment can be carried out in the usual way.

If holes or recesses have to be formed in the shear region of a beam, then the problems become more serious. For this reason beams provided with holes were also included in the tests performed in the course of the experimental research, already mentioned, by CUR Committee C 13 (see Figs. 66, 67 and 68). The tests showed that the safety could be preserved by the provision of a small quantity of stirrup reinforcement, thanks also to the presence of end-blocks on the beams. In this case the problem is mainly that of cracking between the holes. In the case where five holes had been formed (Fig. 67) there was still just no cracking between the holes when the beam had reached its cracking moment at mid-span. The magnitude of the tensile stresses that develop between the holes can be estimated by assuming that the entire shear force in the end region under consideration is resisted only by the concrete "bridges" between the holes. If an – in itself fairly ordinary – calculation of this kind is carried out in accordance with RVB 1967, section 9, it is found that up to three holes of 16 cm diameter can permissibly be formed in this end region for the permissible stresses applicable in this case. The calculation procedure is summarised in Appendix 3 on this page. Of course, appropriate stirrup reinforcement to ensure adequate shear safety must also be provided.

When holes such as these are made in beams in order to provide passage for pipes carrying hot liquids, it should be borne in mind that heat radiated from the pipes may give rise to a significantly more unfavourable state of stress around the hole than might be supposed on the basis of the present calculation.

Appendix 3

Beams with preformed holes.

From the viewpoint of ensuring adequate strength, cracks between the holes can be accepted only if, for instance, by means of end-blocks the co-operation between the top and the bottom of the beam can reliably be maintained in the shear region.

Cracking will occur in the concrete "bridges" between the holes when:

$$\sigma_{spl} = \frac{x+r}{x} \frac{T}{b_0 h_t}$$

where:

σ_{spl} = tensile strength of the concrete as given by $(10+0.05\,\sigma'_w)$ kg/cm^2, or permissible tensile stress in the concrete;
x = least length of a concrete "bridge" between two holes (in cm);
r = diameter or length of hole (in cm);
T = shear force in the region considered (in kg);
b_0 = thickness of web of beam (in cm);
h_t = total depth of beam (in cm).

The crack width in the concrete "bridges" can be approximately determined from:

$$w = \frac{1}{32} T^2 \frac{t^2 \varnothing}{E_a \cdot A_t^2 \cdot h_t^2 \cdot \tau} \text{ cm}$$

where:

t = stirrup spacing (in cm);
$\varnothing$ = diameter of stirrups (in cm);
E_a = modulus of elasticity of steel (in kg/cm^2);
A_t = cross-sectional area of one bar of stirrup (in cm^2);
τ = bond stress (60 kg/cm^2 for deformed bars).

Having regard to corrosion of the prestressing steel, the designer should investigate how the cracks are located in relation to the steel (see Figs. 67 and 68). The normally required amount of concrete cover to the prestressing steel must not be reduced. With regard to the permissible crack width the rules given in the Netherlands code of practice for reinforced concrete (GBV) may be applied, e.g. $w_{max} \leqslant 0.25$ mm.

In the beams concerned it is necessary to install additional stirrups between the holes in order to preserve adequate strength (safety) with regard to shear force. Holes must of course never be so large or so designed as to impair the compressive zone associated with the bending moment that the cross-section in question must be able to resist at failure.

6 Various causes of damage

(including poor workmanship, frost damage, mishaps during erection, and corrosion)

This chapter deals with cases where the trouble encountered was due chiefly to faults of execution. Frost damage will also be considered. Finally, some cautionary information with regard to corrosion hazards will be given.

In this connection it should be mentioned that the designer should duly consider whether he is not demanding too much on that score. Normal accuracy of execution and the possibilities for achieving this on the site must of course always be kept in view. By way of illustration, attention may be called to the case where too little space has been allowed between the cable sheaths and the formwork. Gravel pockets are liable to form here. Besides, such faulty detailing constitutes an inducement to the man on the job to apply excessive vibration or tamping of the concrete, with the attendant hazard that the sheaths will be displaced or deformed. As already indicated, such deviations give rise to considerable secondary stresses which, together with the weakening caused by the excessively large duct and the poor quality of the concrete, may be a real hazard to the structural member in question.

The quality of the concrete in the structure as a whole should fulfil exacting requirements. For prestressed concrete it is of the greatest importance that the specified concrete quality, i.e., its strength, is indeed obtained. The designer should consider whether the design value of the strength which he adopts in his calculations is really attainable in the actual structure. Attainment of the specified strength must be checked by means of test cubes. These should be 20 cm in size and be tested in accordance with the rules laid down in the Netherlands code of practice for prestressed concrete (RVB 1967). Yet non-standard specimens and test procedures are sometimes employed. These of course lead to incorrect assessment of the quality of the concrete.

It is always advisable to ascertain that the specified concrete strength is indeed obtained in the structure. Destructive or non-destructive testing techniques may be used for the purpose (drilled cores, Schmidt rebound hammer, etc.). Difficulty in attaining the specified strength of the concrete has hitherto occurred mainly in the construction of very deep beams. In such cases the quality of the concrete is liable to vary considerably over the depth of the beam. In some instances the strength, on proceeding from the bottom of the beam to the top, was found to be only 70–50% of the required value.

Joints between precast units which are assembled into a single whole by prestressing call for careful execution. This is often underrated. By way of illustration it may be reported that the quality of the concrete in a number of 3 cm wide joints was checked some time ago. These joints had been filled by caulking with mortar, which had been rammed in. The average strength was found to be 463 kg/cm^2 with a standard deviation of 194 kg/cm^2, i.e., a coefficient of variation of 42%. The characteristic strength with 95% probability of being exceeded is then only 144 kg/cm^2. In a few instances there was considerable variation in the thickness of the joint over the depth of the beam. Besides, the cross-sections of the beam units on each side of the joints did not properly fit each other because of poor dimensional accuracy.

All these factors caused irregular stress distribution, resulting in the occurrence of tensile stresses. These in turn caused spalling of corners. It is advisable always to cater for the possibility that such effects may arise by providing the ends of precast units with reinforcement in the form of meshes. Even with very careful execution the member cannot be expected to possess flexural strength (tensile strength of the concrete in bending) at a joint between precast units, and the Netherlands code (RVB 1967) in fact specifies that no such flexural strength is to be taken into account. The

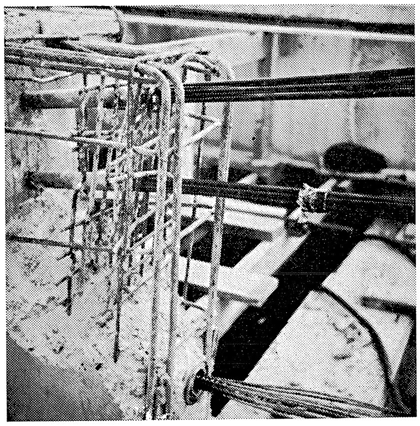


Fig. 69.
Example of poor quality concrete in a precast anchorage block.

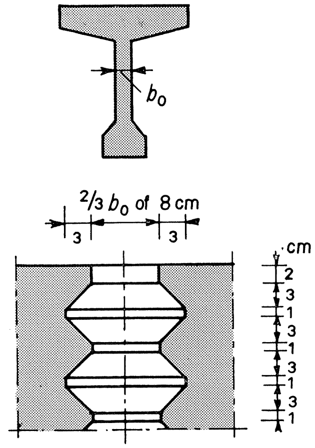


Fig. 70.
Construction of a joint as recommended in R.V.B. 1967.

best possible shear connection between precast units and in-situ concrete joints is obtained only if some kind of mechanical interlocking of the concrete at such joints can be provided. This can, for example, best be achieved by roughening the concrete of the precast unit by thoroughly scratching it with a steel rod before it has hardened. This procedure will, however, be impracticable in most cases, as it can be applied only to unshuttered surfaces.

Recent investigations have shown that the use of checker plate as a formwork surface or the bush-hammering of the concrete face are of little use. For practical purposes such surfaces behave as if they were smooth.

Crenellations which ensure "real" mechanical interlocking of the concrete do of course provide an excellent solution (see Fig. 70).

One of the "ten commandments" relating to prestressed concrete construction states that prestressing implies: compressing together. The required compressive stresses (the precompression) are achieved only if the corresponding shortening of the concrete can take place. This may appear almost too obvious. Yet it is sometimes overlooked, and this is more particularly liable to occur in a case where a precast and already partially prestressed beam is subsequently given its final prestress after being installed in position in the structure.

Some years ago a case of damage occurred in a cooling tower of hyperbolic shape. To strengthen it, a horizontal prestress was to be applied circumferentially around the tower. On account of failure to secure the prestressing wires properly in their correct position, they shifted towards the position where the tower circumference was smallest.

In this context another example calls for mention. In this case beams had been prestressed with Dywidag bars which had been assembled from shorter lengths by splicing with special coupling sleeves. At such splices the duct is always locally widened to provide – in principle – the freedom of movement required for the sleeve when the bar is tensioned and therefore undergoes elongation. The suppliers of the prestressing bars issue clear directives to that effect. Yet in this case the precautions were overlooked: the bars were tensioned from one end of the beam, but the requisite amount of play for the coupling sleeve in the direction of tensioning had not been provided. As a result, the sleeve reached the end of the locally widened part of the tendon duct. With further increase in the force applied at the jack, tension was being introduced only into the part of the bar between the latter and the sleeve, no further prestress being introduced into the rest of the beam. If the tendon extensions had been properly checked, the trouble would doubtless have quickly been detected. However, tensioning was continued, with the result that a substantial proportion of the prestressing force was transmitted into the beam at the point where the coupling sleeve was bearing against the concrete. This caused a substantial area of the concrete to spall off, as shown in Fig. 71.

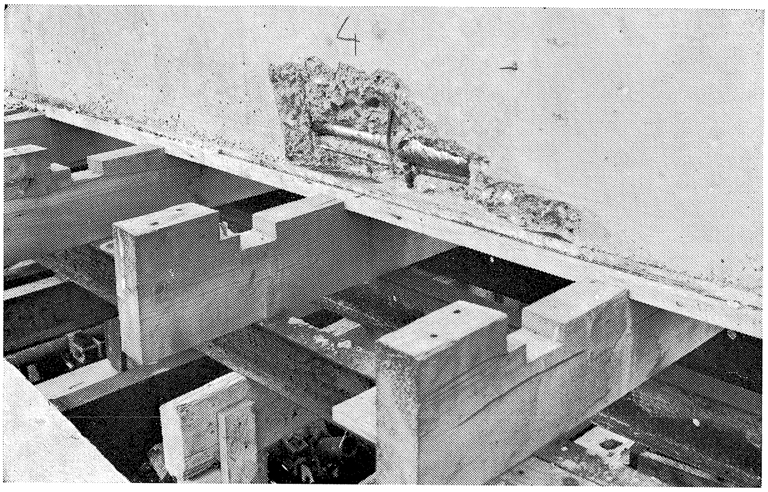


Fig. 71. Concrete has spalled off in consequence of a splicing sleeve bearing against the concrete internally within the cable duct.

Problems which may arise in connection with the grouting of prestressing cable ducts have been investigated in detail by CUR Committee B 6, whose results are presented in CUR-report no. 27 "Investigation into grouting problems in prestressed concrete". Besides giving a full account of the investigation, this report gives some "hints for execution". In this context a good deal of attention is paid to the occurrence of frost damage. Damage due to frost is nevertheless still a fairly frequently encountered phenomenon, both in grouted and in ungrouted cables *) (see Fig. 72). It appears that prolonged exposure to low temperature – just above freezing point –

*) Bolle and Poitevin: "Comportement des mélanges anti-gel pour le remplissage de gaines en attente de câbles". Annales de l'Institut Technique du Bâtiment et des Travaux Publics. November 1964.

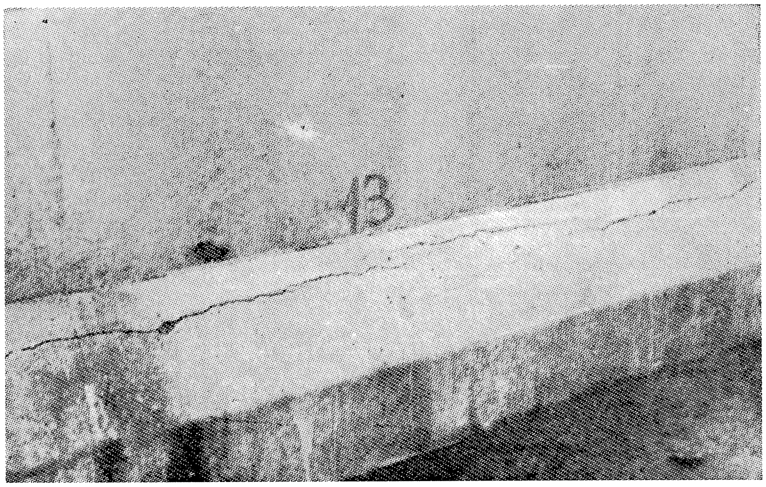


Fig. 72.
Cracking due to freezing of water in a cable duct.

prevents the grout in freshly grouted ducts from hardening. Ungrouted ducts often become filled with rainwater.

This point calls for one further comment. In circumstances where low temperatures prevent grouting, it is normal practice to fill the ducts with an antifreeze (a liquid whose constituents include water and methylated spirit) if it is not possible with certainty to exclude water from them pending final grouting. Laboratory tests have shown that some types of antifreeze can be displaced by rainwater because of the slightly higher specific gravity of the latter. This has on occasion given rise to frost damage in cases where it was supposed that adequate protective precautions had been taken.

Another familiar example of frost damage is associated with the recesses or sockets for the installing of railings in the edges of bridge decks. Such damage can be obviated by providing a temporary filling of not-too-hard consistency.

As already noted in the discussion of pin-and-socket connections, vertical cavities are liable to give rise to frost damage.

This is strikingly exemplified by the following case, which relates to a pipe bridge constructed from reinforced concrete portal frames supporting two lines of precast prestressed concrete beams (Fig. 73). In order to connect these beams securely at their supports, two Dywidag prestressing bars, which cross each other and are inserted through ducts preformed in the beams, were installed at each junction.

When these bars had been tensioned, the ducts were filled with grout, and the recesses at top and bottom of the beams were filled and encased in concrete.

After three winters, damage was found to have occurred, as shown in Figs. 73, 74 and 75. It appeared that the concrete casings over the anchorage recesses for the prestressing bars (see Fig. 74) had become detached from the beams in consequence of shrinkage, climatic influences, etc. As a result of this, water had collected in the recesses, i.e., in the "shrinkage gaps". This water was practically unable to penetrate into the dense concrete of which the beams consisted, but it did make its way into the

Fig. 73.
Part of pipe bridge, showing frost damage.

much more porous grout. Quite possibly there may also have been shrinkage gaps between the hardened grout and the duct walls. Because of the sealing of the recesses at the bottom of the beams, the water remained in the prestressing ducts and accumulated there. When it froze, cracking occurred, and concrete was spalled off at the bottom.

Fig. 74. As a result of shrinkage, climatic effects, etc. the encasing concrete over the recessed tendon anchorage has become detached, permitting ingress of water. The nib on the right was cast monolithically with the beam and has not become detached.

Fig. 75. Water collected in the cable duct, with the result that cracking occurred and a piece of concrete spalled off.

The damage was repaired and a watertight seal was applied at the upper ends of the prestressing bars, at the top of the beams. No further trouble occurred after this had been done.
This case should be borne in mind when dealing with the construction of continuous prestressed concrete bridges. In such structures the venting holes of the cable ducts at the intermediate supports are located at the top of the beam. Here, too, there exists a considerable hazard that water (possibly containing de-icing salts) will penetrate through shrinkage gaps into these ducts.

An example of frost damage to a box-section bridge structure is illustrated in Fig. 76. Here again the damage was due to water having collected in the cavities between the beams. In such cases it is essential to provide a good drainage outlet from each cavity and to ensure that this outlet does not become blocked.

Sometimes deformations due to creep or to temperature effects produce disagreeable surprises. It has already been pointed out – when bearings were being discussed – what types of damage can occur in consequence of temperature effects in a bridge constructed from precast concrete beams with subsequently established structural continuity (Fig. 48).

As a result of creep deformation, long steel railings on bridges may buckle. An

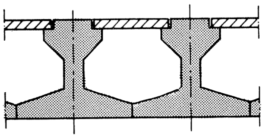


Fig. 76.
Part of cross-section through a hollow bridge deck. In winter the cavities between the precast beams became filled with water and ice, causing the small cover slabs to be forced off.

example of this is illustrated in Fig. 77.*) A calculation showed that buckling was inevitable, even if no more than the normal creep values were considered.

Greep of concrete may also cause buckling of floor tiles, as illustrated in Fig. 78.

Leonhardt **) has reported a case of damage to a prestressed concrete bridge about five years old. The trouble was mainly attributable to temperature effects. The

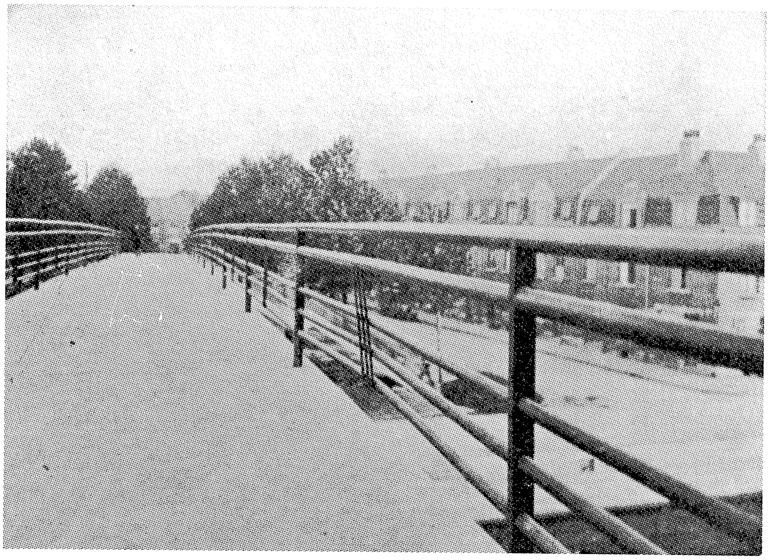


Fig. 77.
Railings buckled due to creep deformation of a bridge.

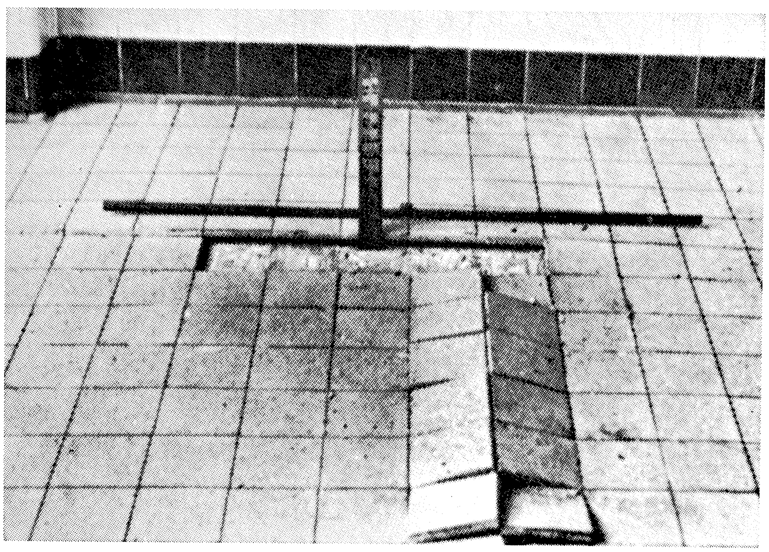


Fig. 78.
Buckling of a tiled floor due to creep of (reinforced) concrete.

*) M. F. Dumas: "Effets des déformations différées des éléments d'une construction sur leur précontrainte." Annales de l'Institut Technique du Bâtiment et des Travaux Publics. November 1964.

**) F. Leonhardt et al.: "Temperaturunterschiede gefährden Spannbetonbrücke". Beton- und Stahlbetonbau, 1965, No. 7.

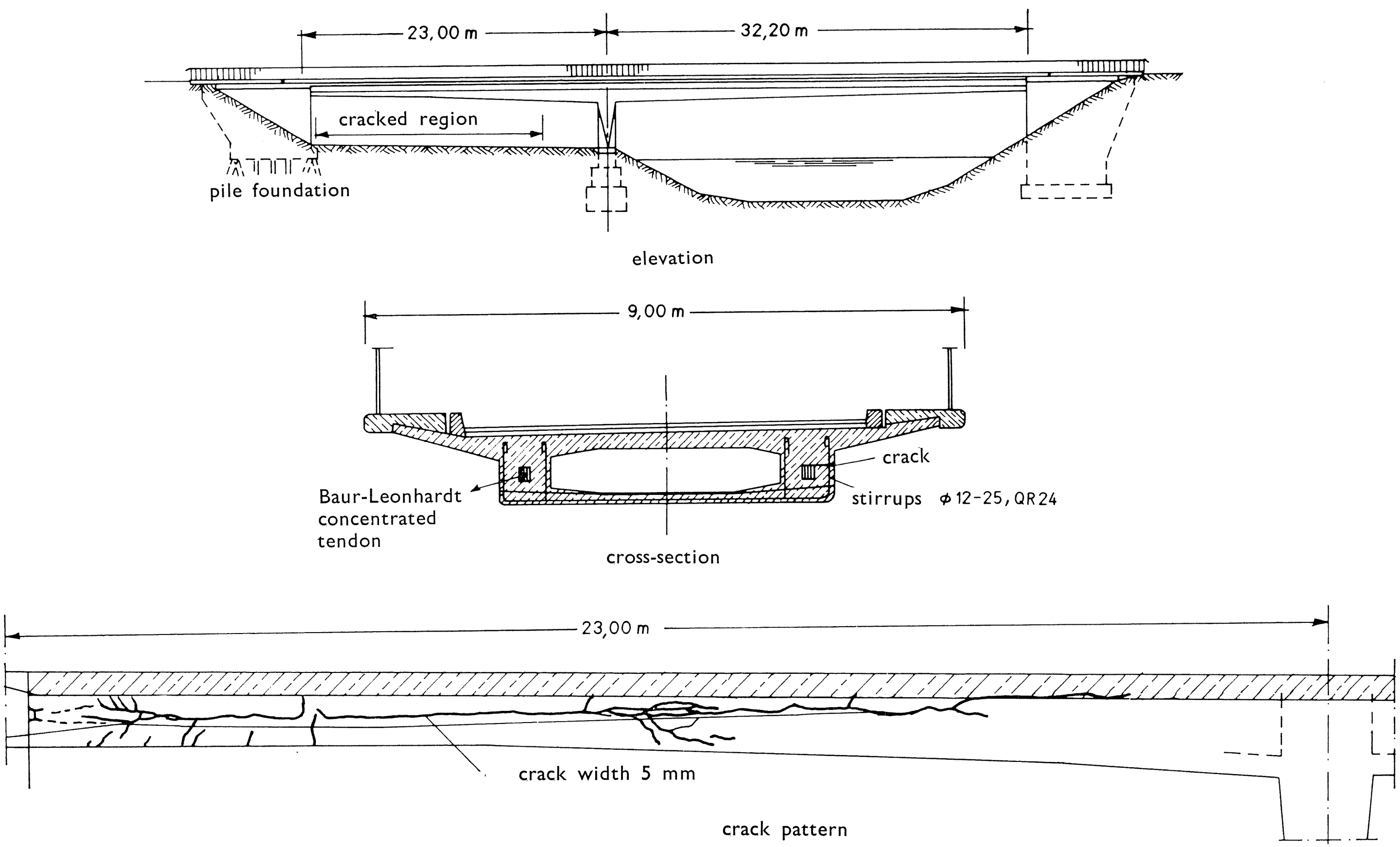


Fig. 79. Cracking of a bridge due to temperature effects (Jagst bridge, described in Beton- und Stahlbetonbau, 1965, No. 7).

essential constructional features of the bridge are presented in Fig. 79. Insufficient stirrup reinforcement to resist the tensile stresses had been provided. Because of this the crack, which was 5–6 mm wide, was able to extend over a considerable length. The bridge was subsequently repaired by the application of a vertical prestress. In this way the cracks, after having been filled with a synthetic resin, were squeezed shut.

From the same publication it emerges that the use of large concentrated prestressing tendons calls for the exercise of due care. Considerable stress concentrations arise in consequence of the major discontinuities in the concrete section. A more distributed prestress would certainly have produced more favourable conditions and would perhaps have limited or even prevented the cracking that occurred. The temperature values which the article reports to have occurred in an asphalt surfacing as a result of the sun's rays are certainly rather high. Besides bringing out the fact that the available information concerning the temperatures liable to occur in bridge structures is still imperfect, this case teaches us that the influence of temperature must not be underrated. It is advisable to provide a sufficient quantity of reinforcing steel to prevent wide-open cracks from developing in structural members in which the magnitude of the tensile stresses is in some doubt.

Another point which requires due attention, more particularly in bridges, is the risk of damage due to collision with vehicles. It happens more frequently than might be supposed that the underside of a bridge superstructure is struck by traffic passing under it (see Fig. 80).

For bridges constructed from precast concrete beams it is advisable to install transverse connecting beams over the carriageways that pass under the bridge. These can prevent the whole of the impact due to a relatively minor collision from being absorbed by one of the main beams alone. Even in the event of a more serious

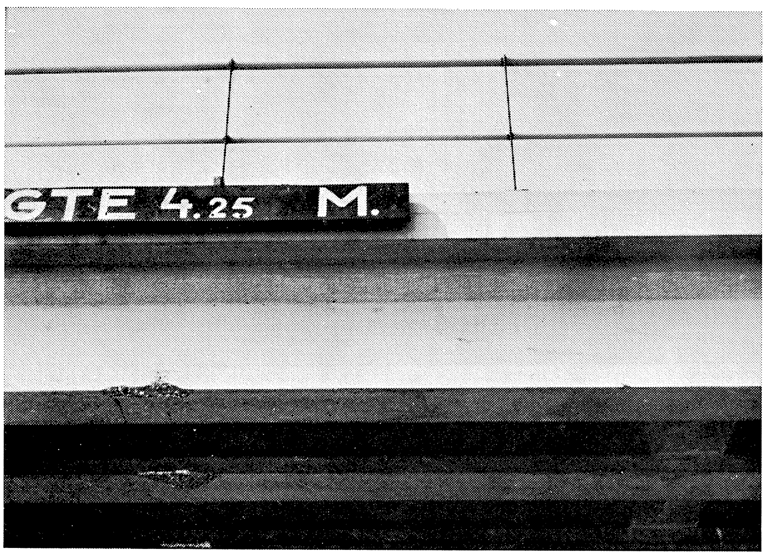


Fig. 80. Damage due to collision on a bridge constructed from precast prestressed concrete beams.

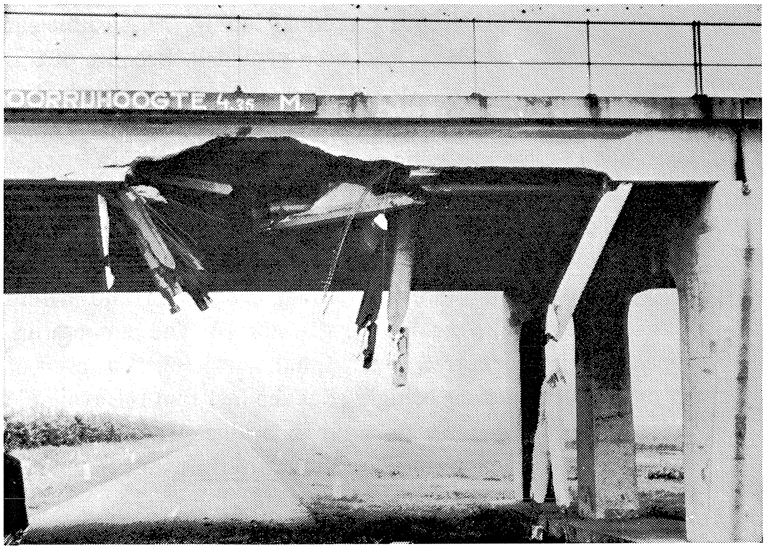


Fig. 81.
The consequences of a serious collision.

collision, the result of which is shown in Fig. 81, the extent of the damage would certainly have been less if the cross-beams had been more favourably located.

Particularly during erection of the bridge, collision with vessels or vehicular traffic is liable to cause serious damage. In the example illustrated in Fig. 82, a colliding vessel knocked off one beam which then in turn dislodged the adjacent beams in a kind of chain reaction.

Other causes may also dislodge beams and produce similar disastrous consequences, as Fig. 83 shows. In this case the beams were seated on rubber bearings and were interconnected at the top only. When one of the last beams had been installed in position, it happened to rest obliquely on the bearing pad. As a result, the beam slipped off, fell over, and overturned the adjacent beam, and so on. If the beams had

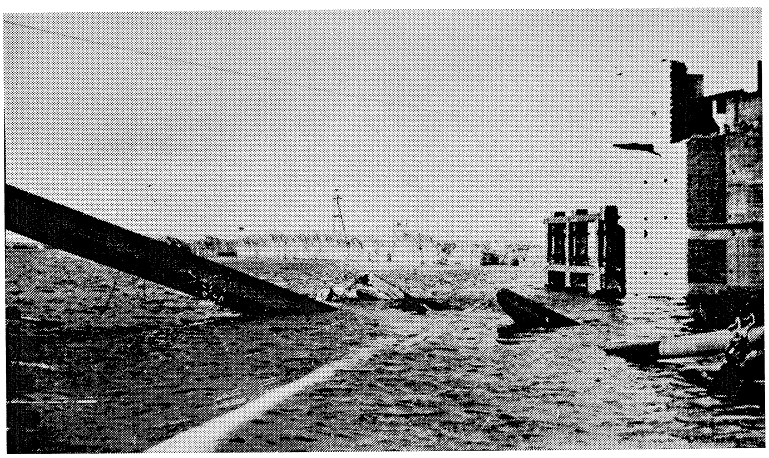


Fig. 82.
The consequences of collision with beams installed but not yet secured in position.

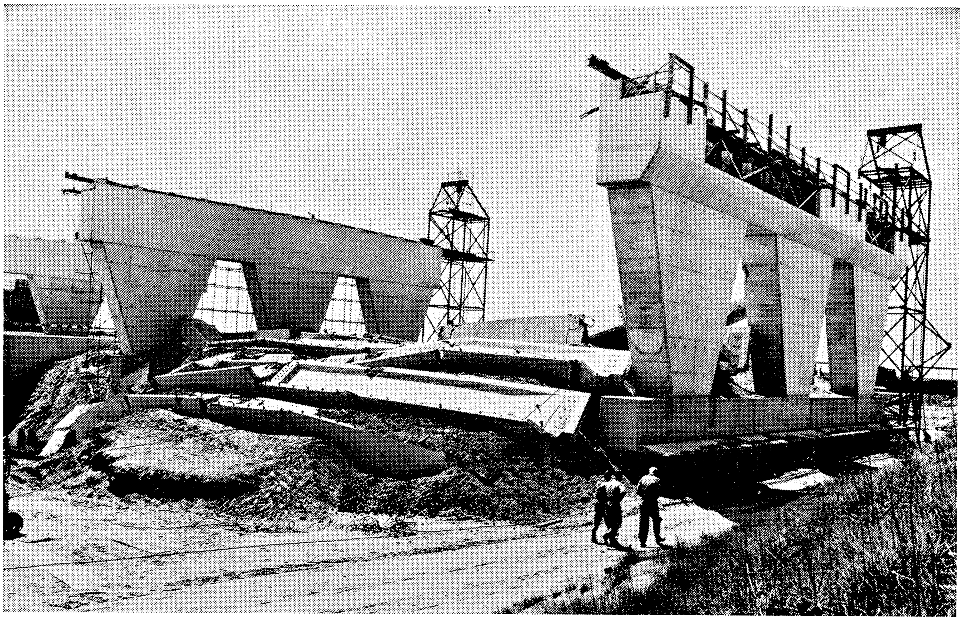


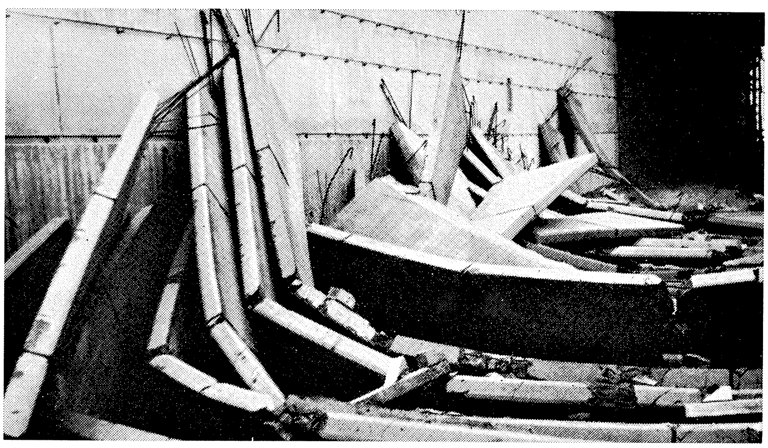
Fig. 83. Toppled beams.

been interconnected at the underside as well as at the top, this mishap could have been avoided.

Another example is illustrated in Fig. 84. Here the last beam to be erected in the construction of a sun baffle grid at the entrance to a highway tunnel overturned.

Fig. 85 shows a series of beams which had been stored side by side. Storage had been carried out very conscientiously. The beams had been interconnected with wire. When the first beam was subsequently removed for despatch, the men forgot to disconnect one wire, with the result that the whole series of beams was pulled over.

Fig. 84.
Beams of a sun baffle grid at a tunnel entrance after collapse due to toppling of the last beam.



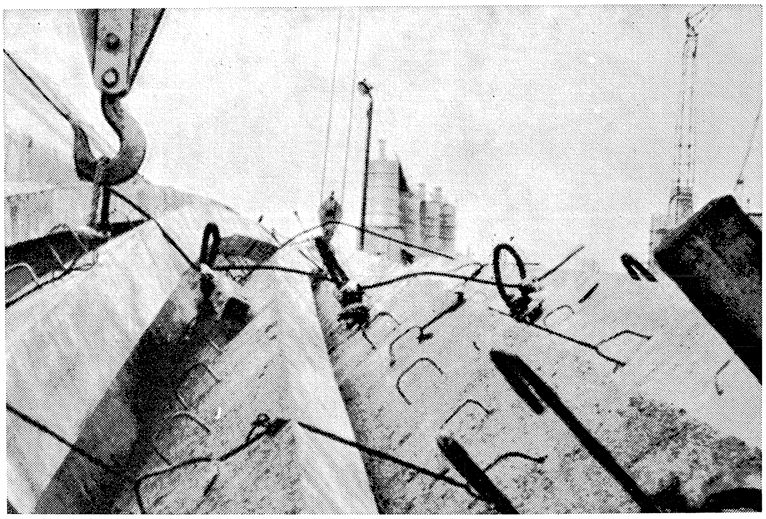


Fig. 85.
A series of beams which tilted and fell over.

In stacking precast concrete beams one upon another, it is necessary to take care that the beams will not, as a result of crushing of the timber battens interposed between them, bear down on the lifting eyes or loops protruding from the beams underneath. Should that happen, the stacked-up beams will inevitably topple over.

Due care must be exercised during erection. Not infrequently a beam is dropped from a crane during lifting. This is more particularly liable to occur with beams in which the centre of gravity of the cross-section at mid-span is located higher than the

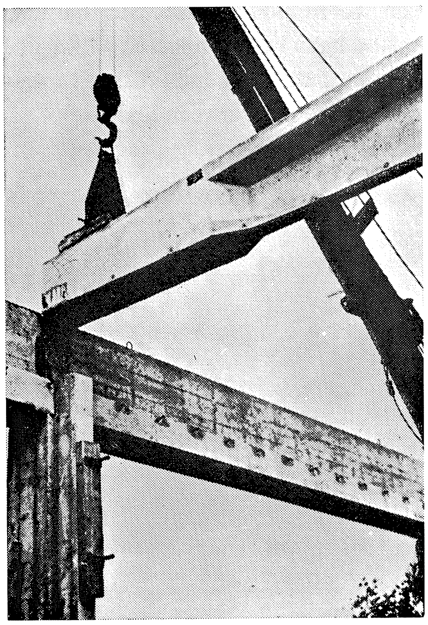


Fig. 86.
Special attachment to raise the level of the slinging point and thus prevent toppling.

centre of gravity at the lifting eyes. This tipping-over hazard can be avoided by placing the point of application of the lifting hook higher up, which can be achieved by an attachment such as that shown in Fig. 86: if it is securely fixed to the beam, it will achieve the desired object.

It has, however, also occurred that ordinary beams of constant cross-section over their whole length have fractured during lifting. The cause was that the hoisting slings were inclined inwards, thus producing a horizontal (compressive) force of such magnitude in the beam as to make it buckle.

There have also been cases where a combination of buckling and tipping-over occurred in prismatic beams under circumstances where horizontal compressive forces could scarcely have played any part. The beams in question were very slender ones, however. Horizontal oscillations produced an unstable condition. It is virtually impossible for this to occur with normal beams. Some further information on the subject is given in Appendix 4 on page 71. A fairly simple check calculation will obviate unpleasant surprises arising from this source.

Finally, something will be said about damage due to special causes. Various research committees of the CUR have reported on these. The fire resistance of prestressed concrete is more particularly envisaged. At the present time a considerable amount of research work on the subject is in progress all over the world. The publications of CUR Committee C 4 should be referred to.*) It is to be noted that beams which have to fulfil fire resistance requirements should always be provided with closed stirrups.

With regard to problems associated with atmospheric attack and weather resistance of concrete and with additives to concrete, reference should be made to the reports of the CUR Committees B 5, B 7 and B 9.**) When additives are employed it is important to satisfy oneself that they contain no constituents which, when present in certain quantities, are harmful to the concrete and/or the steel (e.g., sulphides, chlorides, nitrates).

The prestressing steel as such should also receive the necessary attention. Clearly, for structures which will be exposed to aggressive surroundings, it is essential to make quite sure that the steel is properly protected from corrosive attack. In many cases where damage occurred it was found that no very exacting requirements had been applied to the protection of the steel.

There are, however, two aspects which call for some further comment.

The first of these does not relate to corrosion in the ordinary sense but to the considerable reduction in the strength that prestressing steel is liable to suffer if it is

*) CUR-report no. 13: Brandproeven op voorgespannen betonliggers (Fire tests on prestressed concrete beams).

**) CUR-report no. 22: Weerbestendigheid van beton (Weather resistance of concrete).
CUR-report no. 31: Toevoegingen aan betonspecie (Additives to concrete).
CUR-report no. 32: Technologische invloeden op scheurvorming in beton (Technological influences on cracking in concrete).

,,Betonvereniging", Postbox 61, Zoetermeer, The Netherlands.

welded during execution of the work. In any case, such treatment will introduce "impurities" into the material, thus greatly increasing the corrosion hazard. Hence the following rules should be observed on the site:

1. Do not weld prestressing steel. If welding has to be done in the vicinity of prestressing steel, the latter will be adversely affected if sparks jump on to it. Instances of wire fracture due to this cause have been reported.*)
2. Take care that no splashes of weld metal get on to the prestressing steel, as these, too, may considerably reduce its strength.
3. Never use prestressing steel as an ordinary earth conductor or a lightning conductor and therefore never attach the earth terminal of a welding transformer to it. It is advisable to provide separate rods for earth conductors and lightning conductors.

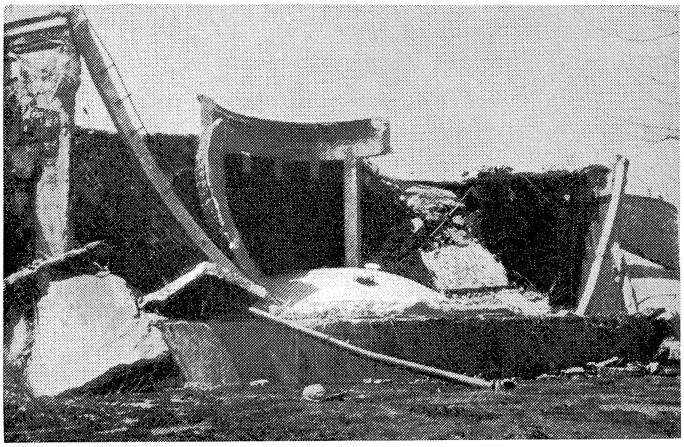


Fig. 87.
Sludge tank (30 m diameter) which collapsed after ten years in consequence of corrosion of prestressing steel.

The second important point in connection with corrosion is the formation of galvanic cells. This constitutes a real danger to the structure, see Fig. 87.

It is a generally known fact that when two different metals are in an electrically conducting environment and make metallic contact with each other, a galvanic cell may be formed. This process may be initiated by a variety of causes. While it has not yet hardened or while it is still wet, the concrete functions a conducting medium (electrolyte). In the galvanic cell the "baser" of the two metals is attacked: metallic ions enter into solution. On the evidence of these facts alone there can thus be no objection to employing prestressing steel in conjunction with baser metals, such as zinc, in concrete.

Of course, the principle of galvanic cells has here been only very briefly indicated. Much more on the subject is to be found in a number of good text-books or reference

*) Ir. G. W. P. van der Heiden: "Een merkwaardig geval van draadbreuk van voorgespannen hardstaalwapening". (A curious case of wire fracture of prestressed steel.) Cement 1965, No. 17–18, p. 425. Amsterdam Netherlands.

books. In most of them, however, the treatment of the subject leads to the conclusion stated at the end of the preceding paragraph. But the most important aspect from the viewpoint of prestressed concrete technology is not considered, namely, that in certain circumstances hydrogen atoms (deriving from the water of the electrolyte) can be liberated at the "nobler" metal. A proportion of these free hydrogen atoms penetrates into the metal and undergoes further propagation in it by diffusion. The hydrogen atoms then combine with one another to form molecules. This phenomenon is accompanied by an increase in volume, whereby high internal stresses are set up in the metal. In prestressing steel these stresses, in combination with the prestress itself, may attain the tensile strength of the steel, with the result that fracture suddenly occurs.

The combination of prestressing steel and baser metals in concrete (e.g., the use of galvanised stirrups in prestressed concrete) therefore also produces a very dangerous situation. The object of the foregoing very incomplete description of the phenomenon has been to utter a warning. Designers who, for whatever reason, wish to incorporate metals other than steel into prestressed concrete structural members are advised to seek expert guidance in the matter.

CUR Committee B 4 has undertaken the study of this subject and will shortly be publishing a report on it.

Appendix 4

Combined buckling and tilting

Precast concrete beams are generally erected with the aid of cranes. The slinging ropes are connected to eyes or loops embedded in the beam. In the course of the handling movements necessary for installing the beam in its final position in the structure it is almost impossible to prevent the beam from undergoing some oscillation. This is associated with deflection in the lateral direction because the line of action of the dead-weight bending moment then no longer coincides with the vertical direction of the beam. This lateral deflection rapidly builds up in magnitude, since the flexural stiffness in that direction is normally relatively low.

The stability of the oscillating beam is determined mainly by the magnitude of the deflection in question. In order that the dead weight of the beam will continue to counteract the oscillating motion, it is necessary that the centre of gravity Z of the deflected beam and the centre of gravity Z_c of the beam cross-section at the slinging points (where the lifting eyes are) should always be located on the same side of a vertical line through those points (which are the points of rotation), see Fig. 88.

If that requirement is fulfilled, there will be stable equilibrium. The above condition is independent of the rotation φ, because the location of the centre of gravity Z and the magnitude of the deflection are both associated with φ in the same way. The magnitude of φ is, however, indeed of importance in so far as considerations of strength are concerned, because the beam must, when in this position, be able to resist the bending moment due to dead weight.

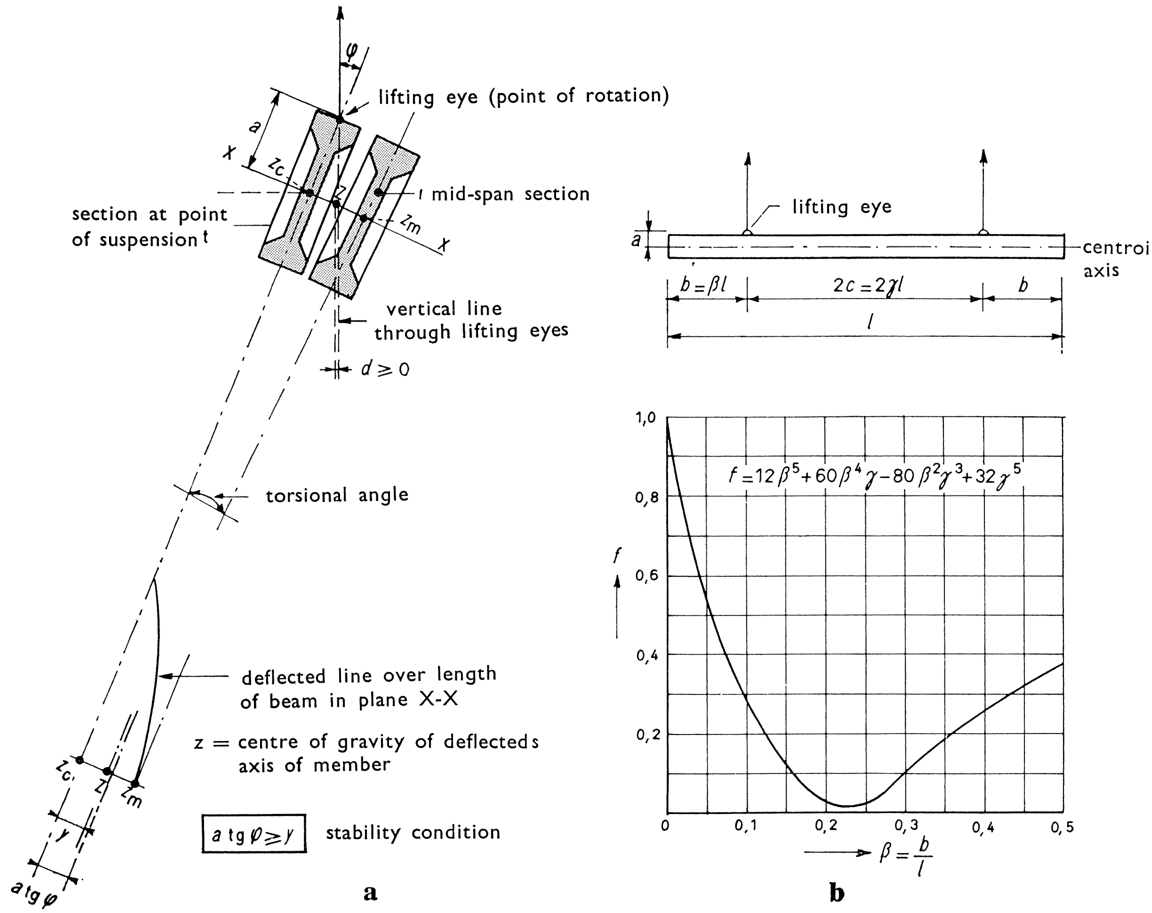


Fig. 88. a. Deflection of a beam due to oscillation during lifting.
b. The factor f for calculating the minimum distance a between the slinging point (lifting eye) and the centre of gravity in connection with combined buckling and tilting in a case where the beam is not lifted at its end (from Gotthard Franz: "Konstruktionslehre des Stahlbetons", Erster Band, Springer-Verlag, Berlin).

Continued rotation of the beam, which would cause it to buckle as a result of tipping over, is obviated by imposing a minimum value for the distance a between the point of rotation of the beam and the centre of gravity of the section ($a \, tg \, \varphi \geqslant y$).

For calculating this distance a *) it is presupposed that the flexural stiffness in the vertical direction of the beam is very large in relation to the flexural stiffness in the lateral direction. The effect of vertical deflection can then be neglected.
The minimum distance a is thus given by:

$$a = f \cdot \left\{ \frac{gl^4}{120EI_d(1-k^2)} + \frac{16}{25}p \right\}$$

where:

a = minimum distance between the slinging point and the centre of gravity of the beam (in m);

g = dead weight of the beam (in t/m);

l = length of the beam (in m);

EI_d = flexural stiffness (or rigidity) of the beam in the lateral direction (in tm^2);

k = effect of torsion of the beam, i.e.,

$$k = g/g_w, \text{ where } g_w = \frac{30}{l^3} \sqrt{EI_d \cdot GI_w}$$

(GI_w is the torsional stiffness of the beam);

p = rise (middle ordinate) of the camber of the beam (in m) (for an approximate calculation see lit. ref. No. 2); also approximately the vertical distance between the centre of gravity at mid-span and the centre of gravity of the end sections of a beam with a ridged profile (see the worked example below);

f = $12\beta^5 + 60\beta^4\gamma - 80\beta^2\gamma^3 + 32\gamma^5$ is a factor which represents the effect of the location of the points of rotation along the length of the beam (see Fig. 88); if the beam is slung at both ends, then $f = 1$.

As a rule the torsional stiffness of the beam is so great that its effect can be neglected. In that case the term k^2 becomes zero.

The above calculation does not take account of all possible unfavourable factors such as, for example, eccentric positioning of the slinging points. A more unfavourable situation will arise also if the beam already has a lateral curvature from the outset or if it is subjected to impact in the vertical direction. For these reasons it is advisable to adopt for the length a always a larger value than obtained from the calculation: for example, the calculated value may be multiplied by 1.25.

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*) 1. Franz, Prof. Dr.-Ing. Gotthard: "Konstruktionslehre des Stahlbetons. Erster Band". Springer-Verlag, Berlin.
2. Ir. B. W. v. d. Vlugt and H. W. Bouwer: "Stabiliteit van geprefabriceerde elementen tijdens montage". (Stability of prefabricated elements during the assembling.) Cement 1962, No. 10, pp. 595–597. Amsterdam Netherlands.

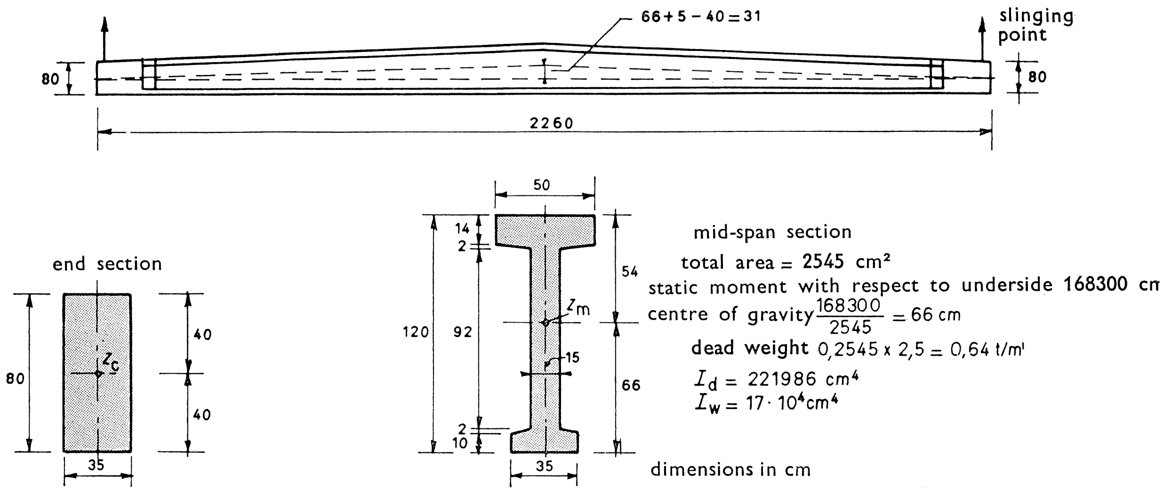

Fig. 89. Combined buckling and tilting of a beam.

Example

Fig. 89 gives particulars of a beam which fractured as a result of combined buckling and tilting during handling. The actual beam after this mishap is illustrated in Fig. 90. The data for the beam in question are:

- dead weight = 0.64 t/m (assumed constant along the beam);
- span $l = 22.60$ m;
- $EI_d = 3{,}500{,}000 \times 0.00222 = 7770$ tm^2;
- $k =$ (approx.) 0.05; $1-k^2 = 0.9975$, which shows that the effect of torsion can be neglected in this case;
- the beam was lifted at slinging points located at its ends, so that $f = 1$;
- depth of beam at the ends = 80 cm; here the centre of gravity is 40 cm above the underside of the beam.

The beam has a ridged profile on top. The mid-span depth is 120 cm; the centre of gravity of the section is here 66 cm above the underside. In addition, the beam has a camber of 5 cm.

Vertical distance of centres of gravity at mid-span and at ends respectively = $66+5-40 = 31$ cm.

On substitution of these data into the above formula, we obtain:

$$a = 1\left\{\frac{0.64 \times 22.6^4}{120 \times 7770 \times 1} + \frac{16}{25} \times 0.31\right\} = 0.38 \text{ m}$$

In actual fact this beam had $a = 0.36$ m, as the lifting eyes for the attachment of the slinging ropes had been recessed in. The beam tipped over suddenly, when it had hardly been lifted free of the formwork. In the photograph (Fig. 90) the ends of the

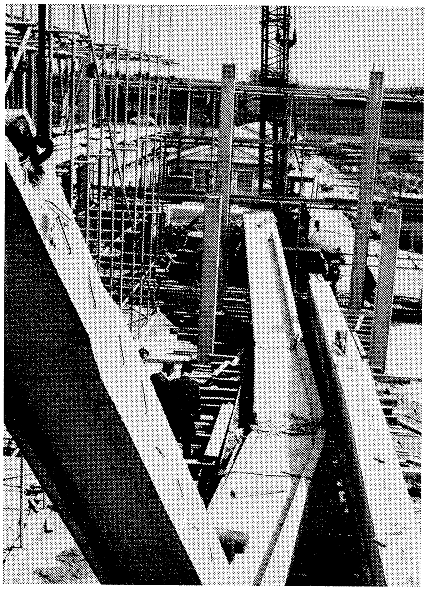

Fig. 90.
Beam after toppling over and fracturing.

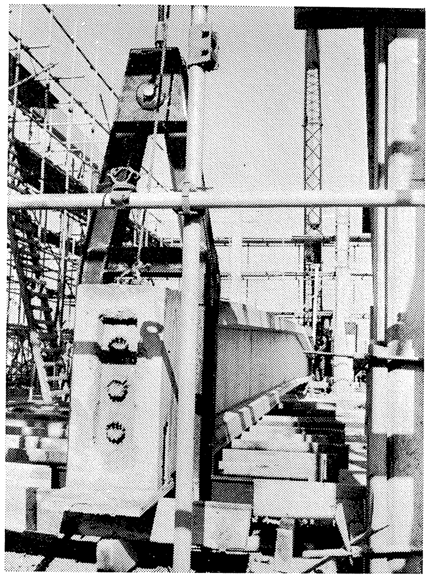

Fig. 91.
The other beams were handled with the aid of this special lifting chair.

beam are shown raised much higher. This was done in an attempt to release a workman who had been trapped under the beam.

The other beams of the same series were handled and erected without much difficulty with the aid of a special lifting chair (see Figs. 91 and 92) which provided $a = 1.25$ m, thus entirely eliminating any risk of buckling and tilting.

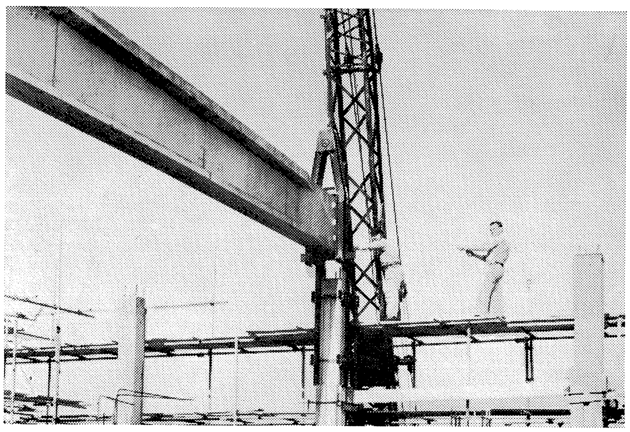

Fig. 92.
Erection of a beam suspended with the aid of lifting chairs: temporary attachments have been provided on the columns to prevent tilting of the beams.