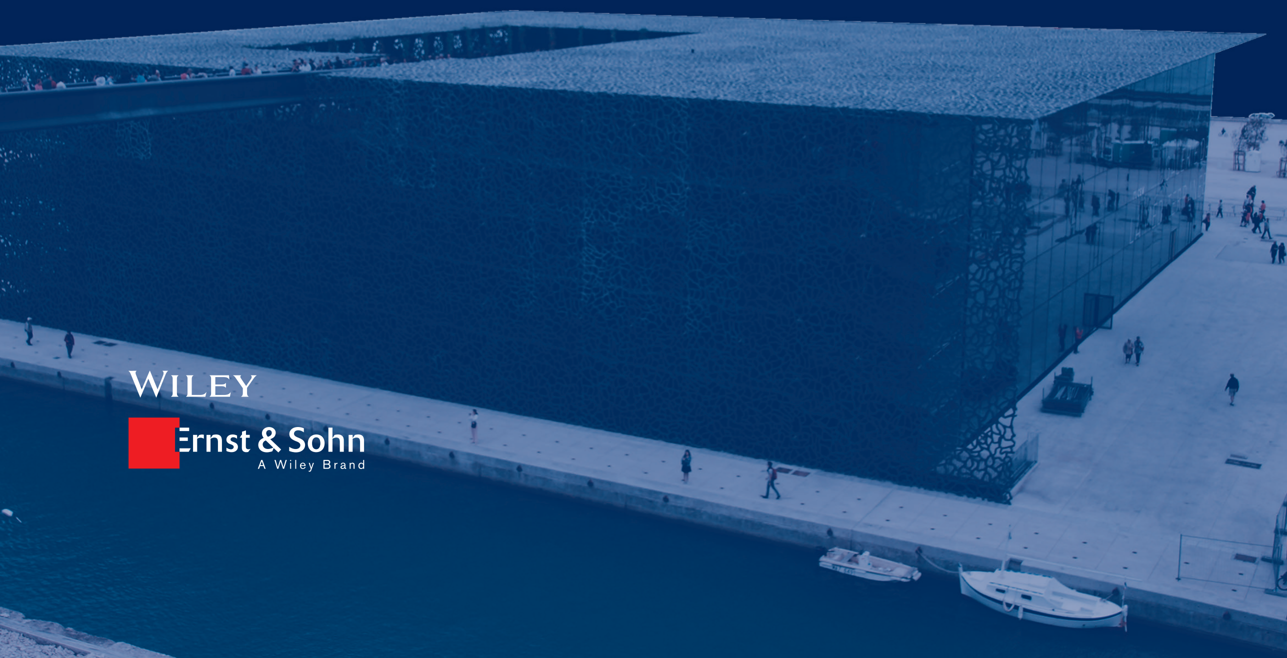
BetonKalender

Ultra-High Performance Concrete UHPC

Fundamentals – Design – Examples

Ekkehard Fehling, Michael Schmidt, Joost Walraven,
Torsten Leutbecher, Susanne Fröhlich



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1 Introduction

1.1 The reason behind this book

The main reason is the revised approach to the design of adhesively bonded strengthening measures for concrete members given in the guideline [1] (q.v. [2]) published by the Deutscher Ausschuss für Stahlbeton DAfStb (German Committee for Structural Concrete). This book explains the design rules of the DAfStb guideline, together with their background, and uses examples to illustrate their use. The scope of the explanations and background information provided here is mainly based on works that have already been published. However, some rules that so far have been dealt with in detail in committee meetings only are elaborated here for the first time.

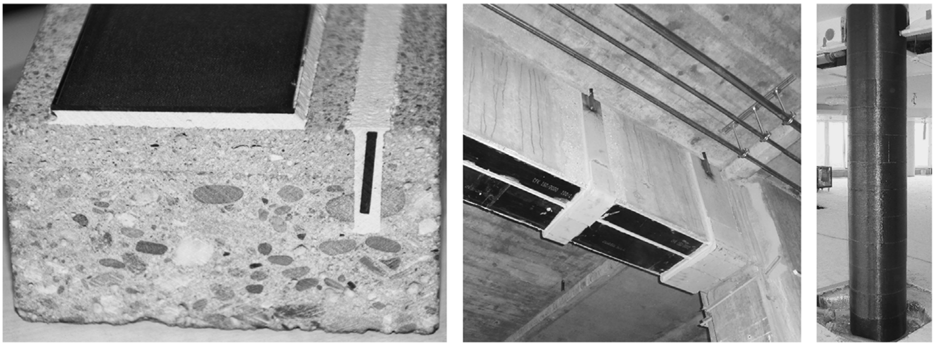
1.2 Strengthening with adhesively bonded reinforcement

The strengthening of concrete members means using constructional measures to restore or improve their load-carrying capacity, serviceability, durability or fatigue strength. The effects of strengthening measures can generally be described in quantitative terms and therefore analysed numerically. Besides numerous other methods (see [3, 4], for example), the subsequent strengthening of existing concrete members can be achieved by using adhesives to bond additional reinforcing elements onto or into those members. This topic of reinforcement bonded with adhesive has been the subject of many contributions to various editions of the *Beton-Kalender* in the past (see [5, 6]). However, design approaches for adhesively bonded reinforcement have continued to evolve (see [7, 8]) and the new DAfStb guideline [1, 2] on this subject revises those design methods and adapts them to our current state of knowledge. In principle, the DAfStb guideline together with a corresponding system approval allows the following concrete member strengthening measures to be carried out:

- Flexural strengthening with externally bonded (surface-mounted) CFRP strips, CF sheets and steel plates
- Flexural strengthening with CFRP strips bonded in slots (near-surface-mounted reinforcement)
- Shear strengthening with externally bonded CF sheets and steel plates
- Column strengthening with CF sheets as confining reinforcement.

Figure 1.1 provides an overview of these methods. The term ‘adhesively bonded’ is used in this book as universal expression comprising both methods ‘externally bonded’ and ‘near-surface-mounted’.

Strengthening of Concrete Structures with Adhesively Bonded Reinforcement: Design and Dimensioning of CFRP Laminates and Steel Plates. First Edition. Konrad Zilch, Roland Niedermeier, and Wolfgang Finckh.



(a) (b) (c)

Fig. 1.1 (a) Externally bonded and near-surface-mounted CFRP strips; (b) flexural strengthening with externally bonded CFRP strips together with shear strengthening in the form of externally bonded steel plates (photo: Laumer Bautechnik GmbH); (c) column strengthening with CF sheets as confining reinforcement (photo: Laumer Bautechnik GmbH)

2 DAfStb guideline

2.1 The reasons for drawing up a guideline

In the past, the product systems as well as the design and installation of adhesively bonded reinforcement were regulated in Germany by national technical approvals and individual approvals. Such approvals contained provisions covering the materials, the design of the strengthening measures, the work on site and the monitoring of products. There were several reasons why it was deemed necessary to revise the design approaches of the earlier approvals.

One of those reasons was the harmonization of standards across Europe, leading to national standards and regulations being successively adapted to the European standards. These developments also render it necessary to adapt the former national approvals to the new generation of standards.

Furthermore, the results of numerous research projects carried out in recent years had only been partly incorporated in the older regulations, which therefore no longer matched the current state of knowledge. Therefore, industry, the building authorities and the German Research Foundation (DFG) made substantial funds available for researching adhesively bonded reinforcement. That led to many scientific projects in the German-speaking countries and adhesively bonded reinforcement gradually becoming a standard method in the building industry. Consequently, all the groups involved regarded the preparation of a universal guideline as indispensable.

2.2 Preparatory work

In order to produce a universal guideline reflecting the current level of knowledge, the German Committee for Structural Concrete (DAfStb) first commissioned a report on the current situation [7] to document and collate national and international knowledge. A database of test results containing almost all the experimental studies carried out nationally and internationally was also set up and compared with the established models and the guidelines available elsewhere in the world. During the drafting of the report it became apparent that the knowledge necessary to produce an effective guideline was lacking in some areas. Therefore, under the direction of the DAfStb, a research project was initiated in which all the groups interested took part. The research work was carried out by the technical universities in Munich and Brunswick, both of which had been working continually on adhesively bonded reinforcement for more than 20 years. The project was financed by the owners of the approvals (Bilfinger Berger AG, Laumer Bautechnik GmbH, Ludwig Freytag GmbH & Co. KG, MC-Bauchemie Müller GmbH & Co. KG, S&P Clever Reinforcement Company AG, Sika Deutschland GmbH, Stocretec GmbH), the Federal Institute for Research on Building, Urban Affairs & Spatial Development (BBSR) plus a number of associations and consulting engineers. Issues surrounding the bond strength under static loads [9] and dynamic loads [10] plus the shear strength [11] were successfully clarified during this project.

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strength f_{Gtk} and the compressive strength f_{Gck}. However, to adjust the values from the bond tests, a system coefficient k_{sys} specific to the product was incorporated in the equation. The strength of the adhesive and this system coefficient can be found in the national technical approvals for the systems and depend on the internal monitoring on the building site. If the tensile and compressive strengths are checked as part of this internal monitoring, then according to the national technical approvals for the systems, values between 21 and 28 N/mm^2 can be assumed for f_{Gtk} and between 75 and 85 N/mm^2 for f_{Gck}. However, these characteristic values must also be obtained in the internal monitoring according to part 3 of the DAfStb guideline following a statistical evaluation. The product-specific system coefficient k_{sys} lies between 0.6 and 1.0 depending on the system.

The concrete can fail in the case of a very low concrete strength and therefore the bond strength of the concrete according to Equation 5.8 governs:

$$\tau_{bck} = k_{bck} \cdot \sqrt{f_{cm}} \tag{5.8}$$

In a similar way to the bond of reinforcing steel (see [102–105], for example), this bond strength is calculated from the square root of the concrete compressive strength and a calibration factor k_{bck}. The system coefficient for the bond failure of the concrete can be taken from the national technical approval for the system. Tests carried out at the Technische Universität München established a characteristic value $k_{bck} = 4.5$.

The factors α_{bG} and α_{bC} were introduced into Equation 5.6 to take account of the long-term durability behaviour of the materials involved. As these are also coefficients specific to particular products, they can again be obtained from the national technical approvals. Many studies of the long-term durability behaviour of concrete have been carried out, and this behaviour is covered by DIN EN 1992-1-1 [20] together with its associated National Annex [21]. Therefore, the long-term effect coefficient α_{bC} for a bond failure in the concrete should lie between 0.85 and 1.0. However, adhesives can exhibit a much lower long term strength in some cases (see [100, 106–108], for example). Depending on the adhesive and the ambient conditions of the application, the long-term effect coefficient α_{bG} for a concrete bond failure lies between 0.50 and 0.85.

5.4 Shear Force Analyses

When analysing the shear capacity, the same requirements apply for near-surface-mounted CFRP strips as for externally bonded strips. This means that as described in Section 3.4.1, verifying the shear capacity should be carried out according to DIN EN 1992-1-1 [20] together with its associated National Annex [21]. As with externally bonded CFRP strips, the area of a near-surface-mounted strip may not be counted as part of the tension reinforcement A_{sl} in Eq. (6.2a) of DIN EN 1992-1-1 [20]. Counting the CFRP strip as part of this reinforcement is not carried out in the DAfStb guideline because only a few shear tests have been carried out on strengthened members without shear reinforcement and so it is difficult to predict the effect of this. If the shear capacity analysis is not satisfied, shear strengthening for near-surface-mounted CFRP strips can be provided as described in Section 3.4.2.

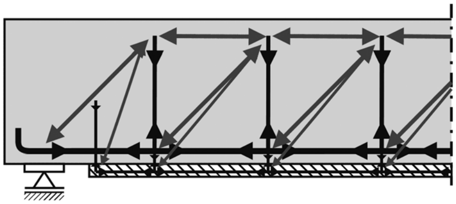

Fig. 5.4 Mechanism for transferring tensile forces from externally bonded reinforcement to flexural compression zone of member by means of truss action

As with externally bonded CFRP strips, an analysis to prevent a concrete cover separation failure, see Section 3.4.3, is required for near-surface-mounted strips as well. Tests (see [11, 54]) have shown that the method described in Section 3.4.3 can also be used for members with near-surface-mounted CFRP strips.

In contrast to externally bonded CFRP strips, debonding at displaced crack edges does not occur with near-surface-mounted strips because the bond behaviour is much more robust. Therefore, the limit given in Section 3.4.1 for additional shear wrapping does not apply for near-surface-mounted CFRP strips. With very high shear loads, however, externally bonded shear straps must ensure that the tensile forces from the externally bonded reinforcement can also be tied back the flexural compression zone of the member with the help of truss action, as Figure 5.4 illustrates.

The limit value τ_{02} to DIN 1045 [94] has turned out to be a suitable variable (see [29]) for the maximum shear capacity without additional externally bonded shear straps. Equation 5.9 expresses this limit (see [11]):

$$V_{\mathrm{Ed}} \leq 0.33 \cdot f_{\mathrm{ck}}^{2/3} \cdot b_{\mathrm{w}} \cdot d \tag{5.9}$$

If this limit value is exceeded, additional externally bonded shear straps are required to confine the strips.

5.5 Fatigue analysis

When checking fatigue for non-static loads, the DAfStb guideline can again be used to verify the bond of flexural strengthening in the form of near-surface-mounted CFRP strips. As the carbon fibres exhibit virtually no signs of fatigue, only the bond needs to be checked for fatigue when using CFRP strips. Besides the fatigue of the strengthening system, the concrete, reinforcing steel and prestressing steel must also be checked according to DIN EN 1992-1-1 [20] in conjunction with its National Annex [21].

In contrast to externally bonded CFRP strips, however, there is no comprehensive analysis concept available for near-surface-mounted strips. Owing to the low number of fatigue tests involving near-surface-mounted CFRP strips (see [27]), a quasi-fatigue strength analysis is the only option here. With so few test results available, it is not possible to specify an S-N curve for near-surface-mounted reinforcement. And as an S-N curve is unavailable, it is not possible to extrapolate for a number of load cycles greater than that given in the test results. Therefore, the analysis can only assume sufficient fatigue resistance for max. $2 \cdot 10^6$ load cycles. Design methods for numbers of load cycles $> 2 \cdot 10^6$ are not covered in the DAfStb guideline.

In this analysis, adequate resistance to fatigue for near-surface-mounted CFRP strips may be assumed for up to $2 \cdot 10^6$ load cycles provided the end anchorage force for a frequent cyclic action to DIN EN 1992-1-1 section 6.8.3 (3), and taking into account the 'shift rule', does not exceed the value $0.6F_{\mathrm{bLRd}}$ (F_{bLRd} to Equation 5.5) and the strip stress range does not exceed a value given by Equation 5.10. The strip thickness t_{L} in mm should be used here so that the result is an admissible stress range in N/mm^2.

$$\Delta\sigma_{\mathrm{L}} \leq \frac{500\,\mathrm{N/mm^2}}{t_{\mathrm{L}}} \tag{5.10}$$

5.6 Analyses for the serviceability limit state

The analyses for the serviceability limit state, which were described for externally bonded CFRP strips in Section 3.6, also apply correspondingly for near-surface-mounted CFRP strips.

It should be pointed out here that owing to their effective and relatively stiff bond behaviour (see Figure 5.1), near-surface-mounted CFRP strips are ideal for retrofitting to control crack widths (see [109], for example). The method for allowing for the crack-limiting effect of near-surface-mounted reinforcement is based on a method proposed in [91], which assumes a bond-related interaction between the internal reinforcement and the near-surface-mounted reinforcement. It is assumed here that the cracks are closed or grouted at the time of strengthening and therefore no significant action effects due to residual stresses and loads are present. In this method it is first necessary to calculate the strip stress due to the load or restraint and assume a crack width. Owing to the assumed crack width, Equation 5.11 can be used to calculate the slip of the internal reinforcement and the near-surface-mounted reinforcement:

$$w_{\mathrm{k}} = 2 \cdot s_{\mathrm{sr}} = 2 \cdot s_{\mathrm{Lr}} \tag{5.11}$$

With the help of the slip it is now possible to determine the mean bond stresses, the crack spacing and the mean strains using the equations given in the DAfStb guideline. The crack width can then be calculated with Equation 5.12:

$$w_{\mathrm{k}} = s_{\mathrm{cr,max}} \cdot (\varepsilon_{\mathrm{Lm}} - \varepsilon_{\mathrm{cm}}) \tag{5.12}$$

If the crack width from Equation 5.11 agrees with the assumption in Equation 5.12, this is the crack width that will occur.

5.7 Detailing

Essentially, near-surface-mounted CFRP strips must comply with the same detailing rules as those for externally bonded strips, which are described in Section 3.7. However, when it comes to the strip spacing, near-surface-mounted CFRP strips must comply not only with a maximum spacing, which is dealt with in the DAfStb guideline in the same way as the externally bonded CFRP strips, but also with a minimum spacing. Further to this minimum spacing there are also enhanced requirements regarding the distance of a strip from the edge of a member.

The DAfStb guideline specifies the minimum spacing a_L for near-surface-mounted CFRP strips by way of Equation 5.13, which is based on the diameter ϕ of the steel reinforcing bars running parallel to the CFRP strips, the clear spacing a_s of these steel reinforcing bars, the maximum aggregate size d_g and the strip width b_L:

$$a_L \geq \begin{cases} d_g & \text{for } a_s \leq 2 \cdot \phi \\ b_L & \text{for } a_s > 2 \cdot \phi \end{cases} \tag{5.13}$$

This minimum spacing is necessary because where individual near-surface-mounted CFRP strips are too close together, one conceivable failure mode involves the strips together with the concrete cover become fully detached from the member (see [27]). The DAfStb guideline therefore includes the rules of [29], which are based on similar rules for internal steel reinforcing bars according to [27]. The final criterion for a minimum distance between CFRP strips is guaranteeing being able to cut the slots without damaging the member, which is also the case with the limits specified above.

A minimum edge distance is necessary because of the risk that the edge of the concrete member could break away if the spacing between a CFRP strip and the free edge of a member is too small and also the risk of damage to the edge of the concrete when cutting the slots. This minimum edge distance is specified in the DAfStb guideline by way of Equation 5.14. This approach was in the detailing rules of an earlier approval [29] and is based on [27].

$$a_r \geq \max \begin{cases} d_g \\ 2 \cdot b_L \end{cases} \tag{5.14}$$

The DAfStb guideline contains another requirement regarding the edge distance for the case where CFRP strips are being bonded to the soffit and the side face at the same time. This is because strips meeting along an edge cause a higher stress in the concrete at this corner.

6 Example 2: Strengthening a beam with near-surface-mounted CFRP strips

6.1 System

6.1.1 General

Owing to a change of use for a single-storey shed, a reinforced concrete downstand beam must carry higher loads and therefore needs to be strengthened. As-built documents with structural calculations to DIN 1045 [94] are available. The downstand beam, which was designed as a simply supported member, is to be strengthened with near-surface-mounted CFRP strips. It is assumed that the beam is free to rotate at its supports. Moderately damp conditions prevail in the building and the loads are primarily static. Figure 6.1 shows the structural system requiring strengthening and Figure 6.2 shows an idealized section through the beam.

6.1.2 Loading

The loads are predominantly static. Three load cases will be investigated for ultimate limit state design:

– **Load case 1** represents the situation prior to strengthening.
– **Load case 2** is the loading during strengthening. The strengthening measures are carried out under the dead load of the beam. Existing fitting-out items will be removed during the strengthening work.
– **Load case 3** represents the loading situation in the strengthened condition.

Table 6.1 lists the actions of the various load cases for the loads given in Figure 6.1.

Load case 3 governs for designing the strengthening measures. The load combination for the ultimate limit state and the load combination for the serviceability limit state under a rare load combination are required for the analyses. These load combinations are given by DIN EN 1990 [24] together with its associated National Annex [25]. The following applies for the ultimate limit state (persistent and transient design situations):

$$\sum_{j\geq 1}\gamma_{\mathrm{G,j}}\cdot G_{\mathrm{k,j}}+\gamma_{\mathrm{P}}\cdot P+\gamma_{\mathrm{Q,1}}\cdot Q_{\mathrm{k,1}}+\sum_{i>1}\gamma_{\mathrm{Q,i}}\cdot\psi_{0,\mathrm{i}}\cdot Q_{\mathrm{k,i}}$$

$$\mathrm{p_d}=\gamma_{\mathrm{G}}\cdot\left(\mathrm{g_{1,k}}+\mathrm{g_{2,k}}\right)+\gamma_{\mathrm{Q}}\cdot\mathrm{q_k}=1.35\cdot(30+5)+1.5\cdot 5.0=122.35\,\mathrm{kN/m}$$

The load for the serviceability limit state is calculated as follows for a rare load combination:

$$\sum_{j\geq 1}G_{\mathrm{k,j}}+P+Q_{\mathrm{k,1}}+\sum_{i>1}\psi_{0,\mathrm{i}}\cdot Q_{\mathrm{k,i}}$$

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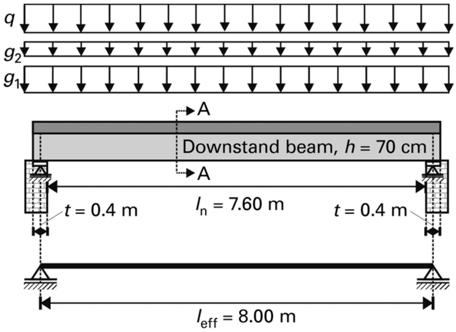

Fig. 6.1 Downstand beam system requiring strengthening

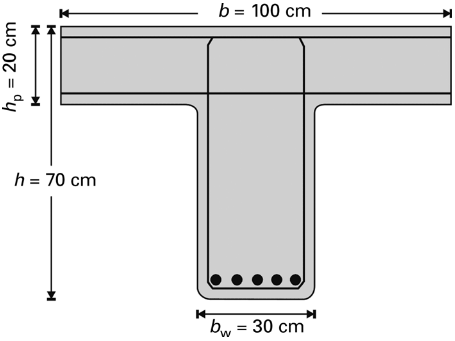

Fig. 6.2 Section through downstand beam, (section A-A)

$$p_{\text{rare}} = g_{1,\text{k}} + g_{2,\text{k}} + q_{\text{k}} = 30 + 5 + 50 = 85\,\text{kN/m}$$

In order to determine the prestrain condition during strengthening, which according to DAfStb guideline [1, 2] part 1 section 5.1.1 (RV 19) must be considered for a quasi-permanent load combination, we get the following for load case 2:

$$\sum_{j\geq 1} G_{\text{k,j}} + P + \sum_{i\geq 1} \psi_{2,\text{i}} \cdot Q_{\text{k,i}}$$

$$p_{\text{perm}} = g_{1,\text{k}} = 30\,\text{kN/m}$$

Table 6.1 Loads on the system in kN/m^2 for the various load cases.

Load case	1	2	3
$g_{1,\text{k}}$ (dead load)	30.0	30.0	30.0
$g_{2,\text{k}}$ (fitting-out load)	5.0	—	5.0
q_{k} (imposed load, category B)	25.0	—	50.0

6.1.3 Construction materials

6.1.3.1 Concrete compressive strength

Concrete of class B35 was able to be ascertained from the as-built documents according to DIN 1045 [94]. Following a test on the member, the result was strength class C30/37. Therefore, the values according to DIN EN 1992-1-1 [20] Tab. 3.1 for C30/37 concrete will be used for the design. This results in a mean concrete compressive strength $f_{cm} = 38\,\text{N/mm}^2$ and a characteristic concrete compressive strength $f_{ck} = 30\,\text{N/mm}^2$.

6.1.3.2 Type and quantity of existing reinforcement

According to the as-built documents, the longitudinal reinforcement is five Ø28 mm ribbed steel reinforcing bars ($A_{sl} = 30.79\,\text{cm}^2$) and shear reinforcement in the form of vertical Ø8 mm links @ 200 mm c/c ($A_{sw}/s = 5.03\,\text{cm}^2$). It is apparent from the documents that the reinforcing steel is grade BSt 500 S (IV S) to [94] or [97]. Consequently, we can assume a yield stress $f_{syk} = 500\,\text{N/mm}^2$ and a modulus of elasticity $E_s = 200\,\text{kN/mm}^2$.

6.1.3.3 Position of existing reinforcement

The as-built documents indicate a concrete cover of min $c = 2.0$ cm, or nom $c = 3.0$ cm, according to DIN 1045 [94]. A survey according to [98] has revealed that the reinforcement is positioned as shown in Figure 6.3.

6.1.3.4 Strengthening system

Commercially available CFRP strips with a characteristic tensile strength $f_{Luk} = 2400\,\text{N/mm}^2$ and modulus of elasticity $E_L = 170\,\text{kN/mm}^2$ are to be bonded in slots for the strengthening. Strips with dimensions of ($t_L \times b_L$) 20×2 mm are to be used. The system includes an appropriate epoxy resin adhesive, for which a tensile strength $f_{Gtk} = 30\,\text{N/mm}^2$ and a compressive strength $f_{Gck} = 90\,\text{N/mm}^2$ will be assumed in the design. The other coefficients specific to this system are $k_{sys} = 0.8$, $k_{bck} = 2.5$, $\alpha_{bc} = 0.9$ and $\alpha_{bG} = 0.5$.

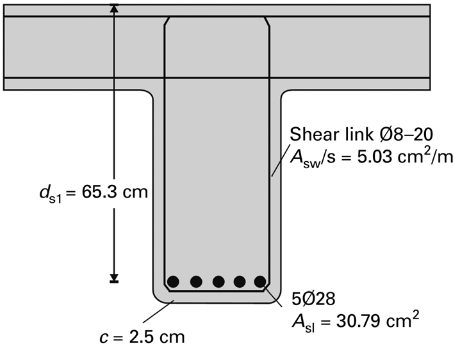

Fig. 6.3 Type and position of existing reinforcement.(other reinforcement omitted for clarity)

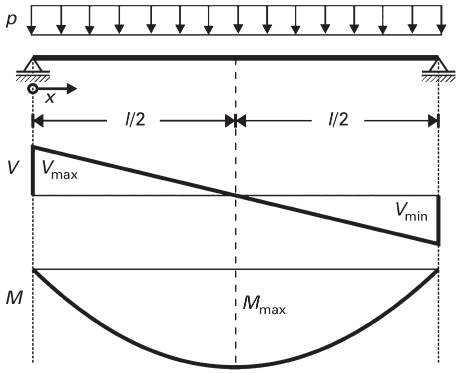

Fig. 6.4 Shear forces and bending moments

6.2 Internal forces

Figure 6.4 shows the basic bending moment and shear force diagrams for the simply supported beam. The actual maximum values for the load combinations relevant to the design are given in Table 6.2.

$$M(x) = \frac{p}{2} \cdot l \cdot x - \frac{p \cdot x^2}{2}$$

$$V(x) = \frac{p}{2} \cdot l - p \cdot x$$

6.3 Determining the prestrain

DAfStb guideline [1, 2] part 1 section 5.1.1 (RV 19) requires that the prestrain be taken into account in the design. This is determined below using the example of the maximum moment. As according to the DAfStb guideline a prestrain should be determined with a quasi-permanent load combination for the serviceability limit state, characteristic material parameters are used in this section.

Table 6.2 Maximum shear forces and bending moments for the relevant load combinations.

Load combination	M_{max}	V_{max}	V_{min}
—	kNm	kN	kN
Load case 3; ULS	978	489	−489
Load case 3; SLS, rare	680	340	−340
Load case 2; SLS, quasi-permanent	240	120	−120

An iterative method is used to determine the prestrain condition in the cross-section. The calculation below uses the internal lever arm of the reinforcing steel, determined iteratively, in order to demonstrate the method briefly. The internal lever arm, which represents the iteration variable, is

$$z_{s1} \approx 0.905 \cdot d_{s1} \approx 0.904 \cdot 653 \approx 590.4\ \mathrm{mm}$$

The tensile force in the steel at the time of strengthening for the maximum moment can be calculated from the moment and the internal lever arm (see Section 3.2 and Figure 3.3):

$$F_{s1} = \frac{M_{0,k}}{z_{s1}} = \frac{240 \cdot 10^6}{590.4} = 406.5\ \mathrm{kN}$$

Following on from that it is possible to determine the prestrain in the reinforcing steel from the area of the reinforcing bars and the modulus of elasticity of the reinforcement:

$$\varepsilon_{s1} = \frac{F_{s1}}{A_{s1} \cdot E_s} = \frac{406.5 \cdot 10^3}{30.79 \cdot 10^2 \cdot 200} = 0.66\ \mathrm{mm/m}$$

Assuming a compressive strain in the concrete $\varepsilon_c > -2$ mm/m and a compression zone contained completely within the slab, the compressive force in the concrete according to Section 3.2 can be calculated approximately using the parabola-rectangle diagram for concrete under compression as follows:

$$\begin{aligned} F_c &= b \cdot x \cdot f_{ck} \cdot \alpha_R = b \cdot \xi \cdot d_{s1} \cdot f_{ck} \cdot \left(-\frac{\varepsilon_c^2}{12} - \frac{\varepsilon_c}{2}\right) \\ &= 1000 \cdot \left(\frac{-\varepsilon_c}{-\varepsilon_c + \varepsilon_{s1}}\right) \cdot 653 \cdot 30 \cdot \left(-\frac{\varepsilon_c^2}{12} - \frac{\varepsilon_c}{2}\right) \end{aligned}$$

Equilibrium of the internal forces results in an equation for calculating the compressive strain in the concrete:

$$\begin{aligned} &F_{s1} = F_c \\ &406.5\ \mathrm{kN} = -1000 \cdot \left(\frac{-\varepsilon_c}{-\varepsilon_c + 0.66}\right) \cdot 653 \cdot 30 \cdot \left(-\frac{\varepsilon_c^2}{12} - \frac{\varepsilon_c}{2}\right) \end{aligned}$$

Solving the equation results in $\varepsilon_c = -0.26$ mm/m. As this value is > –2 mm/m, the above assumption was justified. The relative depth of the compression zone ξ and the depth of the compression zone x can now be determined with the help of the strains. As the depth of the compression zone is less than the depth of the slab, the above assumption – compression zone located fully within slab – was correct.

$$\xi = \frac{-\varepsilon_c}{-\varepsilon_c + \varepsilon_s} = \frac{0.26}{0.26 + 0.66} = 0.28$$

$$x = \xi \cdot d_{s1} = 0.28 \cdot 653 = 182.8\ \mathrm{mm}$$

Using the coefficient k_a (for $\varepsilon_c > -2$ mm/m), calculated according to Section 3.2, it is now possible to determine the internal lever arm z_{s1}:

$$k_a = \frac{8 + \varepsilon_c}{24 + 4 \cdot \varepsilon_c} = \frac{8 - 0.26}{24 - 4 \cdot 0.26} = 0.34$$

$$a = k_a \cdot \xi \cdot d_{s1} = 0.34 \cdot 0.28 \cdot 653 = 62.6\ \text{mm}$$

$$z_{s1} = d_{s1} - a = 653 - 62.6 = 590.4\ \text{mm}$$

As the internal lever arm roughly corresponds to the assumed lever arm, the resistance of the reinforced concrete cross-section at the position of the acting moment is

$$M_{Rk,0} = z_{s1} \cdot F_{s1} = 590.4 \cdot 406.5 \cdot 10^{-3} = 240\ \text{kNm}$$

The prestrain for the concrete therefore amounts to $\varepsilon_{c,0} = -0.26$ mm/m, and for the reinforcing steel $\varepsilon_{s1,0} = 0.66$ mm/m.

6.4 Verification of flexural strength

In the following calculations it is assumed that five strips are required for strengthening. The total strip cross-section is therefore

$$A_L = n_L \cdot t_L \cdot b_L = 5 \cdot 2 \cdot 20 = 200\ \text{mm}^2$$

When strengthening a member by means of near-surface-mounted CFRP strips, the slot dimensions must satisfy certain requirements, which influence the effective structural depth of the strips. According to DAfStb guideline [1, 2] part 3 section 4.4.1 (3), the depth of each slot in the concrete is

$$t_s \leq c - \Delta c_{dev}$$

The allowance Δc_{dev} is made up as follows according to Section 5.2:

$$\Delta c_{dev} = \Delta c_{tool} + \Delta c_{slot} + \Delta c_{member} = 1 + 2 + 2 = 5\ \text{mm}$$

With a concrete cover $c = 25$ mm, the ensuing slot depth is $t_s = 20$ mm, which is exactly the same as the strip width b_L. The effective structural depth of the CFRP strip according to DAfStb guideline [1, 2] part 1 Eq. (RV 6.53) depends on the depth of the slot and is

$$d_L = h - \left(t_s - \frac{b_L}{2}\right) = 700 - \left(20 - \frac{20}{2}\right) = 690\ \text{mm}$$

The maximum strain that may be assumed in the design is determined from DAfStb guideline [1, 2] part 1 Eq. (RV 6.52) using the characteristic tensile strength of the strip f_{Luk}, the safety factor for strip failure γ_{LL} and the coefficient k_ε:

$$\varepsilon_{LRd,max} = \kappa_\varepsilon \cdot \varepsilon_{Lud} = \kappa_\varepsilon \cdot \frac{f_{Luk}}{\gamma_{LL} \cdot E_L} = 0.8 \cdot \frac{2400}{1.2 \cdot 170 \cdot 10^3} = 9.41\ \text{mm/m}$$

The flexural strength is checked at mid-span for the maximum moment. In the following calculations it is assumed that the maximum strain in the strip can be exploited. As the strain in the strip $\varepsilon_{LRd,max} > f_{yd}/E_s$, we shall continue to assume that the reinforcing steel is yielding. Therefore, the tensile force in the reinforcing steel and the tensile force in the externally bonded reinforcement are

$$F_{s1d} = \frac{A_{s1} \cdot f_{yk}}{\gamma_s} = \frac{30.79 \cdot 10^2 \cdot 500}{1.15} = 1338.6\,\text{kN}$$

$$F_{LRd} = \varepsilon_{LRd,max} \cdot A_L \cdot E_L = 9.41 \cdot 200 \cdot 170 \cdot 10^3 = 320.0\,\text{kN}$$

The prestrain at the level of the near-surface-mounted CFRP strips is calculated using the prestrain in the reinforcement steel determined in Section 6.3:

$$\varepsilon_{L,0} = \varepsilon_{s1,0} + \frac{d_L - d_{s1}}{d_{s1}} \cdot \left(\varepsilon_{s1,0} + \varepsilon_{c,0}\right) = 0.66 + \frac{690 - 653}{653} \cdot (0.66 + 0.26) = 0.71\,\text{mm/m}$$

The total strain in the cross-section at the level of the strips is therefore

$$\varepsilon_{L,0} + \varepsilon_{LRd,max} = 0.71 + 9.41 = 10.12\,\text{mm/m}$$

Assuming a compressive strain in the concrete $\varepsilon_c < -2$ mm/m and that the compression zone is contained completely within the slab, the compressive force in the concrete can be expressed as follows according to Section 3.2:

$$F_{cd} = b \cdot x \cdot f_{cd} \cdot \alpha_R = b \cdot \xi \cdot d_L \cdot f_{ck} \cdot \frac{\alpha_{cc}}{\gamma_c} \cdot \left(1 + \frac{2}{3 \cdot \varepsilon_c}\right)$$

$$= 1000 \cdot \left(\frac{-\varepsilon_c}{-\varepsilon_c + \varepsilon_{L,0} + \varepsilon_{LRd,max}}\right) \cdot 690 \cdot 30 \cdot \frac{0.85}{1.5} \cdot \left(1 + \frac{2}{3 \cdot \varepsilon_c}\right)$$

Equilibrium of the internal forces enables the strain in the concrete to be subsequently calculated:

$$F_{s1d} + F_{Ld} = F_{cd}$$

Iteration results in $\varepsilon_c = -2.47$ mm/m. As this value is greater than the maximum compressive strain in the concrete $\varepsilon_{cu} = -3.5$ mm/m and also less than $\varepsilon_c = -2$ mm/m, the above assumption was justified. The relative depth of the compression zone ξ and the depth of the compression zone x can now be determined with the help of the strains. As the depth of the compression zone is less than the depth of the slab, the above assumption – compression zone located fully within slab – was correct.

$$\xi = \frac{-\varepsilon_c}{-\varepsilon_c + \varepsilon_{L,0} + \varepsilon_L} = \frac{2.47}{2.47 + 0.71 + 9.41} = 0.196$$

$$x = \xi \cdot d_L = 0.196 \cdot 690 = 135.4\,\text{mm}$$

Using the coefficient k_a (for $\varepsilon_c < -2$ mm/m), which is the result according to Section 3.2, it is now possible to determine the internal lever arms:

$$k_a = \frac{3 \cdot \varepsilon_c^2 + 4 \cdot \varepsilon_c + 2}{6 \cdot \varepsilon_c^2 + 4 \cdot \varepsilon_c} = \frac{3 \cdot 2.47^2 - 4 \cdot 2.47 + 2}{6 \cdot 2.47^2 - 4 \cdot 2.47} = 0.39$$

$$a = k_a \cdot \xi \cdot d_L = 0.39 \cdot 0.196 \cdot 690 = 53.0\,\text{mm}$$

$$z_{s1} = d_{s1} - a = 653 - 53.0 = 600.0\,\text{mm}$$

$$z_L = h - a = 690 - 53.0 = 637.0\,\text{mm}$$

The moment capacity of the strengthened reinforced concrete cross-section is therefore

$$M_{Rd} = z_{s1} \cdot F_{s1d} + z_L \cdot F_{LRdL} = (1338.6 \cdot 600 \cdot 10^{-3} + 320 \cdot 637 \cdot 10^{-6}) = 1006.9\,\text{kNm}$$

As the moment capacity is greater than the acting moment of 978 kNm, the design is verified.

6.5 Bond analysis

6.5.1 Analysis point

According to DAfStb guideline [1, 2] part 1, RV 6.1.3.3 (RV 2), or Fig. RV 6.12, the analysis should be carried out, as described in section 5.3, at the point at which the CFRP strip is first required for loadbearing purposes. To do this we determine the point on the unstrengthened member at which the existing reinforcing steel reaches its yield point under the loads in the strengthened condition (load case 3). So we must first determine the bending moment at which the reinforcing steel begins to yield. The tensile force and the strain in the reinforcing steel for this situation are

$$F_{s1d} = \frac{A_{s1} \cdot f_{yk}}{\gamma_s} = \frac{30.79 \cdot 10^2 \cdot 500}{1.15} = 1338.6\,\text{kN}$$

$$\varepsilon_{s1} = \frac{f_{yd}}{E_s} = \frac{435}{200\,000} = 2.175\,\text{mm/m}$$

Assuming a compressive strain in the concrete $\varepsilon_c > -2$ mm/m and a compression zone contained completely within the slab, the compressive force in the concrete can be expressed as follows according to Section 3.2:

$$F_c = b \cdot x \cdot f_{ck} \cdot \alpha_R = b \cdot \xi \cdot d_{s1} \cdot f_{cd} \cdot \left(-\frac{\varepsilon_c^2}{12} - \frac{\varepsilon_c}{2}\right)$$

$$= 1000 \cdot \left(\frac{-\varepsilon_c}{-\varepsilon_c + \varepsilon_{s1}}\right) \cdot 653 \cdot 30 \cdot \frac{0.85}{1.5} \cdot \left(-\frac{\varepsilon_c^2}{12} - \frac{\varepsilon_c}{2}\right)$$

Equilibrium of the internal forces enables the strain in the concrete to be subsequently calculated:

$$F_{s1} = F_c$$
$$1338.6\,\text{kN} = -1000 \cdot \left(\frac{-\varepsilon_c}{-\varepsilon_c + 2.175}\right) \cdot 653 \cdot 30 \cdot \frac{0.85}{1.5} \cdot \left(-\frac{\varepsilon_c^2}{12} - \frac{\varepsilon_c}{2}\right)$$

Solving the equation results in $\varepsilon_c = -0.94$ mm/m. The relative depth of the compression zone ξ and the depth of the compression zone x can now be determined with the help of the strains. As the depth of the compression zone is less than the depth of the slab, the above assumption – compression zone located fully within slab – was correct.

$$\xi = \frac{-\varepsilon_c}{-\varepsilon_c + \varepsilon_s} = \frac{0.94}{0.94 + 2.175} = 0.30$$

$$x = \xi \cdot d_{s1} = 0.30 \cdot 653 = 195.9\,\text{mm}$$

Using the coefficient k_a (for $\varepsilon_c > -2$ mm/m), i.e. the result according to Section 3.2, it is now possible to determine the internal lever arm z_{s1}:

$$k_a = \frac{8 + \varepsilon_c}{24 + 4 \cdot \varepsilon_c} = \frac{8 - 0.94}{24 - 4 \cdot 0.94} = 0.35$$

$$a = k_a \cdot \xi \cdot d_{s1} = 0.35 \cdot 0.30 \cdot 653 = 68.6\,\text{mm}$$

$$z_{s1} = d_{s1} - a = 653 - 68.6 = 584.4\,\text{mm}$$

The moment at which the reinforcing steel begins to yield is therefore

$$M_{\text{Rdy},0} = z_{s1} \cdot F_{s1} = 584.4 \cdot 1338.6 = 780.3\,\text{kNm}$$

The point at which the existing steel reinforcement reaches its yield point under the loads in the strengthened condition (load case 3) is found by solving the parabolic moment equation of Section 6.2:

$$x\left(M_{\text{Rdy},0}\right) = \frac{1}{2} - \sqrt{\frac{1^2}{4} - 2 \cdot \frac{M_{\text{Rdy},0}}{p_d}} = \frac{8}{2} - \sqrt{\frac{8^2}{4} - 2 \cdot \frac{780.3}{122.35}} = 2.20\,\text{m}$$

According to DAfStb guideline [1, 2] part 1, RV 6.1.3.3 (RV 2), or Fig. RV 6.12, the analysis point should be determined taking into account the shifted tensile force envelope. The 'shift rule' is calculated according to DIN EN 1992-1-1 section 9.2.1.3:

$$a_l = z \cdot (\cot\theta - \cot\alpha)/2 = 0.9 \cdot 656 \cdot (1.67 - 0)/2 = 491.8\,\text{mm}$$

The angle of the strut for the shear design is taken here from Section 6.6. The analysis point is therefore found to be at $x = 1.71$ m.

6.5.2 Acting strip force

As considering the prestrain in the bond analysis leads to a lower bond stress, it is first necessary to check whether the prestrain can be included. The prestrain can be considered if the cross-section is already cracked at this point. As the actual member was not inspected, it is assumed in the following calculations that the cross-section is cracked, provided the quasi-permanent load prior to strengthening has caused cracks to form.

$$M_{\mathrm{LF1,perm}} \geq M_{\mathrm{cr}}$$

The quasi-permanent moment at the analysis point for load case 1 to which the unstrengthened cross-section was subjected – taking into account the 'shift rule' and with $\psi_2 = 0.3$ to DIN EN 1990 [24] and its associated National Annex [25] – is therefore

$$M_{\mathrm{LF1,perm}}(x = 1.71 + a_{\mathrm{l}} = 2.2) = \frac{g_{1,\mathrm{k}} + g_{2,\mathrm{k}} + \psi_2 \cdot q_{\mathrm{k}}}{2} \cdot 1 \cdot \mathrm{x} - \frac{\left(g_{1,\mathrm{k}} + g_{2,\mathrm{k}} + \psi_2 \cdot q_{\mathrm{k}}\right) \cdot \mathrm{x}^2}{2} =$$
$$= \frac{30 + 5 + 0.3 \cdot 25}{2} \cdot 8.0 \cdot 2.2 - \frac{(30 + 5 + 0.3 \cdot 25) \cdot 2.2^2}{2} = 271.15\,\mathrm{kNm}$$

The cracking moment for the cross-section can be calculated, for example, according to DAfStb guideline [1, 2] part 1, RV 6.1.1.3.3 Eq. (RV 6.5), as described in Section 3.3.3.2:

$$M_{\mathrm{cr}} = \kappa_{\mathrm{fl}} \cdot f_{\mathrm{ctm}} \cdot W_{\mathrm{c,0}} = 1.0 \cdot 2.9 \cdot 31.8 = 92.2\,\mathrm{kNm}$$

In this calculation the tensile strength of the concrete was taken from DIN EN 1992-1-1 Tab. 3.1 and the section modulus calculated as $W_{\mathrm{c,0}} = 31.8 \cdot 10^6\,\mathrm{mm}^3$. The moment under quasi-permanent loading prior to strengthening is greater than the cracking moment and so it is assumed that the cross-section is already cracked.

$$M_{\mathrm{LF1,perm}} = 21.21\,\mathrm{kNm/m} < M_{\mathrm{cr}} = 29.87\,\mathrm{kNm/m}$$

The force in the strip taking into account the prestrain and the 'shift rule' is calculated below. Table 6.3 lists the strains and internal forces at this point.

Table 6.3 Strains and internal forces at bond analysis point.

x	M_{Ed}	$\varepsilon_{\mathrm{s,0}}$	$\varepsilon_{\mathrm{c,0}}$	ε_{L}	ε_{s}	ε_{c}	F_{LEd}	F_{sEd}	F_{cEd}
m	kNm	mm/m	mm/m	mm/m	mm/m	mm/m	kN	kN	kN
2.2	780.3	0.48	−0.19	1.76	2.10	−0.93	59.77	1294.84	−1354.55

6.5.3 Bond resistance

First of all, the bond length of the near-surface-mounted CFRP strip is required to determine the bond resistance. The bond length is the result of the analysis point in Section 6.5.1 minus the distance of the strip from the centre of the support. To make it easier to cut the slot, the distance of the strip from the edge of the support is specified as 200 mm. According to Figure 6.1, the distance from the edge of the support to the centre of the support is another 200 mm. The bond length available is therefore

$$l_{\mathrm{bL}} = x - a_{\mathrm{L}} = 1710 - 200 - 200 = 1310\,\mathrm{mm}$$

To determine the bond strength, the maximum bond stress in the adhesive and the maximum bond stress in the concrete are required according to DAfStb guideline [1, 2] part 1 Eqs. (RV 8.13) and (RV 8.14), using the variables from Section 6.1.3:

$$\tau_{\mathrm{bGk}} = k_{\mathrm{sys}} \cdot \sqrt{\left(2 \cdot f_{\mathrm{Gtk}} - 2 \cdot \sqrt{\left(f_{\mathrm{Gtk}}^2 + f_{\mathrm{Gck}} \cdot f_{\mathrm{Gtk}}\right)} + f_{\mathrm{Gck}}\right) \cdot f_{\mathrm{Gtk}}}$$

$$\tau_{\mathrm{bGk}} = 0.8 \cdot \sqrt{\left(2 \cdot 30 - 2 \cdot \sqrt{\left(30^2 + 90 \cdot 30\right)} + 90\right) \cdot 30} = 24\,\mathrm{N/mm^2}$$

$$\tau_{\mathrm{bck}} = k_{\mathrm{bck}} \cdot \sqrt{\mathrm{f_{cm}}} = 4.5 \cdot \sqrt{38} = 27.7\,\mathrm{N/mm^2}$$

The design value of the bond stress is now calculated with the long-term effect coefficients and the safety factor according to DAfStb guideline [1, 2] part 1 Eq. (RV 8.12):

$$\tau_{\mathrm{bLd}} = \frac{1}{\gamma_{\mathrm{BE}}} \cdot \min\begin{cases} \tau_{\mathrm{bGk}} \cdot \alpha_{\mathrm{bG}} \\ \tau_{\mathrm{bck}} \cdot \alpha_{\mathrm{bc}} \end{cases} = \frac{1}{1.3} \cdot \min\begin{cases} 24.0 \cdot 0,5 \\ 27.7 \cdot 0.85 \end{cases} = \frac{1}{1.3} \cdot 12 = 9.23\,\mathrm{N/mm^2}$$

The tensile force per strip that can be anchored via the composite action between CFRP strip and concrete member can be calculated for $l_{\mathrm{bL}} > 115$ mm to DAfStb guideline part 1 Eq. (RV 6.56):

$$F_{\mathrm{bLRd}} = b_{\mathrm{L}} \cdot \tau_{\mathrm{bLd}} \cdot \sqrt[4]{a_{\mathrm{r}}} \cdot \left(26.2 + 0.065 \cdot \tanh\left(\frac{a_{\mathrm{r}}}{70}\right) \cdot (l_{\mathrm{bL}} - 115)\right) \cdot 0.95$$

$$F_{\mathrm{bLRd}} = 20 \cdot 9.23 \cdot \sqrt[4]{50} \cdot \left(26.2 + 0.065 \cdot \tanh\left(\frac{50}{70}\right) \cdot (1310 - 115)\right) \cdot 0.95 = 34.44\,\mathrm{kN}$$

The edge distance of the strip a_{r} here is such that it is also equal to the centre-to-centre spacing of the strips. The spacing and edge distance chosen in this way also comply with the requirement according to DAfStb guideline part 1, RV 8.2.1 (see Section 5.7 of this book).

$$a_r = \frac{b_w}{n_L + 1} = \frac{300}{5 + 1} = 50\,\text{mm}$$

The design value of the bond strength of all externally bonded reinforcement is obtained by multiplying the tensile force that can be anchored per strip by the number of strips. For simplicity, the most unfavourable edge distance of the outer strips was also applied to the other, inner, strips.

$$F_{\text{bLRd,sum}} = n_L \cdot F_{\text{bLRd}} = 5 \cdot 34.44 = 172.22\,\text{kN}$$

6.5.4 Bond analysis

The design value of the bond strength is greater than the acting strip force and so the bond analysis is regarded as verified:

$$F_{\text{LEd}} = 59.77\,\text{kN} \leq F_{\text{bLRd,sum}} = 172.22\,\text{kN}$$

6.6 Shear analyses

6.6.1 Shear capacity

First of all we shall attempt to analyse the shear capacity of the downstand beam according to DIN EN 1992-1-1 [20] and its associated National Annex [21]. Checking the capacity of the strut in the concrete is the first step. To do this, the design shear force is determined according to DIN EN 1992-1-1 [20, 21] section 6.2.1 (8):

$$V_{\text{Ed,red,max}} = V_{\text{Ed}} - p_{\text{Ed}} \cdot a_i = 489.0 - 122.25 \cdot 0.20 = 464.6\,\text{kN}$$

The maximum strut angle used in the design is obtained from DIN EN 1992-1-1 [20, 21] Eq. (6.7aDE):

$$1.0 \leq \cot\theta \leq \frac{1.2}{1 - V_{\text{Rd,cc}}/V_{\text{Ed}}} \leq 3.0$$

$$1.0 \leq \frac{1.2}{1 - 131.5/464.6} \leq 3.0 \Rightarrow \cot\theta = 1.67$$

The shear resistance $V_{\text{Rd,cc}}$ to DIN EN 1992-1-1 [20, 21] Eq. (6.7bDE) is used here:

$$V_{\text{Rd,cc}} = c \cdot 0.48 \cdot f_{\text{ck}}^{1/3} \cdot b_w \cdot z$$

$$V_{\text{Rd,cc}} = 0.5 \cdot 0.48 \cdot 30^{1/3} \cdot 300 \cdot 0.9 \cdot 653 = 131.5\,\text{kN}$$

The maximum shear resistance, which is limited by the strength of the strut, is calculated using DIN EN 1992-1-1 [20, 21] Eq. (6.9):

$$V_{\text{Rd,max}} = \frac{\alpha_{\text{cw}} \cdot b_w \cdot z \cdot \nu_1 \cdot f_{\text{cd}}}{\cot\theta + \tan\theta} = \frac{1.0 \cdot 300 \cdot 0.9 \cdot 653 \cdot 0.75 \cdot 17}{1.67 + 1/1.67} = 989.8\,\text{kN}$$

The maximum shear resistance is greater than the design shear force and so the analysis of the strut in the concrete is verified.

$$V_{\mathrm{Rd,max}} = 989.8\,\mathrm{kN} \geq V_{\mathrm{Ed,red,max}} = 464.6\,\mathrm{kN}$$

When analysing the load-carrying capacity of the internal shear links, or rather the tie, the design shear force to DIN EN 1992-1-1 [20, 21] section 6.2.1 (8) may be taken as

$$V_{\mathrm{Ed,red,s}} = V_{\mathrm{Ed}} - p_{\mathrm{Ed}} \cdot (a_{\mathrm{i}} + d) = 489.0 - 122.25 \cdot (0.10 + 0.653) = 384.7\,\mathrm{kN}$$

As a simplified approach, the analysis at this point uses the same strut angle as for the analysis of the strength of the strut in the concrete. When analysing the tie, the smaller strut angle leads to a lower load-carrying capacity, which therefore lies on the safe side. The shear resistance (limited by the yield stress of the shear reinforcement) is calculated using DIN EN 1992-1-1 [20, 21] Eq. (6.8).

$$V_{\mathrm{Rd,s}} = \left(\frac{A_{\mathrm{sw}}}{s}\right) \cdot z \cdot f_{\mathrm{ywd}} \cdot \cot\theta = 0.503 \cdot 0.9 \cdot 653 \cdot 435 \cdot 1.67 = 215.0\,\mathrm{kN}$$

The design shear force is greater than the resistance of the shear reinforcement, so the **analysis** of the tie is **not satisfied** and shear strengthening will be required.

$$V_{\mathrm{Rd,s}} = 215.0\,\mathrm{kN} \geq V_{\mathrm{Ed,red,s}} = 384.7\,\mathrm{kN}$$

6.6.2 Shear strengthening

Externally bonded full shear wrapping made from grade S235JR steel, nominal dimensions $t_{\mathrm{Lw}} = 6\,\mathrm{mm}$ and $b_{\mathrm{Lw}} = 80\,\mathrm{mm}$ at a centre-to-centre spacing $s_{\mathrm{Lw}} = 600\,\mathrm{mm}$, will be used for the shear strengthening. The yield stress of grade S235JR steel according to DAfStb guideline [1, 2] part 2 is $f_{\mathrm{yk}} = 0.8 \cdot 235\,\mathrm{N/mm^2} = 188\,\mathrm{N/mm^2}$, and the modulus of elasticity $E_{\mathrm{Lw}} = 200\,000\,\mathrm{N/mm^2}$.

The additional shear force that can be accommodated is calculated according to DAfStb guideline [1, 2] part 1 Eq. (6.108):

$$V_{\mathrm{Rd,Lw}} = \frac{A_{\mathrm{Lw}}}{s_{\mathrm{Lw}}} \cdot z \cdot f_{\mathrm{Lwd}} \cdot \cot\theta$$

The area of shear strengthening is calculated according to DAfStb guideline [1, 2] part 1 Eq. (6.109):

$$\frac{A_{\mathrm{Lw}}}{s_{\mathrm{Lw}}} = \frac{2 \cdot t_{\mathrm{Lw}} \cdot b_{\mathrm{Lw}}}{s_{\mathrm{Lw}}} = \frac{2 \cdot 6 \cdot 80}{600} = 1.6\,\mathrm{mm^2/mm}$$

The capacity of the shear strengthening f_{wLd} is determined depending on the material and the type of strengthening. As the downstand beam to be strengthened is a T-beam, only full wrapping is permitted according to DAfStb guideline part 1, RV 6.2.6 (RV 2). The strength of full wrapping in steel is the minimum of the yield stress and the stress that can be transferred across any laps:

$$f_{\text{Lwd,GS}} = \min\{f_{\text{yd}}; f_{\text{Gud,Lw}}\}$$

A lap is planned on the soffit of the beam in accordance with DAfStb guideline [1, 2] part 1 Fig. RV 9.2. According to DAfStb guideline section RV 9.2.7.2 (RV 7), 260 mm is therefore available for this lap length. The maximum length of lap that can be counted according to DAfStb guideline part 1 Eq. (RV 6.112) is

$$l_{\text{ü,max}} = 0.121 \cdot \sqrt{E_{\text{Lm}} \cdot t_{\text{L}}} = 0.121 \cdot \sqrt{200\,000 \cdot 6} = 132.6\,\text{mm}$$

As $l_{\text{ü,max}} < l_{\text{ü}}$, the stress that can be transferred at the lap is calculated according to DAfStb guideline [1, 2] part 1 Eq. (RV 6.113):

$$f_{\text{Lwd,GS}} = \frac{1.004}{\gamma_{\text{BG}}} \cdot \sqrt{\frac{E_{\text{L}}}{t_{\text{Lw}}}} = \frac{1.004}{1.3} \cdot \sqrt{\frac{200\,000}{6}} = 141.0\,\text{N/mm}^2$$

The strength of a steel shear strap to be used in the calculations is therefore

$$f_{\text{Lwd,GS}} = \min\{f_{\text{yd}}; f_{\text{Gud,Lw}}\} = \min\{188; 141\} = 141.0\,\text{N/mm}^2$$

The additional shear force that can be accommodated can be calculated using DAfStb guideline [1, 2] part 1 Eq. (RV 6.108):

$$V_{\text{Rd,Lw}} = \frac{A_{\text{Lw}}}{s_{\text{Lw}}} \cdot z \cdot f_{\text{Lwd}} \cdot \cot\theta = 1.6 \cdot 0.9 \cdot 653 \cdot 141 \cdot 1.67 = 221.42\,\text{kN}$$

The total load-carrying capacity of the tie is therefore given by DAfStb guideline [1, 2] part 1 Eq. (RV 6.107):

$$V_{\text{Rd}} = V_{\text{Rd,s}} + V_{\text{Rd,Lw}} = 215.0 + 221.4 = 436.4\,\text{kN}$$

The load-carrying capacity of the tie is now greater than the design shear force and so the design with the shear strengthening is verified.

$$V_{\text{Rd}} = 436.4\,\text{kN} \geq V_{\text{Ed,red,s}} = 384.7\,\text{kN}$$

To complete the analysis, it is only necessary to check the fasteners for the steel which are required to anchor the shear straps in the compression zone (see Figure 3.10).

6.6.3 Check for concrete cover separation failure

When checking for a concrete cover separation failure, it is first necessary to calculate the shear resistance of a member without shear reinforcement. The shear resistance of a member without shear reinforcement is obtained from the maximum of Eqs. (6.2a) and (6.2b) in DIN EN 1992-1-1 [20, 21]. The design shear resistance according to Eq. (6.2a) is

$$V_{Rd,c} = \left[C_{Rd,c} \cdot k \cdot (100 \cdot \rho_l \cdot f_{ck})^{1/3} + 0.12 \cdot \sigma_{cp}\right] \cdot d \cdot b_w$$

The following shear resistance is calculated using the variables in Eq. (6.2a) according to DIN EN 1992-1-1 or its National Annex. It should be noted here that according to DAfStb guideline part 1 section 6.2.2 (RV 7) and DIN EN 1992-1-1 Fig. 6.3, the externally bonded reinforcement may not be counted as part of the longitudinal reinforcement.

$$k = 1 + \sqrt{\frac{200}{d}} = 1 + \sqrt{\frac{200}{653}} = 1.55 \leq 2.0$$

$$\sigma_{cp} = N_{Ed}/A_c = 0$$

$$C_{Rd,c} = \frac{0.15}{\gamma_c} = \frac{0.15}{1.5} = 0.10$$

$$\rho_l = \frac{A_{sl}}{d \cdot b_w} = \frac{3079}{653 \cdot 300} = 1.57\% \leq 2\%$$

$$V_{Rd,c} = \left[0.10 \cdot 1.55 \cdot 1.0 \cdot (1.57 \cdot 30)^{1/3}\right] \cdot 653 \cdot 300 = 109.94\,\text{kN}$$

The minimum shear resistance of a member without shear reinforcement is given by DIN EN 1992-1-1 Eq. (6.2b) as

$$\begin{aligned} V_{Rd,c} &= \left[\frac{0.0525}{\gamma_c} \cdot \sqrt{k^3 \cdot f_{ck}} + 0.12 \cdot \sigma_{cp}\right] \cdot d \cdot b_w \\ &= \left[\frac{0.0525}{1.5} \cdot \sqrt{1.55^3 \cdot 30}\right] \cdot 653 \cdot 300 = 73.91\,\text{kN} \end{aligned}$$

The design shear resistance of this member without shear reinforcement is therefore $V_{Rd,c} = 109.94$ kN.

The limit beyond which no shear wrapping at the end of the strip is necessary is calculated using DAfStb guideline [1, 2] part 1 Eq. (RV 6.121) depending on the distance of the strip from the centre of the support a_L according to Section 6.5.3:

$$V_{Rd,c,LE} = 0.75 \cdot \left(1 + 19.6 \cdot \frac{(100 \cdot \rho_{sl})^{0.15}}{a_L^{0.36}}\right) \cdot V_{Rd,c}$$

$$V_{Rd,c,LE} = 0.75 \cdot \left(1 + 19.6 \cdot \frac{(1.57)^{0.15}}{400^{0.36}}\right) \cdot 109.94 = 282.52\,\text{kN}$$

As the acting shear force is greater than the limit according to the DAfStb guideline, shear wrapping at the end of the strip is essential.

Table 6.4 Strains and internal forces for determining force acting on end strap.

x	ε_L	ε_s	ε_c	F_{LEd}	F_{sEd}	F_{cEd}
m	mm/m	mm/m	mm/m	kN	kN	kN
0.892	1.11	1.02	−0.43	37.61	630.59	−668.08

$$V_{Ed} = 489.0\,\text{kN} \leq V_{Rd,c,LE} = 282.52\,\text{kN/m}$$

The force acting on the end strap is calculated according to DAfStb guideline part 1 section RV 9.2.6:

$$F_{LwEd,end} = F^*_{LEd} \cdot \tan\theta = 37.61 \cdot \frac{1}{1.67} = 22.5\,\text{kN}$$

where F^*_{LEd} is the fictitious strip tensile force at the end of the strip plus the ‘shift rule’. This means that the strip force is required at the point $x = a_L + a_l = 400 + 491.8 = 891.8$ mm. This strip force and the associated strains are listed in Table 6.4 and were determined iteratively without taking the prestrain into account because this has a favourable effect here but it is not certain that the cross-section is cracked at this point.

The force acting on the end strap is carried by the end strap of the shear strengthening. For this reason, this strap will be somewhat wider. The additional width necessary is $b_{Lw} = 20$ mm and the additional resistance of the strap can be calculated with the following equation:

$$F_{LwRd,end} = 2 \cdot t_{Lw} \cdot b_{Lw} \cdot f_{Lwd} = 2 \cdot 6 \cdot 20 \cdot 141 = 33.84\,\text{kN}$$

The resistance is greater than the action of 22.5 kN and so the design is verified. To avoid a concrete cover separation failure, the end strap of the shear strengthening must therefore have dimensions of ($b_{Lw} \times t_{Lw}$) 100 × 6 mm.

6.7 Analyses for the serviceability limit state

Analyses of crack width and deformation are not carried out in this example. It is merely verified that the necessary stresses are complied with. According to DAfStb guideline part 1 section 7.2, described in Section 3.6 of this book, the strains in the strip and the reinforcing steel must be limited as follows for a rare load combination:

$$\varepsilon_s \leq \frac{f_{yk}}{E_s} = \frac{500}{200\,000} = 2.5\,\text{mm/m}$$

$$\varepsilon_L \leq 2\,\text{mm/m}$$

Under a rare load combination, we get the following maximum moment at mid-span:

$$M_{E,rare} = 680\,\text{kNm/m}$$

The prestrains $\varepsilon_{s,0} = 0.66$ mm/m, $\varepsilon_{c,0} = -0.26$ mm/m and $\varepsilon_{L,0} = 0.71$ mm/m are calculated as explained in Section 4.3. The strains $\varepsilon_L = 1.18$ mm/m, $\varepsilon_s = 1.76$ mm/m and $\varepsilon_c = -0.53$ mm/m were determined iteratively with the characteristic strengths and the following two conditions:

$$M_R = M_{E,rare}$$

$$F_{s1} + F_L = -F_c$$

As the ultimate strains for the strip and the reinforcing steel are not exceeded, the design is verified.

7 Design of column strengthening with CF sheets

7.1 Principles

As with other materials, triaxial compression loads on concrete lead to an increase in the compression that can be accommodated in the direction of the largest principal stress. Just a hydrostatic lateral pressure amounting to 20% of the uniaxial strength f_{cm} of concrete results in a doubling of the admissible compressive stress; and the admissible deformations also increase considerably. In contrast to a specific load applied in the transverse direction, the effect of confining reinforcement resulting from the prevention of lateral strain is regarded as a passive lateral pressure. Owing to the large deformation capacity of the reinforcing steel, the normal situation in compression members with helical reinforcement, for example, is that the disintegration of the concrete microstructure leads to failure of the member (in a similar way to a triaxial compression test with hydrostatic lateral pressure). If the confining effect is achieved by including transverse reinforcement in the form of fibre-reinforced materials with a virtually linear elastic behaviour, then the lateral pressure rises continuously until the confining reinforcement fails in tension. Figure 7.1 shows a schematic representation of the effect of CFRP wrapping compared with a cross-section containing confining steel reinforcement and an unconfined section.

When it comes to describing the loadbearing behaviour numerically, a distinction has to be made between the load-carrying capacity of the cross-section, which essentially depends on the material properties and therefore can be described by tests (e.g. multiaxial compression tests) on small-format specimens, and the load-carrying capacity of the member, which besides the material properties is also dependent on the geometry of the member and the loading. Only in the case of a concentric load on a short column, in which the influence of slenderness can be excluded, is the load-carrying capacity of the cross-section equal to that of the member.

The development of the principles for designing confined concrete members is attributed to the French engineer *Armand Considére* [110, 111], who in 1902 patented a method for casting concrete elements with a high axial compressive strength. The particular feature of this method was that a metal helix, with closely spaced windings, was placed around the core of the concrete member. On the basis of his experimental studies, Considére formulated an initial addition function that considered the increase in the load-carrying capacity due to the confining reinforcement.

As early as 18 September 1909, the ‘Circular decree concerning the design of concrete columns with confining iron bars’ valid for the Kingdom of Prussia permitted an increase in the load due to the confining effect of helical transverse reinforcement according to Considére’s method. The effect of confining reinforcement was subsequently described in numerous publications.

In the German language the studies by *Müller* [112] and *Menne* [113] are the most important. The design method in DIN 1045 (see [94], for example) for confined compression members was based on their investigations and remained valid and unchanged for more than 25 years. *Müller’s* work was primarily based on tests on

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