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SSF-Ingenieure GmbH

RFCS RFS-P2-08065 INTAB+

Design Guide

Economic and Durable Design of Composite Bridges with Integral Abutments

03.05.2010

1st Edition (Version 0.2)

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The present document and others related to the research project RFS-PR-04120 INTAB "Economic and Durable Design of Composite Bridges with Integral Abutments" and the successive dissemination project RFS – P2 - 08065 INTAB+ "Economic and Durable Design of Composite Bridges with Integral Abutments", which have been co-funded by the Research Fund for Coal and Steel (RFCS) of the European Community, can be accessed for free on the following web site:

http://www.bridgedesign.de

The project reports will be available in the near future free of charge in the EU bookshop:

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Cover picture:

BAB A8 bridge structure 5 near Harlaching, Germany, 2010 (© RWTH)

Preface

This design manual is an outcome of the research project RFS-PR-04120 INTAB "Economic and Durable Design of Composite Bridges with Integral Abutments" (Feldmann, et al., 2010) and the successive dissemination project RFS – P2 - 08065 INTAB+ "Economic and Durable Design of Composite Bridges with Integral Abutments" (Feldmann, et al., 2012) which have been co-funded by the Research Fund for Coal and Steel (RFCS) of the European Community.

Within the RFCS research project essential knowledge has been acquired to enhance the competitiveness of steel and composite bridges with integral abutments and this has been incorporated in the design manual at hand which has been also presented in the frame of several seminars and workshops.

The authors of this design manual gratefully acknowledge the support and financial grant of the Research Fund for Coal and Steel (RFCS) of the European Community.

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The authors would like to thank the further project partners of the original project INTAB "Economic and Durable Design of Composite Bridges with Integral Abutments" for their technical contribution to this design guide. In particular thanks to

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Peter Collin, Olli Kerokoski, Hans Petursson
Ramböll Sverige AB (Ramböll)

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Aachen, May 2010

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1 Introduction

1.1 Motivation

Bridges are of vital importance to the European infrastructure and composite bridges already became a popular solution in many countries and a well-established alternative to concrete bridges. Their competitiveness depends on several circumstances such as site conditions, local costs of material and staff and the contractor's experience. One outstanding advantage of composite bridges compared to concrete bridges is that the steel girders can carry the weight of the formwork and the fresh concrete during casting.

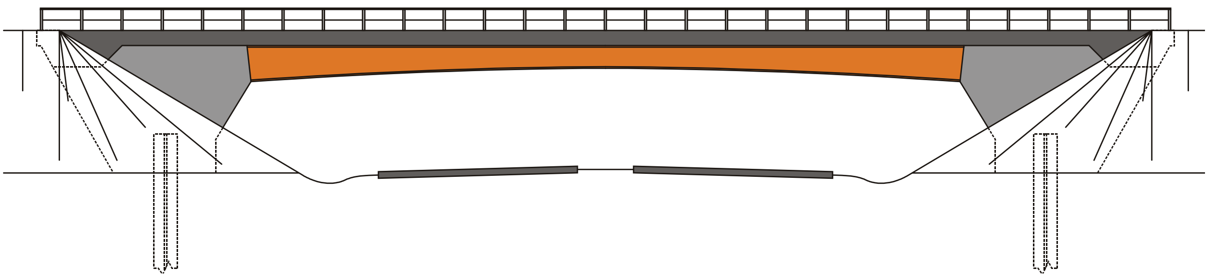


Figure 1-1: Composite bridge with integral abutments

Another major advantage is the savings in construction time, which reduces the traffic disturbance, consequently saves money for the contractor but even more for the road users; a fact that for a long time has been neglected. Recently this factor is increasingly drawn into focus as latest studies show the necessity of taking not only the simple production costs but also the construction time and the maintenance costs into account when deciding for a specific bridge type.

Thus nowadays the following demands are imposed on bridge structures, which are all met by the construction of composite bridges

- low production and maintenance costs
- short construction time, saving costs for traffic disturbances
- bridge construction without essential interference of the traffic under the bridge
- minimised traffic disturbance for maintenance

All these needs are met by integral abutment bridges as well. In addition, this bridge type holds the potential to outclass traditional bridges with transition joints as it does not only reduce production and maintenance costs but saves economic and socio-economic costs as well.

- The superstructure can be designed quite slender, which decreases the construction height and the earthworks respectively. This leads to a further decrease of material, fabrication, transport and construction costs.
- Frame bridges allow in certain spans for the elimination of the middle support. This simplifies the construction of the bridge without essential interference of the traffic under the bridge, as the road has not to be closed.
- Due to the absence of bearings and joints, the maintenance costs can be decreased significantly.

This design guide is addressed to designers, constructors, owners and authorities to help them during the whole process of decision making, planning, design and construction of integral abutment bridges.

Figure 1-2: Composite bridge with integral abutments by SSF, A73 (Munich), Germany

1.2 Advantages

Over years, engineers have become more aware of the disadvantages associated with the use of expansion joints and bearings. Joints are expensive to buy, install, maintain and repair. Joints and malfunctioning expansion bearings can also lead to unanticipated structural damage. These problems with joints are one of the main reasons why the interest for integral abutment is large. The advantages are:

Construction costs:	It is often more economical to construct integral abutment bridges instead of bridges with joints and bearings. The construction time can often be reduced, since fewer piles are needed, and the time consuming installation of expansion joints and bearings are eliminated. Due to the embedded superstructure the construction of the abutment especially its foundation becomes more economical because the abutment is fixed in horizontal direction. Horizontal load cases like earth pressure and breaking forces of vehicles/trains are carried straight over the superstructure into the ground.
Maintenance costs:	Leaking expansion joints is one of the most common reasons to corrosion problems. Expansion joints and bearings have to be maintained, repaired and replaced. Integral bridges have no expansion joints or bearings and are therefore less expensive to maintain.
Modification costs:	It is easier and cheaper to modify an integral bridge, for instance widening.
Driving quality:	No expansion joints, means no bump when vehicles enter or leave a bridge. This gives a smoother ride for the passengers and the noise level is reduced.
Earthquake resistance:	The most common cause of damage to a bridge due to seismic events is loss of girder support. That problem is eliminated in an integral bridge construction.

1.3 Application range

In general, integral bridges are defined as

- single-span or continuous multiple-span bridges
- constructed without movable transverse deck joints at piers or abutments

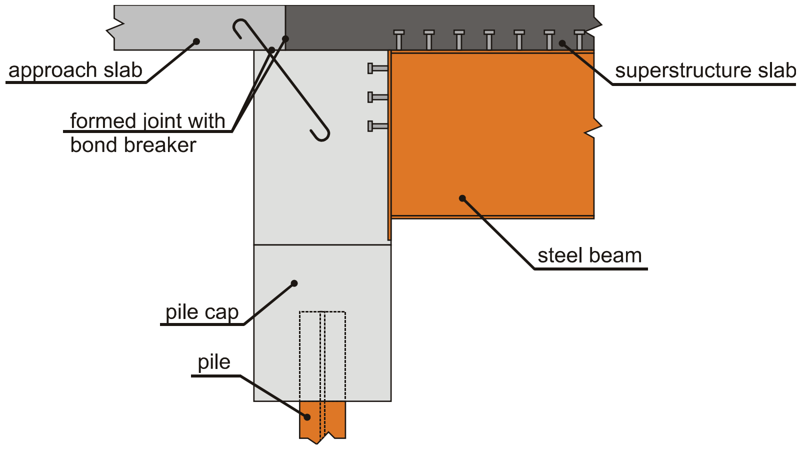


Figure 1-3: Integral abutment concept

Semi-integral bridges are defined as

- single-span or continuous multiple-span bridges
- with abutments supported on rigid foundations
- with a superstructure moving longitudinally independent from the abutments

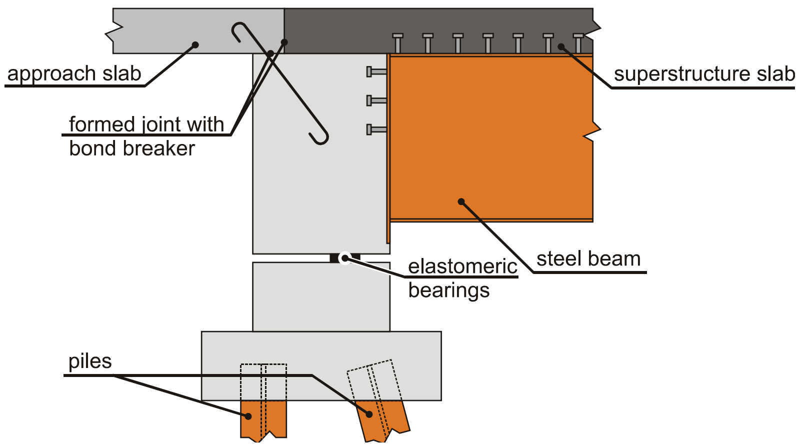


Figure 1-4: Semi-integral abutment concept

1.4 Philosophies

Integral abutment bridges may be generally designed based on two different concepts

1. Low flexural stiffness of piles / low degree of restraint

Especially in the USA, abutments and piers are supported by single rows of flexible piles. The bridge structure can be considered to be a continuous frame. As the columns are quite flexible, the continuous superstructure may be assumed to have simple or hinged supports. Consequently, except for the design of the continuity connections at abutments and piers, frame action can be ignored when analysing the superstructure for superimposed dead and live loads (Burke Jr, 2009). Furthermore, as only low moments need to be conducted through the abutment's corner, the design of that detail becomes rather simple.

2. High flexural stiffness of piles / high degree of restraint

Figure 1-5: BAB A8 bridge structure 5 (SSF), foundation with high flexural stiffness

The more slender the superstructure is intended to be constructed, the stiffer the substructure system has to be (Braun, et al., 2006). In order to increase the corner moment of the bridge and to shift up the field moment, the horizontal member (the continuous superstructure) is partly restraint by the stiff vertical members. Based on that concept, quite slender structures without middle supports can be designed (Figure 1-5). Reference values for the slenderness of road as well as railway bridges are given in Table 1.1.

Table 1.1: Common slenderness of superstructures for road and railway bridges (Braun, et al., 2006)

	construction	**abutment** $l_{sup}/h_{abutment}$	**field** l_{sup}/h_{field}	**without haunch** l_{sup}/h_{field}
Road bridges	reinforced concrete	12-18	20-25	18-21
	prestressed concrete	15-19	24-30	20-25
	steel composite	15-19	25-35	21-25
Railway bridges	reinforced concrete	10-15	20-25	16-18
	prestressed concrete	15-20	20-25	-
	steel composite	15-18	25-30	18-21

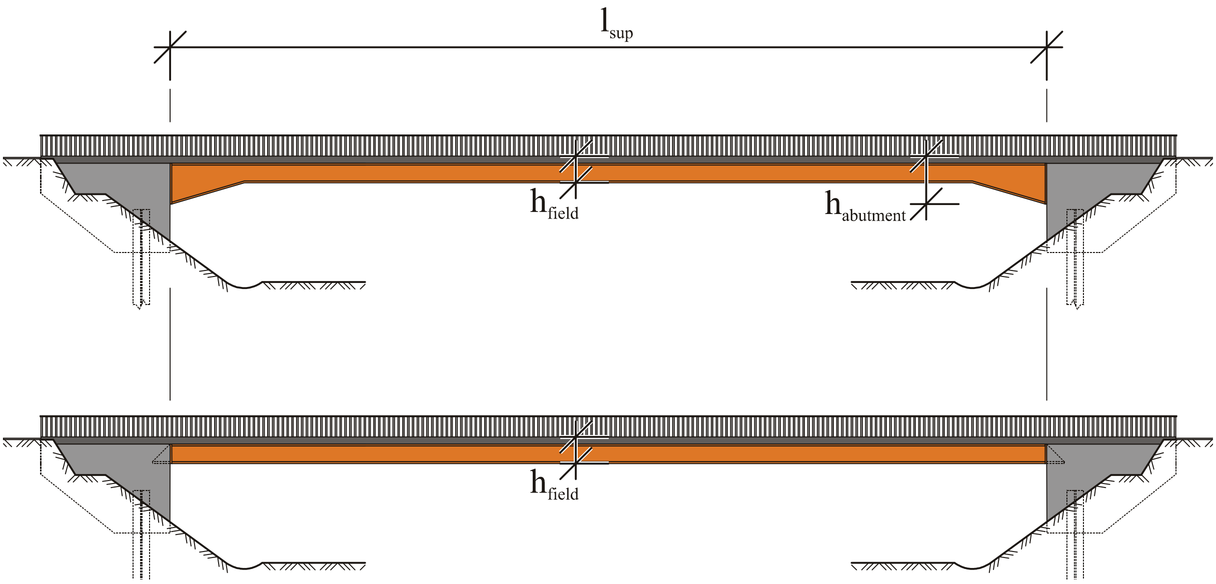


Figure 1-6: Integral abutment bridge, definition of slenderness (with / without haunch)

1.5 Systems

The typical frame reaches over 1 field and is founded on footings (Figure 1-7). In case of large spans, a footing is to be preferred for reason of its flexible horizontal bedding as constraints resulting from temperature and lowering of pillars can be absorbed in a much better way by a yielding structure.

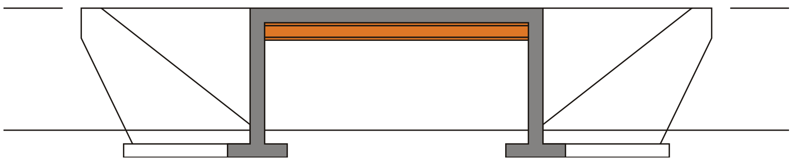


Figure 1-7: Integral abutment bridge

For esthetical reasons but also to improve overview conditions for the vehicles, an implementation of inclined abutment sides is possible (Figure 1-8). Inclining the abutments to the back creates an effectively smaller mid-span moment as the superstructure is dimensioned at span l_{s2} which results in optically very slender superstructures.

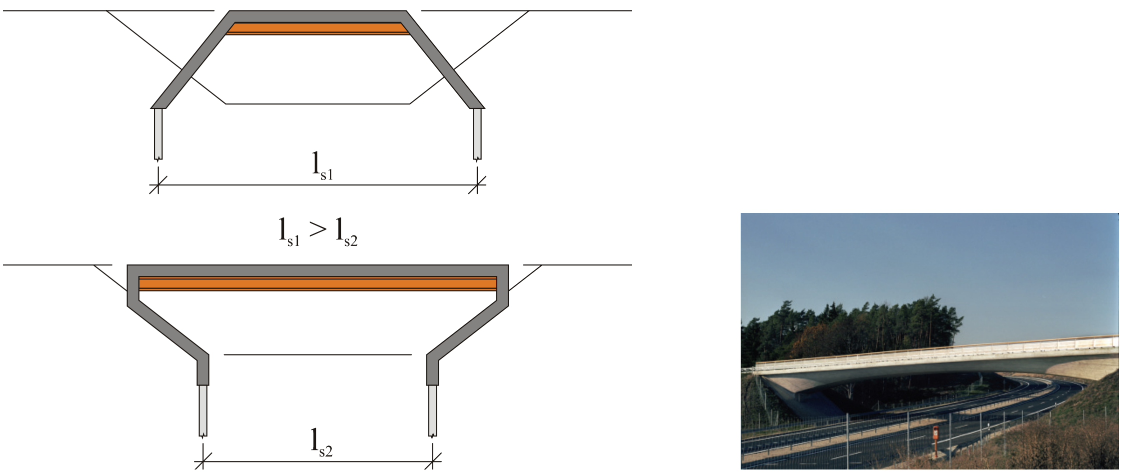


Figure 1-8: Integral abutment bridge with inclined abutments

In case of multi-span structures, it is also advisable to set the pillars and the abutments on footings in any case (Figure 1-8). Alternatively, the abutments can be separated from the superstructure by bearings; called a semi-integral structure (Figure 1-10). Tough, this kind of bearing system loses some of the advantages. High breaking forces from railway traffic, for example, can only be absorbed by pillar blocs underneath the piers with tolerable deformations. With an integral abutment, breaking forces are transferred directly by the pillar bloc underneath the abutment into the backfill (Figure 1-11).

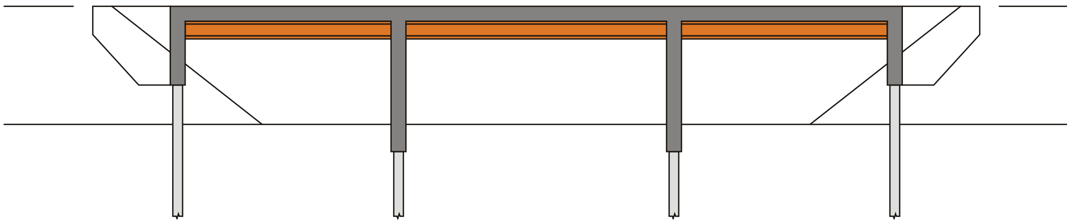


Figure 1-9: Multi span integral abutment bridge

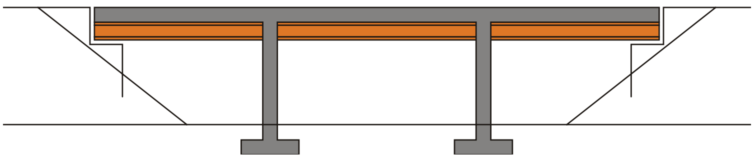


Figure 1-10: Multi span semi-integral abutment bridge

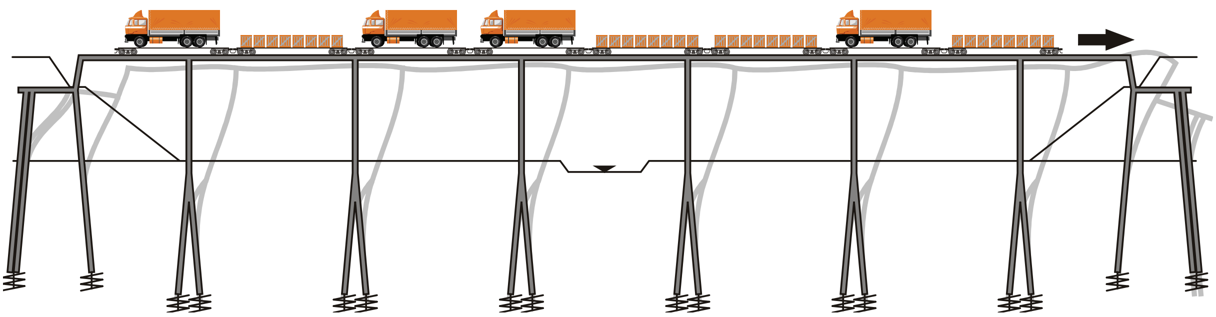


Figure 1-11: Structure deformed due to railway braking force

For long bridges that reach over low valleys, frame bridges are divided into blocs. At the centre, the different sections are equipped with a rigified pillar bloc which is intended to absorb the high breaking forces (Figure 1-12). Deformation between sections can be absorbed without expansion joints.

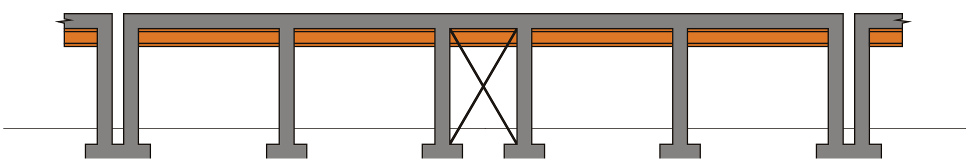


Figure 1-12: Block unit, long railway viaduct

1.6 Materials and cross sections

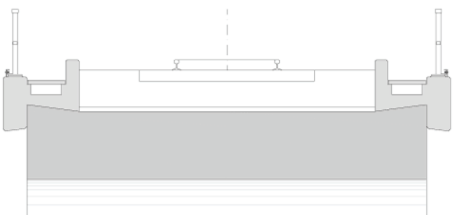
In general, common construction materials are used for frame bridges: concrete, reinforcement, pre-stressing tendons and structural steel. In contrast to bridges with bearings, market shares diverge (Braun, et al., 2006).

Reinforced concrete:	monolithic constructions as slabs and T-beams
	spans up to ca. 20 m, occasionally over 50 m
Pre-stressed concrete:	monolithic construction as slab, T-beams or box sections
	mixed structures with prefabricated elements in reinforced concrete
	spans up to 30 m
Composite method:	monolithic constructions as single or multi-span T-beams with open or closed profiles
	mixed structures with composite prefabricated element girders; pure composite constructions with semi-prefabricated element slabs and cast in-situ supplement are the exception
	spans over 30 m

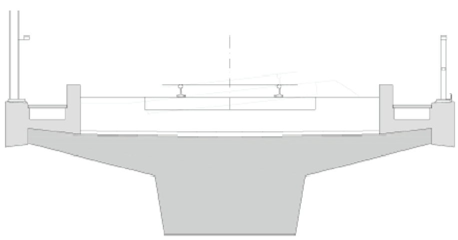
The usual cross sections are illustrated in Figure 1-13 and Figure 1-14.

For railway bridges reinforced concrete frames with short or average span widths are most commonly used. Depending on the client's design wishes and the span width, the form of the cross section of the superstructure beam is either a slab (Figure 1-13 a) or a T-beam (Figure 1-13 b). For superstructures in pre-stressed concrete, a T-beam can be used, too. If the superstructure is holding two tracks, the cross section breaks up into a double-web T-beam.

If great slenderness of the superstructure beam is required, for example to maintain the necessary clearance above water, steel composite frames are used. Hereby the VFT® construction method has established itself as already at the construction stage a framing effect can be produced very easily by the prefabricated concrete slab (Schmitt, et al., 2010). The number of VFT®-girders amounts to two pieces for a single-tracked cross section and to four pieces for a double-tracked superstructure (Figure 1-13 d+e).



a)



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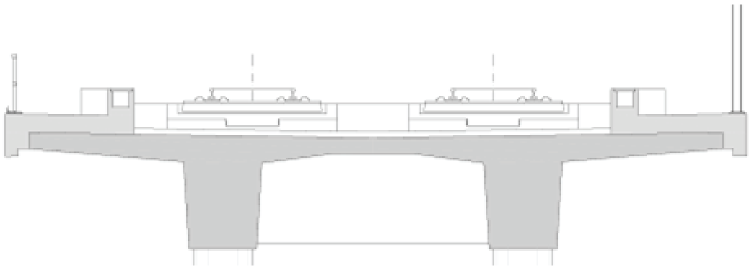

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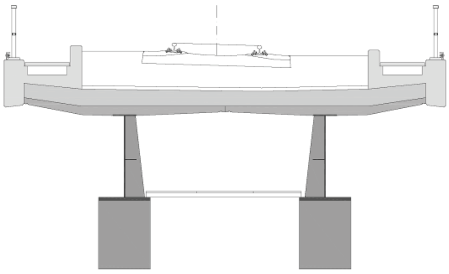
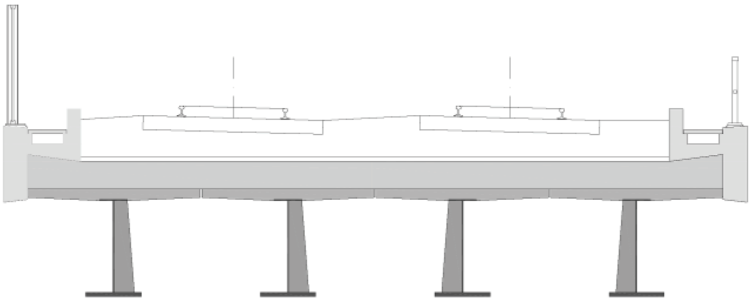
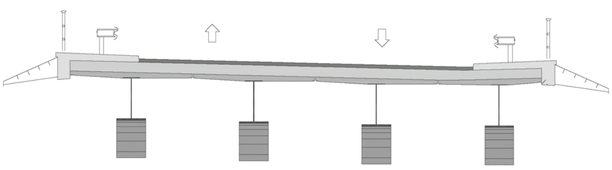




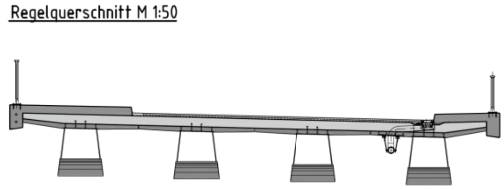
d) e)

Figure 1-13: Cross section for frame bridges for railway traffic

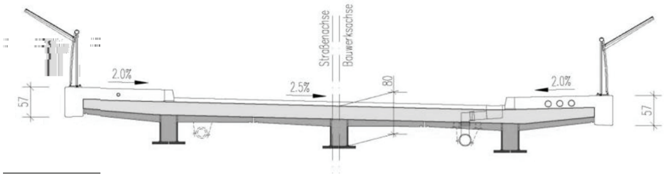
Also for road bridges, the cast in-situ frame presents a very economical alternative as a formwork can be built without difficulty. Depending on the span width, full slabs are used for short spans and divided cross sections for larger span widths in order to reduce dead weight. Cross sections are divided into multi-web T-beams to limit dead weight masses. For construction above traffic routes, the partially prefabricated composite girders, either fabricated at the plant and delivered to the construction site or fabricated on or close to the site are used. They form the main bearing element and at the same time the formwork of the cast in-situ slab. Already at the construction stage, the framing effect for concreting of the cast in-situ slab is achieved by the connecting reinforcement in the prefabricated element at the framing corner. In case of larger spans and greater slenderness, the prefabricated composite element (VFT®) girders are used (Schmitt, et al., 2001). Lately, for road bridges also cross sections with halved rolled profiles, the so called VFT-WIB® girders, were put into application (Figure 1-14). With regards to production costs and maintenance this technique is comparable to the pre-stressed concrete prefabricated elements method (Seidl, et al., 2009).



Prefabricated composite elements (VFT®)

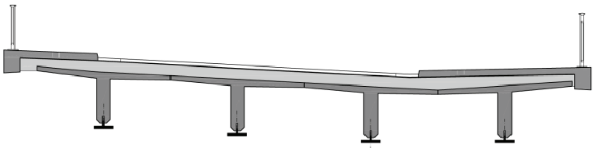


Prefabricated elements with box girder

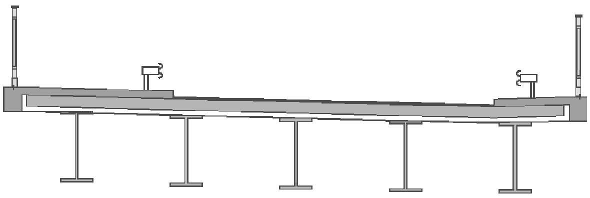


Prefabricated composite element girder with rolled girders in concrete (VFT-WIB®)

with two steel girders in the cross section



with one steel girder in the cross section



Prefabricated composite element with rolled girders (VFT®)

Figure 1-14: Typical cross sections for road bridges in frame construction method

2 Definition of bridge / parts of the bridge

The following items have been adopted as standard nomenclature for use in this design guide:

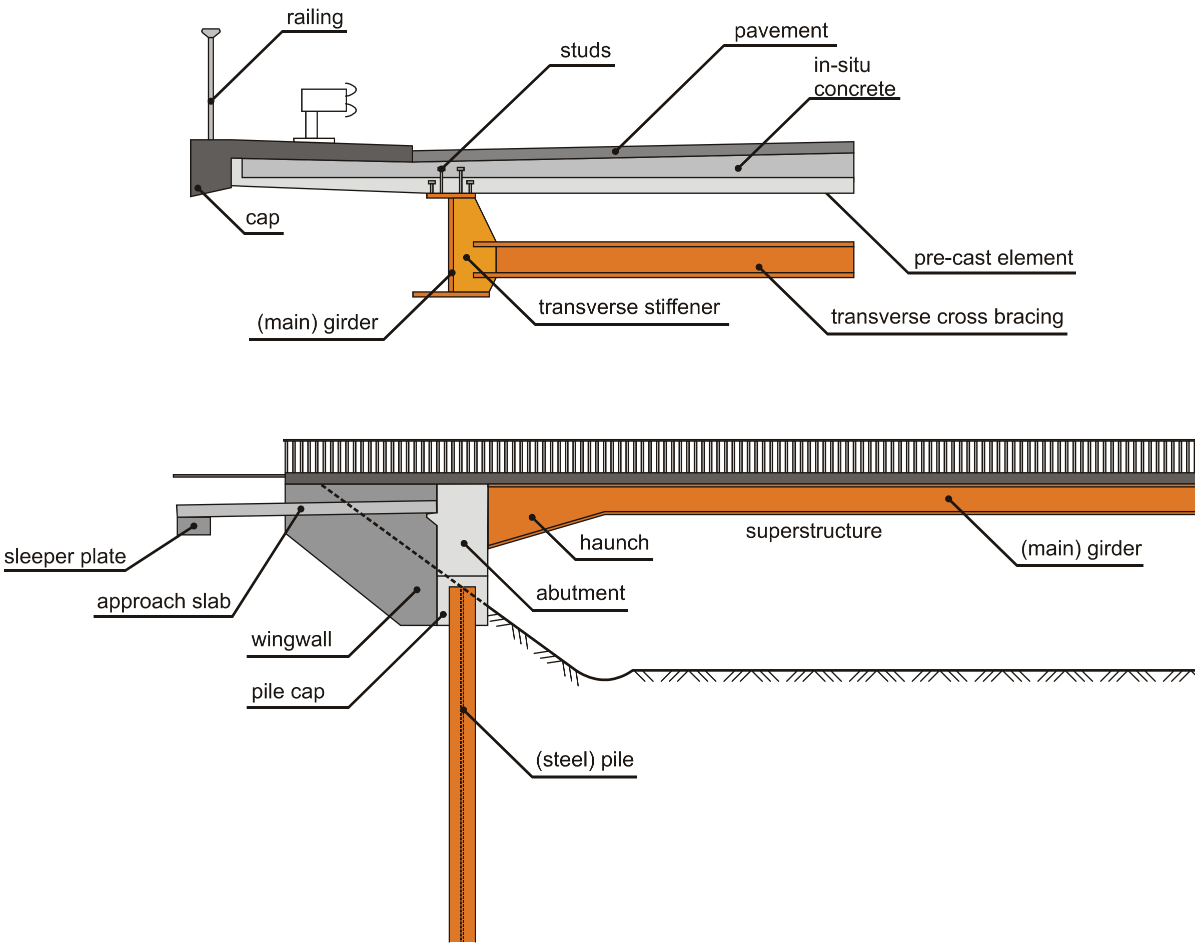


Figure 2-1: Nomenclature

3 Design Overview / Flowchart

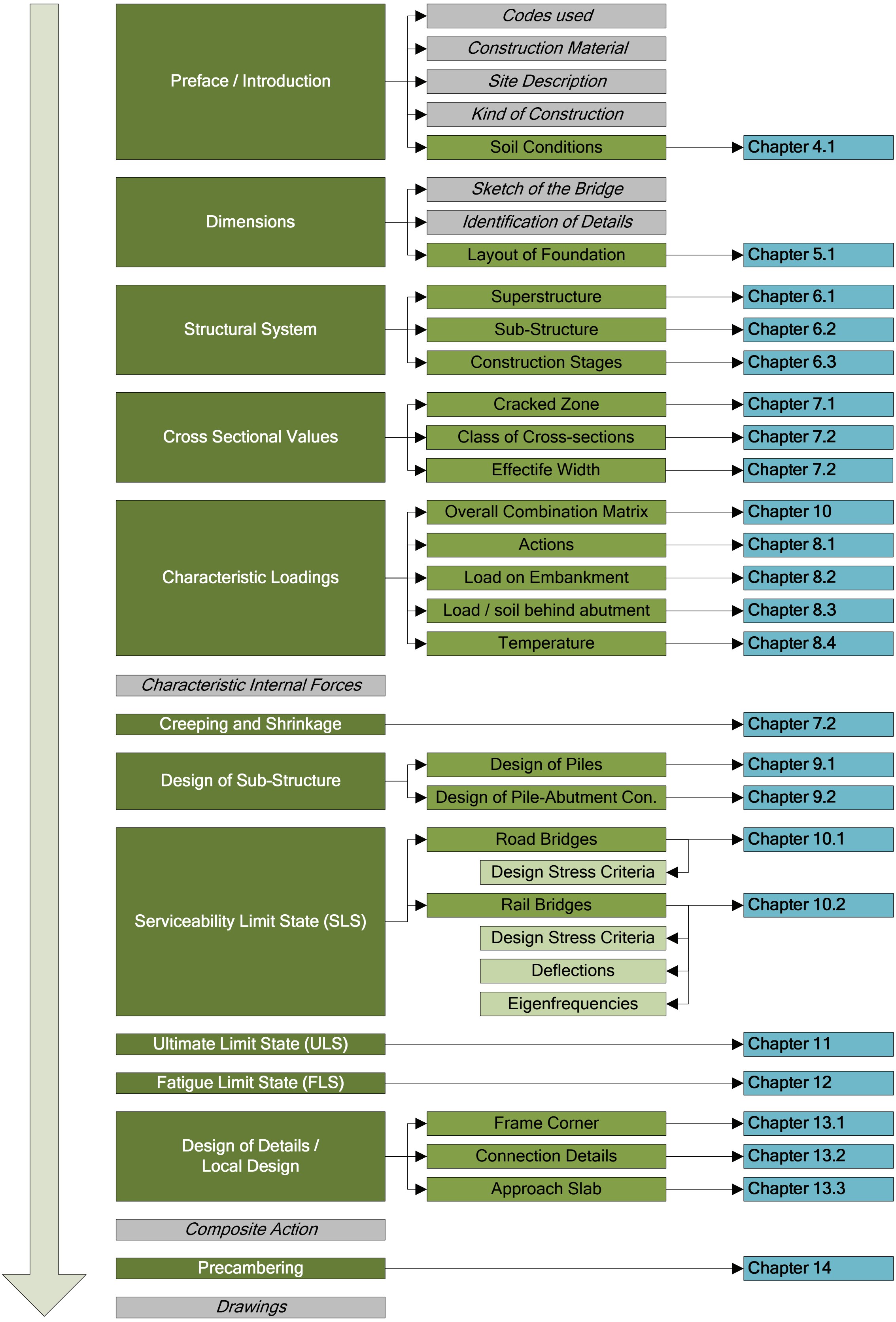


Figure 3-1: Design procedure

4 Preface / Introduction

4.1 Soil conditions (hard soil, soft soil, values)

Soil conditions are generally taken from

- site description
- foundation survey.

<u>In-situ soil / foundation:</u>

For the design, one of the following values is needed for each layer i of in-situ soil, to be taken from the foundation survey / site description:

- $E_{s,i}$ characteristic values of constrained modulus / soil Young's Modulus per layer i
- $k_{s,i}$ coefficient of subgrade reaction per layer i

Here it has to be taken into account, that $E_{s,short}$ (for short term loading) is n times larger as $E_{s,long}$ (for long term loading) (see chapter 10.2.2). An adequate value for n needs to be defined in accordance with the soil expert. Furthermore, the settlement of each axis of the bridge to be applied during design has to be defined by the soil expert as well.

Some design programs allow for the definition of different soil layers i, based on corresponding values of $E_{s,i}$. If this is not possible and springs, based on corresponding values of $k_{s,i}$, have to be applied, $k_{s,i}$ may be taken from the foundation survey or may be calculated (based on $E_{s,i}$) as shown in chapter 6.2.1.

<u>Backfill:</u>

For the design, the following values are needed for backfill:

- K_0 coefficient of earth pressure at rest
- K_a active earth pressure coefficient
- K_p passive earth pressure coefficient

For the determination of these earth pressure coefficients the following values are needed, to be taken from the foundation survey / site description:

- c cohesion of soil
- δ structure-ground interface friction angle (δ_a, δ_p, δ_0) (angle of shearing resistance between ground and wall)
- φ soil friction angle
- a adhesion between soil and wall

5 Dimensions

5.1 Layout of foundation

Different kinds of foundations may be adapted to integral abutment bridges:

- piled foundations
- shallow foundations

Their application is discussed in detail in the following.

5.1.1 Piled foundation, single piles

As already discussed before (see chapter 0), piles for integral abutment bridges may be designed based on two different concepts:

1. Sufficient vertical capacity and low flexural stiffness.

   The stiffness of the piles should be low in order to minimise the flexural effects due to lateral movements and rotations of the abutments. This leads to:

   - simple frame corner design,
   - design close to design of conventional bridge.

   Therefore flexible steel piles orientated about weak axis bending are used, favourably in pre-bored holes filled with loose sand.

2. Sufficient vertical capacity and high flexural stiffness.

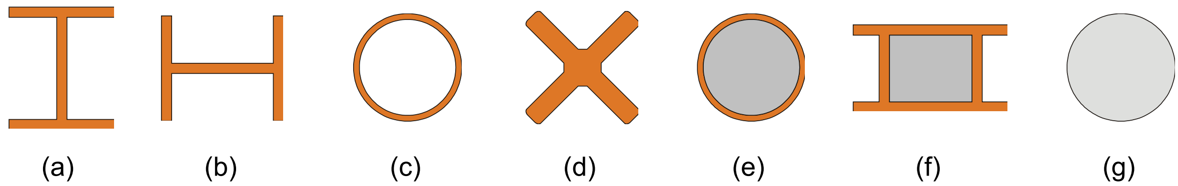
   The stiffness of the piles should be high in order to increase the corner moment of the bridge and to shift up the field moment. This leads to:

   - slender structure,
   - waiving of middle support.

   Therefore stiff cast-in-place concrete piles are used.

For both concepts, the piles are generally distributed evenly in a single line.

Several different pile materials and cross sections have been used in integral bridges. Steel is the most common used pile material in integral bridges. The most common steel-cross-sections are H-piles (see Figure 5-1 (a), (b) respectively). In Germany, mainly concrete is used as a pile material. Composite fibre materials are not used at the moment, but might be a competitive alternative in the near future. Figure 5-1 illustrates the cross-sections that are described in the following sections.



(a) H-pile strong axis bending
(b) H-pile weak axis bending
(c) Steel pipe pile
(d) X-pile rotated 45°
(e) Steel pipe filled with concrete (CIP)
(f) Rectangular FRP pile filled with concrete
(g) Reinforced concrete pile

Figure 5-1: Illustration of different steel / composite pile / concrete cross-sections

5.1.1.1 Steel Piles

Steel piles can take cyclic stresses at least up to their yield stress capacity, provided that the used cross-section will not experience local buckling. If the piles have sufficient rotation capacity and plastic hinges are allowed, then it would be possible to tolerate strains which exceed the yield strain. Plastic strains can however lead to low-cycle fatigue failure and this should be taken into consideration when the piles are designed (see chapter 12.1). The influence of corrosion on steel piles must also be taken into account, since the unaffected cross section area will be time dependent.

5.1.1.2 H-piles

H-piles seem to be the first choice for integral bridges in the USA, especially in longer bridges (Burke Jr, 2009). The opinion of how the H-piles shall be orientated, in weak or strong axis bending, is varying. However, nowadays they are mostly orientated for weak axis bending.

The reason for this is mainly to minimize the stresses in the abutments. For a given displacement of the abutment, a pile oriented for strong axis bending will induce higher stresses in the abutment than a pile oriented for weak axis bending. It is also done in order to make sure that local buckling of the flanges shall not occur, even if the soil is not supporting the pile laterally. (Arsoy, 2000) (Maruri, et al., 2005)

5.1.1.3 Pipe piles

Steel pipe piles are an alternative to steel H-piles. Cyclic load tests performed by Arsoy (Arsoy, 2000) on H-piles and pipe piles with the same width show that pipe piles probably would survive the cyclic loading, and the abutment seems to be the first part to fail if there would be a failure. With the tested piles having the same width, the pipe pile had a 71% larger area than the H-pile, and the moment of inertia was almost 7 times higher. One conclusion drawn by Arsoy is that stiff piles, like pipe piles, shall not be recommended for integral abutment bridges. Regarding to Arsoy, cross-sections with lower flexural stiffness are preferable, like H-piles oriented in weak axis bending, as Arsoy follows the US-concept of low flexural stiffness.

5.1.1.4 X-piles

Cross-shaped steel piles, X-piles, have been used for example in integral bridges in Sweden. The X-shaped piles are driven in a straight line and rotated 45° in order to minimize the bending stresses, see Figure 5-1 (d) (Petursson, et al.).

Figure 5-2: X-piles used for Leduån bridge in Sweden, surrounded by loose sand

5.1.1.5 Reinforced Concrete Piles

In Germany concrete piles are quite common. In general they can be categorized as precast or cast-in-place piles, whereas precast concrete piles may be either conventionally reinforced or prestressed. Their quality is controlled by e.g. dynamical loading tests or integrity checking. Piles and pile cap are connected via connecting reinforcement and form a monolithic structure (see Figure 5-3).

Figure 5-3: Concrete piles used for Entenpfuhler bridge in Germany (connecting reinforcement)

5.1.1.6 Steel Pipes Filled with Concrete

Cast-In-Place piles (CIP) are driven steel pipes which are later filled with concrete and some reinforcement in the upper part of the piles. Like the concrete piles, these piles are generally used in short bridges only. Minnesota Department of Transportation allows these piles in integral bridges with a bridge length less than 45 m (Huang, et al., 2004). In Finland rather large pipe piles which are filled with reinforced concrete are used. This stiff pile will induce high bending moments in the connection between pile and super structure. In Germany CIP piles are used in predrilled holes with diameter 1000 mm.

5.1.2 Piled foundation, sheet piles

Sheet pile walls as foundation members should be taken into consideration, if the sheet pile wall needs to be constructed anyway. Then the existing sheet pile wall can be used as piling system for the integral abutment bridge, additional piling is no longer necessary.

Besides its function as a retaining structure to accommodate the horizontal earth pressures from the soil behind the wall the sheet pile abutment is used also to transfer the vertical design loads from the bridge deck to the underlying subsoil. This requires the sheet piles to be driven deep enough into load bearing subsoil and special care should be applied for the verification of the vertical load bearing capacity and interaction with the retaining function.

Due to the high stiffness of the sheet pile wall, special care needs to be taken for the construction of the abutment-sheet pile connection. On the one hand, slippage between sheet pile wall and abutment needs to be minimized, e.g. by the application of vertical studs, welded to the sheet pile wall. On the other hand, particular attention has to be paid to the problem of concrete cracking within the region of the sheet pile wall.

5.1.3 Shallow foundations

If the subsoil allows for application of shallow foundations, this foundation type represents the most economical solution. The foundations can be centred below the abutment wall, as there is no backing moment induced by soil self-weight necessary for stability reasons. This reduces the abutments self-weight compared to that of self-supporting abutments, which in turn reduces the loading on the foundations, resulting in smaller foundations.

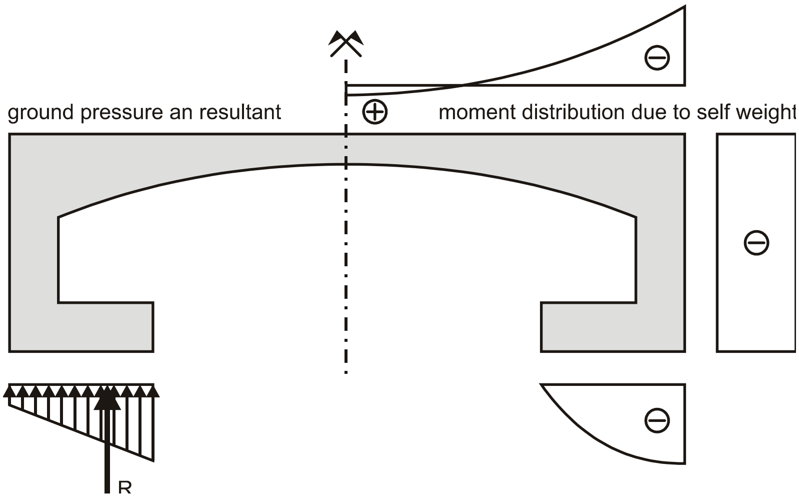


Figure 5-4: Integral abutment bridge on “tip toes” (Braun, et al., 2006)

In case of a recessed abutment, the resultant of ground pressure generates a backing moment, which results into a corner moment in the abutment, allowing for construction of slender structures (see Figure 5-4) (Braun, et al., 2006).

As shallow foundations have not been investigated within the scope of the INTAB project, their design is not covered by this design guide. However, some remarks are given in the following, mainly based on comprehensive studies performed by (Mahlo, et al., 2008).

The foundations need to be designed in such a way, that

- the normal stresses in the bottom of foundation do not exceed the allowable bearing load

- the friction stresses in the bottom of the foundation do not exceed the value of sliding friction (allowing for an adequate safety margin)
- the resulting normal force acts in the kern area of the bottom of foundation

Design models for shallow foundations are often based on the assumption that no horizontal movement of the foundation occurs. However the tangential actions activate a frictional resistance in the bottom of the foundation as well as a mobilised earth resistance at the front end of the foundation (see chapter 8.3, “Load / soil behind abutment ” as well). In both cases the reaction is based on the realised movement. Therefore these effects need to be taken into consideration.

The determination of the horizontal displacement of shallow foundations taking into consideration the frictional resistance in the bottom of the foundation is described in detail in (Mahlo, et al., 2008). Furthermore the application of springs, representing the horizontal bedding, is illustrated. This horizontal bedding is crucial for ULS / SLS design of large integral abutment bridges.

6 Structural system

6.1 Superstructure: Grid model / recommendations for design

Composite frames should be modelised by a grid model including longitudinal beams representing the composite girders and subsidiary cross girders perpendicular to the longitudinal beams representing the in-situ concrete plate. In specific cases e.g. in very skewy structural systems the in-situ concrete plate needs to be modelised by a plate model. The additional flexural stiffness of the plate has to be taken into account due to designing the longitudinal beams. The abutments can be modelised FE including the wingwalls.

The framing system is built in several construction stages. It is important for a correct design that all these stages are implemented in the design model as well.

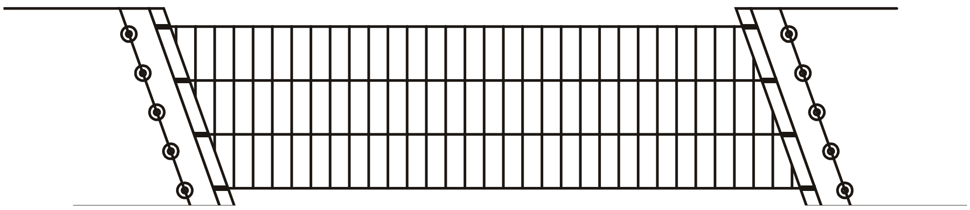


Figure 6-1: Bird view on structural model using beam elements for the superstructure

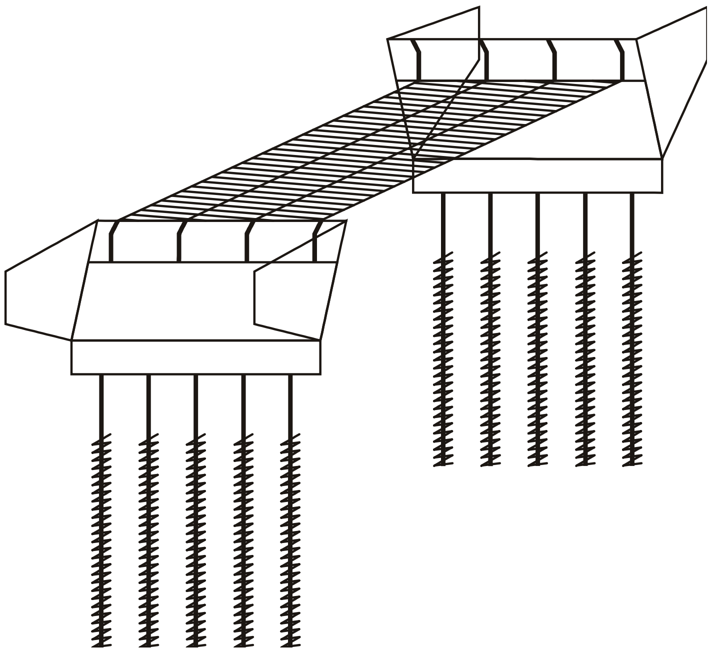


Figure 6-2: 3D view on a typical frame system with gird modelised superstructure, FE abutments and embedded pile elements

6.2 Sub-Structure

6.2.1 Horizontal springs supporting piles

This chapter as well as chapter 8.3 refers to definitions of various geotechnical terms, to avoid misinterpretation between structural and geotechnical engineers. They are summarised in Table 6.1.

Table 6.1: Definition and dimension of terms used in analysis of laterally loaded piles

Description	Symbol	Definition	Dimension
pile diameter Pfahldurchmesser pålens tvärmått	D_s D_s d		[mm]
depth Tiefe jorddjup	z		[mm]
modulus of subgrade reaction - sidomotstånd	K	K=p/y	[N/mm²]
characteristic value of constrained modulus / soil Young's Modulus Steifemodul sättningsmodul	E_s E_s E_k		[MN/m²]
soil spring stiffness	K_s	K_s=F/y	[N/mm]
coefficient of subgrade reaction Bettungsmodul bäddmodul	k_s k_s k	k_s=P/y, k_s=K/D, k_s=E_s/D	[N/mm³]
lateral coefficient of subgrade reaction - tillväxtfaktor	n_h		[N/mm³]
active earth pressure coefficient Beiwert für den wirksamen aktiven horizontalen Erddruck koefficient för aktivt jordtryck	K_a		[-]
coefficient of earth pressure at rest Ruhedruckbeiwert koefficient för vilojordtryck	K_0		[-]
passive earth pressure coefficient Beiwert für den wirksamen passiven horiz. Erdwiderstand koefficient för passivt jordtryck	K_p		[-]

For the elastic bedding of foundation piles, the soil surrounding the piles is represented by linear springs. The possible distribution of spring constants over the depth is brought down to two cases (loose / dense soil), which are considered separately.

Soil surrounding piles is represented by linear springs, see Figure 6-3. These springs should be applied to the pile in regions, where sufficient lateral support by the surrounding soil is expected.

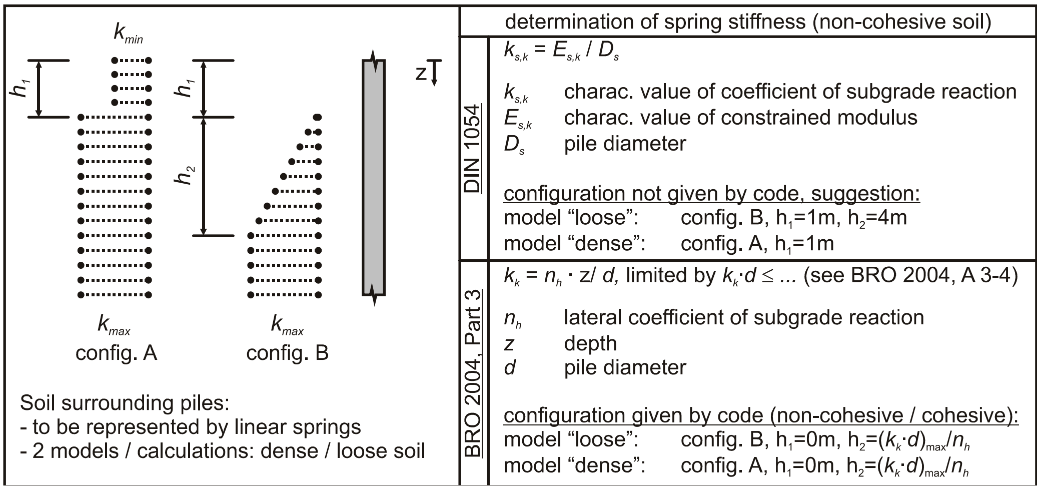


Figure 6-3: Application of springs behind piles

Two set of calculations for design should be performed, taking into account the upper and lower limits of k_s or E_s given by the geological survey. As (EN 1997-1, 2005) does not provide a specific procedure, the following is recommended.

<u>According to DIN 1054:2005-01 (DIN 1054, 2005):</u>

$$k_{s,k} = \frac{E_{s,k}}{D_s} \tag{6.1}$$

where $k_{s,k}$ characteristic value of coefficient of subgrade reaction

$E_{s,k}$ characteristic value of constrained modulus

D_s pile diameter (for D_s >1,00 m, set D_s = 1,00m)

The distribution of $E_{s,k}$ / $k_{s,k}$ over the depth should be based on the geotechnical survey. Alternatively, the following configurations may be applied:

- loose soil: config. B: 1m-5m: 0..$k_{s,max}$, 5m-bottom: $k_{s,max}$
- dense soil: config. A: 0m-1m: $k_{s,min}$, 1m-bottom: $k_{s,max}$

<u>According to Bro 2004 (Bro 2004, 2004):</u>

$$k_k = \frac{n_h \cdot z}{d} \tag{6.2}$$

where k_k characteristic value of modulus of subgrade reaction

n_h lateral coefficient of subgrade reaction

d pile diameter

with a limitation of $k_k \cdot d$. This approach is equivalent to config. B (see Figure 6-3) with h_1=0m, h_2=$(k_k \cdot d)_{\max} / n_h$, $(k_k \cdot d)_{\max}$ acc. to (Bro 2004, 2004), Annex 4, Table 2.

<u>Example:</u>

(DIN 1054, 2005):

given: $E_{s,min}$ = 30 MNm/m², $E_{s,max}$ = 60 MN/m² (geotechnical survey)

$$E_{s,\min} = 30\,MNm/m^2 \;\Rightarrow k_{s,\min} = \frac{30\,MNm/m^2}{0.9m} = 33.33\,MN/m^3$$

$$E_{s,\max} = 60\,MNm/m^2 \;\Rightarrow k_{s,\min} = \frac{60\,MNm/m^2}{0.9m} = 66.67\,MN/m^3$$

The 2D-spring to be applied in the model has the stiffness:

$$k_{s,\min,2D} = 33.33\,MNm/m^3 \cdot 0.9m = 30.00\,MN/m^2$$

$$k_{s,\max,2D} = 66.67\,MNm/m^3 \cdot 0.9m = 60.00\,MN/m^2$$

(Bro 2004, 2004):

given: $n_{h,min}$ = 12 MN/m², $n_{h,max}$ = 18 MN/m² (geotechnical survey)

$$n_{h,\min} = 12\,MNm/m^2 \;\Rightarrow k_{k,\min} = \frac{12\,MN/m^2}{0.9m} \cdot z = 13.33\,MN/m^3$$

$$n_{h,\max} = 18\,MNm/m^2 \;\Rightarrow k_{k,\max} = \frac{18\,MN/m^2}{0.9m} \cdot z = 20.00\,MN/m^3$$

The 2D-spring to be applied in the model has the stiffness:

a) material: sand ($k_k \cdot d \leq 12\,MN/m^2$)

$k_{s,\min,2D} = 13.33\,MNm/m^3 \cdot 0.9m \cdot z = 12.0\,MN/m^2 \cdot z$, constant for $z > 1.00m$

$k_{s,\max,2D} = 20.00\,MNm/m^3 \cdot 0.9m \cdot z = 18.0\,MN/m^2 \cdot z$, constant for $z > 0.67m$

b) material: crushed rock ($k_k \cdot d \leq 50\,MN/m^2$)

$k_{s,\min,2D} = 13.33\,MNm/m^3 \cdot 0.9m \cdot z = 12.0\,MN/m^2 \cdot z$, constant for $z > 4.17m$

$k_{s,\max,2D} = 20.00\,MNm/m^3 \cdot 0.9m \cdot z = 18.0\,MN/m^2 \cdot z$, constant for $z > 2.78m$

6.3 Construction stages

6.3.1 Temporary supports

Using temporary supports during construction, the moment due to self-weight is redistributed from the single steel girder to the composite superstructure.

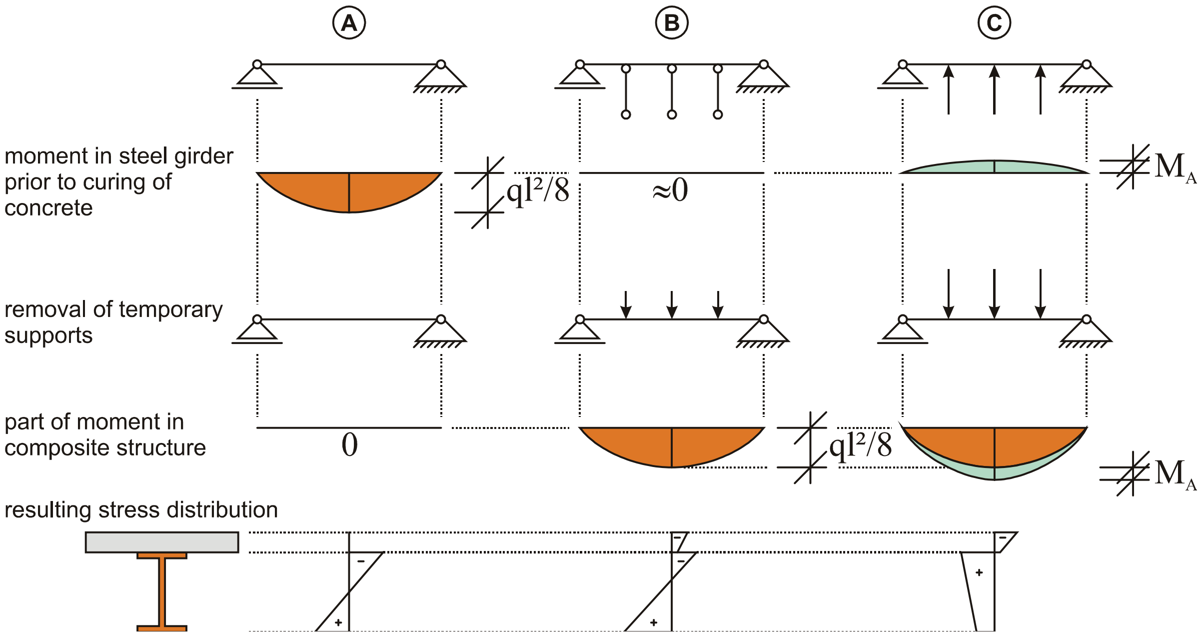


Figure 6-4: Temporary supports during construction, internal moment due to self-weight

A. The steel girder is not supported during construction. The self-weight of the concrete slab as well as the girder itself is carried by the steel girder only. Additional permanent loads as well as life loads, applied after curing of concrete, are acting on the composite structure.
B. The steel girder is supported by temporary supports during construction. Therefore the steel girder stays nearly unloaded during construction. After curing of concrete, the temporary supports are removed; all loads (permanent loads, additional permanent loads, life loads) are acting on the composite structure.
C. The superstructure is constructed as described under (B). Before pouring the concrete, the temporary supports are precambered, producing a negative bending moment (pre-stress) in the steel girder.

This distribution of internal forces has to be taken into account during the design process.

In the case of a pre-fabricated superstructure / partly pre-fabricated superstructure, the designer can in some extend benefit from the advantages of method B. and method C without accepting the disadvantages in the form of temporary supports on construction site.

6.3.2 Time of restraint

Beside the influence of temporary supports, the casting sequence has an effect on the moment distribution in the final structure as well.

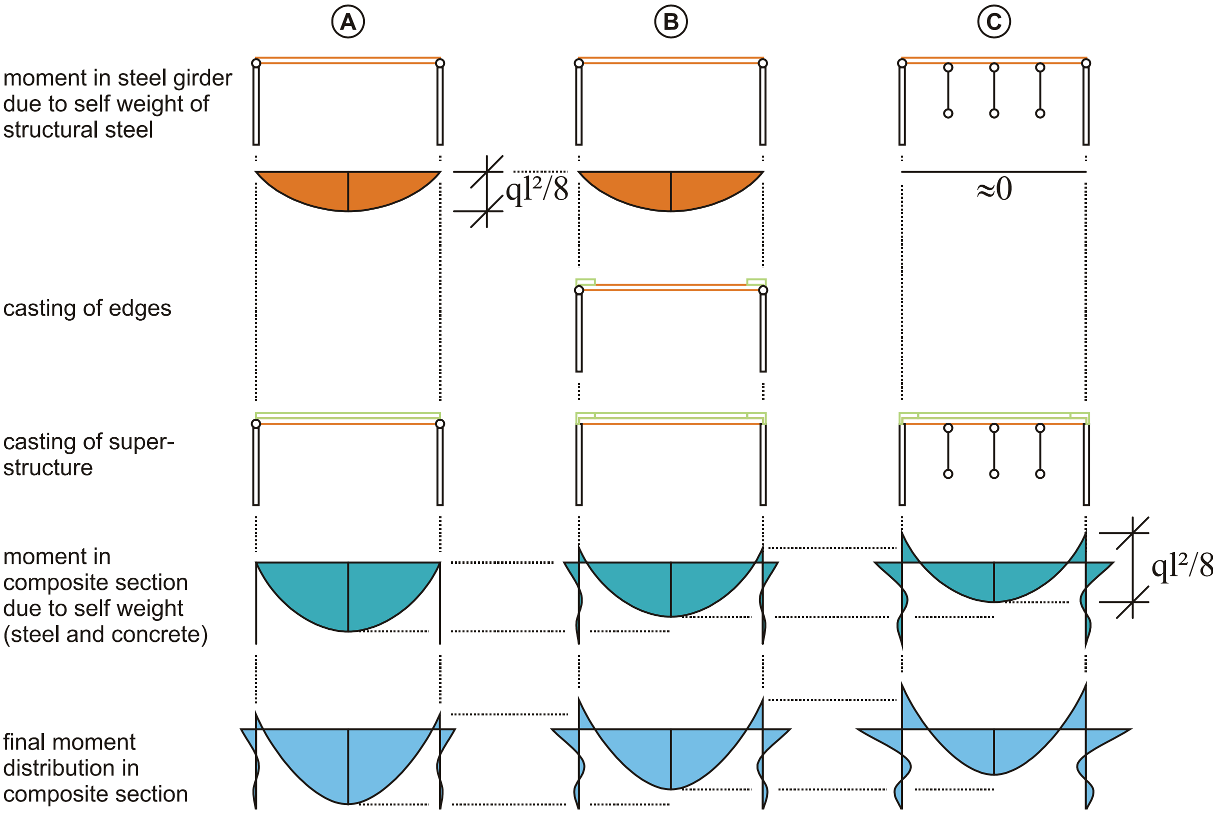


Figure 6-5: Influence of casting sequence / time of restraint

A. The whole slab is cast at once. The self-weight of the concrete slab as well as the girder itself is carried by the simply supported beam. Additional permanent loads as well as life loads, applied after curing of concrete, are acting on the restraint composite structure.
B. Before casting the slab, the steel girder is restraint by casting the edges. The self-weight of the steel girder is carried by the simply supported beam; the self-weight of the concrete slab is carried by the restraint beam. After casting the slab; all loads (permanent loads, additional permanent loads, life loads) are acting on the restraint composite structure.
C. The steel girder is supported by temporary supports during construction. Therefore the steel girder stays almost strainless during construction. Before casting the slab, the steel girder is restraint by casting the edges. The self-weight of the steel girder as well as the self-weight of the concrete slab is carried by the restraint beam. After casting the slab; all loads (permanent loads, additional permanent loads, life loads) are acting on the restraint composite structure.

This degree of restraint during different construction phases as well as in the final structure has to be taken into account during the design process.

Especially if method (A) is chosen, special care has to be taken to avoid cracks in the concrete close to the abutment (continuity connection). Here the fresh concrete could be stressed and cracked due to

a) a substantial temperature drop occurring during initial concrete setting
b) flexural stressing of fresh composite section during concrete placement, as concrete is placed close to the abutments first

To address this problem, several concrete-placement procedures are being used in the USA. These procedures are itemized as follows (Burke Jr, 2009):

- placing continuity connections at sunrise (to avoid a))
- placing deck slabs and continuity connections at night (to avoid a))
- placing continuity connections after deck slab placement (to avoid b))
- using crack sealers

However, Ohio DOT has had reasonable success constructing short single- and multiple-span continuous integral bridges less than 90m long while allowing contractors to place deck slab concrete continuously from one abutment to the other (Burke Jr, 2009).

6.3.3 Backfill

The time of placement of backfill has to be taken into account during design (construction stage). Therefore it has to be specified by the designer by means of a backfill instruction (Braun, et al., 2006).

7 Cross sectional values

7.1 Cracked zone

For ULS calculations, the effective cross section has to be determined considering cracks in concrete. The design bending resistance shall be determined by rigid-plastic theory only where the effective composite cross-section is in Class 1 or Class 2 and where pre-stressing by tendons is not used. The design for bending as well as tension has to be performed neglecting the concrete tensile strength if the component is subjected to tension (DIN FB 104, 2009) (chapter 4.7.2)(EN 1994-2, 2005) (chapter 6.2). This leads to the plastic stress distribution as given in Figure 7-1. For design, in regions of sagging bending, the section consists of concrete as well as structural steel; in regions of hogging bending (e.g. close to the abutment), the section consists of structural steel as well as reinforcement steel.

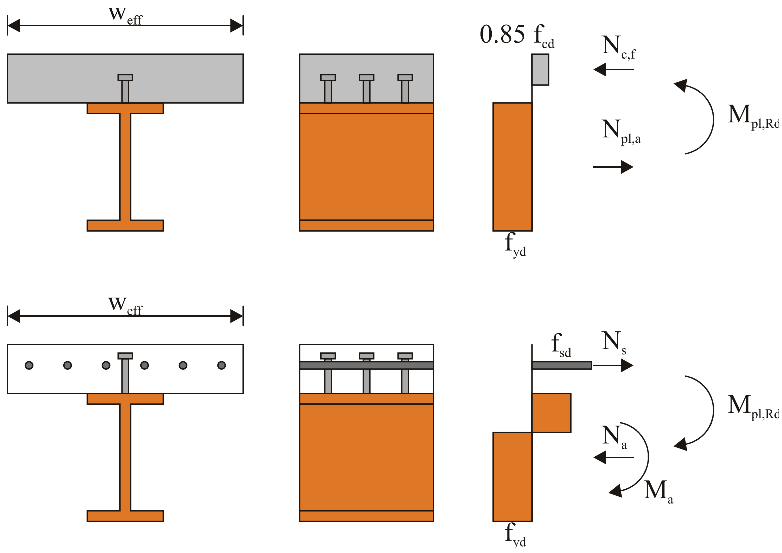


Figure 7-1: Examples of plastic stress distributions for a composite beam with a solid slab and full shear connection in sagging and hogging bending

The assumptions to be made in the calculation of $M_{pl,Rd}$ are given in(EN 1994-2, 2005) (chapter 6.2.1.2), which are summarised in Figure 7-1. Furthermore the full interaction between structural steel, reinforcement, and concrete needs to be guaranteed.

For the SLS determination of stresses, the following effects have to be taken into consideration:

- shear deformation of wide flanges
- creep and shrinkage of concrete
- cracking as well as concrete contribution between cracks
- pre-tensioning
- installation process and load history
- temperature influence
- subsoil movement

7.2 Classification of cross-sections

The classification of cross sections is described in detail in(EN 1994-2, 2005) (chapter 5.5). A composite section is classified according to the least favourable class of its steel elements in compression. The approach as given by (EN 1993-1-1, 2005) is applied here as well. In addition, the following points need to be considered:

- cracked concrete is not allowed to be taken into consideration
- the supporting effect of concrete on the prevention of local steel plate buckling may be taken into consideration as long as defined boundary conditions are satisfied

For example a steel compression flange that is restrained from buckling by effective attachment to a concrete flange by shear connectors may be assumed to be in Class 1 if the spacing of connectors is in accordance with (EN 1993-1-1, 2005) (chapter 6.6.5.5.).

A steel outstand flange of a filler beam deck should be classified in accordance with (EN 1993-1-1, 2005) (table 5.2), a web in Class 3 that is encased in concrete may be represented by an effective web of the same cross-section in Class 2.

7.3 Effective width for verification of cross-sections

The effective width of the concrete flange for verification of cross-sections should be determined in accordance (EN 1994-2, 2005) (chapter 5.4.1.2) taking into account the distribution of effective width between supports and mid-span regions.

7.4 Creepage and shrinkage

Creepage of concrete results in a relocation of partial inner forces, as the steel profile and the reinforcement offer resistance to time-dependant plastic concrete deformations. Permanent loading results into a redistribution of partial inner forces from concrete to steel.

Shrinkage of concrete results into residual stresses as shown in Figure 7-2. In statically indeterminate systems, this primary residual stress state causes deformations and constraints, which are called “secondary prestresses”. The primary and secondary effects have to be taken into consideration regarding SLS crack width limitations.

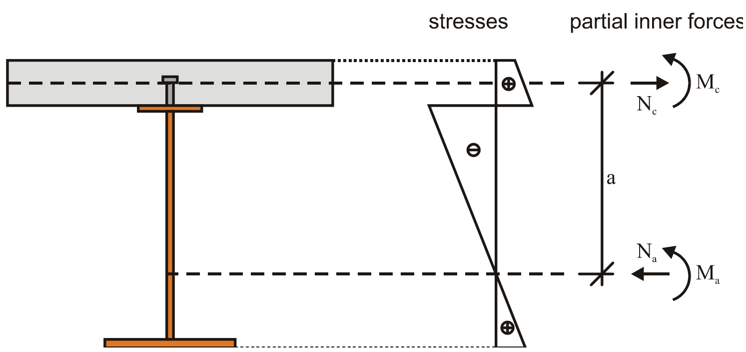


Figure 7-2: Primary residual stresses due to shrinkage

This redistribution of inner forces / stresses can be considered by

- n-value approach (transformed section method) (Eibl, 1999),
- iterative internal stress redistribution.

8 Characteristic loadings

8.1 Actions

Generally, actions are classified as

- Permanent actions
  e.g. self-weight of structural members, fixed equipment and indirect actions such as shrinkage
- Variable actions
  e.g. traffic loads, wind loads
- Accidental actions
  e.g. vehicle impact.

The following loads and load cases are taken into consideration here:

Table 8.1: Loads taken into consideration

type of loading	name of load case	load case n°	reference
permanent loads	*self-weight*	*LC1*	
	construction load (e.g. railing)	*LC2*	
	backfill surcharge on foundations	*LC3*	
	earth pressure at rest	LC4	chapter 8.3
secondary stresses	shrinkage	LC5	chapter 7.2
differential settlement	settlement at supports	LC6, LC7	chapter 4.1
live loads	traffic load on backfill / embankment	LC8-LC11	chapter 8.2
	traffic on bridge	*LC12-LC13* *LC16-LC18*	
	braking, acceleration	LC14-LC15	chapter 10.2.2
	fatigue	*LC19*	
temperature	constant and linear temperature change	LC20-LC27	chapter 8.4
	earth pressure due to constant temperature change	LC28-LC29	chapter 8.3
wind load	*wind on structure and traffic*	*LC30*	

The following loads and load cases are not taken into consideration within the scope of this design guide:

- earthquake load
- snow loading (e.g. on canopied bridges)
- stream and ice pressure, debris torrents
- ice action load
- collision load

8.2 Traffic load on backfill / embankment

Vertical traffic load on the embankment acts as

- horizontal loading on the backwall (LC9, LC11)
- horizontal loading on the wingwalls (LC8, LC10)

For loading, the same traffic loads as the ones acting on the bridge have to be taken into consideration (EN 1991-2, 2003) (DIN FB 101, 2009).

To convert the vertical load q into a horizontal load, it has to be multiplied by the coefficient for vertical loading $K_{0,q}$, see Annex 1.

$$\sigma_0(z) = q \cdot K_{0,q} \tag{8.1}$$

Unlike the loading due to self-weight of soil, this loading does not change with the depth.

8.3 Load / soil behind abutment back wall

An application of linear springs is not possible in this case, as

- the deformations are too large which calls for the use of non-linear springs at least in the upper part of the abutment for wall movements $v_h > v_{p,50}$. However, these non-linear springs contradict the principle of loadcase superposition;
- soil just supports the abutment at one side, thus the forces in the springs may become negative.

Therefore the soil is represented by an external loading (LC28, LC29), which is combined with the conventional temperature load cases (LC20 – LC27). Here it has to be distinguished between a winter (b) and a summer (a) case, which are based on active and mobilised passive earth pressure respectively. Furthermore a permanent load case (LC4), representing the at rest condition of the backfill, needs to be applied, see Figure 8-2.

In winter times, when the wall moves away from the soil due to contradiction, the earth pressure tends to the limiting value of active earth pressures $\sigma_a(z)$. As this limiting value is already activated by a relatively small movement v_a of the abutment, it provides the basis for the variable winter load case LC29. Accordingly the limiting value of passive earth pressure $\sigma_p(z)$ could serve as basis for the variable summer load case LC28. However this approach is far too conservative for bridges with small and medium spans, as the complete passive earth pressure is by far not activated during summer times. Therefore an approach proposed by Vogt is adopted to determine the so-called “mobilised” passive earth pressure $\sigma_{p,mob}(z)$ on the backwall, based on the maximum movement of the abutment during summer times.

Initially, the differential movement of the abutment during summer and winter time needs to be determined. This can be done by application of two load cases, which are used only for that purpose, see Figure 8-1.

	LC		Type	γ
temperature for earth pressure	100	const temperature, elongation	γ_Q	1,00
	101	const. Temperature, contraction	γ_Q	1,00

Figure 8-1: Temperature load cases for determination of soil pressure behind abutments

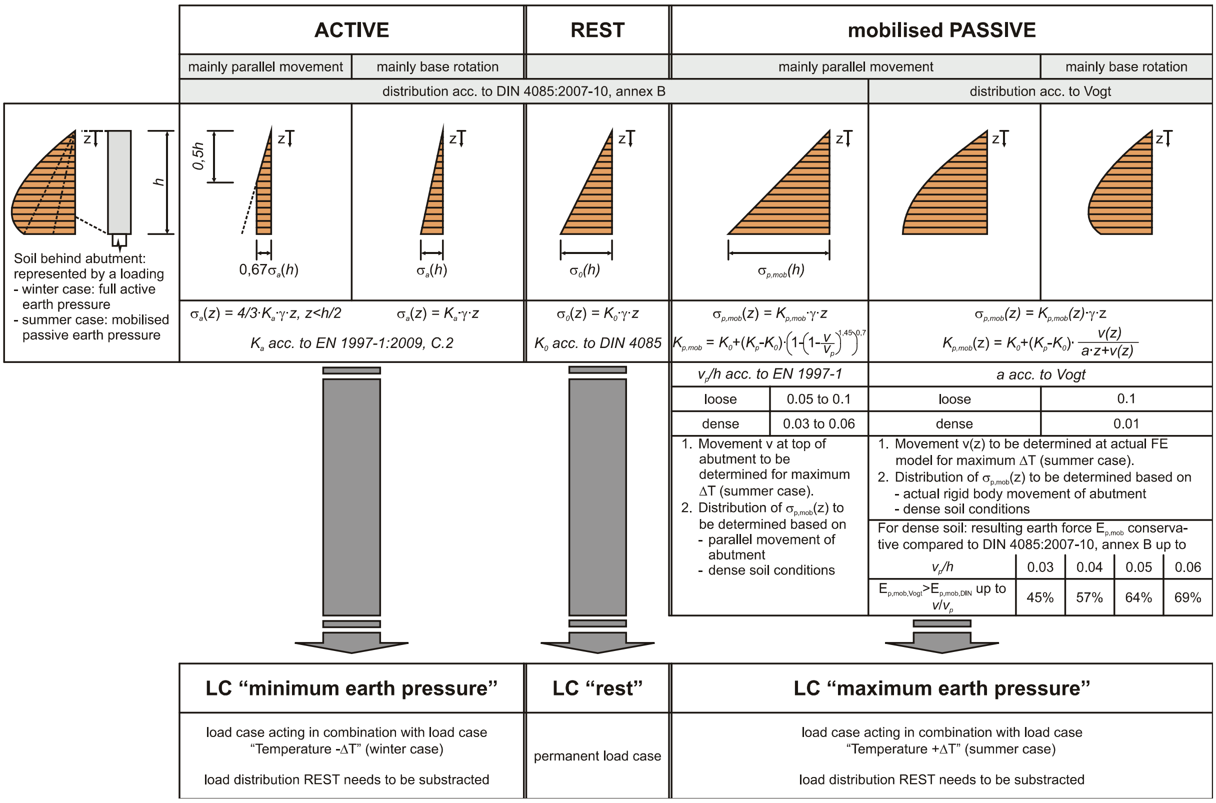


Figure 8-2: Application of loading behind abutments

Three load cases are applied:

- LC “rest” (LC4) → permanent load case
- LC “minimum earth pressure” (LC29) → to be applied in combination with LC “Temperature –ΔT”
- LC “maximum earth pressure” (LC28) → to be applied in combination with LC “Temperature +ΔT”

<u>LC “rest” (LC4)</u>

1. determination of K_0 (see Annex 1)
2. determination of $\sigma_0(z) = K_0 \cdot \gamma \cdot z$
3. application of $\sigma_0(z)$ as permanent loading

<u>LC “minimum earth pressure” (LC29)</u>

1. determination of K_a (acc. to (EN 1997-1, 2005), C.1 or C.2, see Annex 1)
2. determination of $\sigma_a(z)$

- $\sigma_a(z) = K_a \cdot \gamma \cdot z$ (mainly base rotation)
- $\sigma_a(z) = \frac{4}{3} K_a \cdot \gamma \cdot z$ for $z < \frac{h}{2}$; $\sigma_a(z) = \frac{2}{3} K_a \cdot \gamma \cdot z$ for $z \geq \frac{h}{2}$ (parallel movement)

(The resulting total forces are the same, thus both cases can be applied)

3. application of $\sigma_a(z) - \sigma_0(z)$ as a loadcase

<u>LC "maximum earth pressure" (LC28)</u>

Generally, two cases may be considered:

	mobilised resistance acc. to Vogt	mobilised resistance acc. to (DIN 4085, 2007)
1.	determination of movement of abutment due to maximum temperature loading +ΔT	
	$v(z)$ over abutment height (needs to be determined by calculation on the particular system; conservatively, earth behind abutment is neglected)	v_{max} (on top of abutment, may be determined by means of a hand calculation)
2.	determination of v_p (acc. to (EN 1997-1, 2005))	
3.	determination of K_p (acc. to (EN 1997-1, 2005), C.1 or C.2, see Annex 1)	
4.	determination of $K_{p,mob}$	
	$K_{p,mob,Vogt}(z) =$ $K_0 + (K_p - K_0) \cdot \frac{v(z)}{a \cdot z + v(z)}$ where a = 0.1 for loose soil a = 0.01 for dense soil to gain a safe sided approach, a=0.01 should generally be assumed acc. to [4]	$K_{p,mob,DIN} =$ $K_0 + (K_p - K_0) \cdot \left[1 - \left(1 - \frac{v_{\max}}{v_p}\right)^{1.45}\right]^{0.7}$ (acc. to (DIN 4085, 2007))
5.	determination of $\sigma_p(z)$	
	$\sigma_{p,mob,Vogt}(z) = K_{p,mob,Vogt}(z) \cdot \gamma \cdot z$	$\sigma_{p,mob,DIN}(z) = K_{p,mob,DIN} \cdot \gamma \cdot z$
6.	application of $\sigma_{p,mob,Vogt}(z) - \sigma_0(z)$ or $\sigma_{p,mob,DIN}(z) - \sigma_0(z)$ as a loadcase	

ULS consideration:

For parallel movement and dense soil, loading based on Vogt is higher (compared to (DIN 4085, 2007)) at least up to $v < 0.45 \cdot v_p$

SLS consideration:

For parallel movement and dense soil, loading based on (DIN 4085, 2007) is higher (compared to Vogt) at least up to $v < 0.45 \cdot v_p$

8.4 Temperature Loading

(EN 1990/A1, 2006) "Actions on structures – bridge application" does not cover explicitly the assessment of thermal actions on bridges. Therefore (EN 1991-1-5, 2003) "Actions on structures - Part 1-5: General actions - Thermal actions", covering the assessment of thermal actions to be used in the structural design of buildings and civil engineering works due to exposure to daily or seasonal climatic changes and variations has to be applied.

For the purposes of (EN 1991-1-5, 2003), bridge decks are grouped as follows:

- Type 1 Steel deck:
    - steel box girder
    - steel truss or plate girder
- Type 2 Composite deck
- Type 3 Concrete deck:
    - concrete slab
    - concrete beam
    - concrete box girder

Within the scope of this design guide, only Type 2 bridge decks are considered.

(EN 1991-1-5, 2003) distinguishes between two different temperature load types:

- uniform temperature component $\Delta T_{N,exp/con}$
  The uniform temperature component depends on the minimum and maximum temperature which a bridge will achieve. This results in a range of uniform temperature changes which results in a change in element length.
- temperature difference component $\Delta T_{M,heat/cool}$
  Over a prescribed time period heating and cooling of a bridge deck's upper surface will result in a maximum heating (top surface warmer) and a maximum cooling (bottom surface warmer) temperature variation, which is taken into consideration by these 2 load cases.

In case of frame structures, it is necessary to regard both the temperature difference $\Delta T_{M,heat/cool}$ and the maximum range of uniform bridge temperature component $\Delta T_{N,exp/con}$ acting simultaneously.

$$\Delta T_{M,heat}\left(or\ \Delta T_{M,cool}\right) \qquad + \varpi_N \Delta T_{N,\exp}\left(or\ \Delta T_{N,con}\right) \tag{8.2}$$

$$\varpi_M \Delta T_{M,heat}\left(or\ \Delta T_{M,cool}\right) \qquad + \Delta T_{N,\exp}\left(or\ \Delta T_{N,con}\right) \tag{8.3}$$

where ω_N reduction factor of uniform temperature component for combination with temperature difference component

ω_M reduction factor of temperature difference component for combination with uniform temperature component

The National annex may specify numerical values of ω_N and ω_M. If no other information is available, the recommended values for ω_N and ω_M are:

ω_N = 0,35

ω_M = 0,75

This results in the following temperature load cases:

Summer-cases (always to be combined with LC28):

LC20 $\Delta T_{M,heat} + \varpi_N \Delta T_{N,\exp}$

LC21 $\Delta T_{M,cool} + \varpi_N \Delta T_{N,\exp}$

LC22 $\varpi_M \Delta T_{M,heat} + \Delta T_{N,\exp}$

LC23 $\varpi_M \Delta T_{M,cool} + \Delta T_{N,\exp}$

Winter-cases (always to be combined with LC29):

LC24 $\Delta T_{M,heat} + \varpi_N \Delta T_{N,cox}$

LC25 $\Delta T_{M,cool} + \varpi_N \Delta T_{N,con}$

LC26 $\varpi_M \Delta T_{M,heat} + \Delta T_{N,con}$

LC27 $\varpi_M \Delta T_{M,cool} + \Delta T_{N,con}$

9 Design of sub-structure

9.1 Design of piles

9.1.1 Geotechnical works

The following standards apply to the execution of piles:

- (EN 1536, 1999) Execution of special geotechnical work. Bored piles
- (EN 12063, 1999) Execution of special geotechnical work. Sheet pile walls
- (EN 12699, 2000) Execution of special geotechnical work. Displacement piles

Rules for the structural design of piles subjected to axially and laterally loading are given in (EN 1997-1, 2005), chapter 7.8.

(EN 1993-5, 2007) "Design of steel structures - Part 5: Piling" provides principles and application rules for the structural design of bearing piles and sheet piles made of steel.

Depending on the aggressiveness of the media surrounding steel piles corrosion is to be taken into account in the design by a reduction of thickness. Corrosion rates are given by (EN 1993-5, 2007) but may also be given in the National Annex of each country.

9.1.2 Steel piles ULS

According to (EN 1993-5, 2007) the following failure modes should be verified for a steel pile

- failure due to bending and/or axial force;
- failure due to overall flexural buckling, taking account of the restraint provided by the ground and by the supported structure at the connections to it;
- local failure at points of load application;
- fatigue.

(EN 1997-1, 2005), Section 7 presents guidelines to design pile foundations. According to Chapter 7.2 for example the following limit states shall be considered: structural failure of the pile in compression, tension, bending, buckling or shear.

(EN 1997-1, 2005) Chapter 7.3 presents the required actions and design situations and *Chapter 7.4* the design methods and design considerations. The design may be based on analytical calculation methods. *Chapter 7.6* regards axially loaded piles.

(EN 1997-1, 2005) Chapter 7.7 gives rules to design transversely loaded piles. The transverse resistance of a pile or pile group shall be calculated using a compatible set of structural effects of actions, ground reactions and displacements. The analysis of a transversely loaded pile shall include the possibility of structural failure of the pile in the ground. The calculation of the transverse resistance of a long slender pile may be carried out using the theory of a beam loaded at the top and supported by a deformable medium characterised by a horizontal modulus of subgrade reaction.

(EN 1997-1, 2005) Chapter 7.8 presents the structural design of piles. Piles shall be verified against structural failure. The structure of piles shall be designed to accommodate all the situations to which the piles will be subjected, including corrosion conditions, installation (ground conditions such as boulders or steeply inclined bedrock surfaces), driveability and transportation. Slender piles passing through thick deposits of very weak soil shall be checked against buckling.

For failure of the piles and their connections to the structure the design should be done according to (EN 1993-5, 2007) or (EN 1994-2, 2005). If the soil provides insufficient lateral restraint, the slenderness criterion for overall buckling may be assumed to be satisfied if N_{Ed} / N_{cr}= 0,10, where N_{cr} is the critical value of the axial force N_{Ed}. In addition to the imperfections given in chapter 5.3 of (EN 1993-1-1, 2005) consideration should be given to supplementary initial imperfections (e.g. due to joints or installation) in accordance with (EN 12699, 2000) and (EN 14199, 2005).
An approach to decide buckling length l_{cr} is given in (EN 1993-5, 2007), chapter 5.3.3.

9.1.3 Steel piles SLS

According to (EN 1993-5, 2007) the following criteria in SLS should be considered:

- limits to vertical settlements or horizontal displacements necessary to suit the supported structure;
- vibration limits necessary to suit structures directly connected or adjacent to the bearing piles.

The global analysis should be based on a linear elastic model of the structure, and a soil-structure model.

It is to be shown that no plastic deformations occur in the structure as a result of serviceability loading.

The piles below the abutment are subject to displacements and bending moments because of the relative displacements between the pile and the surrounding soil. At the same time, they are supported horizontally by that same soil. Consequently, they cause a special case regarding the behaviour of the structure.

If the abutments are used to transfer horizontal loads to the embankment, the same phenomenon has to be considered.

9.1.4 Grouped piles

According to (EN 1997-1, 2005), the group effect needs to be taken into consideration for the design of

- compression piles;
- tension piles;
- transversely loaded piles.

<u>Piles under compression</u>

Regarding the compressive ground resistance, two failure mechanisms shall be taken into account for piles in groups:

- compressive resistance failure of the piles individually;
- compressive resistance failure of the piles and the soil contained between them acting as a block.

The design resistance shall be taken as the lower value caused by these two mechanisms. The compressive resistance of the pile group acting as a block may be calculated by treating the block as a single pile of large diameter.

Piles under tension

For tension piles, the group effect, which may reduce the effective vertical stresses in the soil and hence the shaft resistances of individual piles in the group, shall be considered when assessing the tensile resistance of a group of piles.

Here two failure mechanisms shall be considered:

- pull-out of the piles from the ground mass;
- uplift of the block of ground containing the piles.

For isolated tensile piles or a group of tensile piles, the failure mechanism may be governed by the pull-out resistance of a cone of ground, especially for piles with an enlarged base or rock socket. Normally the block effect will govern the design tensile resistance if the distance between the piles is equal to or less than the square root of the product of the pile diameter and the pile penetration into the main resisting stratum.

Transversely loaded piles

The group effect shall be considered as well when assessing the resistance of transversely loaded piles. It should be considered that a transverse load applied to a group of piles may result in a combination of compression, tension and transverse forces in the individual piles.

For transversely loaded piles in groups where the pile heads are all displaced in equal measure, (DIN 1054, 2005) Annex E gives information regarding the distribution of forces by means of reduction ratios.

9.2 Design of pile-abutment connection

9.2.1 Concrete piles

Fully integral abutment bridges

Concrete piles as well as rigid pile-abutment connections are designed in accordance to (EN 1992-2, 2005) or (DIN FB 102, 2009).

Semi-integral abutment bridges

For the construction of semi-integral bridges, hinged joints may be used. The hinge transfers only vertical and shear forces to the piles but no bending moments. An example of a bridge with abutments constructed with this technique is Gillies Street Bridge, Australia. Figure 9-1 shows a modified sketch of one of the abutments from this bridge and the hinged connection between the abutment and the concrete piles that were used. The precast beam of the superstructure has been replaced for illustration purpose by a steel beam.

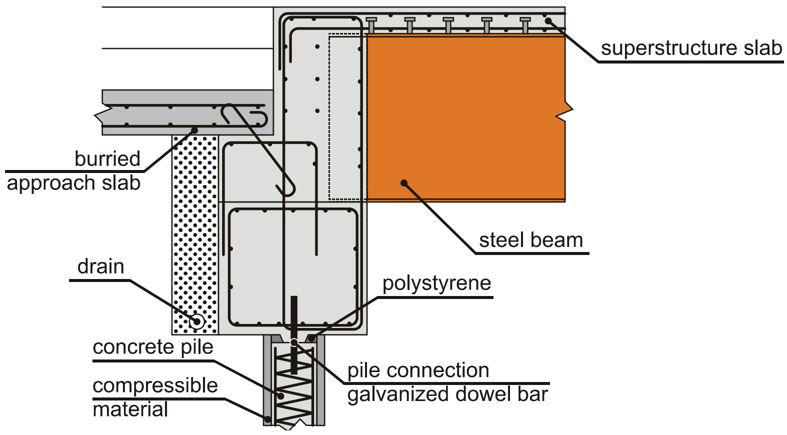


Figure 9-1: Abutment with hinged-piles (acc. to (Connal, 2004))

The pin connection was made of galvanized dowel bars, which were anchored in both the concrete pile and the pile cap. Polystyrene sheets were used as joint filling in order to avoid crushing of the concrete when the pile cap is rotating due to the applied moments. To make sure that the lateral forces are not getting to high in the top of the concrete piles, the upper 2 m were wrapped with 50 mm thick compressible material (e.g. foam).

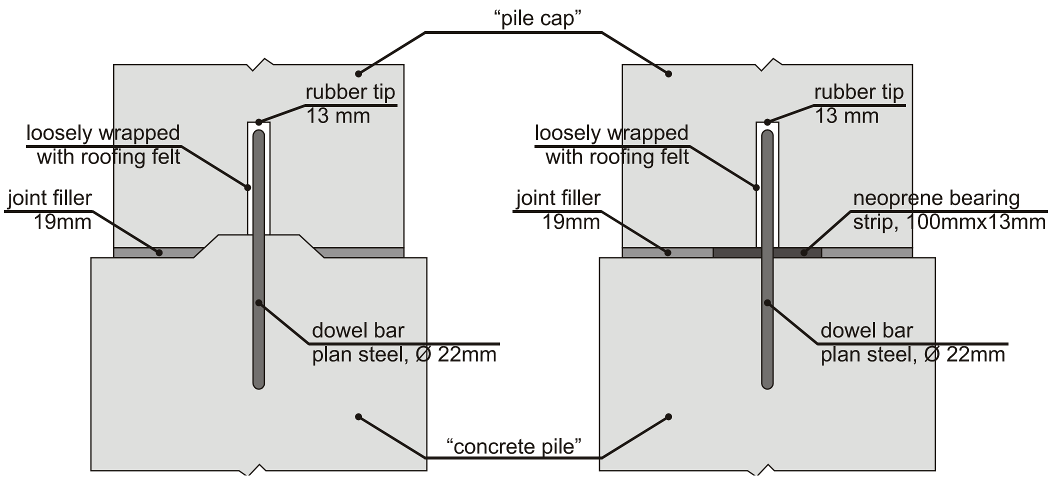


Figure 9-2: VDOT original hinge (left) / modified hinge (right)

A similar type of hinge has been developed by Virginia Department of Transportation (VDOT), based on a shear key along the joint as shown in Figure 9-2 (left). Tests performed by Arsoy showed, however, that the hinge did not work as a hinge. The abutment and the pile cap rotated as one singular unit until the shear key failed. The bond between the upper and lower part was almost as strong as if they would have been cast together. Therefore the hinge was modified as shown in Figure 9-2 (right).

The modified hinge is more flexible to rotations and consists of strips of neoprene along both sides of the line of dowels. The rest of the joint is filled with some joint filler, for example sponge rubber. The vertical forces are transferred from the upper part of the abutment through the neoprene and down into the pile cap, the dowels transfer the shear forces.

9.2.2 Steel piles

<u>Fully integral abutment bridges</u>

In the past, some states in the USA preferred a welded connection between piles and girders, see Figure 9-3. However, the major disadvantage of this type of connection is that the piles have to be driven very close to their planned position, as the girders shall be welded on top of them. This means that piles often must be driven within a tolerance of 2-3 cm, and this can be hard to achieve in difficult pile driving condition (Conboy, et al., 2005) (Yannotti, et al., 2005).

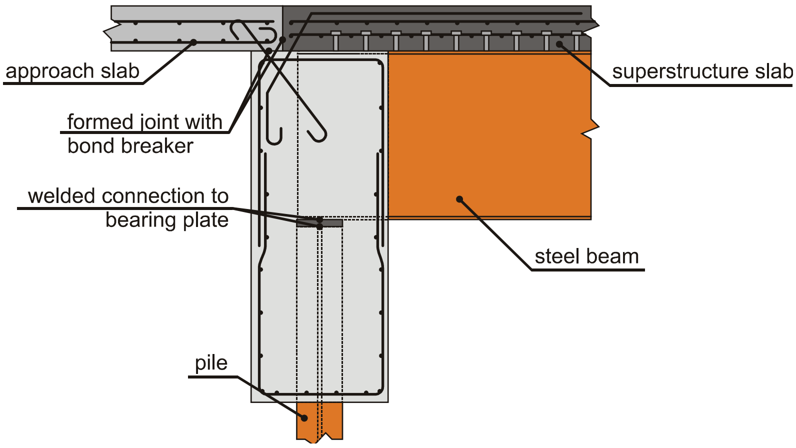


Figure 9-3: Welded connections between piles and girders

Therefore nowadays another way of constructing a rigid connection between piles and girders is used. Initially the driven piles are covered with a pile cap or a lower part of the abutment backwall. The girders are mounted on top of the pile cap and fixed to the abutments on levelling bolts that are anchored in the pile cap, see Figure 9-4. These levelling bolts may be replaced by exactly levelled steel pressure plates (two for each girder) which allow for a horizontal correction as well. A vertical correction may be realized by lining plates. However, a tilting of the steel beam needs to be avoided during construction in any case. The ends of the girders are later surrounded by concrete, when the top of the abutment backwall is cast.

In particular if settlement of the foundation has to be expected, a possibility of horizontal adjustment has to be provided, the height has to be controlled metrological during construction.

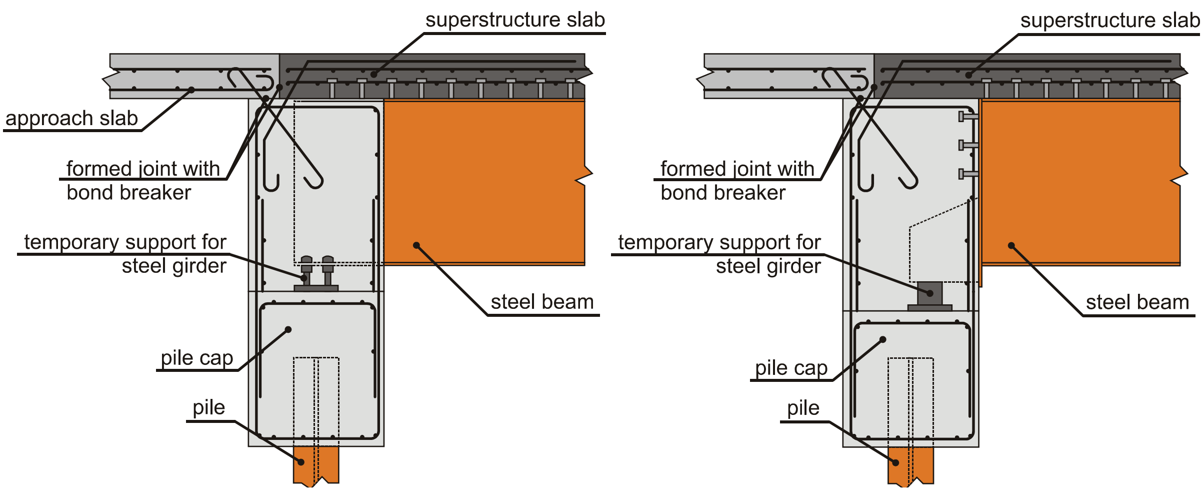


Figure 9-4: Girder mounted on levelling bolts / pressure plate on top of a pile cap

It has been proven that constructions without welds between piles and girders are easier to construct, and no differences in performance have been detected (Conboy, et al., 2005).

Semi-integral abutment bridges

The same solutions as shown in chapter 9.2.1 may be used for steel piles as well. The hinge then might be placed between the pile cap and the abutment (see Figure 9-4).

Within the scope of the INTAB project (Feldmann, et al., 2010), another type of hinged connection was developed and tested (see Figure 9-5).

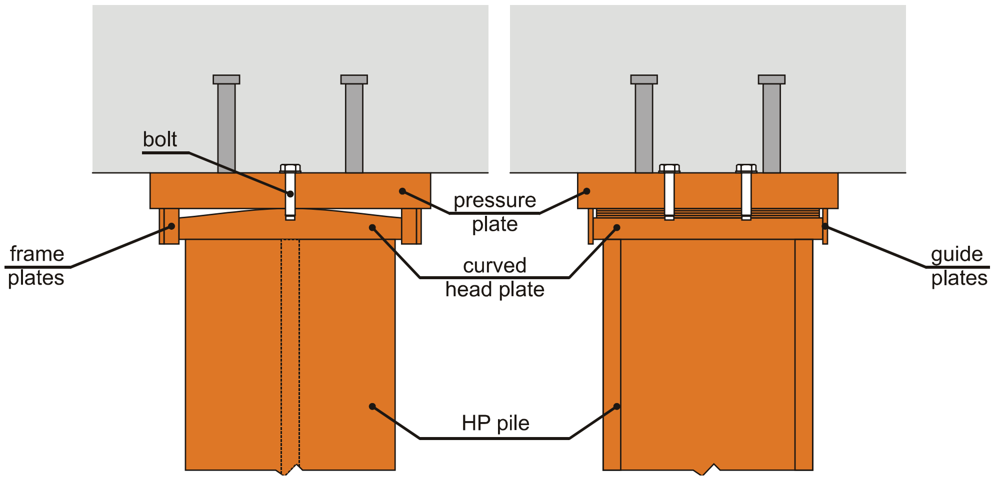


Figure 9-5: Hinged connection, type “INTAB”

A curved head plate is welded on top of the piles, enclosed by a pressure plate with welded on frame plates. The shear forces are transferred by the fame plates, the bolt just serves as assembling aid. Static as well as cyclic tests performed during the INTAB project could proof, that

- the hinge works as a perfect hinge, no moment is transferred
- no sign of fatigue failure occurred after more than 43800 cycles, which simulates the thermal movements during 120 years. The concrete close to the hinge did not crack, the steel did not show any signs of fatigue failure.
- local tensile stresses are low

9.2.3 Sheet piles

Integral abutment foundations based on sheet pile systems may particularly come into operation if a sheet pile wall needs to be constructed anyway (see Figure 9-6).

Figure 9-6: Soleuvre bridge, South Motorway, Luxembourg, (conventional bridge) (ArcelorMittal Long Commercial, 2003)

To provide a standardized solution in current practice, a new connection between sheet pile wall and superstructure has been developed and tested within the scope of the INTAB project (Feldmann, et al., 2010) (see Figure 9-7). The new system offers the following advantages:

- an existing sheet pile wall can be used as piling system for the integral abutment bridge, additional piling is not necessary
- the superstructure and the abutment can be cast at once, saving construction time and avoiding construction joints
- the degree of restraint is lowered to enable the designer to reduce the reinforcement in the corner

However, attention has to be paid regarding the following points:

- A slippage between sheet pile wall and abutment has to be avoided. Vertical studs, welded to the sheet pile wall, can eliminate this problem.
- The load carrying capacity as well as the rotational capacity of the connection highly depends on the stiffness of the sheet pile wall and the embedded length of the sheet pile wall into the abutment.
- Reinforcement needs to be placed horizontally through the sheet pile wall to increase the moment bearing capacity of the connection and to avoid cracking and spalling of the concrete, acting as a substitute for stirrups as used in pile caps.

For testing, a sheet pile system AZ 13 has been chosen, offering a higher degree of restraint than a HP-piling system but a lower degree of restraint than a concrete pile system (see Table 9.1).

Tests performed on the connection have shown that the tested abutment – sheet pile connection should be restricted to:

$M_{Ed,max}$ = 200 kNm/m

Δ_{max} = 20 mm

which is sufficient for bridges up to a length of 100m.

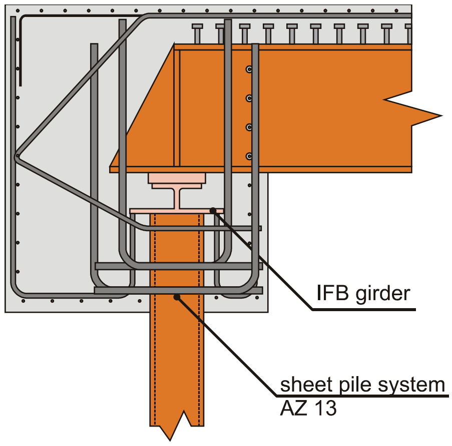


Figure 9-7: Sheet pile connection, type "INTAB"

Regarding the design, the sheet pile wall needs to be considered beyond the outer edges of the abutment (see Figure 9-8), as the adjacent wall adds significantly to the stiffness of the system. A comparison of different foundation systems is given in Table 9.1.

Table 9.1: stiffness of different foundation systems

foundation	stiffness I_{yy}	
5 x HP 305 x 95, weak axis	32,645 [cm^4]	steel pile
Sheet pile wall, AZ 13, w=5.00m	*98,500 [cm^4]*	*acc. to data sheet*
	137,900 [cm^4]	taking into consideration adjacent wall
2 x Ø 90cm	6,441,247 [cm^4]	concrete pile

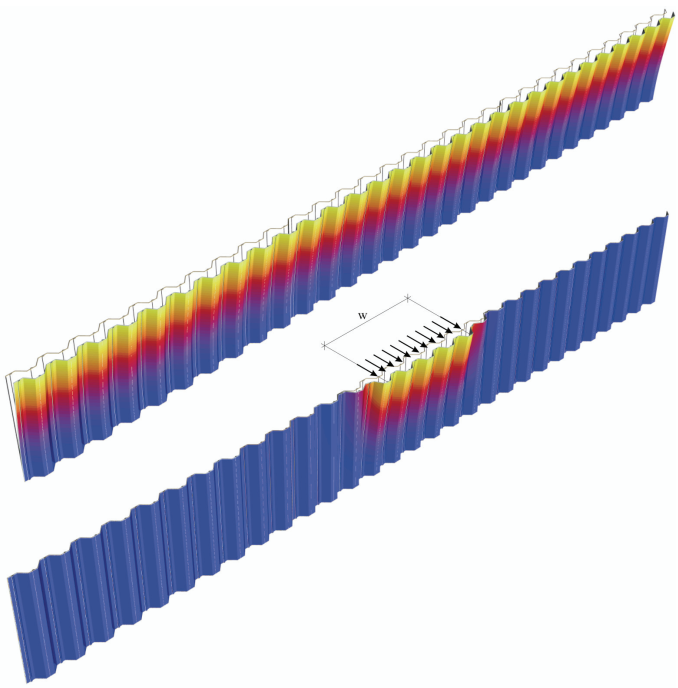


Figure 9-8: Sheet pile deformation

10 Serviceability Limit State (SLS)

The serviceability limit state combinations of actions used are:

1. Characteristic combination

$$\sum_{j\geq 1} G_{k,j} + P_k + Q_{k,1} + \sum_{i>1} \psi_{0,i} \cdot Q_{k,i} \tag{10.1}$$

2. Frequent combination

$$\sum_{j\geq 1} G_{k,j} + P + \psi_{1,1} \cdot Q_{k,1} + \sum_{i>1} \psi_{2,i} \cdot Q_{k,i} \tag{10.2}$$

3. Quasi-permanent combination

$$\sum_{j\geq 1} G_{k,j} + P + \sum_{i\geq 1} \psi_{2,i} \cdot Q_{k,i} \tag{10.3}$$

These combinations are completed by (EN 1990/A1, 2006) as well as (DIN FB 101, 2009) by the

4. Non-frequent combination

$$\sum_{j\geq 1} G_{kj} + P + \psi_{1,\inf q} \cdot Q_{k,1} + \sum_{i>1} \psi_{1i} \cdot Q_{ki} \tag{10.4}$$

whereas (DIN FB 101, 2009) writes $\psi_{1,infq}$ as ψ'_1.

The partial factors γ as well as the combination factors ψ are given in Table 10.1.

Table 10.1: Combination matrix ULS / SLS / FLS

					partial factor γ ULS STR / GEO F		STR / GEO A		EQU		SLS	FLS	comb. factor ULS / SLS				note
		LC		Type	sup	inf	sup	inf	sup	inf			ψ_0	ψ_1	ψ_2	ψ_1'	
permanent loads	perm.	1	self weight	$\gamma_{Gsup/Ginf}$	1,35	1,00	1,00	1,00	1,05 (1,35)	0,95 (1,00)	1,00	1,00	-	-	-	-	
	perm.	2	construction load	$\gamma_{Gsup/Ginf}$													
	perm.	3	backfill surcharge on foundations (vertical)	$\gamma_{Gsup/Ginf}$													
	perm.	4	earth pressure at rest (horizontal)	$\gamma_{Gsup/Ginf}$													
secondary prestress	perm.	5	shrinkage	γ_P / γ_{SH}	1,00	1,00	1,00	1,00	1,00	1,00	1,00	-	-	-	-	-	EN 1992-1-1 p. 23/24
load due to differential settlement		6	settlement of support 1	γ_{Gset}	1,20 / 1,35 (1,00)	0,00	-	-	-	-	1,00	-	-	-	-	-	γ = 1.2 (lin. el. calc.) γ = 1.35 (non-lin. el. calc.)
		7	settlement of support 2	γ_{Gset}													
		xx	settlement of support x	γ_{Gset}													
live loads	Alternative A1	8	traffic on backfill, press. on wing wall / foundation of abut. 1	γ_Q	1,50	0,00 (1,00)	(1,00) (0,00)	0,00	1,50	0,00	1,00	-	0,75	0,75	0,00 (0,20)	(0,80)	γ = 1,5: for "other" actions, e.g. soil comb. factors for TS tandem axle[a]
		9	traffic on backfill, press. on end wall / foundation of abut. 1	γ_Q													
		10	traffic on backfill, press. on wing wall / foundation of abut. 2	γ_Q													
		11	traffic on backfill, press. on end wall / foundation of abut. 2	γ_Q													
	Alternative A2	12	LM1, TS tandem axle lane 1	γ_Q	1,35 (1,50)	0,00	1,00	0,00	1,35 (1,50)	0,00	1,00						γ = 1,35: for predom. traffic actions comb. factors for TS tandem axle
		13	LM1, TS tandem axle lane 2	γ_Q													
	Alternative A3	14	braking	γ_Q													
		15	acceleration	γ_Q													
	Alternative A4	16	LM1, traffic basic load	γ_Q	1,35 (1,50)	0,00	1,00	0,00	1,35 (1,50)	0,00	1,00	-	0,40	0,40	0,00 (0,20)	(0,80)	comb. factors for evenly distr. loading
		17	LM1, UDL overload lane 1	γ_Q													
		18	LM1, UDL overload lane 2	γ_Q													
		19	LM3, fatigue	γ_Q	-	-	-	-	-	-	-	1,00	-	-	-	-	DIN EN 1991-2 D.2
temperature	Alternative A5	20	ω x expansion, top warmer[b]	γ_Q	1,50 (1,35)	0,00	1,00	0,00	1,50 (1,35)	0,00	1,00	-	0,60[d] (0,80)	0,60	0,50	(0,80)	γ = 1,5: for "other" actions, e.g. temp.
		21	ω x expansion, bottom warmer[b]	γ_Q													
		22	expansion, ω x top warmer[b]	γ_Q													
		23	expansion, ω x bottom warmer[b]	γ_Q													
		24	ω x contraction, top warmer[c]	γ_Q													
		25	ω x contraction, bottom warmer[c]	γ_Q													
		26	contraction, ω x top warmer[c]	γ_Q													
		27	contraction, ω x bottom warmer[c]	γ_Q													
	Alternative A6	28	mob. earth pressure due to expansion[b]	γ_Q	1,50	0,00	1,00	0,00	1,50	0,00	1,00	-	0,60[d] (0,80)	0,60	0,50	(0,80)	γ = 1,5: for "other" actions, e.g. soil comb. factors for temperature
		29	mob. earth pressure due to contraction[c]	γ_Q													
wind		30	wind load on structure / traffic	γ_Q	1,50	0,00	1,00	1,00	-	-	1,00	-	0,60 (0,50)	0,20 (0,50)	0,00	(0,60)	γ = 1,5: for "other" actions, e.g. wind

Cells with two factors:
factors without brackets are in accordance to EN 1990:2002/A1:2005
factors with brackets are in accordance to DIN Fachbericht 102

[a]: traffic on backfill is a combination of both load models - on the safe hand side, the resulting load cases are combined, using the com. factors for TS tandem axle

[b]: (20 or 21 or 22 or 23) and 28 always act together

[c]: (24 or 25 or 26 or 27) and 29 always act together

[d]: acc. to EN 1990:2002/A1:2005, this value may be reduced to 0 in some specific cases for ULS design EQU, STR and GEO- such a case is NOT given here!

10.1 Road bridges

10.1.1 Design stress criteria

Stresses at serviceability limit states should be determined from a linear elastic analysis, with the use of the appropriate section properties. The distribution of permanent weight and stiffness, non-uniform distribution resulting from changes in plate thickness, stiffening etc., creep and shrinkage of the concrete, erection scheme and load history, temperature effects and soil-abutment interaction should be taken into account.

The nominal stresses in all steel elements of the bridge resulting from characteristic load combinations should be limited as follows:

$$\sigma_{Ed,ser} \leq \frac{f_y}{\gamma_{M,ser}} \tag{10.5}$$

$$\tau_{Ed,ser} \leq \frac{f_y}{\sqrt{3}\ \gamma_{M,ser}} \tag{10.6}$$

$$\sqrt{\sigma_{Ed,ser}^{2} + 3\tau_{Ed,ser}^{2}} \leq \frac{f_y}{\gamma_{M,ser}} \tag{10.7}$$

In general, these design checks do not become controlling, as normally ULS governs the design.

Where relevant, allowance for the effects of shear lag in wide flanges, for secondary effects implied by deflections or for effects from transverse loads should be made.

Further the nominal stress range σ_{fre} due to the representative values of variable loads specified for the frequent load combination should be limited to 1,5 $f_y/\gamma_{M,ser}$, see (EN 1993-1-9, 2005).

The tension stresses in the reinforcement should be limited for the non-frequent load combination to:

$$\sigma_{Ed,ser} \leq \frac{0{,}8 f_{sk}}{\gamma_s} \tag{10.8}$$

In general, this design check does not become controlling, as the crack width verification governs the design.

Exceeding creep and micro cracking should be restricted by limiting the concrete compression stress for the non-frequent load combination to:

$$\sigma_{Ed,ser} \leq \frac{0{,}6 f_{ck}}{\gamma_c} \tag{10.9}$$

With a minimum of 1% shear reinforcement, embedding the compression zone, the concrete compression stress limitation may be exceeded by 10%.

10.2 Rail bridges

10.2.1 Design stress criteria

See chapter 10.1.1.

10.2.2 Deflections, deflections caused by braking

Horizontal loads caused by braking and acceleration of traffic can be introduced directly into the backfill. Therefore the active earth pressure acting behind the abutment should be taken into consideration.

The characteristic value of constrained modulus / soil Young's Modulus E_s may be increased to $E_{s,short}$ in accordance with the soil expert (see chapter 4.1).

10.2.3 Eigenfrequencies

For the design of railway bridges several codes - both European and national - are considered. In (EN 1991-2, 2003), implemented in (DIN FB 101, 2009), specific railway loads are defined - the static load models LM71 and SW/0. Dynamic effects are taken into account by application of a load factor Φ increasing the static loads. However this approach is only allowed in case that resonance effects are not expected. If resonance effects might occur – for example in case of high train velocities - a dynamic analysis has to be carried out. (EN 1991-

2, 2003) provides a flow chart for determining whether a dynamic analysis is required or not (see EN 1991-2, chapter 6.4.3, figure 6.9). One of the main input values in this procedure is the first natural frequency of the bridge.

The restraint superstructure however generally offers lower natural frequencies than its unrestraint counterpart, which often superseded the dynamic analysis.

Additional specifications about bridge design and dynamic calculations are found in a document provided by the German railway operator (DB Netz AG, 2003). Furthermore the European research project DETAILS “Design for optimal life cycle costs (LCC) of high speed railway bridges by enhanced monitoring systems” (Blasi, et al., 2011) has been carried out, aiming at an improvement of design, safety and durability of steel-concrete composite solutions for the realization of railway bridges in high-speed networks.

11 Ultimate Limit State (ULS)

All relevant ULS design checks specified in EC2 and EC3 need to be carried out for composite bridges and its members. Consequently each element and cross section of the integral bridge needs to comply with the requirements given for the concrete, reinforcement and structural steel. The used calculation methods must respect loss of resistance and ductility linked to local buckling of the structural steel members and cracking and local damage of concrete. In addition the shear studs need to be verified for the ULS design.

The following ultimate limit states are required to be verified:

- EQU Loss of static equilibrium of the structure or a structural element
- STR Failure or excessive deformation of a structure or structural element
- GEO Failure or excessive deformation of the ground where the strengths of soil or rock are significant in providing resistance
- FAT Fatigue failure of the structure or structural elements.

The effects of actions depends on the combinations of actions that can occur and(EN 1990, 2002) (EN 1990/A1, 2006) give expressions for the effects for three classes of combination of actions at the ultimate limit state:

1. Fundamental combinations (for persistent and transient situations)

The design value may be determined from either expression a) or (for STR and GEO) from the less favourable of b). The National Annex must be consulted for guidance on which method to use. (DIN FB 101, 2009) refers to expression (11.1) only.

$$\sum_{j\geq 1}\gamma_{G,j}\cdot G_{k,j}+\gamma_P\cdot P_k+\gamma_{Q,1}\cdot Q_{k,1}+\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i} \tag{11.1}$$

or

$$\begin{cases}\sum_{j\geq 1}\gamma_{G,j}\cdot G_{k,j}+\gamma_P\cdot P_k+\gamma_{Q,1}\cdot\psi_{0,1}\cdot Q_{k,1}+\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i}\\ \sum_{j\geq 1}\xi_j\gamma_{G,j}\cdot G_{k,j}+\gamma_P\cdot P_k+\gamma_{Q,1}\cdot Q_{k,1}+\sum_{i>1}\gamma_{Q,i}\cdot\psi_{0,i}\cdot Q_{k,i}\end{cases} \tag{11.2}$$

2. Combinations for accidental situations

$$\sum_{j\geq 1}\gamma_{GA,j}\cdot G_{k,j}+\gamma\cdot P+A_d+\left(\psi_{1,1}\, or\, \psi_{2,1}\right)\cdot Q_{k,1}+\sum_{i>1}\psi_{2,i}\cdot Q_{k,i} \tag{11.3}$$

Here, $\gamma_{GA,j}$ is introduced by (DIN FB 101, 2009) only. Furthermore (DIN FB 101, 2009) specifies the use of $\psi_{1,1}$.

3. Combinations for seismic situations

$$\sum_{j\geq 1}G_{k,j}+P+\gamma_1\cdot A_{Ed}+\sum_{i\geq 1}\psi_{2i}\cdot Q_{ki} \tag{11.4}$$

Here, γ_1 is introduced by (DIN FB 101, 2009) only.

The partial factors γ as well as the combination factors ψ are given in Table 10.1.

12 Fatigue Limit State (FLS)

12.1 General

The resistance of composite structures to fatigue has to be verified where the structures are subjected to repeated fluctuations of stresses. In particular for railway bridges, FLS often governs design.

The following parts of the structure need to be checked:

- headed studs
- structural steel
- concrete and reinforcement

The internal forces and moments are determined by elastic global analysis of the structure. For road bridges simplified methods according to (EN 1992-2, 2005) and (EN 1993-2, 2006), based on Fatigue Load Model 3 of (EN 1991-2, 2003) may be used for verifications of fatigue resistance. For railway bridges the characteristic values for load model 71 according to (EN 1991-2, 2003) should be used.

The design checks for FLS are the same as to be performed for conventional composite bridges with bearings and joints. However, if steel piles are used as foundation members, special care has to be taken regarding their low cycle fatigue resistance. As these effects have been investigated thoroughly within the scope of the INTAB project, they are discussed in the following shortly.

12.2 Low cycle fatigue of steel piles

Low cycle fatigue (LCF) is fatigue caused by strain cycles involving plastic deformations. As the vertical piling of integral abutments resist the lengthening and shortening of bridge superstructures responding to temperature changes, the piling of long integral bridges can be subjected to stresses which can exceed the yield stress of the pile material. These alternating high strains have to be taken into consideration regarding the fatigue design of piles as well.

Regarding the resistance of the piles against LCF, commonly a strain-based approach is used. The number of cycles until failure, N_f, for a certain strain cycle can be estimated according to Coffin-Manson's universal slope equation (Huang, et al., 2004) or the extrapolated ε-N_f curves. Therefore, the Wöhler-curves given by (EN 1993-1-9, 2005) are modified as follows:

- the Wöhler-curves are extrapolated since they are illustrated only for N_f larger than 1000
- the curves are converted from stresses into strains

Regarding the determination of strain spectra $\Delta\varepsilon$, special care has to be taken concerning

- the determination of the internal forces in the pile caused by forced displacements due to temperature effects, taking the reduced stiffness of the partly plastified pile into account
- the determination of the resulting strains, whereas non-linear effects have to be considered.

To raise awareness especially regarding the determination of correct strain spectra, these non-linear effects are described in detail for H-piles subjected to bending about the weak axis in the following chapter.

12.2.1 H-piles

In the following, an H-pile subjected to bending about the weak axis is considered, causing a fixed-end moment at the pile-abutment interface. The derivation of the formula for the maximum strain ε_{max} is based on an elastic-ideal plastic material; the contribution of the web is neglected.

Initially, the strain in outermost fibre of the flanges is determined, based on the equilibrium.

For $M<=M_{el}$:

$$\varepsilon_{outer} = \frac{1}{E} \cdot \frac{M}{W_{el,z}} = \frac{1}{E} \cdot \frac{3 \cdot M}{t_f \cdot w^2} = \frac{f_y}{E} \cdot \frac{M}{M_{el}} \tag{12.1}$$

For $M>M_{el}$:

$$\varepsilon_{outer} = \frac{f_y}{E} \cdot \frac{w}{h_{el}} = \frac{f_y}{E} \cdot \left(3 - \frac{6 \cdot M}{t_f \cdot w^2 \cdot f_y}\right)^{-\frac{1}{2}} = \frac{f_y}{E} \cdot \left(3 - 2 \cdot \frac{M}{M_{el}}\right)^{-\frac{1}{2}} \tag{12.2}$$

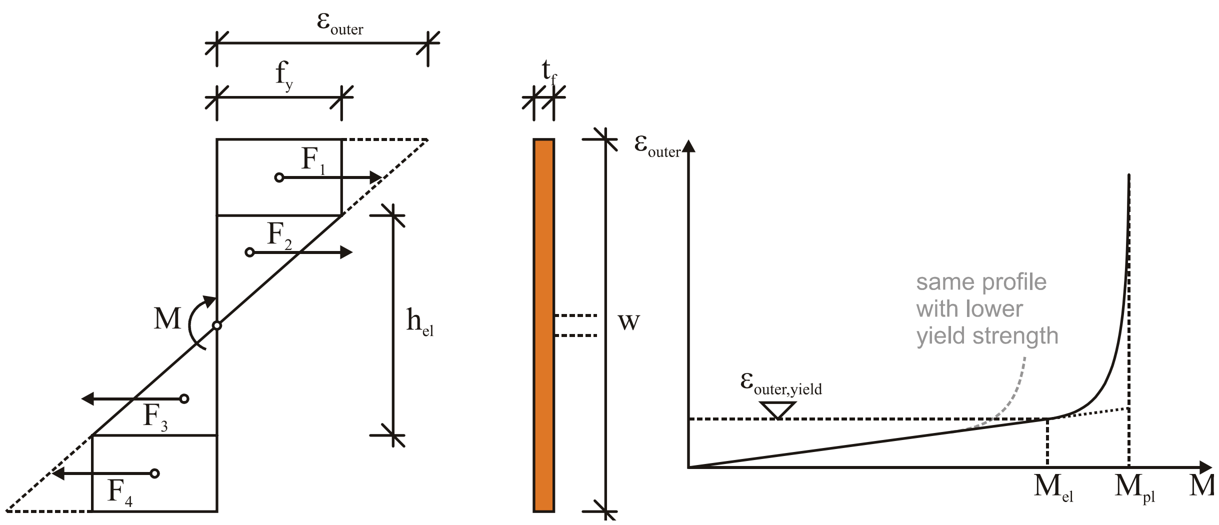


Figure 12-1: partly plastified H-pile, partial inner forces

Up to an internal force of M_{el}, the strain in the outermost region of the profile ε_{outer} increases linearly with the applied moment M. After exceeding M_{el}, ε_{outer} starts to increase disproportionate to the applied moment.

The deformation of a cantilever based on these strains is given by:

$$f = f_{el} + f_{pl} \tag{12.3}$$

$$f_{el} = \frac{1}{3} \cdot \frac{F \cdot L^3}{EI_{zz}} \tag{12.4}$$

$$f_{pl} = -\frac{1}{2} \cdot \frac{F \cdot L^3}{EI_{zz}} \cdot \left(1 - 3\frac{M_{el}{}^2}{F^2 \cdot L^2}\left(1 - \frac{2}{a}\right) - \frac{4}{a \cdot F \cdot L} \cdot M_{el}\right) \geq 0$$

$$a = \sqrt{3 - \frac{2 \cdot F \cdot L}{M_{el}}} \tag{12.5}$$

Based on these equations, the maximum outer strain ε_{outer} is plotted against a given deformation δ (see Figure 12-2).

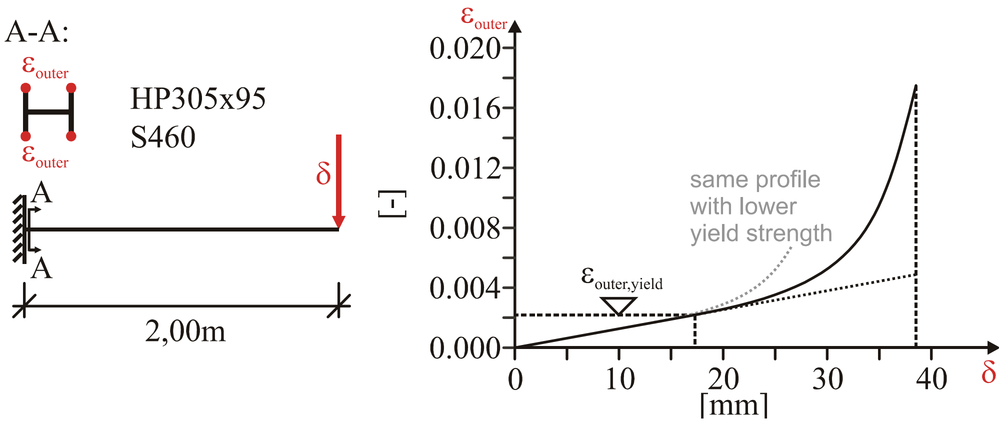


Figure 12-2: strain ε_{outer} caused by deformation δ

As long as the strains in the outer fibres of the profile are below yield strain, the strain in the outermost region of the profile ε_{outer} increases linearly with the applied deformation δ. After exceeding $\varepsilon_{outer,yield}$, ε_{outer} starts to increase disproportionate to the applied deformation. This effect has to be taken into consideration when determining strain spectra for LCF design.

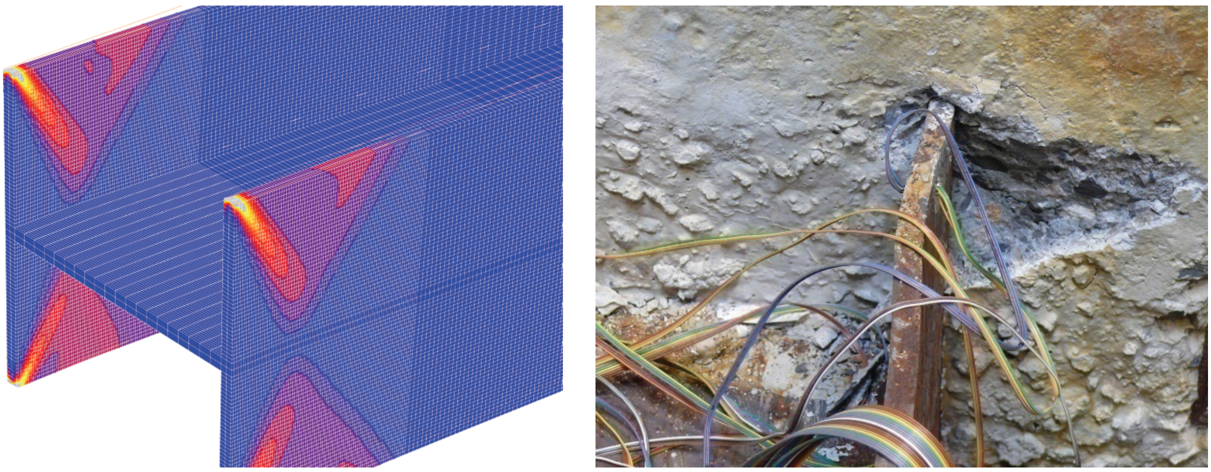


Figure 12-3: equivalent strains (FE), local buckling (LCF test)

Furthermore, two other effects have to be taken into consideration:

- higher strains close to the support have to be considered, as the strains are increased here due to local clamping effects (see Figure 12-3)
- higher strains due to buckling have to be taken into account for the fatigue calculations (see Figure 12-3); therefore an approach has been developed by (Conboy, et al., 2005) (Maruri, et al., 2005)

To take into account all these effects, a geometrical and material non-linear analysis has to be performed. On the safe hand side, an elastic / ideal-plastic material law can be applied.

12.3 Tubular piles

Tubular piles are tested at LTU, (Petursson, et al., 2010) and results are very promising to allow plastic strains in piles due to the seasonal temperature variation.

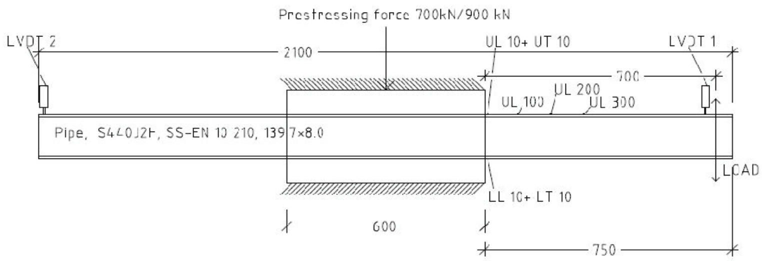


Figure 12-4: Test set-up (Petursson, et al., 2010)

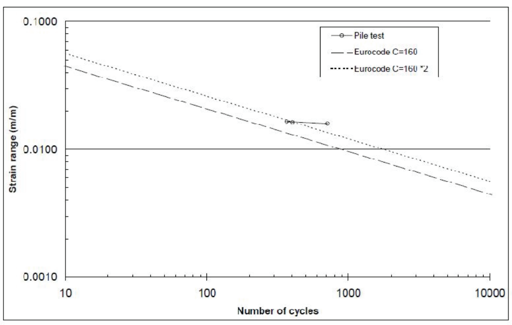
The detail category $\Delta\sigma_c$ acc. to (EN 1993-1-9, 2005) is specified for to the fatigue strength at 2 million cycles. The number of constant amplitude cycles to failure N_f at the nominal stress range is calculated using the formulae:

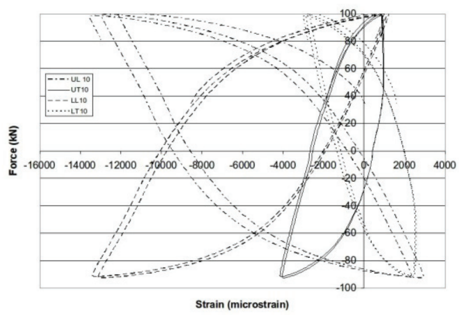
$$N_f = \frac{\Delta\sigma_c^m}{\Delta\sigma_c} \cdot 2 \cdot 10^6 \tag{12.6}$$

where m = 3 in the range of interest here. For tubular section $\Delta\sigma_c$ = 160 MPa (the longitudinal seam weld of the pile is in the neutral axis). The nominal elastic stress in Eq. (12.6) is a uni-axial nominal stress and can be converted to a nominal strain

$$\Delta\varepsilon = \frac{\Delta\sigma}{E} \tag{12.7}$$

where E = 210 GPa is the elastic modulus. After rearranging equations above the plastic strain is plotted in Figure 12-5 and compared with the test results.





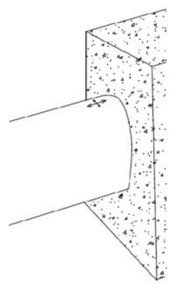


Figure 12-5: Test results compared with design values according to (EN 1993-1-9, 2005) / strain varies in compression and tension between -13500 µstrain to 2400 µstrain.

The number of cycles to failure in low cycle fatigue is often described with the Coffin-Manson relationship

$$\frac{\Delta\varepsilon_{eq}}{2} = \frac{\sigma'_f}{E} \cdot \left(2N_f\right)^b + \varepsilon'_f \cdot \left(2N_f\right)^c \tag{12.8}$$

The first part of the equation is dominant at elastic strain levels and the second at plastic strain levels. The coefficients and the exponents in the expression are material dependent and many experiments have been carried out to calibrate them for different materials.

The total strain range in the example of the Leduån bridge, designed by Ramböll and monitored by LTU, is 1353 (= 1634 – 282) μstrains corresponding to a stress range of 0.8 f_{yk}. The adoption of a strain range $\pm 2f_{yk}/E$ implies that we could have a 5 times larger strain range. Assume that the strain range due to traffic remain the same if the bridge is made longer, due to the facts that we make the girders larger and also put in more supports. This means that the strain range could be increased with 4*1353 = 5412 μstrains. As the bridge length 40 m gives a strain range of 294+136=430 μstrains, the length could be increased with 40*(5412/430) = 500 m. Other criteria than the fatigue criterion also have to be checked, and different load factors in other codes might lead to slightly different results, but the calculations based on the pile testing indicate that a bridge length up to 500 m seems possible, with regard to fatigue due to pile straining.

13 Design of details / local design

13.1 Frame Corner

The following design checks are covered here:

- local concrete compression,
- punching,
- local bending of plate.

13.1.1 Geometric relations

The dimensioning is based on a struts-and-tie model.

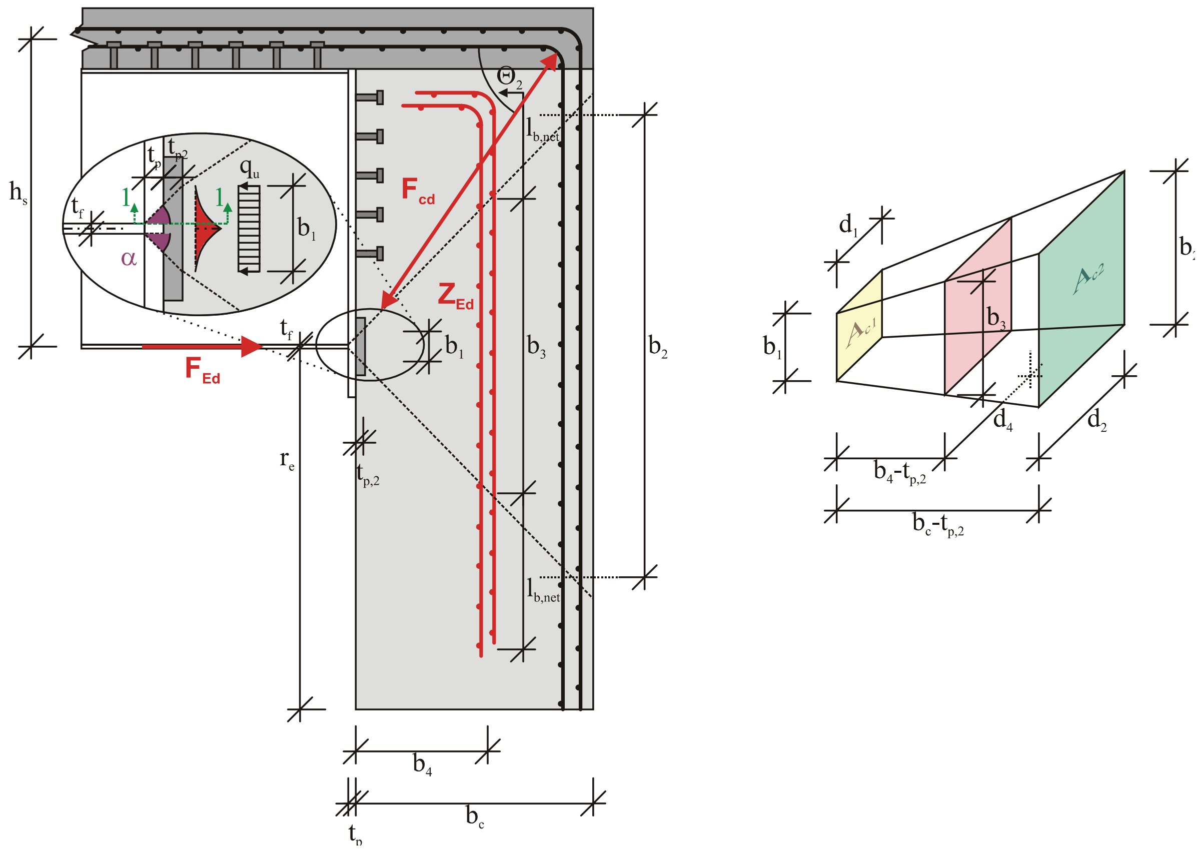


Figure 13-1: geometry, corner

$$\alpha = 45^\circ \text{ acc. to (EN 1993-1-5, 2006)} \quad (13.1)$$

$$b_1 = t_f + 2 \cdot \left(t_p + t_{p,2}\right) \cdot \tan\alpha \quad (13.2)$$

$$b_2 = \min\begin{cases} 3 \cdot b_1 \\ \left(b_c - t_{p,2}\right) + b_1 \\ 2 \cdot \left(r_e - 50mm\right) + t_f \end{cases} \quad (13.3)$$

$$b_3 = b_1 + \frac{b_4 - t_{p,2}}{b_c - t_{p,2}} \cdot \left(b_2 - b_1\right) \quad (13.4)$$

$$d_1 = b_f + 2 \cdot (t_p + t_{p,2}) \cdot \tan\alpha \tag{13.5}$$

$$d_2 = \min\begin{cases} 3 \cdot d_1 \\ (b_c - t_{p,2}) + d_1 \end{cases} \tag{13.6}$$

$$d_4 = d_1 + \frac{b_4 - t_{p,2}}{b_c - t_{p,2}} \cdot (d_2 - d_1) \tag{13.7}$$

$$A_{c1} = b_1 \cdot d_1 \tag{13.8}$$

$$A_{c2} = b_2 \cdot d_2 \tag{13.9}$$

where t_f flange thickness, main girder

b_f flange width, main girder

t_p thickness of end plate

$t_{p,2}$ thickness of pressure plate

b_c concrete width = abutment depth

b_4 position of tensile splitting reinforcement

13.1.2 Local Design – ULS

The cross-section at the end of the composite girder at the beginning of the frame corner has to be designed as 2-point cross-section. That means that only the headed plate is acting in compression and the reinforcement in the concrete plate is acting in tension.

For partially loaded areas, local crushing and transverse tension forces shall be considered acc. to (EN 1992-1-1, 2004) (chapter 6.7).

1. Local crushing

As the local compression force is determined based on the struts-and-tie model as given above, the following formula may be used:

$$F_{Ed} = \frac{M_{Ed}}{h_s} \leq \min\begin{cases} 0.85 \cdot 3.0 \cdot f_{ck}/\gamma_c \cdot A_{c1} \\ 0.85 \cdot f_{ck}/\gamma_c \cdot A_{c1} \cdot \sqrt{A_{c2}/A_{c1}} \end{cases} \tag{13.10}$$

where M_{Ed} corner moment

h_s static height

If the design check cannot be fulfilled, the width of the pressure plate may be adapted.

2. Transverse tension

$$Z_{Ed} = \frac{1}{2} \cdot F_{Ed} \cdot \left(1 - \frac{b_1}{b_4 - t_{p,2}}\right) \cdot \left(1 - \frac{d_1}{b_4 - t_{p,2}}\right) \leq A_{sZ} \cdot \frac{f_{sk}}{\gamma_s} \tag{13.11}$$

where A_{sZ} area of reinforcement

Z_{Ed} tensile force in reinforcement

The anchorage length $l_{b,net}$ needs to be sufficient.

3. Local crushing, diagonal strut

The diagonal strut with a width of $2 \cdot d_{br}/2 \cdot \sin\Theta_2$ is anchored back by the corner reinforcement.

$$F_{Ed} \cdot \frac{1}{\sin\Theta_2} = \frac{M_{Ed}}{h_s} \cdot \frac{1}{\sin\Theta_2} \leq F_{cd} = f_{cd} \cdot (b_w \cdot a_c) \tag{13.12}$$

where $a_c = d_{br} \cdot \sin\Theta_2$

d_{br} mandrel diameter

b_w beam distance

and $b_c \geq h_s$

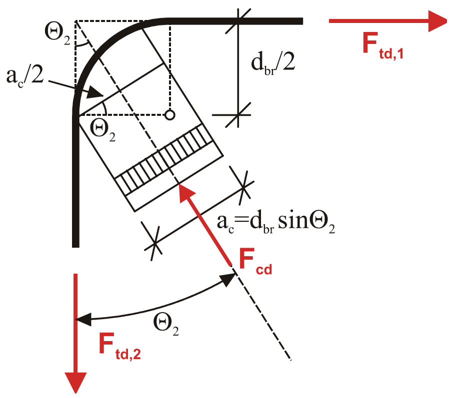


Figure 13-2: geometry, diagonal strut

4. Punching

For very slender abutments, where punching may become decisive, the relevant design check needs to be performed as well.

13.1.3 Local Design – SLS, concrete compression

The compressive stress in the concrete shall be limited in order to avoid longitudinal cracks, micro-cracks or high levels of creep, where they could result in unacceptable effects on the function of the structure (see (EN 1992-1-1, 2004) (chapter 7.2)).

$$F_{Ed} \leq 1.1 \cdot k_1 \cdot f_{ck} \cdot A_{c1} \quad \text{(characteristic combination)} \quad \text{(min. 1\% reinforcement)} \tag{13.13}$$

$$F_{Ed} \leq 1.1 \cdot k_2 \cdot f_{ck} \cdot A_{c1} \quad \text{(quasi-permanent combination)} \tag{13.14}$$

where k_1 according to National Annex of (EN 1992-1-1, 2004), recommended: 0.6

k_2 according to National Annex of (EN 1992-1-1, 2004), recommended: 0.45

13.1.4 Local Design – SLS, crack width design

As crack width design generally governs the design of the frame corner, the approach as given in (EN 1992-2, 2005) is recapitulated here briefly.

For the limitation of crack width, the general considerations of (EN 1992-1-1, 2004) (chapter 7.3.1) apply to composite structures. The limitation of crack width depends on the exposure classes according to (EN 1992-2, 2005) (chapter 4).

As a simplified and conservative alternative, crack width limitation to acceptable width can be achieved by limiting bar spacing or bar diameters as long as at least the minimum reinforcement defined in eq. (13.15) is provided (see (EN 1994-2, 2005) (chapter 7.4.2).

$$A_s = k_s \cdot k_c \cdot k \cdot f_{ct,eff} \cdot A_{ct} / \sigma_s \qquad (13.15)$$

where $f_{ct,eff}$ mean value of the tensile strength of the concrete effective at the time when cracks may first be expected to occur (see (EN 1992-1-1, 2004))

k coefficient which allows for the effect of non-uniform self-equilibrating stresses
recommended: 0.8

k_s coefficient which allows for the effect of the reduction of the normal force of the concrete slab due to initial cracking and local slip of the shear connection
recommended: 0.9

k_c coefficient which takes account of the stress distribution within the section immediately prior to cracking and is given by

$$k_c = \frac{1}{1 + h_c/(2 \cdot z_o)} + 0.3 \leq 1.0$$

h_c thickness of the concrete flange, excluding any haunch or ribs

z_o vertical distance between the centroids of the un-cracked concrete flange and the uncracked composite section, calculated using the modular ratio n_0 for short-term loading

σ_s maximum stress permitted in the reinforcement immediately after cracking. This may be taken as its characteristic yield strength f_{sk}. A lower value, depending on the bar size, may however be needed to satisfy the required crack width limits. This value is given in (EN 1994-2, 2005) (table 7.1)

A_{ct} area of the tensile zone (caused by direct loading and primary ef fects of shrinkage) immediately prior to cracking of the cross section. For simplicity the area of the concrete section within the effective width may be used

Maximum bar diameter and maximum bar spacing depend on the stress σ_s in the reinforcement and the design crack width. Maximum bar diameters are given in (EN 1994-2, 2005) (table 7.1) and maximum bar spacing in (EN 1994-2, 2005) (table 7.2).

In composite beams where the concrete slab is assumed to be cracked and not pre-stressed by tendons, stresses in reinforcement increase due to the effects of tension stiffening of concrete between cracks compared with the stresses based on a composite section neglecting concrete. The tensile stress in reinforcement σ_s due to direct loading may be calculated according to (EN 1994-2, 2005) (chapter 7.4.3). Where bonded tendons are used, design should follow (EN 1992-1-1, 2004) (chapter 7.3), where σ_s should be determined taking into account tension stiffening effects.

13.1.5 Local design – steel plate bending

The end plate has to be checked for local steel plate bending. Therefore a stress verification against local bending of the steel plate is performed, whereas the stresses in the plate have to stay elastic.

$$\sigma_{Ed} \leq \frac{f_{yp}}{\gamma_a} \tag{13.16}$$

$$\sigma_{Ed} = \frac{M_{u,1-1}}{W_{p_a}} \qquad \text{with } W_p = \frac{t_f^2 \cdot d_1}{6}$$

$$M_{u,1-1} = M_{u,\max} - F_{Sd} \cdot \frac{t_f}{8}$$

$$M_{u,\max} = q_u \cdot \frac{b_1^2}{2} \qquad \text{with } q_u = \frac{F_{Ed}}{b_1}$$

$$\Rightarrow \qquad \sigma_{Ed} = F_{Ed} \frac{3 \cdot \left(b_1 - 1/4 \cdot t_f\right)}{t_f^2 \cdot d_1} \tag{13.17}$$

13.1.6 Local design – fatigue

The fatigue check of the detail “end plate weld” is performed according to (EN 1993-1-9, 2005) as given in eq. (13.18).

$$\frac{\gamma_{Ff} \cdot \Delta\sigma_{E,2}}{\Delta\sigma_C / \gamma_{Mf}} \leq 1.0 \tag{13.18}$$

Where the verification for fatigue is based on damage equivalent stress ranges, in general a range $\Delta\sigma_E$ should be determined from

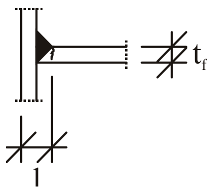
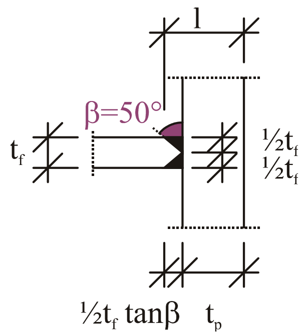
$$\Delta\sigma_E = \lambda \cdot \phi \cdot \left|\sigma_{\max,f} - \sigma_{\min,f}\right| \tag{13.19}$$

where $\sigma_{max,f}$ and $\sigma_{min,f}$ maximum and minimum stresses based on the load models as given by the relevant codes

	(see chapter 12.1)
λ	damage equivalent factor
	for road bridges: (EN 1992-2, 2005) (chapter 9.5.2)
	for railway bridges: (EN 1992-2, 2005) (chapter 9.5.3)
ϕ	damage equivalent impact factor
	for road bridges: ϕ = 1.0
	for railway bridges: (EN 1991-2, 2003) (chapter 6.4.5),
	see chapter 10.2.3

The detail category is given by (EN 1993-1-9, 2005) (table 8.5), see Table 13.1. The partial factors are given by EN 1993-1-9, Table 3.1 (γ_{Mf}).

Table 13.1: detail categories

Detail category		$l = 1/2 \cdot t_f \cdot \tan\beta + t_p$		
	80	$l \leq 50mm$	all t	
	71	$50mm < l \leq 80mm$	all t	
	63	$80mm < l \leq 100mm$	all t	
	56	$100mm < l \leq 120mm$	all t	

13.1.7 Stability

The following stability checks need to be performed for composite beams:

- resistance to lateral-torsional buckling (EN 1994-2, 2005) (chapter 6.4)
- resistance to shear buckling and in-plane forces applied to webs (EN 1994-2, 2005) (chapter 6.2.2)
- resistance to buckling of flanges (EN 1993-1-5, 2006) (chapter 4)

All steel flanges in compression should be checked for lateral stability according to (EN 1993-1-1, 2005). However, a steel flange that is attached to a concrete or composite slab by shear connection properly in accordance with (EN 1994-2, 2005) may be assumed to be laterally stable, provided that lateral instability of the concrete slab is prevented.

The shear buckling resistance $V_{b,Rd}$ of an uncased steel web should be determined in accordance with (EN 1993-1-5, 2006) (chapter 5). No account should be taken of a contribution from the concrete slab, unless a more precise method than the one of (EN 1993-1-5, 2006) (chapter 5) is used and unless the shear connection is designed for the relevant vertical force.

Plate buckling effects due to direct stresses at the ultimate limit state in flat compression elements (flanges) need to be considered where these elements are under compression. This is the case for the lower flange of the girders at the support as well as close to middle supports (hogging moment).

Furthermore the construction stages need to be taken into consideration as well. Steel beams before the hardening of concrete should be verified according to (EN 1993-1-1, 2005) and (EN 1993-2, 2006).

13.2 Connection details

Integral abutments which are fixed rigidly with a composite girder implement two construction stages. The reinforcement in the retaining wall has to be guided due to this construction stages (see chapter 6.3, “Construction stages”):

- placing of the girders;
- casting of the frame corner;
- casting of the slab (structural system → frame).

An example is given in the following figures.

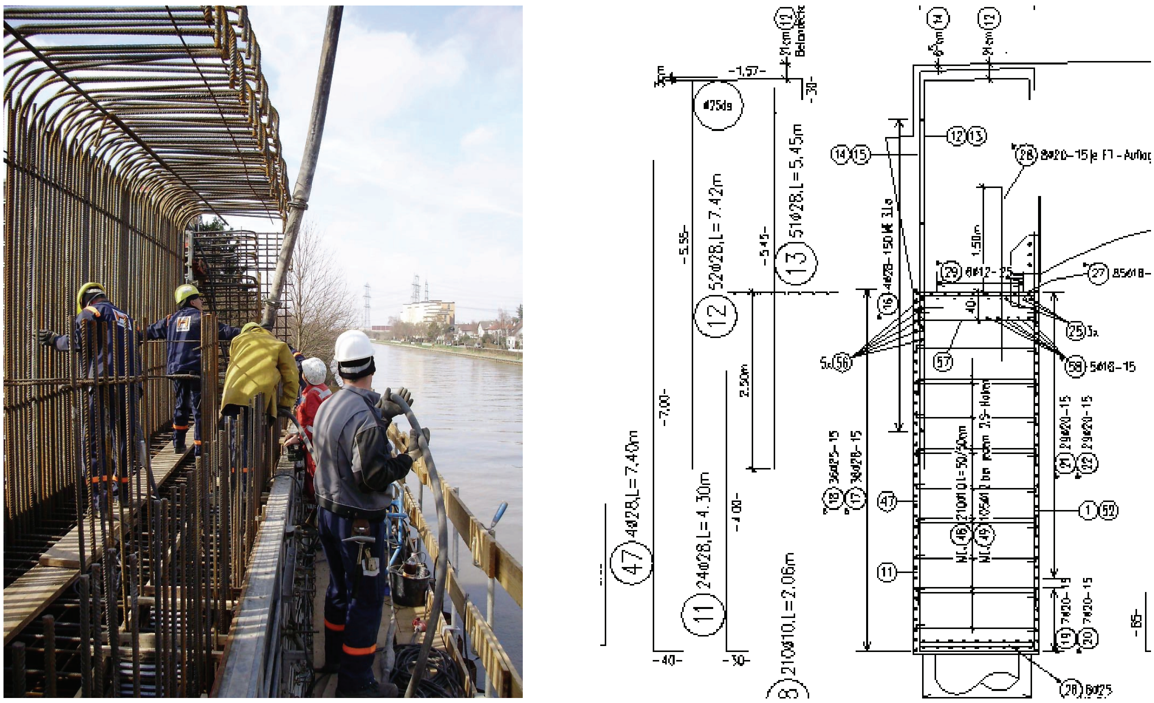


Figure 13-3: placement of reinforcement

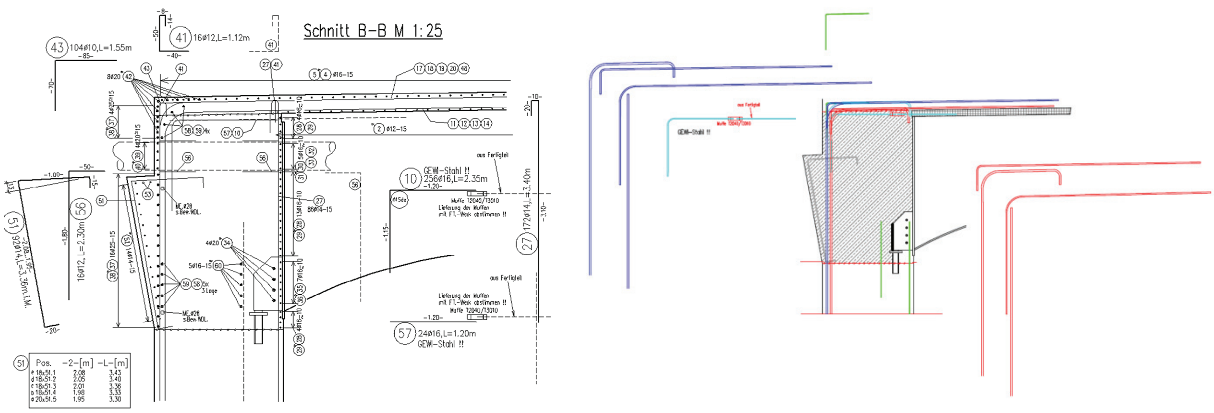


Figure 13-4: configuration of corner reinforcement

13.3 Approach slab

Approach slabs are installed to make the approach onto the bridge smooth and comfortable. They are tied to the bridge and function as ramps from rigidly supported bridge abutments to consolidating approach embankments and thereby serve to help retain smoother riding surfaces and reduce vehicular impacts. The main objective is to span over the disturbed ground between a bridge superstructure and the road pavement in order to accommodate expected elevation differences between these two elements and provide satisfactory ride comfort. However their performances are limited. Only a certain difference can be accommodated and eventually failure of the slab and the approach system in general can occur.

To avoid local settlement at the junction between approach slab and pavement caused by high stresses / pressures concentration, a sleeper plate may be located at the end of the actual approach slab.

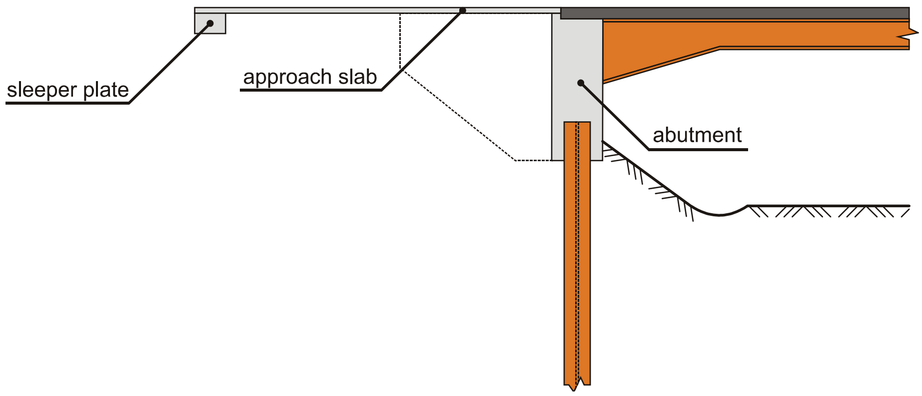


Figure 13-5: bridge approach

Further beneficial effects of approach slabs are

- they prevent vehicular traffic from consolidating traffic adjacent to abutments
- they decrease life load surcharge on abutment backfill
- they help to control bridge deck drainage
- they help to minimize erosion of the backfill

In effect, approach slabs minimize the amount of continual maintenance that is necessary adjacent to bridges constructed without them. (Burke Jr, 2009)

However, if designed inappropriately, after some time problems occur which are usually simply referred to as “bump at the end of the bridge”, worsening the riding comfort. The approach slab indeed appears to settle, bend and/or crack.

13.3.1 Failure modes

To quantify this problem, three different types of failure have been identified:

1. Too high slope; when the relative gradient (settlement δ / approach length L_{apr}) exceeds 1/200 ride discomfort appears
2. Abrupt changes in slope; it is a local equivalent of a too high slope and is mainly the result of slab bending.
3. Cracks; ground support decrease or inappropriate design leads to cracking and eventually destruction of the slab. Most problems are encountered at extremities and joints (Cai, et al., 2005)

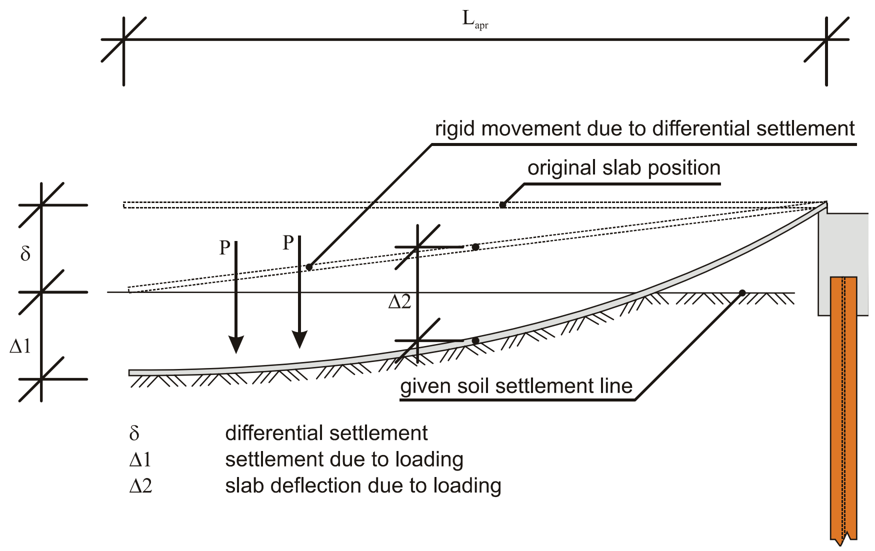


Figure 13-6: bridge approach

Causes for these failures are given in the following table.

Table 13.2: Overview of approach slab failures and their causes

	Main causes		
	Geotechnical		**Structural**
Type of Failure **(user point of view)**	**Relative settlement** • Settlement of embankment • Settlement of foundation soil • Local settlement at extremities	**Void** • Abutment movements (=compression / decompression cycles) • Erosion	**Design** **(approach slab & bridge)**
Too high average slope	when not properly supported the slab sinks as the embankment fill settles		• insufficient length • weak end supports
Abrupt change in slope	diminution and / or local loss of ground support excessive slab bending		• low rigidity
Cracks	diminution and/or local loss of ground support excessive stresses		• low resistance • stiff connections with abutment

They can be classified as geotechnical and structural causes, whereas the geotechnical causes can be reduced to relative settlement (differential settlement between bridge and adjacent road) and void (under the approach slab).

13.3.2 Relative settlement

Regarding the problem of differential settlement, it has to be distinguished between two types of settlement:

- settlement
    - settlement of the embankment fill
    - settlement of the natural soil under embankment extra load
- settlement of bridge structure

<u>Settlement of embankment fill</u>

Embankments are made of materials processed during construction. They are therefore naturally subject to time dependent post compression (White, et al., 2005).

This however can be minimized by making use of adapted materials and construction techniques. According to (White, et al., 2005), the ideal material properties for construction of an embankment are the following:

- Easily compacted, to facilitate construction
- Elastic behaviour
- No time dependent properties (consolidation)

Granular fill will therefore perform much better than cohesive soils but other parameters can also play an important role as well.

Compaction work during construction should be done very carefully.

Moisture content of the soil during construction was found to be of importance (Mekkawy, et al., 2005) since high percentages can induce sudden collapse of certain types of materials.

Use of geosynthetic materials is a possible alternative that also has great advantages regarding the drainage capacity of the system (Mekkawy, et al., 2005) (Horvath, 2002).

<u>Settlement of the natural soil under embankment extra load</u>

Under extra load from both traffic and embankment weight it is likely that compressible natural soil like clays or silts will consolidate. This seems to be one of the main causes for approaches failure (White, et al., 2005). Different techniques are available to reinforce the soil and limit the effects of this phenomenon. It is also of interest to use lightweight materials for the embankment.

13.3.3 Void

In addition to the settlement, the soil can locally be removed under approach slabs, leaving a void or gap. The decrease or loss of the soil support has serious consequences for the slab whose bearing capacity diminishes eventually leading to excessive bending and cracks.

This void may be caused by

- erosion of backfill caused by inappropriate drainage and water management and / or faulty joints (Mekkawy, et al., 2005)
- cyclic compression/decompression due to abutment movements under thermal stresses inevitably leads to subsidence of the backfill near the abutment (Horvath, 2002)

Erosion of backfill

Two types of erosion can affect performances:

- surface erosion
- “internal” erosion of soil under approaches

Surface erosion must be limited by judicious water management where runoff is directed away from embankment and joints. Therefore surface water collected on the bridge needs to be conducted away from the embankment in an appropriate way. Here a rain gutter embedded into the pavement on the abutment might be used. Otherwise it needs to be guaranteed that no water is able to infiltrate the joint between the approach slab and the abutment.

“Internal” erosion is function of drainage. A model of bridge abutment drainage was developed at Iowa University to study different materials and drainage systems (total of 13 tests). Three systems showed particularly interesting performances:

- geocomposite drain + backfill reinforcement +moisture content above bulking
- tire chips behind bridge abutment (best flow but difficult construction)
- porous backfill (relatively limited flow but very good stability though and very simple use)

A similar kind of backfill and drainage system is given by (RiZ-ING, 2007) (WAS7) as shown in Figure 13-7. The sketch has been modified to comply with an integral abutment bridge.

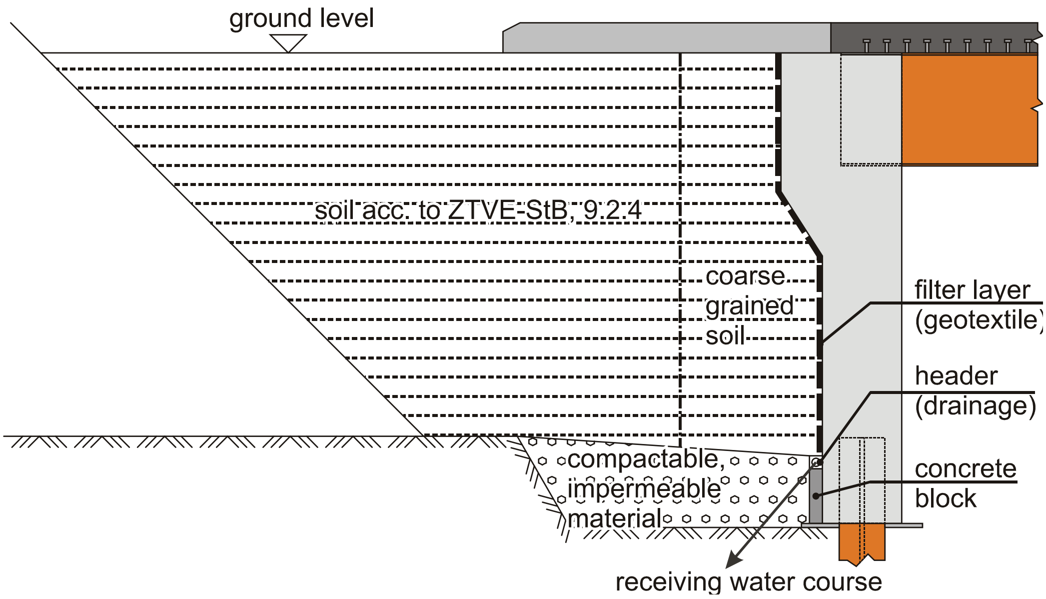


Figure 13-7: drainage of backfill acc. to (RiZ-ING, 2007), WAS7

Cyclic compression / decompression

To reduce the problems caused by cyclic compression / decompression, two conditions must be met:

- avoid the inward movement of fill
- limit the pressure resulting from the outward movement

The inward movement of the soil can be avoided by the construction of a self-stable embankment. This can be practically achieved by geosynthetic reinforcement or the use of geofoam which, through its light weight, also presents an advantage in diminishing load on natural soil.

An inclusion of compressible material between abutment and embankment can limit the pressure resulting from abutment outward movement. This inclusion can also be used to improve the drainage system.

Therefore the approach slab needs to be designed in an appropriate way to accommodate a certain decrease or loss of the supporting soil underneath. An inappropriate design might have serious consequences, leading to excessive bending, stresses and cracks in the approach slab.

13.3.4 Cracks

Studies have been carried out to find the relations between the soil settlement and the bearing capacities of the approach slabs, in order to improve the design of the slabs.

It was found in a study in New Jersey (Nassif, et al., 2002) that the approach slabs were cracking on all lanes because of weak ground support. The thickness of the plate was determined to be the most important parameter influencing the slab resistance. But for economic reasons feasible thickness is limited and two new designs with an embedded and deep beam were presented as an alternative solution.

13.3.5 Approach slab details

A study from Louisiana (Cai, et al., 2005) pointed out the fact that higher flexural rigidity is required for longer slabs without much ground support. For this purpose ribbed slabs were found to be an interesting solution.

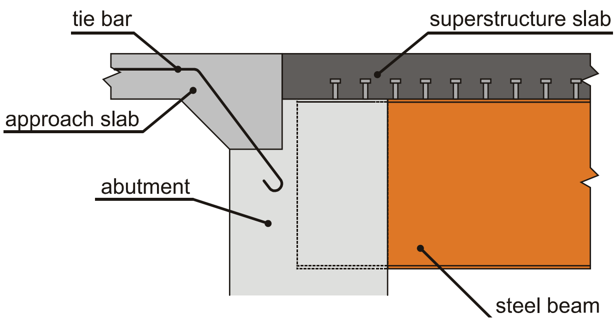


Figure 13-8: design propositions from Louisiana (Cai, et al., 2005)

The connection to the abutment was also considered. To allow for bending and to avoid crushing of the concrete at the joints, hinged or loose connections should be preferred. The authors concluded that the latter cannot be used with integral bridges because of the necessity to transfer lateral movements of the abutment, so doweled connections as shown in Figure 13-8 are favoured.

Especially in Germany, transitions without any joints are common for bridges with small and medium spans (<44m), where the asphalt has to accommodate all deformations.

Therefore the deformations need to comply with the following boundary conditions (Berger, et al., 2003):

- horizontal expansion < 25mm
- horizontal contraction <12.5mm
- vertical differential movement <5mm

Approach slabs are installed if the total deformation exceeds 20mm.

For elongations < 10mm, no special measures have to be provided according to (Berger, et al., 2003) (see Figure 13-9). The asphalt surface layer is notched, the joint is grouted.

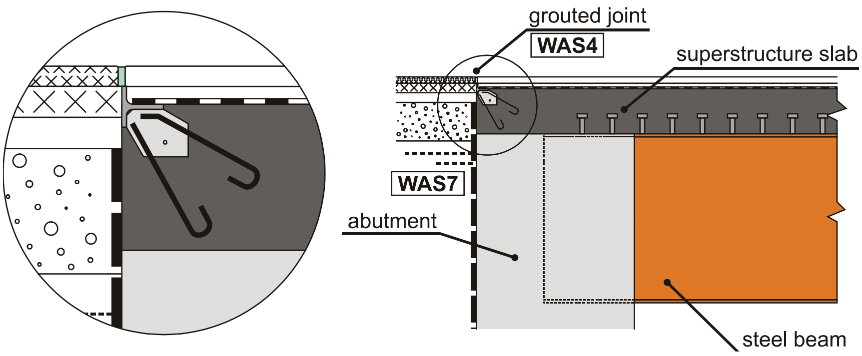


Figure 13-9: design propositions acc. to (RiZ-ING, 2007), WAS4

For elongations < 20mm, an asphalt joint according to (ZTV-ING, 2003) has to be placed between superstructure and adjacent roadway (see Figure 13-10). The frost-proof footing beam should have at least a width of 0.80m (Berger, et al., 2003).

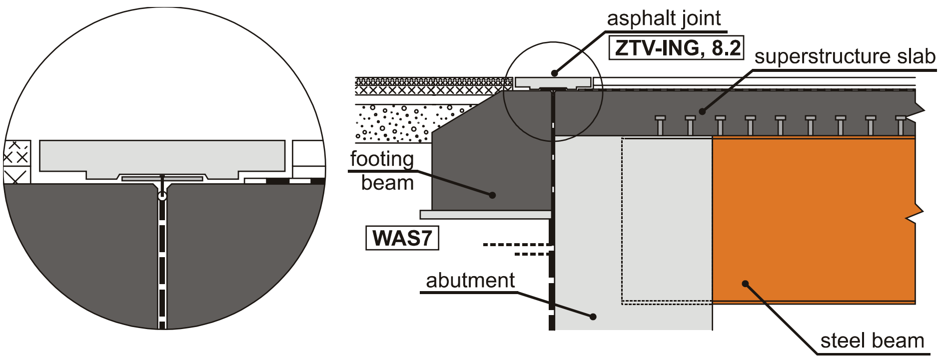


Figure 13-10: asphalt joint acc. to (ZTV-ING, 2003), ZTV-ING 8.2

For elongations > 20mm but smaller than 25mm, an asphalt joint according to (ZTV-ING, 2003) is combined with an approach slab (see Figure 13-11). The length of the slab is calculated according to eq. (13.20), the height is specified with h_{slab}=50 cm (Berger, et al., 2003).

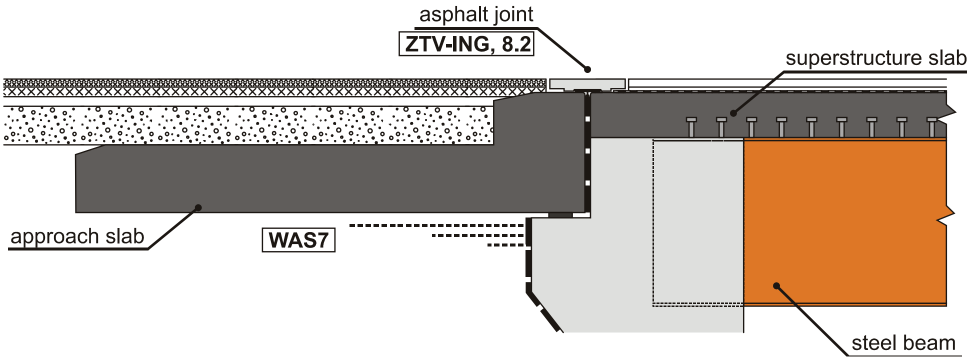


Figure 13-11: approach slab with asphalt joint acc. to (ZTV-ING, 2003), ZTV-ING 8.2

For elongations up to 65mm, the approach slab has to be connected to the superstructure by a watertight expansion joint acc. to (RiZ-ING, 2007) (see Figure 13-12). The length of the slab is calculated according to eq. (13.20), the height is specified with h_{slab}=50 cm (Berger, et al., 2003).

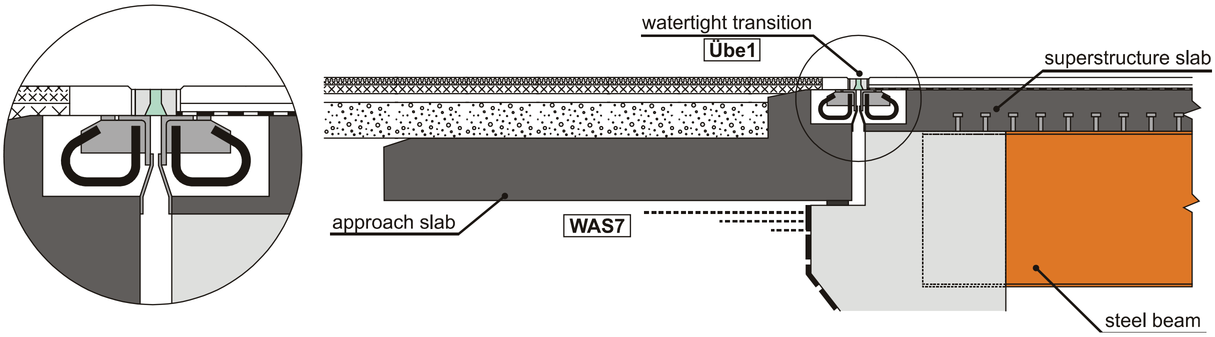


Figure 13-12: approach slab with joint acc. to (RiZ-ING, 2007), Übe1

The length of the approach slab is calculated as follows (Berger, et al., 2003):

$$l_{approachslab} \geq h_w + l_{bearing} \geq 3.60m \tag{13.20}$$

where h_w = height of settlement-effective backfill

$\approx 1.0 \cdot h_{abutment}$ for displaceable abutments (deep foundation)

$\approx 0.6 \cdot h_{abutment}$ for nearly undisplaceable (shallow foundation)

$l_{bearing}$ = required bearing length of approach slab, $\approx 0.2 \cdot h_w$

14 Precambering

According to (EN 1990/A1, 2006), the load combinations to be taken into account for the determination of precamber need to be defined for each individual project. For example, DIN FB for composite structures (DIN FB 104, 2009) recommends the permanent load combination.

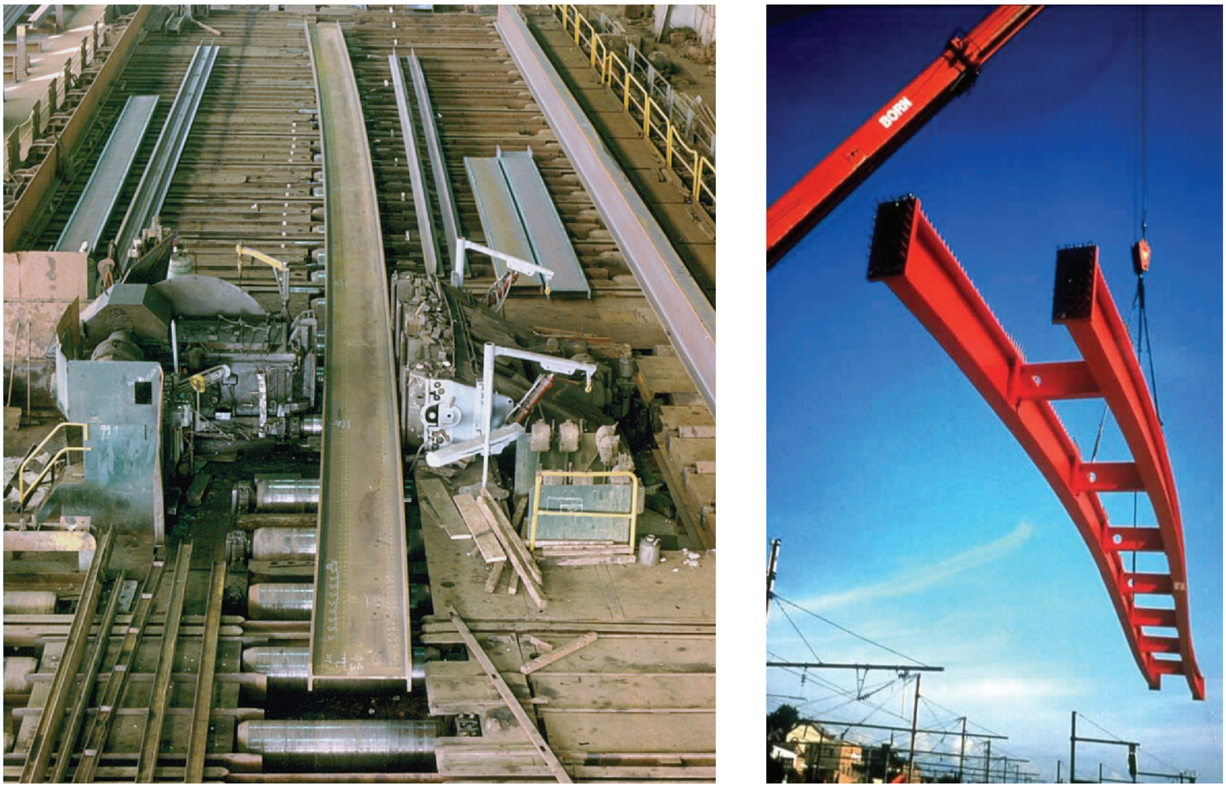


Figure 14-1: precambering of steel girders, end product (Hechler, 2010)

It is important to note that for the definition of a camber the realistic deformation of the construction is crucial. This covers the choice of the Young's modulus as well as the consideration of the cracked concrete (tension stiffening of concrete).

Annex 1: Determination of coefficients of earth pressure

K_a, K_p according to EN 1997-1:2009:

Values of the earth pressure coefficients may be taken from (EN 1997-1, 2005), Annex C.1, figures C.1.1 to C.1.4 for K_a and from figures C.2.1 to C.2.4 for K_p. They are approximately on the safe side.

Alternatively, the numerical procedure described in EN (EN 1997-1, 2005), Annex C.2 may be used. It is summarized in the following.

To determine the values of earth pressure coefficients for the non-cohesive backfill (chapter 8.3), only K_γ needs to be determined ($K_p = K_\gamma$ / $K_a = K_\gamma$).

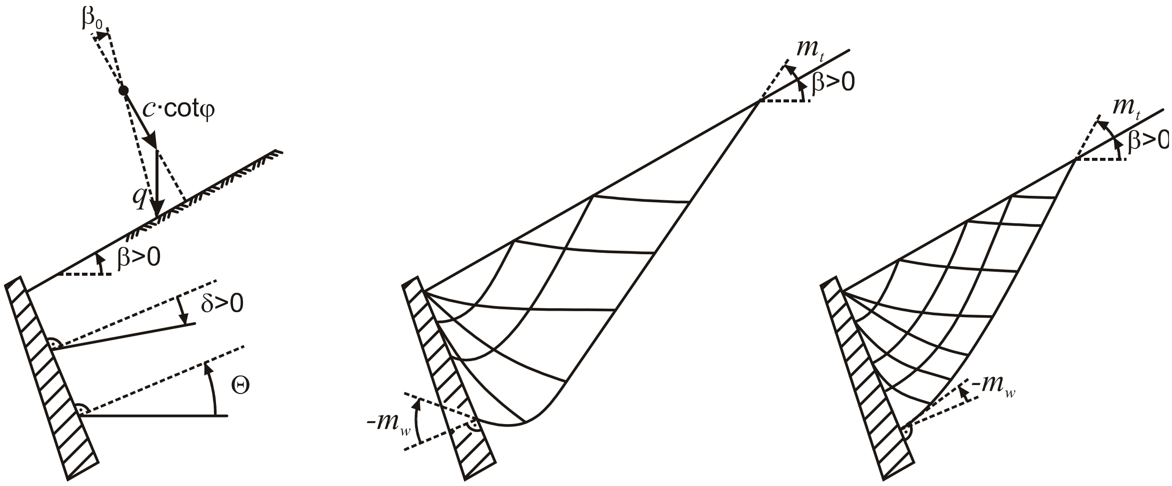


Figure 0-1: Definitions concerning wall and backfill inclination, surcharges and slipline geometry

Determination of K_p

- φ, c, δ, a inserted as positive values, $\delta = \delta_p$
- β_0 is defined from the vectorial sum of q (not necessarily vertical) and $c\ cot\varphi$ (normal to surface)
- for c = 0 and surface load q = vertical or zero, $\beta_0 = \beta$

Determination of K_a

- φ, c, δ, a inserted as negative values, $\delta = \delta_a$
- $\beta_0 = \beta$

The following symbols are used:

a	adhesion between soil and wall
c	cohesion
δ	structure-ground interface friction angle (δ_a, δ_p) (angle of shearing resistance between ground and wall)
φ	soil friction angle

K_c	coefficient for cohesion
K_n	coefficient for normal loading on the surface
K_q	coefficient for vertical loading
K_γ	coefficient for the soil weight
q	general uniform surcharge pressure, per area unit of the actual surface (not necessarily vertical)
p	vertical uniform surcharge pressure, per area unit in a horizontal projection

β_0, β, Θ, m_w, m_t as given in Figure 0-1

δ and a must be chosen so that

$$\frac{a}{c} = -\frac{\tan\delta}{\tan\varphi} \tag{0.1}$$

φ <> 0:

1. determine m_t and m_w (in [rad])

$$\cos(2m_t + \varphi + \beta_0) = -\frac{\sin\beta_0}{\sin\varphi} \tag{0.2}$$

$$\cos(2m_w + \varphi + \delta) = \frac{\sin\delta}{\sin\varphi} \tag{0.3}$$

2. determine v (in [rad])

$$v = m_t + \beta - m_w - \Theta \geq 0 \tag{0.4}$$

If this condition is not (even approximately) fulfilled, e.g. for a smooth wall and a sufficiently sloping soil surface when β and φ have opposite signs, it may be necessary to consider using other methods. This may also be the case when irregular surface loads are considered.

3. determine K_q, K_c, K_γ based on K_n

$$K_n = \frac{1 + \sin\varphi\sin(2m_w + \varphi)}{1 - \sin\varphi\sin(2m_t + \varphi)} \cdot \exp(2v \cdot \tan\varphi) \tag{0.5}$$

$$K_q = K_n \cdot \cos^2\beta \tag{0.6}$$

$$K_c = (K_n - 1) \cdot \cot\varphi \tag{0.7}$$

$$K_\gamma = K_n \cdot \cos\beta \cdot \cos(\beta - \Theta) \tag{0.8}$$

The expression for K_γ is on the safe side. While the error is unimportant for active pressures it may be considerable for passive pressures with positive values of β.

For comparative reasons, K_q can be written as:

$$K_q = K_\gamma \cdot \frac{\cos\beta}{\cos(\beta - \Theta)} \tag{0.9}$$

<u>$\varphi = 0$:</u>

1. determine m_t and m_w (in [rad])

$$\cos(2m_t) = -\frac{p}{c} \cdot \sin\beta \cdot \cos\beta \tag{0.10}$$

$$\cos(2m_w) = \frac{a}{c} \tag{0.11}$$

2. determine v (in [rad])

$$v = m_t + \beta - m_w - \Theta \geq 0 \tag{0.12}$$

If this condition is not (even approximately) fulfilled, e.g. for a smooth wall and a sufficiently sloping soil surface when β and φ have opposite signs, it may be necessary to consider using other methods. This may also be the case when irregular surface loads are considered.

3. determine K_q, K_c, K_γ

$$K_q = \cos^2\beta \tag{0.13}$$

$$K_c = 2v + \sin(2m_t) + \sin(2m_w) \tag{0.14}$$

$$K_\gamma = \cos\Theta + \frac{\sin\beta \cdot \cos m_w}{\sin m_t} \tag{0.15}$$

<u>K_0 according to DIN 4085:2007 / BRO 2004:</u>

A procedure for the determination of K_0 is not given by (EN 1997-1, 2005). Therefore national rules have to be applied.

<u>DIN 4085:2007:</u>

The determination of K_0 is described in detail in (DIN 4085, 2007), chapter 6.4.

It is summarized in the following.

For harmonization with (EN 1997-1, 2005), the following symbols have been changed:

α	(DIN)	→	Θ	(EN)	
K_{0gh}	(DIN)	→	$K_{0,\gamma}$	(EN)	(for soil behind abutment, chapter 8.3)
K_{0ph}	(DIN)	→	$K_{0,q}$	(EN)	(for embankment loading, chapter 8.2)

The following symbols are used:

δ_0 structure-ground interface friction angle (angle of shearing resistance between ground and wall)

φ soil friction angle

$K_{0,\gamma}$ coefficient for soil weight

$K_{0,p}$ coefficient for vertical loading

β, Θ as given in Figure 0-1

β>0:

$\delta_0 \leq \beta - \Theta$ has to be guaranteed

for $\Theta = 0$ and $\delta_0 = \beta = \varphi$, $K_{0,\gamma} = \cos^2 \varphi$, else proceed with 1.

β<0:

set $\delta_0 = -\Theta$

1. check, if simplified equation can be applied

if $\Theta = \beta = \delta_0 = 0$

$$K_{0,\gamma} = 1 - \sin\varphi \tag{0.16}$$

else, proceed with 2.

2. determine K_1

$$K_1 = \frac{\sin\varphi - \sin^2\varphi}{\sin\varphi - \sin^2\beta} \cdot \cos^2\beta \tag{0.17}$$

3. determine tan α_1

$$\tan\alpha_1 = \sqrt{\frac{1}{1/K_1 + \tan^2\beta}} \tag{0.18}$$

4. determine f

$$f = 1 - \left|\tan\Theta \cdot \tan\beta\right| \tag{0.19}$$

5. determine $K_{0,\gamma}$ (coefficient for soil weight)

$$K_{0,\gamma} = K_1 \cdot f \cdot \frac{1 + \tan\alpha_1 \cdot \tan\beta}{1 + \tan\alpha_1 \cdot \tan\delta_0} \tag{0.20}$$

6. determine $K_{0,q}$ (coefficient for vertical loading /traffic on embankment)

$$K_{0,q} = K_{0,\gamma} \cdot \frac{\cos\Theta \cdot \cos\beta}{\cos(\Theta - \beta)} \tag{0.21}$$

According to (DIN 4085, 2007), the ratio $K_{0,q}/K_{0,\gamma}$ for earth pressure at rest is the same as the one for active earth pressure $K_{a,q}/K_{a,\gamma}$.

In (EN 1997-1, 2005), the ratio $K_{0,q}/K_{0,\gamma}$ is not given. However, the ratio $K_{a,q}/K_{a,\gamma}$ which actually is given (see eq. (0.9)) differs to the one given in (DIN 4085, 2007) (see eq. (0.21)). For reasons of code consistency, the ratio $K_{0,q}/K_{0,\gamma}$ as proposed by (DIN 4085, 2007) should be used here.

BRO 2004:

Values for K_0 are given in table 21-1, (Bro 2004, 2004) for different soils (see Table 0.1).

Table 0.1: Earth pressure coefficients for different soils acc. to (Bro 2004, 2004)

Soil	Dead weight [kN/m^3]		Earth pressure coefficient		
	above ground water level	below ground water level	at rest	active	passive
			K_0	K_a	K_p
Blasted rock	18	11	0.34	0.17	5.83
Gravel	20	13	0.36	0.22	4.60
LECA	5	0	0.43	0.27	3.70
Styrofoam	1	0	0.40	0	-

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16 List of figures