ArcelorMittal Sheet Piling

ArcelorMittal

Knife Edge Support

Schneidenlagerung

Design of the transfer of vertical loads into a steel sheet pile according to the German National Technical Approval Z-15.6-235 (2021)

Deutsches Institut für Bautechnik

DIBt

Eine vom Bund und den Ländern gemeinsam getragene Anstalt des öffentlichen Rechts

Zulassungs- und Genehmigungsstelle für Bauprodukte und Bauarten

Allgemeine Bauartgenehmigung

Datum: 09.12.2021

Geschäftszeichen: I 12-1.15.6-16/21

Nummer:
Z-15.6-235

Geltungsdauer
vom: **8. Dezember 2021**
bis: **8. Dezember 2026**

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Gegenstand dieses Bescheides:
Stahlbetonholm mit Schneidenlagerung zur Einleitung von Vertikal- und Horizontalkräften in Stahlspundwandbohlen der Firma ArcelorMittal nach DIN EN 1992-1-1 mit DIN EN 1992-1-1/NA

Der oben genannte Regelungsgegenstand wird hiermit allgemein bauaufsichtlich genehmigt.
Dieser Bescheid umfasst 14 Seiten und vier Anlagen auf insgesamt 11 Anlagenseiten.
Der Gegenstand ist erstmals am 25. November 2011 zugelassen worden.

DIBt


Content

1. The Knife Edge Support

1.1. Introduction

Steel sheet piles have been used for over 100 years to build reliable and cost-effective permanent and temporary structures such as quay walls and breakwaters in ports, locks, riverbank reinforcement on rivers and canals, retaining walls on road and rail infrastructures, and so on.

Steel sheet pile structures are subjected mainly to horizontal loads induced by earth and/or water pressure. In most cases minor vertical loads are transferred to the walls, for instance as a result of the vertical component of the earth pressures or battered anchors,….

However, there are specific situations in which a sheet pile structure is designed to resist additional significant vertical loads, similarly to HP piles, to transfer the loads into the soil through friction and / or point resistance. Loads can be static or dynamic, permanent or variable, depending on the origin: cranes on quay walls, buildings when sheet piles act as a foundation, traffic for bridge abutments, etc.

A capping beam designed based on the Knife Edge Support (KES) method will yield a cost-effective solution to transmit horizontal and substantial vertical loads to the ground through a steel sheet pile. This innovative design concept also simplifies the execution since it does not require any additional welding of stirrups or shear connectors at the job-site to ensure the load transmission from the superstructure to the steel sheet pile.

Main applications where significant vertical loads may have to be transferred to steel sheet piles are

- quay walls,
- underground car parks,
- bridge abutments,
- locks.

The National Technical Approval (NTA), which is called nowadays '*Allgemeine Bauartgenehmigung*' in German, was granted by the German authorities DIBt[1] with the number Z-15.6-235. It is based on an extensive research and development programme lead by ArcelorMittal's R&D department in Luxembourg, and carried out in collaboration with the University of Darmstadt. During this project several full scale tests were performed to analyse the vertical and horizontal load transmission throughout the connection, and to compare its behaviour to a standard reinforced capping beam.

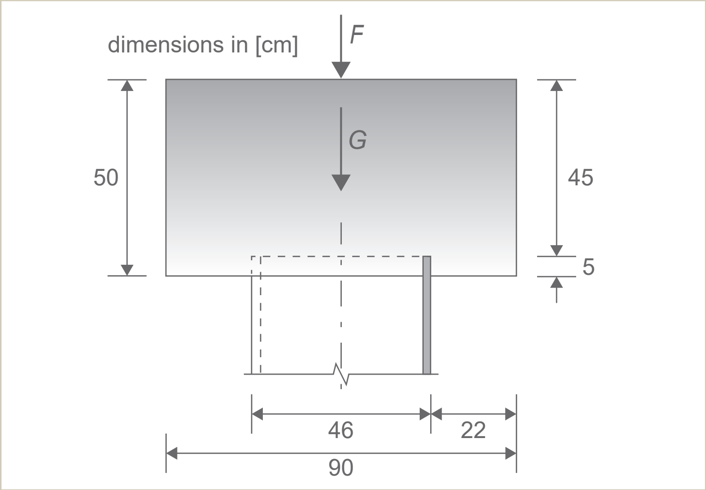


Figure 1.1. Example used for the design comparison between EN 1992 and the NTA.

As an example (see Figure 1.1.), a capping beam designed according to the German standard DIN 1045, or to the European code EN 1992, allows a maximum vertical load of 625 kN/m, while the same capping beam designed according to the NTA allows a vertical load of 1475 kN/m, which is an increase of 136%.

The KES has been tested and approved for static and 'non-static' vertical loads, as well as for static horizontal loads. The following definitions apply in the scope of this document

- static action: action that does not cause significant acceleration of the structure or structural members[2],
- non-predominantly-static action (fatigue): action defined in the German standards as '*nicht vorwiegend ruhende Last*', which refers to a non-static load that may lead to a fatigue phenomenon after a number of repeated actions (cyclic load), but which does not fall into the dynamic category of loads,
- dynamic action: action that causes significant acceleration of the structure or structural members[3] and must be designed accordingly.

Picture 1. Bridge abutments where traffic loads from the bridge are transmitted to the steel sheet piles.

[1] Stahlbetonholm mit Schneidenlagerung zur Einleitung von Vertikal- und Horizontalkräften in Stahlspundwandbohlen der Firma ArcelorMittal nach DIN EN 1992-1-1. mit DIN EN 1992-1-1/NA. Allgemeine Bauartgenehmigung Z-15.6-235. 8/12/2021. DIBt (Deutsches Institut für Bautechnik) Berlin, Germany.
[2] According to EN 1990.
[3] Based on EN 1992.

The KES has not been verified for uplifting forces, nor for external torsional moments. The reinforced concrete body should always fulfil the minimum reinforcement criteria required in the local regulations and national standards, as well as any other geometrical requirements. The rules given in the NTA are to be considered as minimum requirements to follow.

The design and construction of the reinforced concrete capping beam based on the NTA fulfils the design criteria from following European and German standards

- EN 1992-1-1: 2011-01,
- DIN EN 1992-1-1 / NA: 2013-04,
- EN 1993-5: Eurocode 3 Part 5,
- EN 10248-1: 2006-05,
- DIN 1045-1: 2008-08 (replaced by DIN EN 1992-1-1:2011-01 DIN EN 1992-1-1/NA: 2011-01, DIN EN 1992-1-1/NA:2013-04, DIN EN 1992-3: 2011-01, DIN EN 1992-3/NA:2011-01)
- DIN 1045-2: 2008-08,
- DIN 1045-3: 2012-03,
- DIN 1055-100: 2001-03 (replaced by DIN EN 1990: 2010-12, DIN EN 1990/NA: 2010-12, DIN 1055-2:2010-11).

ArcelorMittal also developed the software *VLoad®* to simplify the design according to the German NTA. *VLoad* allows the designer to calculate quickly the connection between the concrete capping beam and the sheet pile section, as well as to prepare drawings of the necessary steel reinforcement, including the geometry of the capping beam. This user-friendly software is available for free for download on ArcelorMittal's website at

https://sheetpiling.arcelormittal.com/en/download-center/software

For all further clarifications, please contact the technical department of ArcelorMittal in Luxembourg (sheetpiling@arcelormittal.com).

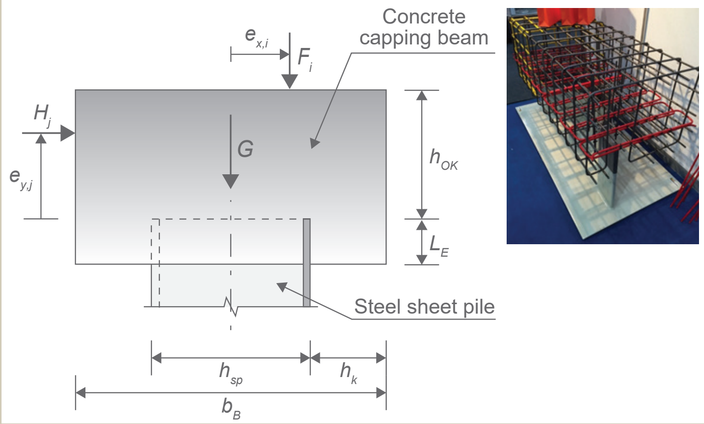


Figure 1.2. Knife Edge Support capping beam (left: sketch – right: prototype).

1.2. Scope of application

When designing a capping beam on top of a steel sheet pile wall according to the NTA, two cases should be distinguished

- **simply supported capping beam**
  (noted '**simple connection**' from this point forward).

  The sheet pile wall is slightly embedded into the concrete capping beam but does not transfer any bending moment to the sheet pile.
- **restrained capping beam**
  (noted '**fixed connection**' from this point forward).

  The sheet pile wall is sufficiently embedded into the concrete capping beam so that it is able to transfer bending moments to the sheet pile.

The NTA considers horizontal capping beams. Capping beams with a slope up to 5% in the longitudinal axis of the wall (see Figure 1.4.) can be designed with the 'fixed connection' method. However, the top and bottom surface of the capping beam in a plane perpendicular to the plane of the wall must be horizontal.

According to the NTA, concrete listed below can be used

- concrete strength classes
  (according to Table 3.1. EN 1992-1-1:2004) for design
  - minimum strength: class C 20/25 ($f_{ck} = 20$ MPa)
  - maximum strength: class C 30/37 ($f_{ck} = 30$ MPa)[4)]
- exposure classes
  - all classes specified in the European standards, except for abrasion class XM 1, XM 2 and XM 3[5)]
- mixture
  - maximum aggregate size $d_g \leq 16$ mm

In Germany, the steel rebars used for the concrete shall fulfil the criteria of steel grade B 500 B according to DIN 488-1:2009-08. In other countries, rebars may have to comply with other standards, but their properties have to be equivalent to the steel grade B 500 B.

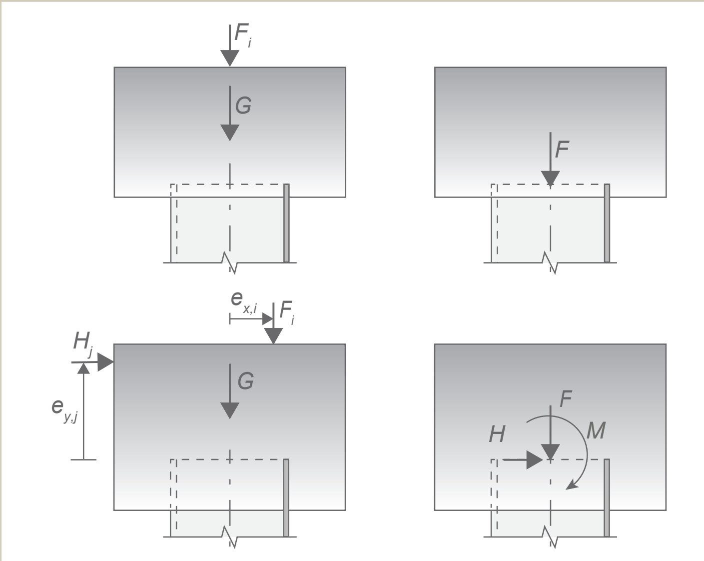


Figure 1.3. 'Simple connection' capping beam (top). 'Fixed connection' capping beam (bottom).

[4)] Classes of concrete above C 30/37 can be used for execution, but for the design $f_{ck} \leq 30$ MPa.
[5)] For more information about Exposure classes, please see Table 4.1. on EN 1992-1-1: 2004.

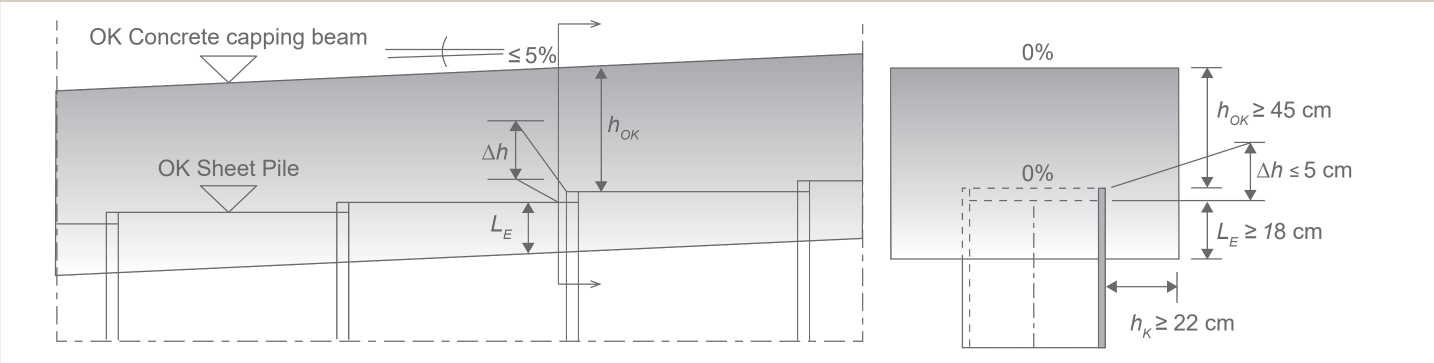


Figure 1.4. Capping beam with sloped surface restricted to the 'fixed connection' design method.

The NTA covers only Z- and U-shape steel sheet pile sections from ArcelorMittal listed in its Annex 1. Jagged walls and box-piles are not covered by the NTA. For combined walls with box-piles, a safe-sided preliminary approach could consider only the 'standard' sheet piles, but this approach may be too conservative.

Note

- The steel grade of the sheet pile does not have an influence on the KES design. However, steel sheet piles must be manufactured and delivered according to the European standard EN 10248.
- The common interlocks of double and triple U-type steel sheet piles have to be crimped or welded to prevent slippage in the interlocks.
- Classes of concrete above C 30/37 can be used for execution, but for the design $f_{ck} \leq 30$ MPa.
- The NTA is valid for capping beams that do not exceed the temperature of +60°C, with exceptional short term temperature of +80°C being acceptable.

1.3. Capping beam types

It is necessary to differentiate two types of reinforced concrete capping beams when designing a KES. Following geometrical requirements apply to both types (see Figure 1.2.).

- minimum height of the capping beam above the top of the sheet pile
  $h_{OK} \geq 45$ cm
- minimum recommended concrete cover[6)]
  $c_{min} \geq 40$ mm
  tolerance $\Delta c \leq \pm 15$ mm
- concrete lateral overhang
  $h_K \geq 22$ cm

1. **Simple connection** capping beams (Figure 1.5.) are not able to transmit bending moments. Therefore, it can only be used if the external loads transmitted to the sheet pile wall are vertical and centered on the neutral axis of the sheet pile.
   Geometrical requirements
   - minimum embedment depth of the sheet pile
     $L_E \geq 5$ cm
   - minimum height of capping beam
     $h \geq 50$ cm $(= h_{OK} + L_E)$

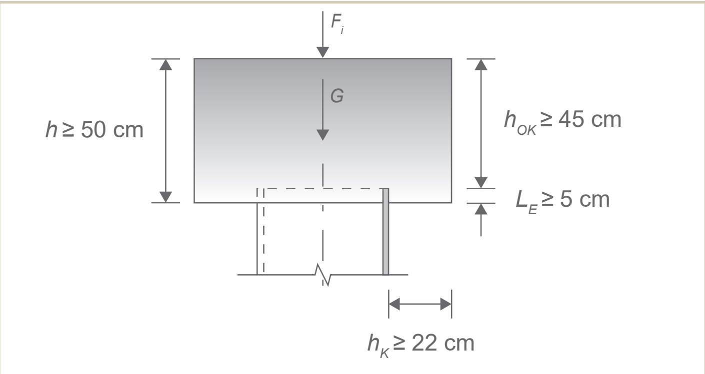


Figure 1.5. Simple connection capping beam.

2. **Fixed connection** capping beams (Figure 1.6.) are required in situations with horizontal loads and/or where eccentric vertical loads occur (vertical load F_k not aligned with neutral axis of the sheet pile).
   Geometrical requirements
   - embedment depth of the sheet pile L_E
     18 cm $\leq L_E \leq$ 33 cm[7)]
     tolerance: $\Delta L_E \leq \pm 3$ cm
   - minimum height of capping beam
     $h \geq 63$ cm $(= h_{OK} + L_E)$

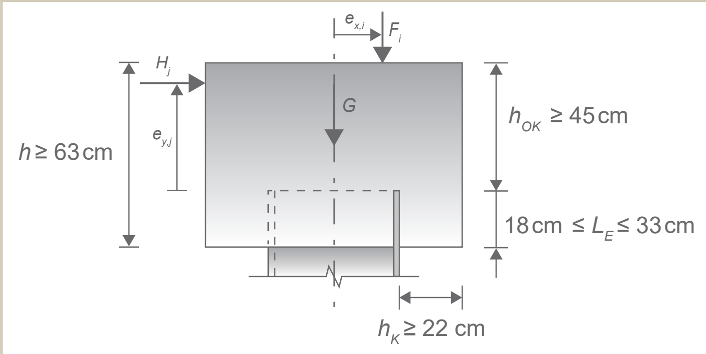


Figure 1.6. Fixed connection capping beam.

4) Classes of concrete above C 30/37 can be used for execution, but for the design $f_{ck} \leq 30$ MPa.
5) For more information about Exposure classes, please see Table 4.1. on EN 1992-1-1: 2004.
6) These values are minimum recommendations from the NTA, but the design may have to comply with more stringent requirements of local regulations or national standard.
7) If the embedment length is above 33 cm, only 33 cm can be considered in the design calculations. Additional reinforcing bars may be required for the overhang.

2. Design

From the structural design point of view it is important to highlight the fact that the NTA covers only the following loading situations

- static loads, horizontal and vertical loads
- non-predominantly-static loads, also described in the Eurocode as a loading situation where fatigue governs the design, but exclusively vertical loads

Purely dynamic design is out of scope of the NTA and is not dealt with in this document.

Note

The design values shown in the Annexes of the NTA have been calculated for a concrete of class C 30/37 with $f_{ck} = 30$ MPa, as well as with a minimum embedment length L_E. Conversion factors for lower concrete classes and / or other embedment length are listed in Chapter 7.

2.1. Loads, actions and combination of actions

2.1.1. Loads

a) Formulas

Horizontal loads	$H = \sum H_i$
Vertical loads	$F = G + \sum F_i$
Moments	$M = \sum F_i e_{x,i} + \sum H_j e_{y,j}$ with $e_{y,j} > 0$

b) Convention

Figure 2.1. highlights the convention used in the NTA

- vertical loads F_i are positive downwards,
- horizontal loads H_i are positive from left to right,
- moments M_i are positive clockwise,
- origin of the x-axis is located on the neutral axis of the steel sheet pile, positive on the right side of the axis,
- origin of the y-axis is located on the top of the steel sheet pile, positive upwards,
- eccentricity on the x-axis can be positive or negative,
- eccentricity on the y-axis can only be positive (no horizontal load can act below the top of the steel sheet pile).

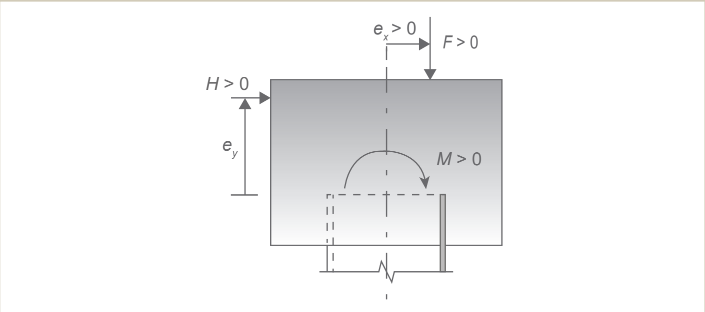


Figure 2.1. Convention: positive loads and distances.

2.1.2. Combination of actions & design values

2.1.2.1. Static situation

In a static loading case (standard situation), the design is calculated using the Ultimate Limit State (ULS), combination of actions for persistent or transient design situations[8] (EN 1990: 2002, Section 6.4.3.2)

$$E_d = \sum_{j \geq 1} \gamma_{G,j} \; G_{k,j} + \gamma_{Q,1} \; Q_{k,1} + \sum_{i > 1} \gamma_{Q,i} \; \psi_{0,i} \; Q_{k,i}$$

E_d is function of F, M and H described above.

2.1.2.2. Fatigue situation

In a non-predominantly-static loading, the following combination is proposed in the NTA

$$E_{d,frequ} = G_k + \psi_1 \left(Q_{k,1} + Q_{k,1,NR}\right) + \sum \psi_{2,i} \left(Q_{k,i} + Q_{k,i,NR}\right)$$

$$E_{d,frequ,NR} = \psi_1 Q_{k,1,NR} + \sum \psi_{2,i} Q_{k,i,NR}$$

with $i \geq 2$.

In this specific situation, $E_{d,frequ}$ is a function of F and M described above.

Note

Index NR stands for '*nicht-vorwiegend-ruhende Last*' in German language = non-predominantly-static load.

2.1.3. Influence of the concrete class and the embedment length on the design resistance

The resistance of the KES may be influenced by following choices

- a concrete class that is different from C 30/37 (*reminder:* 20 MPa ≤ f_{ck} ≤ 30 MPa),
- in the case of a fixed connection, an embedment exceeding 18 cm (*reminder:* 18 cm ≤ L_E ≤ 33 cm).

The resistance values and the conversion factors are given in Annex 1 and Annex 2 of the NTA (see Chapters 6 and 7). **Following values should be converted before using the verification formulae of the NTA shown in the next chapters.**

[8] According to EN 1990 :2002 ; EN 1991 ; EN 1992 ; EN 1993 ; EN 1997.

$$F_{Rd,m} = F_{Rd,m}\{Annex 1\}\cdot\left(\frac{f_{ck}}{30}\right)$$

$$M_{Rd,S} = M_{Rd,S}\{Annex 1\}\cdot\left(\frac{f_{ck}}{30}\right)$$

$$M_{Rd,K} = M_{Rd,K}\{Annex 1\}\cdot\left(\frac{f_{ck}}{30}\right)^{2/3}\cdot\left(\frac{L_E-3}{15}\right)$$

$$H_{Rd,K} = H_{Rd,K}\{Annex 2\}\cdot\left(\frac{f_{ck}}{30}\right)^{2/3}$$

$$k_{QK} = k_{QK}\{Annex 2\}\cdot\left(\frac{15}{L_E-3}\right)$$

$$k_{BM} = k_{BM}\{Annex 2\}\cdot\left(1.1-\frac{L_E}{180}\right)$$

with
f_{ck} in MPa
L_E in cm

2.2. Verifications

As mentioned before, the KES connection is a simplified method to optimize the design of concrete capping beams resting on top of sheet pile walls. Two methods can be utilised for this purpose: the classical analytical method and one based on a diagram.

It is necessary to consider the full range of vertical and horizontal forces, as well as their combinations. The two critical extremes are the maximum vertical load $(F_{d,sup})$[9] and the minimum vertical load $(F_{d,inf})$. Both values, and their associated design moments $M_d\{F_{d,sup}\}$ and $M_d\{F_{d,inf}\}$ have to be verified.

The statical verifications of the connection are focused on load transfer through the concrete body into the embedded sheet pile depth. In the same way as most Eurocode calculations, the principle of the verifications is that the design value of the effect of the actions E_d shall be lower or equal to the design value of the corresponding resistance R_d

$$E_d \le R_d$$

2.2.1. Static loading

$$F_d \le F_{Rd}$$
$$H_d \le H_{Rd}$$
$$M_d \le M_{Rd}$$

2.2.1.1. Simple connection

A simple connection capping beam is only able to transmit vertical loads. Therefore, the verification is quite straight-forward

$$F_d \le F_{Rd,m}$$

where $F_{Rd,m}$ is given in the tables of Chapter 6. $F_{Rd,m}$ is a value determined from the results of the laboratory tests.

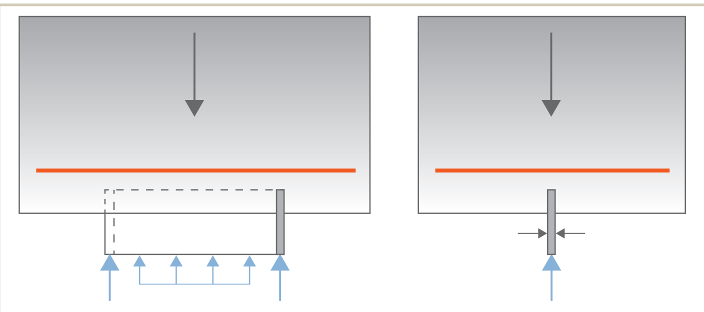


Figure 2.2. Simple connection capping beam model.

2.2.1.2. Fixed connection

A fixed connection capping beam is able to transmit horizontal and vertical loads, as well as moments (derived from eccentric loads). In this case, the verification is slightly more complex.

2.2.1.2.1. Analytical verification

The system is modelled as follows:

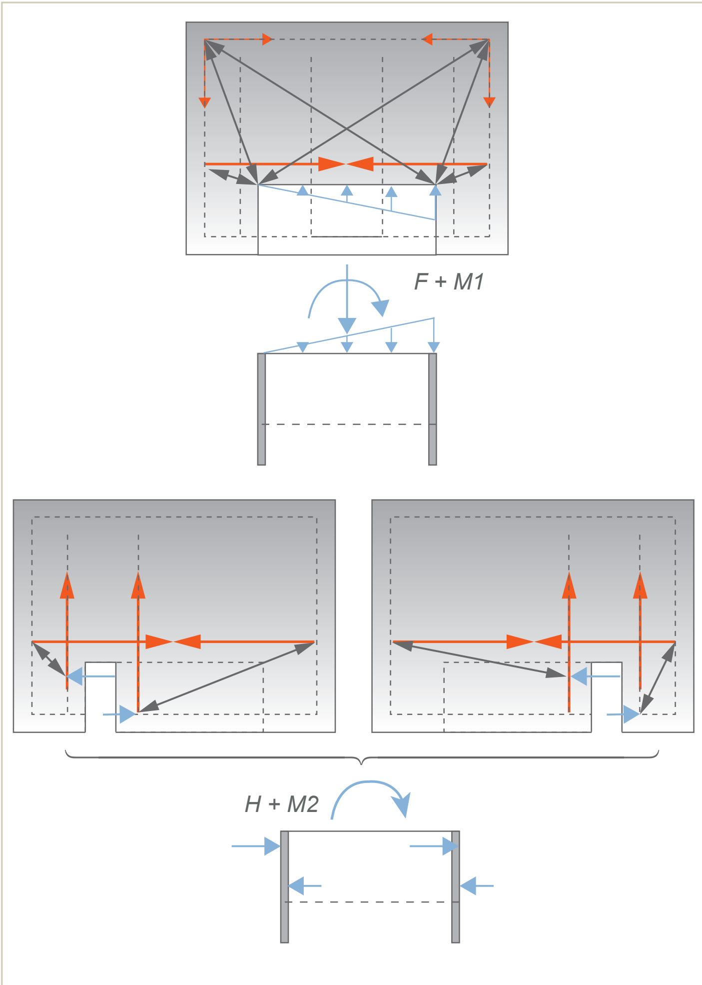


Figure 2.3. Detailed diagram and verification example (dotted lines).

[9] According to the '*Allgemeine Bauartgenehmigung*' Z-15.6-235. It is important not to mix this value with the Eurocode value G_{sup} that refers to the permanent loads of the superior part of an element (EN 1990: 2002).

The structural verification is based on

a)

$$F_d \leq F_{Rd,m}$$

b)

$$H_d \leq H_{Rd,K}$$

c)

$$M_d \leq M_{Rd}\{F_d\} = M_{Rd,K}\{F_d\} + M_{Rd,S}\{F_d\}$$

with

$$M_{Rd,K}\{F_d\} = M_{Rd,K} \cdot \left(1 - \frac{F_d}{F_{Rd,m}}\right)$$

and

$$M_{Rd,S}\{F_d\} \Rightarrow \begin{cases} \text{if } F_d \leq \dfrac{F_{Rd,m}}{2} \Rightarrow M_{Rd,S}\{F_d\} = 2 \cdot M_{Rd,S} \cdot \left(\dfrac{F_d}{F_{Rd,m}}\right) \\ \text{if } F_d > \dfrac{F_{Rd,m}}{2} \Rightarrow M_{Rd,S}\{F_d\} = 2 \cdot M_{Rd,S} \cdot \left(1 - \dfrac{F_d}{F_{Rd,m}}\right) \end{cases}$$

2.2.1.2.2. Diagram method

The diagram is a graphic representation of the analytical method. It is a function of moments (M) and vertical loads (F), easy to build and to use.

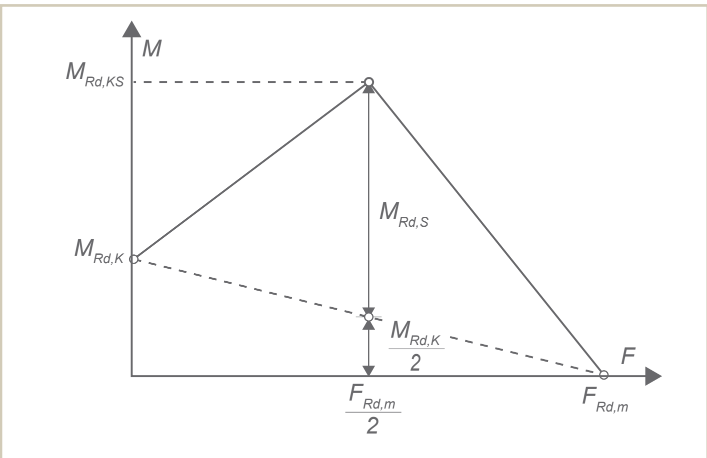


Figure 2.4. Diagram for predominantly static loading.

To build the diagram shown in Figure 2.4., the following points must be calculated.

- 50% of the maximum resistance to vertical forces and maximum moment resistance

$$\left(\frac{F_{Rd,m}}{2} \; ; \; M_{Rd,KS}\right)$$

$$M_{Rd,KS} = \frac{M_{Rd,K}}{2} + M_{Rd,S}$$

- no vertical forces, moment resistance due to the embedment depth ($F = 0$; $M_{Rd,K}$)

$$M_{Rd,K}\{F_d = 0\} = M_{Rd,K} \cdot \left(1 - \frac{0}{F_{Rd,m}}\right) = M_{Rd,K}$$

- maximum resistance to vertical forces and no moment resistance ($F_{Rd,m}$; $M = 0$)

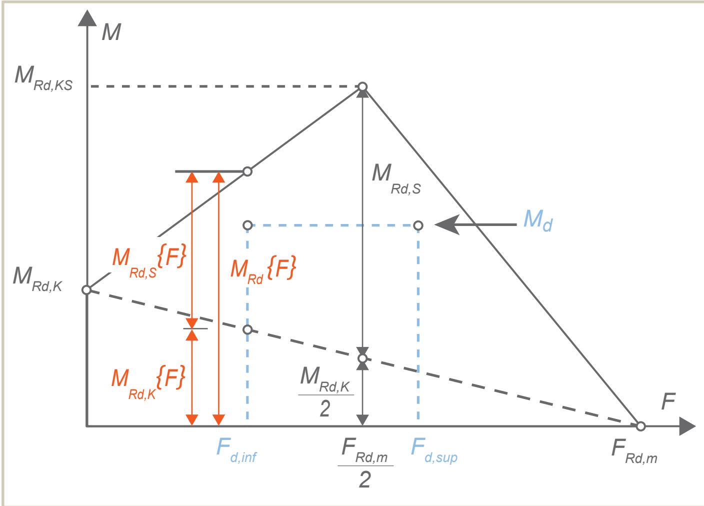


Figure 2.5. Detailed diagram and verification example (dotted lines).

Apart from the three previous points that define the diagram, it is possible to determine the two main moment resistances $M_{Rd,K}\{F_d\}$ and $M_{Rd,S}\{F_d\}$ for any given vertical load F_d between 0 and $F_{Rd,m}$ (see arrows in Figure 2.5.). The conservative assumption deduced from the laboratory tests is that the steel sheet pile takes the maximum possible stresses (for a detailed explanation, see Chapter 3).

Furthermore, in order to determine if the capping beam can resist the design actions, both pair of points ($F_{d,inf}$; $M_d\{F_{d,inf}\}$) and ($F_{d,sup}$; $M_d\{F_{d,sup}\}$) have to remain within the established limits (see dashed lines in Figure 2.5.).

The diagram shows how an increase in vertical loads (i.e. an increase in the size of the capping beam) has a positive effect on the design resistance up to a certain extent (50% of the maximum vertical resistance = $F_{Rd,m}/2$). Past this point, the vertical loads reduce the bending moment resistance.

Additionally, it is possible to determine graphically the maximum resistance $M_{Rd,K}$ that the restrained capping beam can introduce into the sheet pile, and how $M_{Rd,K}\{F_d\}$ decreases linearly when the vertical load rises.

2.2.2. Fatigue situation

The verifications are very similar to the static situation. Even though in this loading state cyclic loads may govern the design, they are transformed into quasi-static loads in order to simplify the calculations. Due to the non-predominantly-static actions, the resistance of the capping beam is reduced:

$$F_d \leq F_{Rd,reduced}$$
$$M_d \leq M_{Rd,reduced}$$

The reduced resistance considers a reduction factor applied on the design resistance of the system which takes into account the concrete contribution and the ratio between the non-static component of the actions and the full action (static + non-static).

Remember that in the fatigue situation, horizontal non-predominantly-static loads are not covered in the NTA. Only vertical non-predominantly-static loads can be applied to the capping beam.

2.2.2.1. Simple connection

The model is identical to the static situation (centered vertical load), except for the additional non-predominantly-static load.

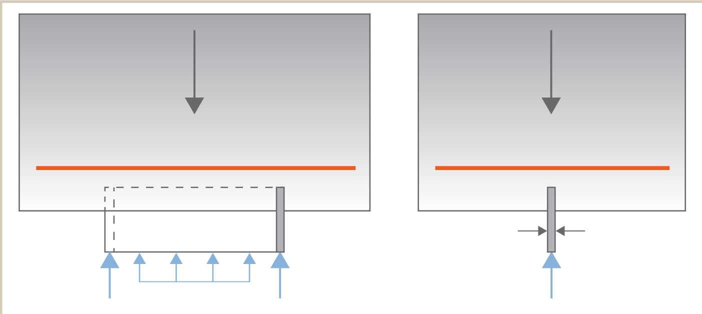


Figure 2.6. Simple connection capping beam model.

The verification is

$$F_{d,frequ} \leq F_{Rd,m,fat}$$

where

$$F_{Rd,m,fat} = r_{fat,FM} \cdot F_{Rd,m}$$

with

$$r_{fat,FM} = \frac{k_c}{1.22 + n_{NR,FM}}$$

where

$k_c = 0.98$ (constant, function of the concrete class)[10)]

and

$$n_{NR,FM} = \frac{F_{d,frequ,NR}}{F_{d,frequ}}$$

2.2.2.2. Fixed connection

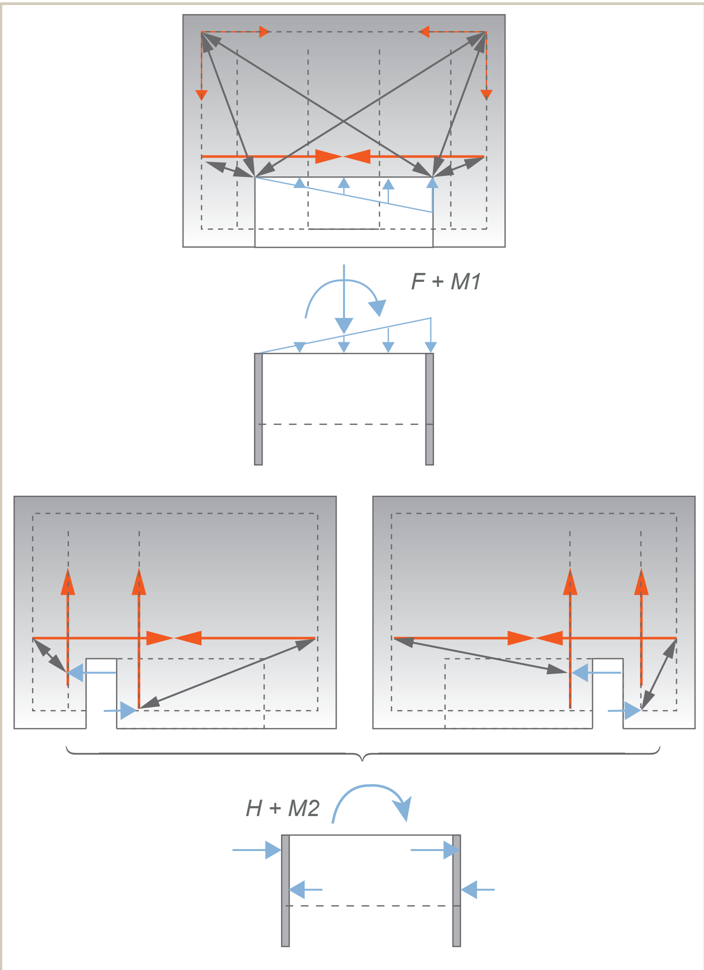


Figure 2.7. Fixed connection model, fatigue situation.

The model is quite similar to the static situation, except for the additional vertical non-predominantly-static load (horizontal non-predominantly-static loads are not allowed). The resistance reduction of vertical loads and moments due to the non-predominantly-static loads is taken into account with the factors $r_{fat,FM}$ and $r_{fat,MK}$

$$r_{fat,FM} = \frac{k_c}{1.22 + \eta_{NR,FM}}$$

$$r_{fat,MK} = \frac{k_c}{1.22 + \eta_{NR,MK}}$$

with k_c constant, function of the concrete class (see 2.2.2.1.)

and

$$n_{NR,FM} = \frac{\dfrac{F_{d,frequ,NR}}{A} + \dfrac{M_{d,frequ,NR}}{W}}{\dfrac{F_{d,frequ}}{A} + \dfrac{M_{d,frequ}}{W}}$$

$$n_{NR,MK} = \frac{M_{d,frequ,NR}}{M_{d,frequ}}$$

with

A cross sectional area of the sheet pile section (see Chapter 6)

W elastic section modulus of the steel sheet pile section (see Chapter 6)

2.2.2.2.1. Analytical method

When the fatigue governs the design, the following verification method shall be followed

$$F_{d,frequ} \leq F_{Rd,m,fat}$$

$$M_{d,frequ}\{F_{d,frequ}\} \leq M_{Rd,fat}\{F_{d,frequ}\}$$

To calculate the reduced resistances, due to the effect of the fatigue, following applies

- vertical forces

$$F_{Rd,m,fat} = r_{fat,FM} \cdot F_{Rd,m}$$

- moments

$$M_{Rd,fat}\{F_{d,frequ}\} = M_{Rd,K,fat}\{F_{d,frequ}\} + M_{Rd,S,fat}\{F_{d,frequ}\}$$

with

$$M_{Rd,K,fat}\{F_{d,frequ}\} = M_{Rd,K,fat} \cdot \left(1 - \frac{F_{d,frequ}}{F_{Rd,m,fat}}\right)$$

$$M_{Rd,K,fat} = r_{fat,MK} \cdot M_{Rd,K}$$

and

$$M_{Rd,S,fat}\{F_{d,frequ}\} \Rightarrow \begin{cases} \text{if } F_{d,frequ} \leq \dfrac{F_{Rd,m,fat}}{2} \Rightarrow M_{Rd,S,fat}\{F_{d,frequ}\} = 2 \cdot M_{Rd,S,fat} \cdot \left(\dfrac{F_{d,frequ}}{F_{Rd,m,fat}}\right) \\ \text{if } F_{d,frequ} > \dfrac{F_{Rd,m,fat}}{2} \Rightarrow M_{Rd,S,fat}\{F_{d,frequ}\} = 2 \cdot M_{Rd,S,fat} \cdot \left(1 - \dfrac{F_{d,frequ}}{F_{Rd,m,fat}}\right) \end{cases}$$

$$M_{d,S,fat} = \mathrm{r}_{\mathrm{fat,FM}} \cdot M_{Rd,S}$$

[10)] The *'Allgemeine Bauartgenehmigung'* does not consider the influence of the concrete class on k_c and retained only $k_c = 0.98$ (conservative approach as this value corresponds to a concrete class C 30/37).

2.2.2.2.2. Diagram method

The diagram is a graphic representation of the analytical method. It is a function of moments M and vertical loads F, easy to build and to use.

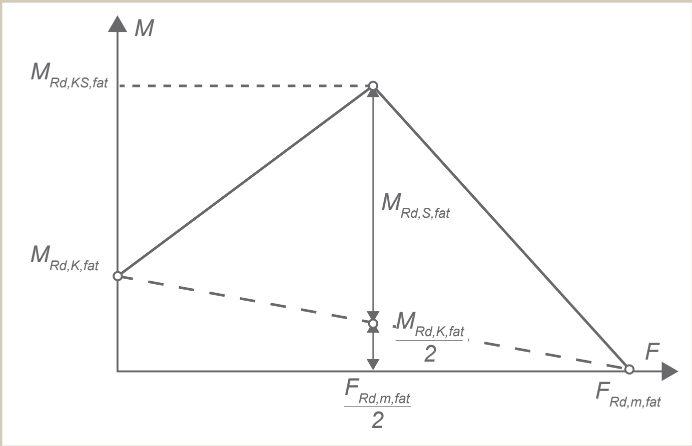


Figure 2.8. Diagram for non-predominantly-static loading.

To build the diagram shown in Figure 2.8., following points must be calculated.

- 50% of the reduced resistance to vertical forces due to the fatigue action, and reduced moment resistance

$$\left(\frac{F_{Rd,m,fat}}{2}\ ;\ M_{Rd,KS,\,fat}\right)$$

$$F_{Rd,m,fat} = r_{fat,FM} \cdot F_{Rd,m}$$

$$M_{Rd,KS,fat} = \frac{M_{Rd,K,fat}}{2} + M_{Rd,S,fat}$$

where

$$M_{Rd,S,fat} = r_{fat,FM} \cdot M_{Rd,S}$$
$$M_{Rd,K,fat} = r_{fat,MK} \cdot M_{Rd,K}$$

- no vertical forces: moment resistance due to the embedment depth ($F = 0$; $M_{Rd,K,fat}$):

$$M_{Rd,K,fat} = r_{fat,MK} \cdot M_{Rd,K}$$

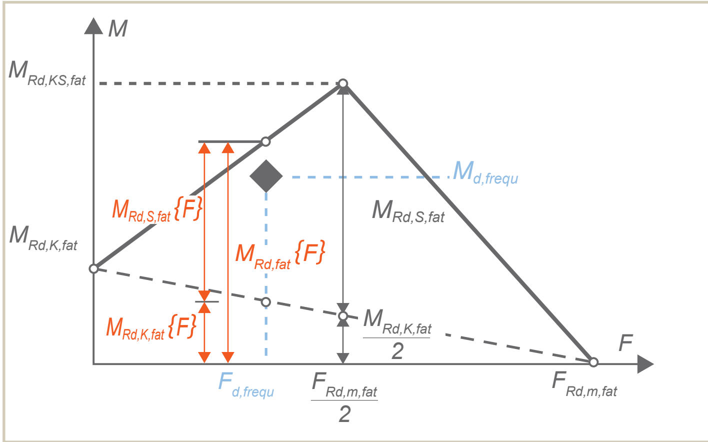


Figure 2.9. Fatigue situation: detailed diagram and verification example (dotted lines).

Apart from the three points that define the diagram, it is possible to determine the two main moment resistances $M_{Rd,K,fat}\{F_{d,frequ}\}$ and $M_{Rd,S,fat}\{F_{d,frequ}\}$ for any given vertical load $F_{d,frequ}$ between 0 and $F_{Rd,m,fat}$ (see arrows on Figure 2.9.). The conservative assumption deduced from the laboratory tests is that the steel sheet pile takes the maximum possible stresses (for a detailed explanation, see section 2.3.).

Furthermore, in order to determine if the capping beam can resist the fatigue loads, the point $(F_{d,frequ}\,;M_{d,frequ}\{F_{d,frequ}\})$ needs to stay within the established limits (see dashed lines on Figure 2.9.).

The diagram shows how an increase in vertical loads (i.e. an increase in the size of the capping beam) has a positive effect on the design resistance up to a certain extent (50% of the maximum vertical resistance $= F_{Rd,m,fat}/2$). Past this point, the vertical loads reduce the bending moment resistance.

Additionally, it is possible to determine graphically the maximum resistance $M_{Rd,K,fat}$ that the restrained capping beam can introduce into the sheet pile, and how $M_{Rd,K,fat}\{F_{d,frequ}\}$ decreases linearly when the vertical load rises.

2.3. Fixed connection: design values for the calculation of the reinforcement

The method considered in the NTA assumes that the KES connection (that is $M_{d,S}$, $M_{Rd,S}$, $M_{Rd,S,fat}$, etc.) takes the maximum possible loading. This assumption lies on the safe side, as $M_{d,S}$ is a critical value when calculating the reinforcing.

2.3.1. Static situation

The moment distribution in the connection follows the expressions

$$M_d = M_{d,S} + M_{d,K}$$

$$\text{if } M_d \le M_{Rd,S}\left\{F_d\right\} \rightarrow \begin{cases} M_{d,S} = M_d \\ M_{d,K} = 0 \end{cases}$$

$$\text{if } M_d > M_{Rd,S}\left\{F_d\right\} \rightarrow \begin{cases} M_{d,S} = M_{Rd,S}\left\{F_d\right\} \\ M_{d,K} = M_d - M_{d,S} \end{cases}$$

with

$M_{Rd,S}\{F_d\}$ according to section 2.2.1.2.1.

2.3.2. Fatigue situation

In the fatigue case, the design actions are

$$F_d^* = 6.21 \cdot F_{d,frequ,NR}$$
$$M_d^* = 6.21 \cdot M_{d,frequ,NR}$$

where

$$M_d^* = M_{d,S}^* + M_{d,K}^*$$

The moment distribution in the connection follows the expressions

$$\text{if } M_d^* \le 6.21 \cdot M_{Rd,S,fat}\left\{F_{d,frequ}\right\} \rightarrow \begin{cases} M_{d,S}^* = M_d^* \\ M_{d,K}^* = 0 \end{cases}$$

$$\text{if } M_d^* > 6.21 \cdot M_{Rd,S,fat}\left\{F_{d,frequ}\right\} \rightarrow \begin{cases} M_{d,S}^* = 6.21 \cdot M_{Rd,S,fat}\left\{F_{d,frequ}\right\} \\ M_{d,K}^* = M_d^* - M_{d,S}^* \end{cases}$$

with

$M_{Rd,S,fat}\{F_{d,frequ}\}$ according to section 2.2.2.2.1.

2.4. Reinforcement guidelines & layout

The German 'National Technical Approval' contains recommendations and minimum requirements to calculate the required concrete reinforcement, which will be detailed in this Chapter. For all other cases or situations not foreseen in the approval, ArcelorMittal recommends following EN 1992:2002 or the national standards, or contacting our technical department.

Reinforcement is necessary to ensure the different load transfer to the steel sheet pile section, as well as to avoid the cracks resulting from the concrete shrinkage and splitting due to localized concentrated forces. The minimum reinforcement suggested is based on DIN EN 1992-1-1 with DIN EN 1992-1-1 / NA in Germany, and should be sufficient for most European countries, but national standards can specify higher amounts of minimum reinforcement.

The maximum allowable diameter of the rebars d_s is 16 mm.

2.4.1. Simple connection

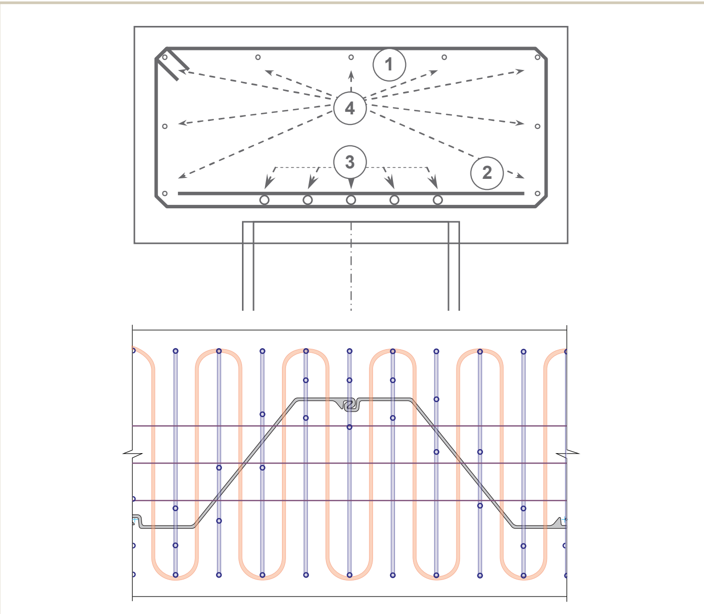


Figure 2.10. Simple connection reinforcement detailing (top: cross section; bottom: plan view).

In this situation 4 different kinds of reinforcement can be distinguished

1. stirrups (Pos. 1): reinforcing bars used for shear reinforcement; typically bent into a U-shape or box-shape and placed perpendicular to the longitudinal reinforcing bars,
2. transversal splitting reinforcement (Pos. 2): reinforcement bars placed perpendicular to the longitudinal reinforcement, to avoid cracking in the joint between concrete and steel sheet pile section. This reinforcement is usually executed as a serpentine,
3. longitudinal splitting reinforcement (Pos. 3): reinforcement bars placed parallel to the longitudinal axis of the wall, to avoid cracking in the joint between concrete and steel sheet pile section. These rebars can be spread only over the depth of the sheet pile! (see Figure 2.12.),
4. longitudinal reinforcement (Pos. 4): minimum steel reinforcement placed in the longitudinal axis of the wall.

2.4.1.1. Stirrups (Pos. 1)

- minimum bar diameter $d_s = 10\ \mathrm{mm}$
- maximum distance between stirrups $a = 15\ \mathrm{cm}$

2.4.1.2. Transversal splitting reinforcement (Pos. 2)

The required reinforcing is calculated with following formula

$$a_{SpQ} = k_{QF} \cdot F_d$$

k_{QF} depends on the section (see Chapter 6).

The steel reinforcement from Pos. 1 can be taken into account for this position, so that in some cases no additional reinforcing bars may be needed.

If $a_{SpQ} > 10\ cm^2/m$ then the reinforcement shall be spread over at least two layers.

2.4.1.3. Longitudinal splitting reinforcement (Pos. 3)

- minimum bar diameter $d_s = 10\ \mathrm{mm}$
- maximum distance between bars $a = 15\ \mathrm{cm}$
- minimum amount of reinforcement bars is 3 bars of $d_s = 10\ \mathrm{mm}$

The longitudinal splitting reinforcing is calculated as follows

$$A_{SpL} = k_{LF} \cdot F_d$$

k_{LF} depends on the section (see Chapter 6).

2.4.1.4. Longitudinal reinforcement (Pos. 4)

- minimum bar diameter $d_s = 10\ \mathrm{mm}$
- maximum distance between bars $a = 15\ \mathrm{cm}$
- minimum amount of reinforcing bars
  - lateral 3 bars of $d_s = 10\ \mathrm{mm}$
  - top 5 bars of $d_s = 10\ \mathrm{mm}$

2.4.2. Fixed connection

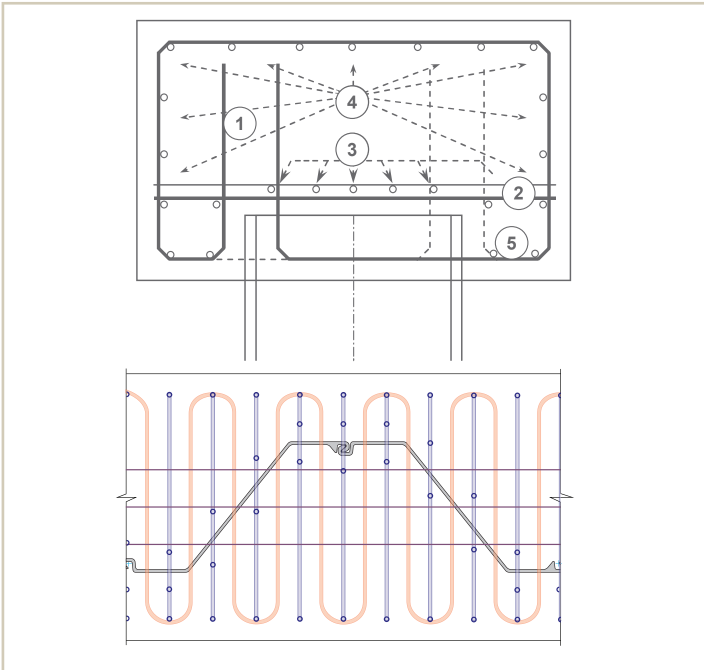


Figure 2.11. Fixed connection reinforcement detailing (top: cross section; bottom: plan view).

In this situation 5 different kinds of reinforcement can be distinguished

1. stirrups (Pos. 1): reinforcing bars used for shear reinforcement; typically bent into a U-shape or box-shape and placed perpendicular to the longitudinal reinforcing bars,
2. transversal splitting reinforcement (Pos. 2): reinforcement bars placed perpendicular to the longitudinal reinforcement, to avoid cracking in the joint between concrete and steel sheet pile section,

3. longitudinal splitting reinforcement (Pos. 3): reinforcement bars placed parallel to the longitudinal axis of the wall, to avoid cracking in the joint between concrete and steel sheet pile section. These rebars can be spread only over the depth of the sheet pile! (see Figure 2.12.),
4. longitudinal reinforcement (Pos. 4): minimum steel reinforcement placed in the longitudinal axis of the wall,
5. longitudinal corbel reinforcement (Pos. 5): reinforcing bars placed in the longitudinal axis of the wall to avoid cracking of the overhang h_k.

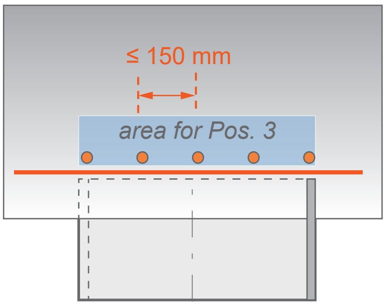


Figure 2.12. Positioning of the rebars from Pos. 3.

2.4.2.1. Stirrups (Pos. 1)

- minimum bar diameter $d_s = 10\ \mathrm{mm}$
- maximum distance between stirrups $a = 15\ \mathrm{cm}$

2.4.2.1.1. Static situation

$$a_{Bü,K} = k_{BM} \cdot M_{d,K} + k_{BH} \cdot H_d$$

where

k_{BM} constant associated to $M_{d,K}$ (see Chapter 7)

k_{BH} constant associated to H_d (see Chapter 7)

2.4.2.1.2. Fatigue situation

$$a_{Bü,K} = k_{BM} \cdot M^*_{d,K}$$

where

k_{BM} constant associated to $M_{d,k}$ (see Chapter 7).

2.4.2.2. Transversal splitting reinforcement (Pos. 2)

- minimum bar diameter $d_s = 10\ \mathrm{mm}$
- maximum distance between stirrups $a = 15\ \mathrm{cm}$
- if the minimum reinforcement $a_{SpQ} + \Delta a_{SpQ}$

$> 10\ \mathrm{cm}^2/\mathrm{m}$, then it shall be spread over at least two layers

2.4.2.2.1. Static situation

The transversal splitting reinforcement is equal to $a_{SpQ} + \Delta a_{SpQ}$

with

$a_{SpQ} = k_{QF} \cdot F_d + k_{QM} \cdot M_{d,S}$

and

$\Delta a_{SpQ} = k_{QK} \cdot M_{d,K} + k_{QH} \cdot H_d$

k_{QK} constant associated to $M_{d,K}$ (see Chapter 7)

k_{QH} constant associated to H_d (see Chapter 7)

2.4.2.2.2. Fatigue situation

The transversal splitting reinforcement is equal to $a_{SpQ} + \Delta a_{SpQ}$

with

$$a_{SpQ} = k_{QF} \cdot F^*_d + k_{QM} \cdot M^*_{d,S}$$

and

$$\Delta a_{SpQ} = k_{QK} \cdot M^*_{d,K}$$

k_{QK} constant associated to $M^*_{d,K}$ (see Chapter 7)

2.4.2.3. Longitudinal splitting reinforcement (Pos. 3)

- minimum bar diameter $d_s = 10\ \mathrm{mm}$
- maximum distance between bars $a = 15\ \mathrm{cm}$
- minimum amount of reinforcement bars is 3 bars of $d_s = 10\ \mathrm{mm}$

The longitudinal splitting reinforcing is calculated as follows, where k_{LF} depends on the section (see Chapter 6).

2.4.2.3.1. Static situation

$$A_{SpL} = k_{LF} \cdot F_d$$

2.4.2.3.2. Fatigue situation

$$A_{SpL} = k_{LF} \cdot F^*_d$$

2.4.2.4. Longitudinal reinforcement (Pos. 4)

- minimum bar diameter $d_s = 10\ \mathrm{mm}$
- maximum distance between bars $a = 15\ \mathrm{cm}$
- minimum amount of reinforcing bars
  - lateral 3 bars of $d_s = 10\ \mathrm{mm}$
  - top 5 bars of $d_s = 10\ \mathrm{mm}$

2.4.2.5. Longitudinal corbel reinforcement (Pos. 5)

- minimum bar diameter $d_s = 10\ \mathrm{mm}$
- minimum quantity of reinforcing bars on each side: 2

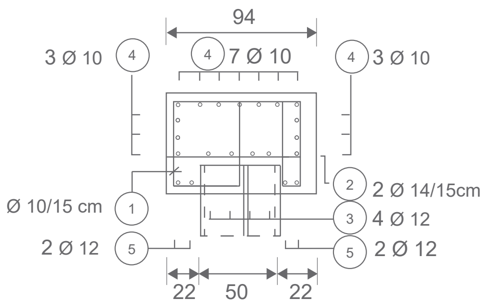


Figure 2.13. Fixed connection: example of a typical reinforced concrete section.

3. Background

Steel sheet pile walls are cost-effective retaining walls even in combination with high vertical loads. Several 'standardized' solutions to transfer those vertical loads into the sheet pile wall existed, such as concrete or steel capping beams.

These well-established design methods can be found for instance in the 1938 edition of the German Larssen Handbook[11)]. Furthermore, in 1973 steel capping beams are discussed in the technical review of the EAU[12)] as a preferred recommendation for construction.

However, these methods seemed to be too conservative, so that in 2004, ArcelorMittal started the procedure for a national technical approval for developing a simple but optimized method to design and execute a capping beam capable of supporting high vertical loads. The challenge was to use less reinforcement steel in order to save construction time and material cost.

ArcelorMittal Research and Development launched a first project in partnership with the University of Darmstadt, Germany, following a scientific approach based on a series of models (full scale and scaled testing programs) to establish a preliminary design concept. Afterwards, the final design approach was elaborated in collaboration with the German construction authority DIBt (Deutsches Institut für Bautechnik). It was peer-reviewed by the renowned German consulting engineers "Wörner und Nordhues Tragwerksplanung GmbH", leading to the first national technical approval that was issued in 2011. In 2014 a revised technical approval was released. It takes into account the latest German and European standards DIN 1045: 2088-08, EN 1992-1-1: 2004 and DIN EN 1992-1-1/NA: 2013-04, and some other modifications in the scope of application.

The technical approval is divided into centric and eccentric load introduction, with static and non-predominantly-static vertical loads (introduced as 'quasi-static' actions), and static horizontal loads. The technical approval is not valid for non-static, nor for dynamic horizontal actions (in those specific cases, please follow the directions given in EN 1992: 2004, the corresponding national standards, or contact the technical department of ArcelorMittal for technical support).

3.1. Laboratory testing

Within the framework of the KES technical approval, various tests were carried out by the Technical University of Darmstadt and double-checked by the consulting engineers.

3.1.1. Small scale tests

In the period from December 2006 to January 2007, small scale tests were carried out at the university. In these tests, the compressive strength under the cutting edge (*'Schneidenlagerung'*) of the sheet pile were examined using different 'cutting' plates and concrete thicknesses.

Therefore, an allowable compressive stress across the cutting edge of seven times f_{cd} can be used for the design concept. It was included in the further calculation method in the form of the usual partial safety factors.

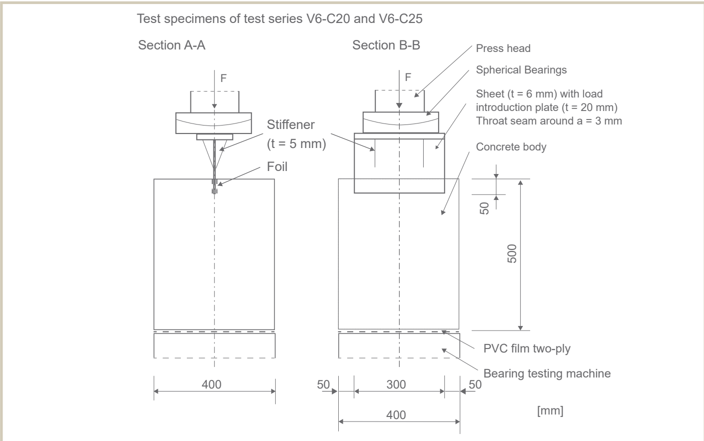
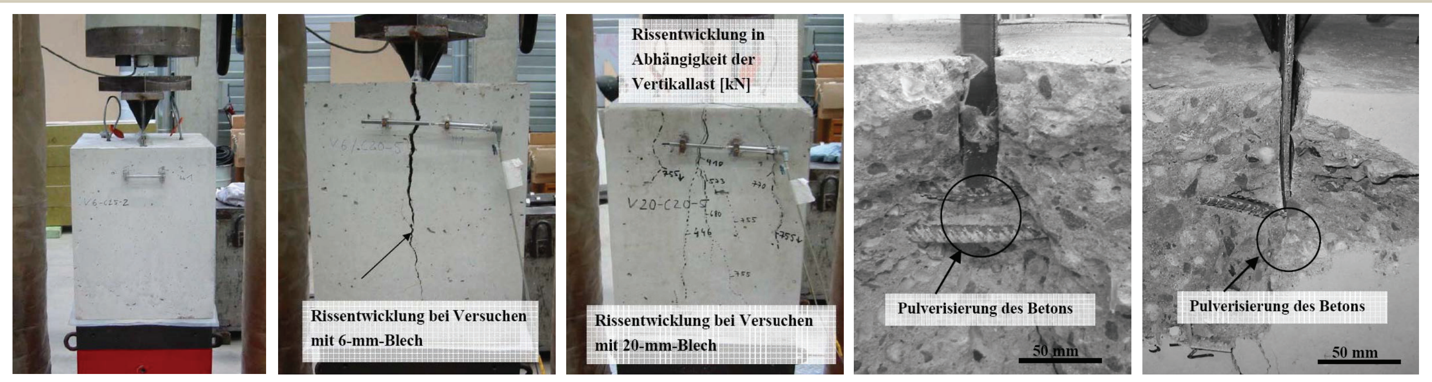


Figure 3.1. Scale tests design.



Picture 2. Results from the small scale tests.

11) Larssen Handbuch. Dortmund-Hörder Hüttenverein Aktiengesellschaft. Ausgabe 1938.
12) Technischer Jahresbericht 1973 des Arbeitsausschusses "Ufereinfassungen" der Hafenbautechnischen Gesellschaft e.V. und der Deutschen Gesellschaft für Erd- und Grundbau e.V. Die Bautechnik. 50. Jahrgang. Heft 12/1973.

3.1.2. Large scale tests

The main objective of these large scale tests was to analyse the local effect of the loading along the 'cutting edge' (the connection between the steel sheet pile and the concrete). The load bearing capacity and the embedment depth was tested for two profiles: a light section PU 6 and a quite strong PU 32.

The university performed 12 large scale tests, from April to October 2007, to verify the capping beam behaviour up to its maximum admissible load (ultimate limit state). Six tests were performed on the PU 6 and six on the PU 32. Each of the following tests on one section had at least one parameter that differs from the first test.

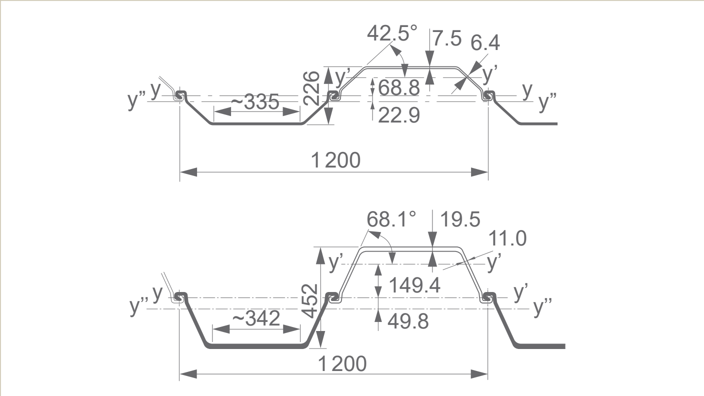


Figure 3.2. Sheet piles used in the laboratory tests. PU 6 (top) and PU 32 (bottom).

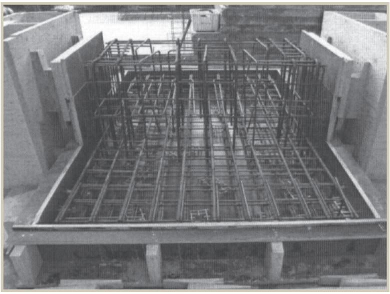
The characteristics of the concrete used to cast the capping beams in these tests were

- class C 20/25, according to EN 1992-1-1:2004 ($f_{ck} = 20$ MPa),
- no air-entraining additives.

In addition to being submitted to their maximum admissible load, the prototypes were tested against fatigue, with the following loading procedure

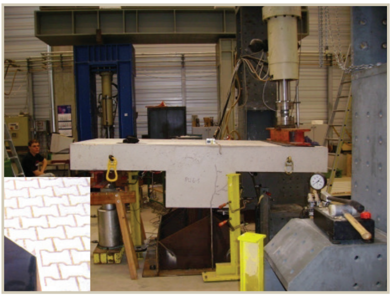
- the first cycle consisted in constantly increasing the applied moment with a step of 5 kNm, at a rate of 5 kNm/min, followed by a break of 1-2 minutes, until reaching 120% of the estimated maximum service load (see Picture 5),
- then, 12 cyclic loadings with a loading rate of 50 kNm/min, and an amplitude varying from 120% of the service load down to -20 kNm, were applied,
- after 12 cycles, the load was raised to 120% of the service load,
- finally, loading steps with a rate up to 5 kNm/min were applied, followed by smaller strain loadings, until the prototype failed.

  The successful tests demonstrated that the capping beam and the connection between the capping beam and the sheet pile section (Knife Edge Support) performed better than an 'average' capping beam designed with the 'standard' methods.

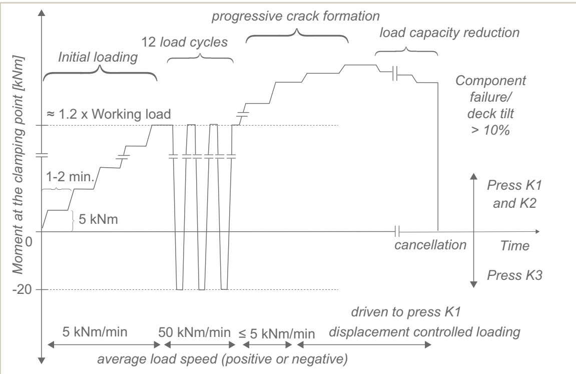


Picture 3. Reinforcement detail.

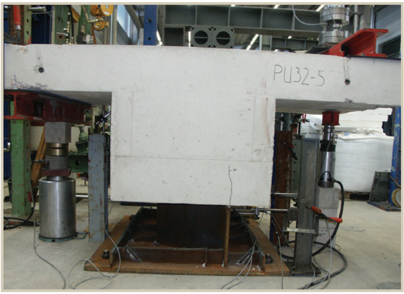
Picture 4. Concrete casting.



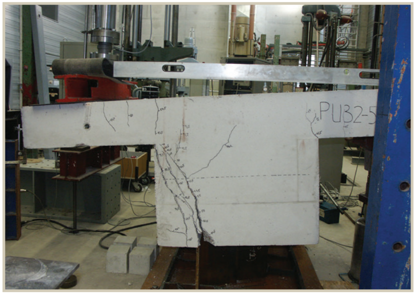
Picture 5. Fatigue test - load cycles.



Picture 6. Loading sequence in the fatigue test.



Picture 7. PU 32 during a loading test.



Picture 8. PU 32 at failure (slope ≥ 10%).

3.2. Elaboration of the design method

The results from the laboratory tests confirmed the proposed design method.

However, additional verifications and tests were requested by the DIBt before issuing the technical approval.

4. Example

To illustrate the verifications in a didactic way, this example is quite basic on purpose. Please read Chapter 2 'Design' before scrolling through the example (the theoretical design assumptions, formulas and coefficients are assumed to be known).

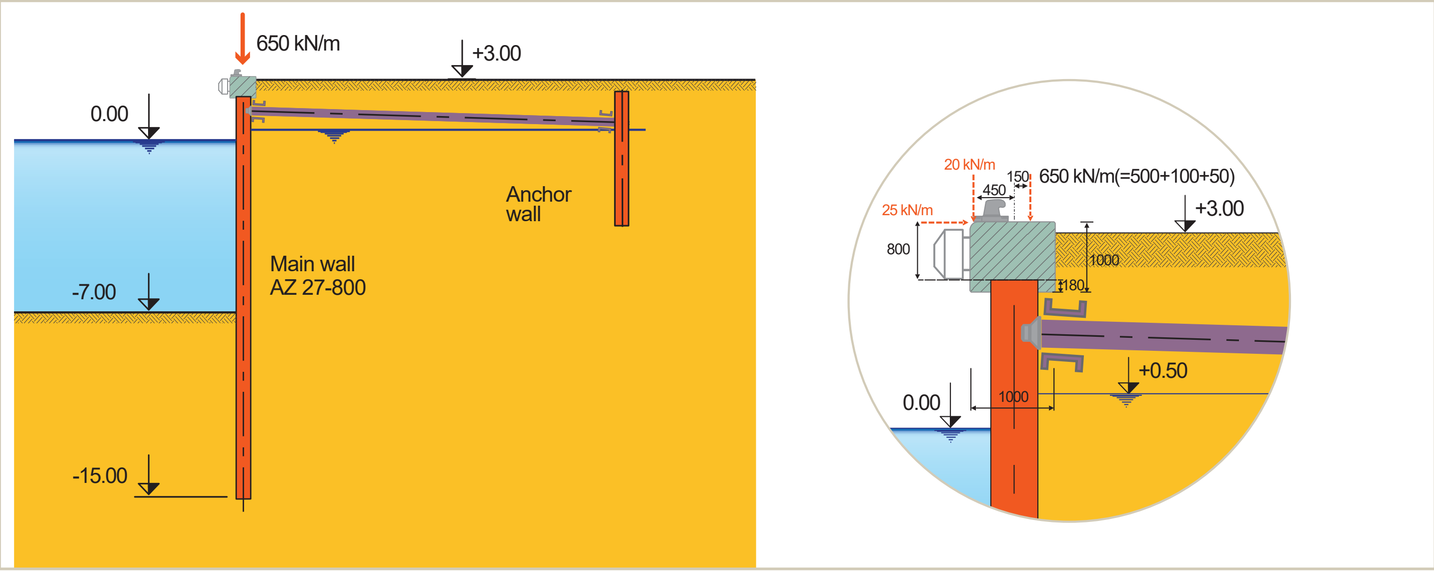


Figure 4.1. Design situation – left: cross-section – right: detail.

4.1. Actions, geometry and assumptions

4.1.1. Loads

- vertical load due to the concrete capping beam

  $G_{concrete} = 25 \text{ kN/m}^3 \cdot 1 \text{ m}^3\text{/m} = 25 \text{ kN/m}$
- vertical load due to a 'fixed' crane

  $F_{crane} = 650 \text{ kN/m}$ (maximum vertical load)

  - permanent $= 50 \text{ kN/m}$
  - variable $= 500 \text{ kN/m}$ $= Q_{k,1}$
  - non-predominantly-static (cyclic) $= 100 \text{ kN/m} = Q_{k,1,\,NR}$

  $e_x = 150 \text{ mm}$

Note

We assume that the variable load and the associated non-predominantly-static load act always concurrently (inseparable actions). However, several variable loads and associated non-predominantly-static loads can be taken into account.

- vertical loads due to the accessories (bollard, etc...)

  $F_{acc} = 20 \text{ kN/m}$

  $e_x = \text{-}450 \text{ mm}$
- horizontal load due to berthing

  $H_{berth} = +25 \text{ kN/m}$

  $e_y = 800 \text{ mm}$

4.1.2. Exposure class

The governing class of exposure of the concrete capping beam for a quay wall can be assumed to be XS 3, which corresponds to areas of tides, splashing and spraying of seawater.

The recommended concrete cover according to the European standard is 55 mm.

4.1.3. Concrete characteristics

Since the class of exposure is XS 3, the minimum class of concrete to be utilised is C 35/45. However, according to the NTA, the maximum concrete strength that can be taken into account is $f_{ck} = 30 \text{ MPa}$ (see Chapter 1.2.).

- concrete class C 35/45 but $f_{ck} = 30 \text{ MPa}$
- $\gamma = 25 \text{ kN/m}^3$

4.1.4. Sheet pile section

Chosen section[13] AZ 27-800.

Section	Width b mm	Height h mm	Thickness t mm	Thickness s mm	Sectional area cm²/m	Mass single pile kg/m	Mass wall kg/m²	Moment of inertia cm⁴/m	Elastic section modulus cm³/m	Static moment cm³/m	Plastic section modulus cm³/m	Class S 240 GP	S 270 GP	S 320 GP	S 355 GP	S 390 GP	S 430 GP	S 460 AP
AZ®-800																		
AZ 23-800	800	474	11.5	9.0	151	94.6	118	55 260	2 330	1 340	2 680	2	2	2	3	3	3	3
AZ 25-800	800	475	12.2	10.0	163	102.6	128	59 410	2 500	1 445	2 890	2	2	2	2	2	3	3
AZ 27-800	800	476	13.5	11.0	176	110.5	138	63 570	2 670	1 550	3 100	2	2	2	2	2	2	2

Figure 4.2. AZ 27-800 section properties from ArcelorMittal General Catalogue.

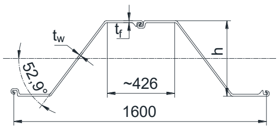
Profil	Skizze	B mm	h mm	t_F mm	t_W mm	A cm^2/m	W cm^3/m	$F_{Rd,m}$ kN/m	$M_{Rd,S}$ kNm/m	$M_{Rd,K}$ kNm/m	k_{LF} $\frac{cm^2}{MN/m}$	k_{QF} $\frac{cm^2/m}{MN/m}$	k_{QM} $\frac{cm^2/m}{kNm/m}$
AZ 23-800		800	474	11.5	9.0	150.6	2330	1792	138.6	31.0	5.41	10.42	0.067
AZ 25-800-0.5		800	475	12.0	9.5	157.0	2415	1868	143.7				
AZ 25-800		800	475	12.5	10.0	163.3	2500	1943	148.8				
AZ 25-800+0.5		800	476	13.0	10.5	169.6	2585	2018	153.8				
AZ 27-800		800	476	13.5	11.0	176.0	2670	2094	158.9				

Figure 4.3. AZ 27-800 section properties from Annex 1 of the 'Allgemeine Bauartgenehmigung' Z-15.6-235. Annex 1 is reproduced in Chapter 6.

4.1.5. Connection sheet pile / capping beam

Due to the unbalanced vertical loads and horizontal loads the support must be designed according to the **'fixed connection'** method.

The embedment length of the sheet pile into the concrete capping beam is assumed to be $L_E = 18$ cm (minimum embedment length, no conversion factors need to be applied in this case).

4.2. Combination of actions

Finding the worst-case scenario for each required verification is quite complex when several loads must be considered. Although the combination leading to the maximum vertical load, the maximum horizontal load and the maximum bending moment can be obvious in quite simple cases, the non-linear interaction between the vertical load, horizontal load and the resisting moment can lead to situations where a combination with lower vertical or horizontal loads may be governing the design!

Hence, it is important to analyse all the possible combinations, which realistically can only be performed with a software.

Below the load case that could be expected to be governing the design.

Note

$\psi_{0,i}$, ψ_1 and $\psi_{2,i}$ chosen in this example are for illustrative purposes only!

Use the adequate values based on national application documents of the Eurocodes and / or national standards.

[13] This is an iterative process. For instance, start with a sheet pile section that fulfils the design criteria of the bending moment. Reminder: the steel grade of the sheet pile does not have an influence on the KES design.

4.2.1. Static situation

Persistent design situation with the following combination of actions

$$F_d = \Sigma\, \gamma_{G,j}\ G_{k,j} + \gamma_{Q,1}\ Q_{k,1} + \Sigma\, \gamma_{Q,i}\ \psi_{0,i}\ Q_{k,i}$$

where

$G_{k1} = 25\ \mathrm{kN/m}$ (weight of the concrete capping beam)

$G_{k2} = 50\ \mathrm{kN/m}$

$G_{k3} = 20\ \mathrm{kN/m}$

$\gamma_{G,j} = 1.35$ for $j = 1$ to 3

$Q_{k1} = 500 + 100 = 600\ \mathrm{kN/m}\ (='Q_{k1} + Q_{k1,NR}')$

Reminder: Q_{k1} and $Q_{k1,NR}$ are inseparable!

$\gamma_{Q1} = 1.50$

$\Rightarrow F_d = 1.35 \cdot 95 + 1.50 \cdot 600 = 1028.25\ \mathrm{kN/m}$

$$H_d = \gamma_{Q1}\ Q_{k1} + \Sigma\, \gamma_{Q,i}\ \psi_{0,i}\ Q_{k,i}$$

where

$Q_{k2} = 25\ \mathrm{kN/m}\ (= H_{berth})$

$\gamma_{Q2} = 1.50$

$\psi_{0,2} = 0.9$

$\Rightarrow H_d = 1.50 \cdot 0.9 \cdot 25 = 33.75\ \mathrm{kN/m}$

$$M_d = \Sigma\, \gamma_{G,j}\, G_{k,j}\, e_{G,j} + \gamma_{Q1}\, Q_{k1}\, e_{Q1} + \Sigma\, \gamma_{Q,i}\, \psi_{0,i}\, Q_{k,i}\, e_{Q,i}$$

where

$G_{k1} = 25\ \mathrm{kN/m}$ $\quad e_{G1} = 0.00\ \mathrm{m}$

$G_{k2} = 50\ \mathrm{kN/m}$ $\quad e_{G2} = 0.15\ \mathrm{m}$

$G_{k3} = 20\ \mathrm{kN/m}$ $\quad e_{G3} = -0.45\ \mathrm{m}$

$\gamma_{G,j} = 1.35$

$Q_{k1} = 500 + 100 = 600\ \mathrm{kN/m}\ (='Q_{k1} + Q_{k1,NR}')$ $\quad e_{Q1} = 0.15\ \mathrm{m}$

$Q_{k2} = 25\ \mathrm{kN/m}\ (= H_{berth})$ $\quad e_{Q2} = 0.80\ \mathrm{m}$

$\gamma_{Q,i} = 1.50$

$\psi_{0,2} = 0.90$

$$\Rightarrow M_d = 1.35 \cdot \left[50 \cdot 0.15 + 20 \cdot (-0.45)\right] + 1.50 \cdot 600 \cdot 0.15 + 1.50 \cdot 0.90 \cdot 25 \cdot 0.80$$

$$= 159.98\ \mathrm{kNm/m}$$

4.2.2. Fatigue situation

Persistent design situation, with the following combination of actions

$$F_{d,frequ} = G_k + \psi_1 \left(Q_{k1} + Q_{k1,NR}\right) + \Sigma\, \psi_{2,i} \left(Q_{k,i} + Q_{k,i,NR}\right)$$

can be rewritten as

$$F_{d,frequ} = \Sigma\, G_{k,j} + \psi_1 \left(Q_{k1} + Q_{k1,NR}\right) + \Sigma\, \psi_{2,i} \left(Q_{k,i} + Q_{k,i,NR}\right)$$

where

$G_{k1} = 25\ \mathrm{kN/m}$ (weight of the concrete capping beam)

$G_{k2} = 50\ \mathrm{kN/m}$

$G_{k3} = 20\ \mathrm{kN/m}$

$Q_{k1} = 500\ \mathrm{kN/m}$

$Q_{k1,NR} = 100\ \mathrm{kN/m}$

$\psi_1 = 0.80$

$\Rightarrow F_{d,frequ} = (25 + 50 + 20) + 0.80 \cdot (500 + 100) = 575.00\ \mathrm{kN/m}$

$$F_{d,frequ,NR} = \psi_1\ Q_{k1,NR} + \Sigma\, \psi_{2,i}\ Q_{k,i,NR}$$

$Q_{k1,NR} = 100\ \mathrm{kN/m}$

$\psi_1 = 0.80$

$\Rightarrow F_{d,frequ,NR} = 0.80 \cdot 100 = 80.00\ \mathrm{kN/m}$

$$M_{d,frequ} = \Sigma\, G_{k,j}\, e_{G,j} + \psi_1 \left(Q_{k1} + Q_{k1,NR}\right) e_{Q1} + \Sigma\, \psi_{2,i}\, (Q_{k,i} + Q_{k,i,NR})\, e_{Q,i}$$

where

$G_{k1} = 25\ \mathrm{kN/m}$ $\quad e_{G1} = 0.00\ \mathrm{m}$

$G_{k2} = 50\ \mathrm{kN/m}$ $\quad e_{G2} = 0.15\ \mathrm{m}$

$G_{k3} = 20\ \mathrm{kN/m}$ $\quad e_{G3} = -0.45\ \mathrm{m}$

$Q_{k1} = 500\ \mathrm{kN/m}$ $\quad e_{Q1} = 0.15\ \mathrm{m}$

$Q_{k1,NR} = 100\ \mathrm{kN/m}$ $\quad e_{Q1,NR} = 0.15\ \mathrm{m}$

$\psi_1 = 0.80$

$Q_{k2} = 25\ \mathrm{kN/m}\ (= H_{berth})$ $\quad e_{Q2} = 0.80\ \mathrm{m}$

$Q_{k2,NR} = 0\ \mathrm{kN/m}$

$\psi_{2,2} = 0.70$

$$\Rightarrow M_{d,frequ} = \left[50 \cdot 0.15 + 20 \cdot (-0.45)\right] + 0.80 \cdot (500 + 100) \cdot 0.15 + 0.70 \cdot (25 + 0) \cdot 0.80$$

$$= 84.50\ \mathrm{kNm/m}$$

$$M_{d,frequ,NR} = \psi_1\ Q_{k1,NR}\ e_{Q,1} = 0.80 \cdot 100 \cdot 0.15 = 12.00\ \mathrm{kNm/m}$$

4.3. Verifications: analytical method

4.3.1. Static situation

a) $F_d \leq F_{Rd,m}$

with

$F_{Rd,m} = 2\,094$ kN/m (see Figure 4.3.)

$F_d \leq F_{Rd,m} \Rightarrow 1\,028.25$ kN/m $\leq 2\,094$ kN/m ✓OK

Optimization factor

$$\frac{F_d}{F_{Rd,m}} = \frac{1\,028.25}{2094} = 0.49$$

b) $H_d \leq H_{Rd,K}$

with

$H_{Rd,K} = 222$ kN/m (see Chapter 7)

$H_d \leq H_{Rd,K} \Rightarrow 33.75$ kN/m ≤ 222 kN/m ✓OK

Optimization factor

$$\frac{H_d}{H_{Rd,K}} = \frac{33.75}{222} = 0.15$$

c) $M_d \leq M_{Rd}\{F_d\}$

with

$M_{Rd,K} = 31.0$ kNm/m (see Figure 4.3.)

and

$M_{Rd,S} = 158.9$ kNm/m (see Figure 4.3.)

$$M_{Rd,K}\{F_d\} = M_{Rd,K} \cdot \left(1 - \frac{F_d}{F_{Rd,m}}\right) = 31.0 \cdot \left(1 - \frac{1\,028.25}{2094}\right) = 15.78 \text{ kNm/m}$$

As

$$\frac{F_{Rd,m}}{2} = 1047 \text{ kN/m} \geq 1\,028.25 \text{ kN/m} = F_d$$

$$\Rightarrow M_{Rd,S}\{F_d\} = 2 \cdot M_{Rd,S} \cdot \left(\frac{F_d}{F_{Rd,m}}\right) = 2 \cdot 158.9 \cdot \frac{1\,028.25}{2094} = 156.05 \text{ kNm/m}$$

$$\Rightarrow M_{Rd}\{F_d\} = M_{Rd,K}\{F_d\} + M_{Rd,S}\{F_d\} = 15.78 + 156.05 = 171.83 \text{ kNm/m}$$

Finally

$M_d = 159.98$ kNm/m $\leq M_{Rd}\{F_d\} = 171.83$ kNm/m ✓OK

Optimization factor

$$\frac{M_d}{M_{Rd}\{F_d\}} = \frac{159.98}{171.83} = 0.93$$

4.3.2. Fatigue situation

a) $F_{d,frequ} \leq F_{Rd,m,fat}$

with

$F_{Rd,m,fat} = r_{fat,FM} \cdot F_{Rd,m}$

and

$$r_{fat,FM} = \frac{k_c}{1.22 + \eta_{NR,FM}}$$

and

$$\eta_{NR,FM} = \frac{\frac{F_{d,frequ,NR}}{A} + \frac{M_{d,frequ,NR}}{W}}{\frac{F_{d,frequ}}{A} + \frac{M_{d,frequ}}{W}}$$

where

$k_c = 0.98$

$A = 176.0$ cm^2/m (see Figure 4.3.)

$W_{el} = 2\,670$ cm^3/m (see Figure 4.3.)

So that

$$\eta_{NR,FM} = \frac{\frac{80.00}{176.0} \cdot 10 + \frac{12.00}{2670} \cdot 1\,000\ [\text{MPa}]}{\frac{575.00}{176.0} \cdot 10 + \frac{84.50}{2670} \cdot 1\,000\ [\text{MPa}]} = 0.1405$$

$$r_{fat,FM} = \frac{0.98}{1.22 + 0.1405} = 0.7203$$

$$F_{Rd,m,fat} = 0.7203 \cdot 2094 = 1\,508.31 \text{ kN/m}$$

$\Rightarrow F_{d,frequ} = 575$ kN/m $\leq F_{Rd,m,fat} = 1\,508.31$ kN/m $\Rightarrow$ ✓OK

$$\frac{F_{d,frequ}}{F_{Rd,m,fat}} = \frac{575}{1\,508.31} = 0.38$$

b) $M_{d,frequ}\{F_{d,frequ}\} \leq M_{Rd,fat}\{F_{d,frequ}\}$

with

$$M_{Rd,fat}\{F_{d,frequ}\} = M_{Rd,K,fat}\{F_{d,frequ}\} + M_{Rd,S,fat}\{F_{d,frequ}\}$$

1) $M_{Rd,K,fat}\{F_{d,frequ}\}$

As

$$\eta_{NR,MK} = \frac{M_{d,frequ,NR}}{M_{d,frequ}} = \frac{12.00}{84.50} = 0.1420$$

$$\Rightarrow r_{fat,MK} = \frac{k_c}{1.22 + \eta_{NR,MK}} = \frac{0.98}{1.22 + 0.1420} = 0.7195$$

and

$M_{Rd,K} = 31.0$ kNm/m

$$\Rightarrow M_{Rd,K,fat} = r_{fat,MK} \cdot M_{Rd,K} = 0.7195 \cdot 31.0 = 22.30 \text{ kNm/m}$$

$$\Rightarrow M_{Rd,K,fat}\{F_{d,frequ}\} = M_{Rd,K,fat} \cdot \left(1 - \frac{F_{d,frequ}}{F_{Rd,m,fat}}\right)$$

$$\Rightarrow M_{Rd,K,fat}\{F_{d,frequ}\} = 22.30 \cdot \left(1 - \frac{575.0}{1\,508.31}\right) = 13.80 \text{ kNm/m}$$

2) $M_{Rd,S,fat}\left\{F_{d,frequ}\right\}$

As

$$F_{d,frequ} = 575.0 \text{ kN/m} \leq 754.2 \text{ kN/m} = \frac{F_{Rd,m,fat}}{2}$$

then

$$M_{Rd,S,fat}\left\{F_{d,frequ}\right\} = 2 \cdot M_{Rd,S,fat} \cdot \left(\frac{F_{d,frequ}}{F_{Rd,m,fat}}\right)$$

As

$M_{Rd,S} = 158.9$ kNm/m (see Figure 4.3.)

$$\Rightarrow M_{Rd,S,fat} = r_{fat,FM} \cdot M_{Rd,S} = 0.7203 \cdot 158.9 = 114.46 \text{ kNm/m}$$

$$\Rightarrow M_{Rd,S,fat}\left\{F_{d,frequ}\right\} = 2 \cdot 114.46 \cdot \left(\frac{575.0}{1\,508.31}\right) = 87.27 \text{ kNm/m}$$

3) $M_{Rd,fat}\left\{F_{d,frequ}\right\}$

$$\Rightarrow M_{Rd,fat}\left\{F_{d,frequ}\right\} = M_{Rd,K,fat}\left\{F_{d,frequ}\right\} + M_{Rd,S,fat}\left\{F_{d,frequ}\right\}$$

$$= 13.80 + 87.27 = 101.07 \text{ kNm/m}$$

Finally

$$M_{d,frequ}\left\{F_{d,frequ}\right\} = 84.50 \text{ kNm/m} \leq 101.07 \text{ kNm/m} = M_{Rd,fat}\left\{F_{d,frequ}\right\}$$ ✓OK

Optimization factor

$$\frac{M_{d,frequ}\left\{F_{d,frequ}\right\}}{M_{Rd,fat}\left\{F_{d,frequ}\right\}} = \frac{84.50}{101.07} = 0.84$$

Note

This load case is not the most unfavourable load case for the fatigue situation! See Chapter 4.5.

4.4. Verifications: diagram method

For the AZ 27-800, a capping beam of class C 30/37 or above and a fixed connection, the graphs elaborated based on Annex 3 and Annex 4 of the '*Allgemeine Bauartgenehmigung*' are shown below. The dot represents the design value of above example in the static, respectively in the 'fatigue' case.

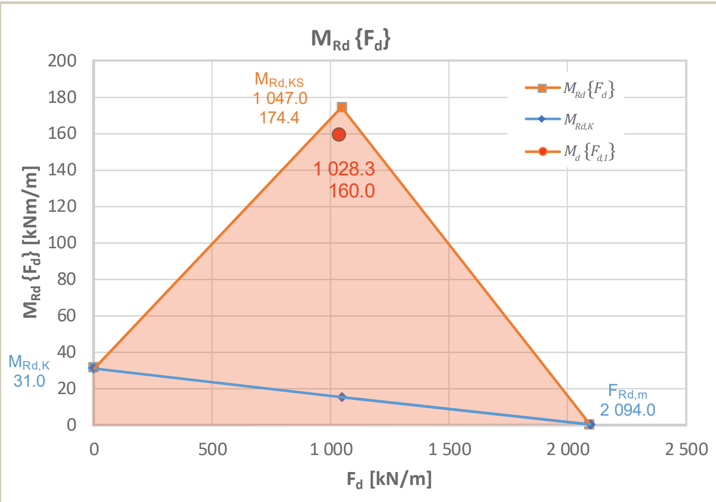


Figure 4.4. Static situation with $M_d\{1\,028.3 \text{ kN/m}\} = 160.0 \text{ kNm/m}$ (AZ 27-800).

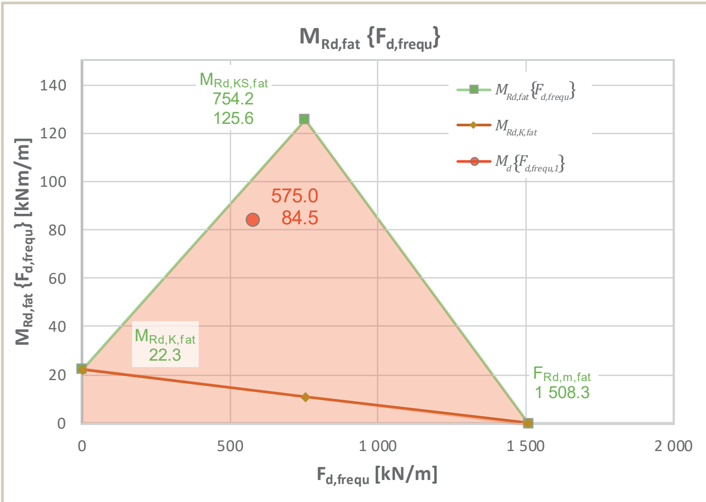


Figure 4.5. Fatigue situation with $M_{d,frequ}\{575.0 \text{ kN/m}\} = 84.5 \text{ kNm/m}$ (AZ 27-800).

4.5. Key conclusion: verification of all load cases

Reminder: the verification of all the possible combinations is quite complex and time-consuming. The combination which seem to be the most unfavourable, for instance the one that takes into account all the loads, is not always the combination that may be governing every verification.

ArcelorMittal strongly recommends using the software VLoad® for the Knife Edge Support analysis, followed if suited or required, by a hand-calculation of the most unfavourable load cases identified with *VLoad*.

For instance, in the quite simple example analysed, the report of *VLoad* shows that load case LC 0059 yields a higher ratio $M_d / M_{Rd,m} = 0.97$ compared to the result calculated in 4.3.1. which is only 0.93!

However, $F_d / F_{Rd,m} = 1\,028.25 / 2\,094 = 0.49$ calculated in previous Chapter is the same as in *VLoad* (LC 0016 & LC 0032).

Table 1. displays the loads F_d and moments M_d for above example for all load cases.

Table 2. displays the details of the combinations of actions for all the load cases (1 to 64!!).

As a conclusion, different load cases may be governing the selection of the reinforcing bars of different positions!

Load case	F_d kN/m	M_d kNm/m	$M_{Rd,K}\{F_d\}$ kNm/m	$M_{Rd,S}\{F_d\}$ kNm/m	$M_{Rd}\{F_d\}$ kNm/m	$M_d / M_{Rd}\{F_d\}$ -
0001	95.0	1.5	29.6	14.4	44.0	0.03
0002	103.8	1.5	29.5	15.7	45.2	0.03
0003	112.5	1.1	29.3	17.1	46.4	0.02
0004	121.3	1.1	29.2	18.4	47.6	0.02
0005	102.0	4.7	29.5	15.5	45.0	0.10
0006	110.8	4.7	29.4	16.8	46.2	0.10
0007	119.5	2.0	29.2	18.1	47.4	0.04
0008	128.3	2.0	29.1	19.5	48.6	0.04
0009	995.0	133.5	16.3	151.0	167.3	0.80
0010	1 003.8	133.5	16.1	152.3	168.5	0.79
0011	1 012.5	136.1	16.0	153.7	169.7	0.80
0012	1 021.3	136.1	15.9	155.0	170.9	0.80
0013	1 002.0	130.4	16.2	152.1	168.2	0.77
0014	1 010.8	130.4	16.0	153.4	169.4	0.77
0015	1 019.5	133.0	15.9	154.7	170.6	0.78
0016	**1 028.3**	**133.0**	**15.8**	**156.1**	**171.8**	**0.77**
0017	95.0	25.5	29.6	14.4	44.0	0.58
0018	103.8	25.5	29.5	15.7	45.2	0.56
0019	112.5	28.1	29.3	17.1	46.4	0.61
0020	121.3	28.1	29.2	18.4	47.6	0.59
0021	102.0	22.4	29.5	15.5	45.0	0.50
0022	110.8	22.4	29.4	16.8	46.2	0.48
0023	119.5	25.0	29.2	18.1	47.4	0.53
0024	128.3	25.0	29.1	19.5	48.6	0.51
0025	995.0	160.5	16.3	151.0	167.3	0.96
0026	1 003.8	160.5	16.1	152.3	168.5	0.95
0027	**1 012.5**	**163.1**	**16.0**	**153.7**	**169.7**	**0.96**
0028	1 021.3	163.1	15.9	155.0	170.9	0.95
0029	1 002.0	157.4	16.2	152.1	168.2	0.94
0030	1 010.8	157.4	16.0	153.4	169.4	0.93
0031	1 019.5	160.0	15.9	154.7	170.6	0.94
0032	**1 028.3**	**160.0**	**15.8**	**156.1**	**171.8**	**0.93**
0033	95.0	1.5	29.6	14.4	44.0	0.03
0034	103.8	1.5	29.5	15.7	45.2	0.03
0035	112.5	1.1	29.3	17.1	46.4	0.02
0036	121.3	1.1	29.2	18.4	47.6	0.02
0037	102.0	4.7	29.5	15.5	45.0	0.10
0038	110.8	4.7	29.4	16.8	46.2	0.10
0039	119.5	2.0	29.2	18.1	47.4	0.04
0040	128.3	2.0	29.1	19.5	48.6	0.04
0041	905.0	120.0	17.6	137.3	155.0	0.77
0042	913.8	120.0	17.5	138.7	156.1	0.77
0043	922.5	122.6	17.3	140.0	157.3	0.78
0044	931.3	122.6	17.2	141.3	158.5	0.77
0045	912.0	116.9	17.5	138.4	155.9	0.75
0046	920.8	116.9	17.4	139.7	157.1	0.74
0047	929.5	119.5	17.2	141.1	158.3	0.75
0048	938.3	119.5	17.1	142.4	159.5	0.75
0049	95.0	28.5	29.6	14.4	44.0	0.65
0050	103.8	28.5	29.5	15.7	45.2	0.63
0051	112.5	31.1	29.3	17.1	46.4	0.67
0052	121.3	31.1	29.2	18.4	47.6	0.65
0053	102.0	25.4	29.5	15.5	45.0	0.56
0054	110.8	25.4	29.4	16.8	46.2	0.55
0055	119.5	28.0	29.2	18.1	47.4	0.59
0056	128.3	28.0	29.1	19.5	48.6	0.58
0057	905.0	150.0	17.6	137.3	155.0	0.97
0058	913.8	150.0	17.5	138.7	156.1	0.96
0059	**922.5**	**152.6**	**17.3**	**140.0**	**157.3**	**0.97**
0060	931.3	152.6	17.2	141.3	158.5	0.96
0061	912.0	146.9	17.5	138.4	155.9	0.94
0062	920.8	146.9	17.4	139.7	157.1	0.93
0063	929.5	149.5	17.2	141.1	158.3	0.94
0064	938.3	149.5	17.1	142.4	159.5	0.94

Table 1. Excerpt of the report from *VLoad*: $M_d / M_{Rd}\{F_d\}$ for all LC (load cases).

Load case		F kN/m	H kN/m	M kNm/m
Permanent and temporary combinations				
0001	1.0 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G	95.0	0.0	-1.5
0002	1.35 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G	103.8	0.0	-1.5
0003	1.0 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G	112.5	0.0	1.1
0004	1.35 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G	121.3	0.0	1.1
0005	1.0 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G	102.0	0.0	-4.7
0006	1.35 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G	110.8	0.0	-4.7
0007	1.0 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G	119.5	0.0	-2.0
0008	1.35 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G	128.3	0.0	-2.0
0009	1.0 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Crane_v_Q	995.0	0.0	133.5
0010	1.35 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Crane_v_Q	1 003.8	0.0	133.5
0011	1.0 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Crane_v_Q	1 012.5	0.0	136.1
0012	1.35 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Crane_v_Q	1 021.3	0.0	136.1
0013	1.0 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Crane_v_Q	1 002.0	0.0	130.4
0014	1.35 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Crane_v_Q	1 010.8	0.0	130.4
0015	1.0 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Crane_v_Q	1 019.5	0.0	133.0
0016	**1.35 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Crane_v_Q**	**1 028.3**	**0.0**	**133.0**
0017	1.0 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Berthing_h_Q	95.0	33.8	25.5
0018	1.35 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Berthing_h_Q	103.8	33.8	25.5
0019	1.0 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Berthing_h_Q	112.5	33.8	28.1
0020	1.35 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Berthing_h_Q	121.3	33.8	28.1
0021	1.0 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Berthing_h_Q	102.0	33.8	22.4
0022	1.35 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Berthing_h_Q	110.8	33.8	22.4
0023	1.0 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Berthing_h_Q	119.5	33.8	25.0
0024	1.35 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Berthing_h_Q	128.3	33.8	25.0
0025	1.0 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Crane_v_Q + 1.5 * 0.90 * Berthing_h_Q	995.0	33.8	160.5
0026	1.35 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Crane_v_Q + 1.5 * 0.90 * Berthing_h_Q	1 003.8	33.8	160.5
0027	**1.0 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Crane_v_Q + 1.5 * 0.90 * Berthing_h_Q**	**1 012.5**	**33.8**	**163.1**
0028	1.35 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Crane_v_Q + 1.5 * 0.90 * Berthing_h_Q	1 021.3	33.8	163.1
0029	1.0 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Crane_v_Q + 1.5 * 0.90 * Berthing_h_Q	1 002.0	33.8	157.4
0030	1.35 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Crane_v_Q + 1.5 * 0.90 * Berthing_h_Q	1 010.8	33.8	157.4
0031	1.0 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Crane_v_Q + 1.5 * 0.90 * Berthing_h_Q	1 019.5	33.8	160.0
0032	**1.35 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Crane_v_Q + 1.5 * 0.90 * Berthing_h_Q**	**1 028.3**	**33.8**	**160.0**
0033	1.0 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G	95.0	0.0	-1.5
0034	1.35 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G	103.8	0.0	-1.5
0035	1.0 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G	112.5	0.0	1.1
0036	1.35 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G	121.3	0.0	1.1
0037	1.0 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G	102.0	0.0	-4.7
0038	1.35 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G	110.8	0.0	-4.7
0039	1.0 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G	119.5	0.0	-2.0
0040	1.35 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G	128.3	0.0	-2.0
0041	1.0 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q	905.0	0.0	120.0
0042	1.35 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q	913.8	0.0	120.0
0043	1.0 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q	922.5	0.0	122.6
0044	1.35 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q	931.3	0.0	122.6
0045	1.0 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q	912.0	0.0	116.9
0046	1.35 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q	920.8	0.0	116.9
0047	1.0 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q	929.5	0.0	119.5
0048	1.35 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q	938.3	0.0	119.5
0049	1.0 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Berthing_h_Q	95.0	37.5	28.5
0050	1.35 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Berthing_h_Q	103.8	37.5	28.5
0051	1.0 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Berthing_h_Q	112.5	37.5	31.1

Load case		F	H	M
		kN/m	kN/m	kNm/m
0052	1.35 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * Berthing_h_Q	121.3	37.5	31.1
0053	1.0 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Berthing_h_Q	102.0	37.5	25.4
0054	1.35 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Berthing_h_Q	110.8	37.5	25.4
0055	1.0 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Berthing_h_Q	119.5	37.5	28.0
0056	1.35 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * Berthing_h_Q	128.3	37.5	28.0
0057	1.0 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q + 1.5 * Berthing_h_Q	905.0	37.5	150.0
0058	1.35 * DL + 1.0 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q + 1.5 * Berthing_h_Q	913.8	37.5	150.0
0059	**1.0 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q + 1.5 * Berthing_h_Q**	**922.5**	**37.5**	**152.6**
0060	1.35 * DL + 1.35 * Crane_v_G + 1.0 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q + 1.5 * Berthing_h_Q	931.3	37.5	152.6
0061	1.0 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q + 1.5 * Berthing_h_Q	912.0	37.5	146.9
0062	1.35 * DL + 1.0 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q + 1.5 * Berthing_h_Q	920.8	37.5	146.9
0063	1.0 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q + 1.5 * Berthing_h_Q	929.5	37.5	149.5
0064	1.35 * DL + 1.35 * Crane_v_G + 1.35 * Bollard_v_G + 1.5 * 0.90 * Crane_v_Q + 1.5 * Berthing_h_Q	938.3	37.5	149.5
Frequent combinations				
0065	Gk + 1.0 * 0.80 * Crane_v_Q	575.0	-	70.5
0066	Gk + 1.0 * 0.80 * Crane_v_Q + 1.0 * 0.70 * Berthing_h_Q	575.0	-	84.5
0067	Gk + 1.0 * 0.80 * Berthing_h_Q	95.0	-	14.5
0068	Gk + 1.0 * 0.70 * Crane_v_Q + 1.0 * 0.80 * Berthing_h_Q	515.0	-	77.5
Frequent combinations of the non-static load contents				
0069	1.0 * 0.80 * Crane_v_Q	80.0	-	12.0

DL = dead load, Gk = characteristic value of the permanent loads

Table 2. Excerpt of the report from *VLoad*: Load cases LC.

Figure 4.6. shows the diagram method for $M_d\{F_d\}$ for all the load cases (dots).

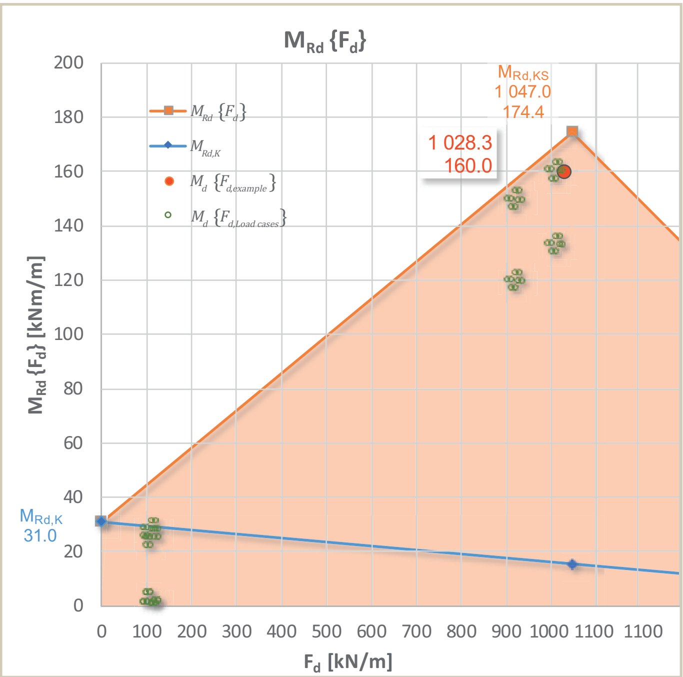


Figure 4.6. Static situation with $M_d\{F_d\}$ for all load cases (AZ 27-800).

4.6. Reinforcement calculation

In order to calculate the necessary reinforcement, it is important to determine $M_{d,K}$ and $M_{d,S}$.

As $M_d = 160.00$ kNm/m > 156.05 kNm/m $= M_{Rd,S}\{F_d\}$

$$\Rightarrow \begin{cases} M_{d,S} = M_{Rd,S}\{F_d\} \\ M_{d,K} = M_d - M_{d,S} \end{cases}$$

Also, in the fatigue situation the moment distribution follows the expression

$$M_d^* = 6.21 \cdot M_{d,frequ,NR} = 6.21 \cdot 12.00 = 74.52 \text{ kNm/m}$$

As

$$M_d^* = 74.52 \text{ kNm/m} \leq 541.95 \text{ kNm/m} = 6.21 \cdot 87.27 = 6.21 \cdot M_{Rd,S,fat}\{F_{d,frequ}\}$$

$$\Rightarrow \begin{cases} M_{d,S}^* = M_d^* \\ M_{d,K}^* = 0 \end{cases}$$

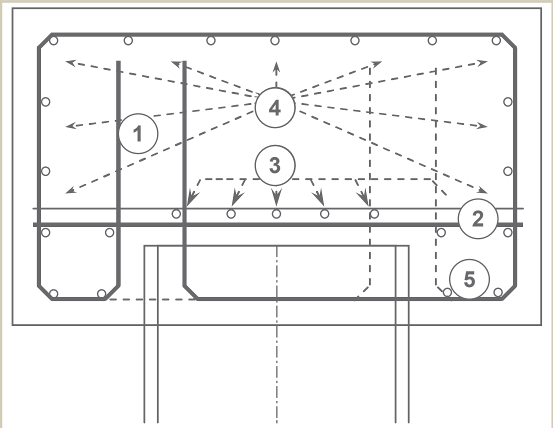


Figure 4.7. Reinforcement detailing for a 'fixed' connection.

4.6.1. Stirrups (Pos. 1)

- minimum bar diameter of $d_s = 10$ mm
- maximum distance between stirrups $a = 15$ cm $\Rightarrow$ 6.67 bars / m

$$a_{s,min} = 6.67 \left(\pi \left(\frac{d_s}{2} \right)^2 \right) = 5.24 \text{ cm}^2/\text{m}$$

4.6.1.1. Static situation

$$a_{Bü,K} = k_{BM} \cdot M_{d,K} + k_{BH} \cdot H_d$$

with

$$k_{BM} = 0.275 \frac{\text{cm}^2/\text{m}}{\text{kNm/m}}$$

$$M_{d,K} = M_d - M_{d,S} = 160.00 - 156.05 = 3.95 \text{ kNm/m}$$

$$k_{BH} = 0.013 \frac{\text{cm}^2/\text{m}}{\text{kNm/m}}$$

$$H_d = 33.75 \text{ kN/m}$$

$$a_{Bü,K} = 0.275 \cdot 3.95 + 0.013 \cdot 33.75 = 1.53 \text{ cm}^2/\text{m}$$

Note: this load case is not the most unfavourable for H_d!

4.6.1.2. Fatigue situation

$$a_{Bü,K} = k_{BM} \cdot M_{d,K}^* = 0.275 \cdot 0 = 0$$

4.6.1.3. Static & fatigue situation

$$a_{min} = 5.24 \text{ cm}^2/\text{m}$$

$$a_{Bü,K} = 1.53 \text{ cm}^2/\text{m}$$

$$\boxed{\Rightarrow a_1 : d_s = 10 \text{ mm} / 15 \text{ cm} \Rightarrow 5.24 \text{ cm}^2/\text{m}}$$

Note

ArcelorMittal recommends using at least $d_s = 12$ mm (every 15 cm).

4.6.2. Transversal splitting reinforcement (Pos. 2)

4.6.2.1. Static situation

The transversal splitting reinforcement is $a_{SpQ} + \Delta a_{SpQ}$

a) $a_{SpQ} = k_{QF} \cdot F_d + k_{QM} \cdot M_{d,S}$

$$k_{QF} = 10.42 \frac{\text{cm}^2/\text{m}}{\text{MN/m}} \quad (\text{AZ } 27-800)$$

$$F_d = 1\,028.25 \text{ kN/m}$$

$$k_{QM} = 0.067 \frac{\text{cm}^2/\text{m}}{\text{kNm/m}} \quad (\text{AZ } 27-800)$$

$$M_{d,S} = 156.05 \text{ kNm/m}$$

$$\Rightarrow a_{SpQ} = 10.42 \cdot \frac{1\,028.25}{1\,000} + 0.067 \cdot 156.05 = 21.17 \text{ cm}^2/\text{m}$$

b) $\Delta a_{SpQ} = k_{QK} \cdot M_{d,K} + k_{QH} \cdot H_d$

$$k_{QK} = 0.230 \frac{\text{cm}^2/\text{m}}{\text{kNm/m}}$$

$$M_{d,K} = 3.95 \text{ kNm/m}$$

$$k_{QH} = 0.023 \frac{\text{cm}^2/\text{m}}{\text{kN/m}}$$

$H_d = 1.5 \cdot 0.9 \cdot 25 = 33.75$ kN/m $(= \gamma_{Q,i} \; \psi_{0,i} \; Q_{k,i}$ in this load case)

Note

H_d = value from the load combination analysed

$(M_d = \sum \gamma_{G,j} G_{k,j} e_{G,j} + \gamma_{Q1} Q_{k1} e_{Q1} + \sum \gamma_{Q,i} \psi_{0,i} Q_{k,i} e_{Q,i})$; this value may differ from the load H_d to consider in the most unfavourable verification!

$$\Rightarrow \Delta a_{SpQ} = 0.230 \cdot 3.95 + 0.023 \cdot 33.75 = 1.68 \text{ cm}^2/\text{m}$$

c) $a_{SpQ} + \Delta a_{SpQ}$

$$a_{SpQ} + \Delta a_{SpQ} = 21.17 + 1.68 = 22.85 \text{ cm}^2/\text{m}$$

4.6.2.2. Fatigue situation

The transversal splitting reinforcement is equal to $a_{SpQ} + \Delta a_{SpQ}$

a) $a_{SpQ} = k_{QF} \cdot F_d^* + k_{QM} \cdot M_{d,S}^*$

$$F_d^* = 6.21 \cdot F_{d,frequ,NR} = 6.21 \cdot 80.00 = 496.80 \text{ kN/m}$$

$$M_d^* = 6.21 \cdot M_{d,frequ,NR} = 6.21 \cdot 12.00 = 74.52 \text{ kNm/m}$$

$$\Rightarrow a_{SpQ} = 10.42 \cdot \frac{496.80}{1\,000} + 0.067 \cdot 74.52 = 10.17 \text{ cm}^2/\text{m}$$

b) $\Delta a_{SpQ} = k_{QK} \cdot M_{d,K}^*$

$$k_{QK} = 0.230 \frac{\text{cm}^2/\text{m}}{\text{kNm/m}}$$

$$M_{d,K}^* = 0 \text{ kNm/m}$$

$$\Rightarrow \Delta a_{SpQ} = 0.230 \cdot 0 = 0 \text{ cm}^2/\text{m}$$

c) $a_{SpQ} + \Delta a_{SpQ}$

$$a_{SpQ} + \Delta a_{SpQ} = 10.17 + 0 = 10.17 \text{ cm}^2/\text{m}$$

4.6.2.3. Static & fatigue situation

$$a_{SpQ} + \Delta a_{SpQ} = 22.85 \text{ cm}^2/\text{m}$$

Note

This load case is not the most unfavourable load case for the verification of Pos. 2 (See Chapter 4.5). Load case LC 0027 governs the design with the maximum value of

$$a_{SpQ} + \Delta a_{SpQ} = 20.85 + 2.94 = 23.79 \text{ cm}^2/\text{m}$$

$$\boxed{\Rightarrow a_2 : 2 \text{ layers of } d_s = 14 \text{ mm} / 12.5 \text{ cm} = 24.63 \text{ cm}^2/\text{m}}$$

4.6.3. Longitudinal splitting reinforcement (Pos. 3)

- minimum bar diameter $d_s = 10$ mm
- maximum distance between bars $a = 15$ cm
- minimum amount of reinforcement bars is 3 bars of $d_s = 10$ mm

4.6.3.1. Static situation

$$A_{SpL} = k_{LF} \cdot F_d$$

$$k_{LF} = 5.41 \frac{\text{cm}^2}{\text{MN/m}} \text{ (AZ 27-800)}$$

$$F_d = 1\,028.25 \text{ kN/m}$$

$$\Rightarrow A_{SpL} = 5.41 \cdot \frac{1\,028.25}{1\,000} = 5.56 \text{ cm}^2$$

4.6.3.2. Fatigue situation

$$A_{SpL} = k_{LF} \cdot F_d^*$$

$$F_d^* = 496.80 \text{ kN/m}$$

$$\Rightarrow A_{SpL} = 5.41 \cdot \frac{496.80}{1\,000} = 2.69 \text{ cm}^2$$

4.6.3.3. Static & fatigue situation

$$A_{SpL} = 5.56 \text{ cm}^2$$

$$\Rightarrow \quad a_3 : 5 \text{ bars of } d_s = 12 \text{ mm} = 5.65 \text{ cm}^2$$

As the 'depth'[14] of the AZ 27-800 is 476 mm

$$\Rightarrow \text{distance of bars} \approx \frac{476}{4} = 11.9 \text{ cm} \le 15 \text{ cm} \text{ (max 15 cm)}$$

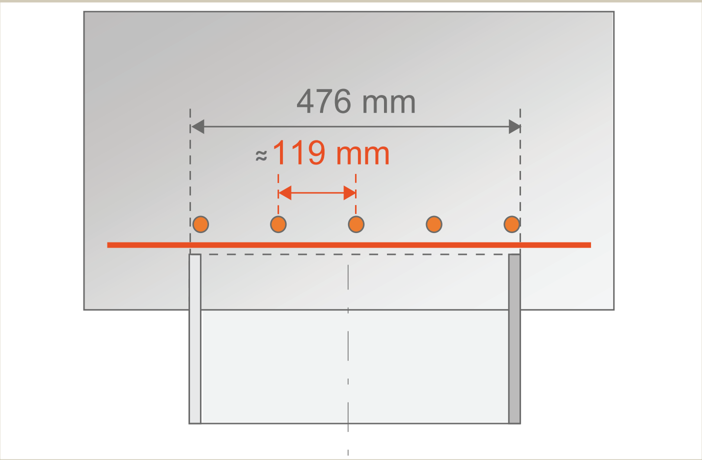


Figure 4.8. Distribution of rebars of Pos. 3.

[14] Labelled as 'height' in the general catalogue.

4.6.4. Longitudinal reinforcement (Pos. 4)

- minimum bar diameter $d_s = 10$ mm
- maximum distance between bars $a = 15$ cm
- minimum amount of reinforcing bars
  - lateral 3 bars of $d_s = 10$ mm
  - top 5 bars of $d_s = 10$ mm

Hence

$$\Rightarrow \text{number of top bars} = \text{RoundUp}\left(\frac{\text{width} - 2 \cdot \text{cover} - 2 \cdot \text{diameter stirrups} - 2 \cdot \frac{\text{diameter bars}}{2}}{\text{a}}\right) + 1$$

$$\Rightarrow \begin{aligned} \text{number of top bars} &= \text{RoundUp}\left(\frac{1\,000 - 2 \cdot 55 - 2 \cdot 10 - 2 \cdot \frac{10}{2}}{150}\right) + 1 \\ &= \text{RoundUp}\,(5.73) + 1 = 7 \end{aligned}$$

Similarly, for lateral bars

$$\Rightarrow \text{number of lateral bars} = \text{RoundUp}\left(\frac{1\,000 - 2 \cdot 55 - 2 \cdot 10 - 2 \cdot \frac{10}{2}}{150}\right) + 1$$

$$= \text{RoundUp}(5.73) + 1 = 7$$

Note

On the sketch from *VLoad*, the bars from the top and from the corbel (Pos. 5) contribute to the lateral reinforcement; hence only 5 bars are labelled as Pos. 4 on the lateral sides!

4.6.5. Longitudinal corbel reinforcement (Pos. 5)

- minimum bar diameter $d_s = 10$ mm
- minimum amount of reinforcing bars at the bottom is 2 bars of $d_s = 10$ mm on each side

4.7. Cross-section and steel reinforcing

The sketches below (Figure 4.9.) from the software *VLoad* show the chosen reinforcing bars. The spacing and quantity of rebars differs slightly from Chapter 4.6. because the software limits the choice of some parameters to predefined values based on the geometry of the chosen sheet pile.

In this example designed with *VLoad*, the spacing between stirrups and reinforcing splitting bars is 11.4 cm, instead of 15 cm from Chapter 4.6.1. and 12.5 cm from Chapter 4.6.2.

This choice increases slightly the amount of steel reinforcement, but simplifies the execution and installation of the stirrups of Pos. 1 and Pos. 2.

Note

From the design, $d_s = 10$ mm every 15 mm is sufficient for Pos. 1, but ArcelorMittal recommends using a diameter of $d_s = 12$ mm.

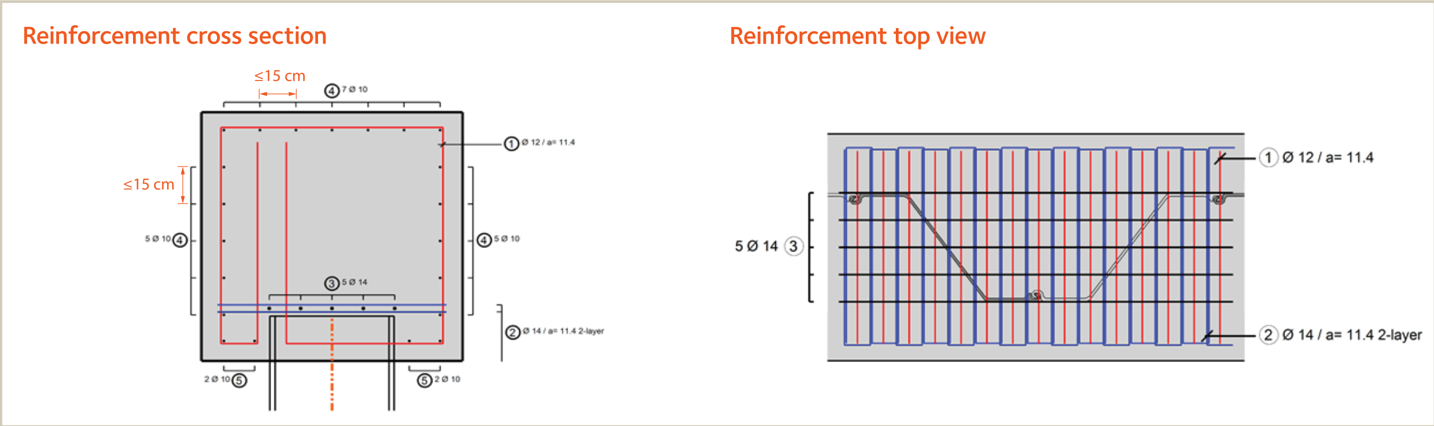


Figure 4.9. Cross section and top view – excerpt from the software *VLoad*.

Overall, the amount of proposed rebars in *VLoad* is shown in the Table 3.

Position	Required reinforcement	Selected reinforcement		Mass of rebars
Pos. 1	5.24 cm²/m	Ø 12 / 11.4 cm	9.92 cm²/m	38.5 kg/m
Pos. 2	23.80 cm²/m	Ø 14 / 11.4 cm | two-layer	27.01 cm²/m	21.0 kg/m
Pos. 3	5.56 cm²	5 Ø 14	7.70 cm²	6.0 kg/m
Pos. 4	nominal	17 Ø 10	13.35 cm²	10.5 kg/m
Pos. 5	nominal	4 Ø 10	3.14 cm²	2.5 kg/m

Table 3. Minimum reinforcement required versus proposed reinforcement in *VLoad*.

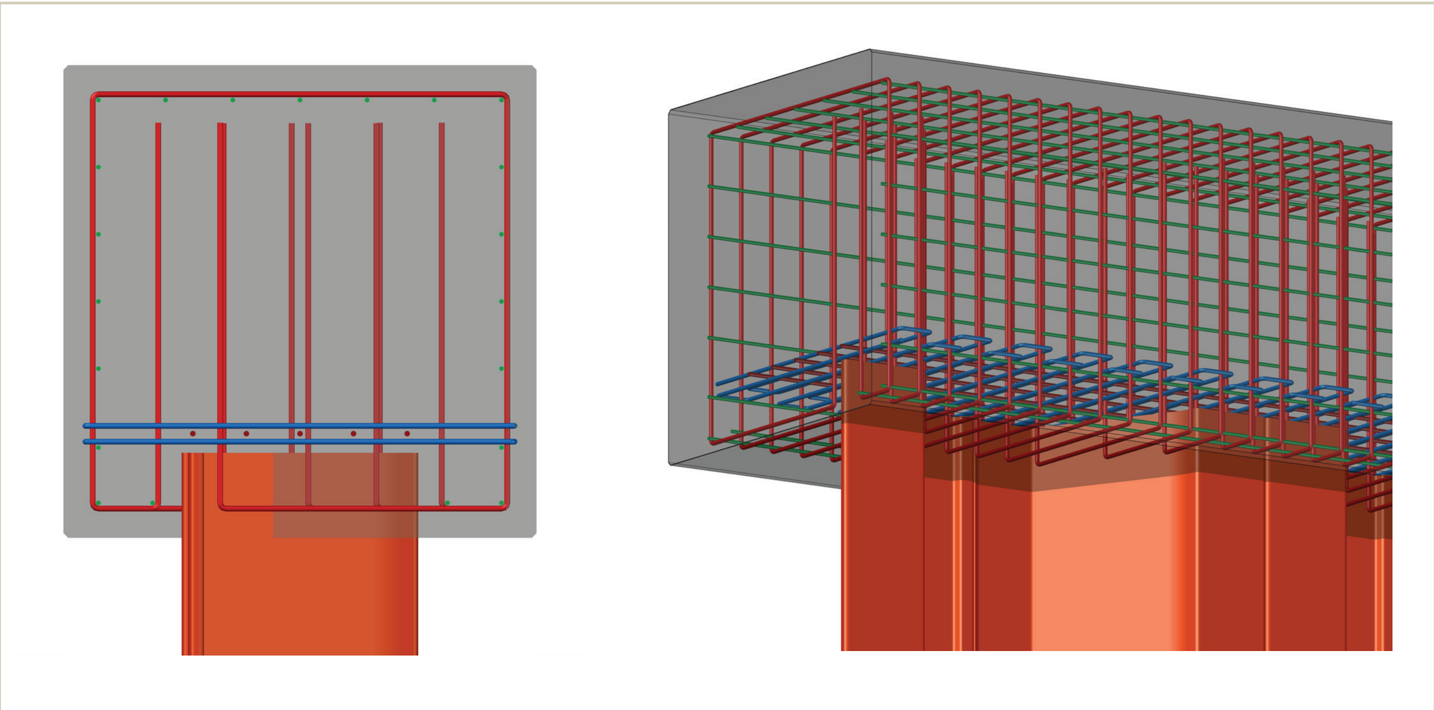


Figure 4.10. 3D view of the reinforcement (typical sketch).

5. Symbols and abbreviations

5.1. Symbols

Symbols used in this document are either based on the National Technical Approval or on the European standards.

$a_{Bü,K}$	area per meter of wall of the stirrup reinforcement
a_{SpQ}	area per meter of wall of the transversal splitting reinforcement
$e_{x,i}$	eccentricity of vertical load i
$e_{y,j}$	eccentricity of horizontal load j ($e_{y,j} > 0$)
f_{ck}	characteristic compressive cylinder strength of concrete at 28 days
k_c	constant, function of the concrete class
k_{BH}	constant associated to H_d
k_{BM}	constant associated to $M_{d,K}$ and $M^*_{d,K}$
k_{LF}	coefficient associated to F_d and F^*_d , depends on the sheet pile section
k_{QF}	coefficient associated to F_d and F^*_d , depends on the sheet pile section
k_{QH}	constant associated to H_d
k_{QK}	constant associated to $M_{d,K}$ and $M^*_{d,K}$
k_{QM}	coefficient associated to $M_{d,S}$ and $M^*_{d,S}$, depends on the sheet pile section
$n_{NR,FM}$	coefficient associated to $F_{Rd,m}$ and $M_{Rd,S}$ that accounts for the influence of the non-static component of an action
$n_{NR,MK}$	coefficient associated to $M_{Rd,K}$ that accounts for the influence of the non-static component of an action
$r_{fat,FM}$	reduction factor applied to $F_{Rd,m}$ and $M_{Rd,S}$
$r_{fat,MK}$	reduction factor applied to $M_{Rd,K}$, considering the influence of the non-static component of an action

A	cross sectional area of the sheet pile section
A_{SpL}	area of longitudinal splitting reinforcement
E_d	design value of effect of actions
$E_{d,frequ}$	design value of effect of actions taking into account static and non-predominantly-static action
$E_{d,frequ,NR}$	design value of the effect of the non-predominantly-static components of actions
F	resultant of vertical loads (action)
F_d	design value of the effect of vertical actions
F_i	vertical load i
$F_{d,frequ}$	design value of the combination of static and non-predominantly-static vertical action
$F_{d,frequ,NR}$	design value of the non-predominantly-static component of the vertical action
$F_{Rd,m}$	design resistance to vertical actions of the sheet pile section
$F_{Rd,m,fat}$	design resistance to vertical actions of the sheet pile section reduced by the effect of the non-predominantly-static vertical action
G	permanent action (weight)
G_k	characteristic value of permanent action
$G_{k,j}$	characteristic value of permanent action j
H	resultant of horizontal loads
H_d	design value of effect of horizontal actions
H_i	horizontal load i
$H_{Rd,K}$	design value of the resistance to horizontal actions
M	bending moments on the capping beam (on neutral axis of sheet pile wall)
M_d	design value of the effect of moments (due to eccentric actions)
$M_{d,frequ}\{F_{d,frequ}\}$	design value of the effect of moments due to the combination of static and non-predominantly-static vertical action

$M_{d,K}$	design value of the effect of moments resisted by the embedment of the sheet pile (corbel)
$M_{d,S}$	design value of the effect of moments transferred directly to the sheet pile (knife edge support)
$M_{Rd}\{F_d\}$	design moment resistance considering the effect of the vertical action
$M_{Rd,fat}\{F_{d,frequ}\}$	design moment resistance considering the effect of the non-predominantly-static vertical action
$M_{Rd,K}$	design value of the bending moment resistance of the sheet pile due to the embedment depth
$M_{Rd,K}\{F_d\}$	design value of the bending moment resistance of the sheet pile due to the embedment depth considering the effect of the vertical action
$M_{Rd,K,fat}\{F_{d,frequ}\}$	design value of the bending moment resistance of the sheet pile due to the embedment depth considering the effect of the non-predominantly-static vertical action
$M_{Rd,S}$	design value of the bending moment resistance of the sheet pile (knife edge support)
$M_{Rd,S}\{F_d\}$	design value of the bending moment resistance of the sheet pile considering the effect of the vertical action
$M_{Rd,S,fat}\{F_{d,frequ}\}$	design value of the bending moment resistance of the sheet pile considering the effect of the non-predominantly-static vertical action
Q_1	leading variable action
Q_i	variable action i
$Q_{k,1}$	characteristic value of the leading variable action
$Q_{k,i}$	characteristic value of the accompanying variable action i
$Q_{k,1,NR}$	non-predominantly-static component of the leading variable action
$Q_{k,i,NR}$	non-predominantly-static component of the variable action i
W	elastic section modulus of the steel sheet pile section
Δa_{SpQ}	additional transversal splitting reinforcement area per meter of wall
$\gamma_{G,j}$	partial factor for permanent action j
$\gamma_{Q,1}$	partial factor for the leading variable action
$\gamma_{Q,i}$	partial factor for variable action i
$\Psi_{0,i}$	factor for combination value of a variable action i
Ψ_1	factor for frequent value of a variable action
$\Psi_{2,i}$	factor for quasi-permanent value of a variable action i

5.2. Abbreviations

KES	Knife Edge Support
NTA	National Technical Approval
DIBt	Deutsches Institut für Bautechnik
ULS	Ultimate Limit State
OK	*Oberkante* = top of the element in the sketches

6. Annex 1 of the NTA. Design values that depend on the sheet pile section

The values of this annex are valid for

- all steel grades of the sheet pile sections
- a capping beam executed with concrete of class C 30/37 (with $f_{ck} = 30$ MPa)

Conversion factors might be applicable for lower concrete classes and/or for different embedment length L_E of the steel sheet pile into the concrete capping beam (see Chapter 7.).

Table 4. lists sections shown in ArcelorMittal's 'General catalogue', edition 2020, where some sections are only available on request. Please refer to the NTA for the whole list of sheet pile sections that are covered.

Section	A	W	$F_{Rd,m}$	$M_{Rd,S}$	$M_{Rd,K}$	k_{LF}	k_{QF}	k_{QM}
	$\mathrm{cm^2/m}$	$\mathrm{cm^3/m}$	$\mathrm{kN/m}$	$\mathrm{kNm/m}$	$\mathrm{kNm/m}$	$\frac{\mathrm{cm^2}}{\mathrm{MN/m}}$	$\frac{\mathrm{cm^2/m}}{\mathrm{MN/m}}$	$\frac{\mathrm{cm^2/m}}{\mathrm{kNm/m}}$
AZ sections								
AZ 18-800	128.6	1 840	1 530	109.5	30.0	5.60	9.81	0.069
AZ 20-800	141.0	2 000	1 678	119.0				
AZ 22-800	153.5	2 165	1 827	128.8				
AZ 23-800	150.6	2 330	1 792	138.6	31.0	5.41	10.42	0.067
AZ 25-800	163.3	2 500	1 943	148.8				
AZ 27-800	176.0	2 670	2 094	158.9				
AZ 28-750	171.2	2 810	2 037	167.2	31.9	5.53	9.45	0.058
AZ 30-750	184.7	3 005	2 198	178.8				
AZ 32-750	198.3	3 200	2 360	190.4				
AZ 12-770	120.1	1 245	1 429	74.1	27.5	4.68	11.30	0.109
AZ 13-770	125.8	1 300	1 497	77.4				
AZ 14-770	131.5	1 355	1 565	80.6				
AZ 14-770-10/10	137.2	1 405	1 633	83.6				
AZ 12-700	123.2	1 205	1 466	71.7	26.0	4.29	10.90	0.111
AZ 13-700	134.7	1 305	1 603	77.6				
AZ 13-700-10/10	140.4	1 355	1 671	80.6				
AZ 14-700	146.1	1 405	1 739	83.6				
AZ 17-700	133.0	1 730	1 583	102.9	29.4	5.00	10.17	0.078
AZ 18-700	139.2	1 800	1 656	107.1				
AZ 19-700	145.6	1 870	1 733	111.3				
AZ 20-700	152.0	1 945	1 809	115.7				
AZ 24-700	174.1	2 430	2 072	144.6	30.5	5.39	9.29	0.067
AZ 26-700	187.2	2 600	2 228	154.7				
AZ 28-700	200.2	2 760	2 382	164.2				
AZ 36-700N	215.9	3 590	2 569	213.6	31.1	4.87	9.52	0.057
AZ 38-700N	230.0	3 795	2 737	225.8				
AZ 40-700N	244.2	3 995	2 906	237.7				
AZ 42-700N	258.7	4 205	3 079	250.2	31.0	4.97	9.25	0.057
AZ 44-700N	272.8	4 405	3 246	262.1				
AZ 46-700N	287.0	4 605	3 415	274.0				
AZ 48-700	288.4	4 755	3 432	282.9	31.2	4.81	9.51	0.058
AZ 50-700	302.6	4 955	3 601	294.8				
AZ 52-700	316.8	5 155	3 770	306.7				
AZ 17*	138.3	1 665	1 646	99.1	27.3	4.41	9.76	0.081
AZ 18	150.4	1 800	1 790	107.1				
AZ 18-10/10	157.2	1 870	1 871	111.3				
AZ 19*	163.8	1 940	1 949	115.4				
AZ 25*	185.0	2 455	2 202	146.1	28.9	4.74	9.24	0.070
AZ 26	197.8	2 600	2 354	154.7				
AZ 28*	211.1	2 755	2 512	163.9				

Section	A cm²/m	W cm³/m	$F_{Rd,m}$ kN/m	$M_{Rd,S}$ kNm/m	$M_{Rd,K}$ kNm/m	k_{LF} $\frac{cm^2}{MN/m}$	k_{QF} $\frac{cm^2/m}{MN/m}$	k_{QM} $\frac{cm^2/m}{kNm/m}$
AU sections								
AU 14	132.3	1 405	1 574	83.6	29.3	6.26	8.61	0.074
AU 16	146.5	1 600	1 743	95.2				
AU 18	150.3	1 780	1 789	105.9	29.9	6.67	8.24	0.062
AU 20	164.6	2 000	1 959	119.0				
AU 23	173.4	2 270	2 063	135.1	29.5	6.25	8.01	0.060
AU 25	187.5	2 500	2 231	148.8				
PU sections								
PU 12	140.0	1 200	1 666	71.4	27.9	5.57	8.68	0.086
PU 12S	150.8	1 260	1 794	75.0				
PU 18-1.0	154.2	1 670	1 835	99.4	29.9	5.90	7.51	0.066
PU 18	163.3	1 800	1 943	107.1				
PU 18+1.0	172.3	1 920	2 050	114.2				
PU 22-1.0	173.9	2 060	2 069	122.6	30.1	5.98	7.07	0.058
PU 22	182.9	2 200	2 177	130.9				
PU 22+1.0	192.0	2 335	2 285	138.9				
PU 28-1.0	206.8	2 680	2 461	159.5	29.4	5.53	7.43	0.056
PU 28	216.1	2 840	2 572	169.0				
PU 28+1.0	225.6	3 000	2 685	178.5				
PU 32-1.0	233.3	3 065	2 776	182.4	29.4	4.92	8.45	0.064
PU 32	242.3	3 200	2 883	190.4				
PU 32+1.0	251.3	3 340	2 990	198.7				
GU sections								
GU 6N	89.0	625	1 059	37.2	26.6	5.05	9.72	0.101
GU 7N	93.7	675	1 115	40.2				
GU 7S	98.2	740	1 169	44.0				
GU 8N	103.1	770	1 227	45.8				
GU 8S	107.8	820	1 283	48.6				
GU 10N	118.5	995	1 410	59.2	26.1	5.11	8.31	0.095
GU 11N	127.9	1 095	1 522	65.2				
GU 12N	137.2	1 200	1 633	71.4				
GU 13N	127.2	1 270	1 514	75.6	29.9	5.73	7.81	0.074
GU 14N	136.5	1 400	1 624	83.3				
GU 15N	145.9	1 530	1 736	91.0				
GU 16N	154.2	1 670	1 835	99.4	29.9	5.90	7.51	0.066
GU 18N	163.3	1 800	1 943	107.1				
GU 20N	172.3	1 920	2 050	114.2				
GU 21N	173.9	2 060	2 069	122.6	30.1	5.98	7.07	0.058
GU 22N	182.9	2 200	2 177	130.9				
GU 23N	192.0	2 335	2 285	138.9				
GU 27N	206.8	2 680	2 461	159.5	29.4	5.53	7.43	0.056
GU 28N	216.1	2 840	2 572	169.0				
GU 30N	225.6	3 000	2 685	178.5				
GU 31N	233.3	3 065	2 776	182.4	29.4	4.92	8.45	0.064
GU 32N	242.3	3 200	2 883	190.4				
GU 33N	251.3	3 340	2 990	198.7				
GU 16-400	197.3	1 560	2 348	92.8	22.2	3.52	7.03	0.087
GU 18-400	220.8	1 785	2 628	106.2				

* Sections available only on request.

Table 4. Design values according to Annex 1 of the German NTA.

7. Annex 2 of the NTA. Constants and conversion factors

The values of following Table 5. are valid for

- all steel grades of the sheet pile sections,
- a capping beam executed with concrete of class C 30/37 (with $f_{ck} = 30$ MPa),
- an embedment length $L_E = 18$ cm.

Conversion factors that are applicable for lower concrete classes (minimum C 20/25) and/or for different embedment length L_E of the steel sheet pile into the concrete capping beam are listed in Table 6. below.

Constant	Value	Unit	Associated data / parameter
$H_{Rd,K}$	222	kN/m	-
k_{QH}	0.023	$\frac{\text{cm}^2/\text{m}}{\text{kN/m}}$	H_d
k_{QK}	0.230	$\frac{\text{cm}^2/\text{m}}{\text{kNm/m}}$	$M_{d,K}$
k_{BH}	0.013	$\frac{\text{cm}^2/\text{m}}{\text{kN/m}}$	H_d
k_{BM}	0.275	$\frac{\text{cm}^2/\text{m}}{\text{kNm/m}}$	$M_{d,K}$

Table 5. Constants and associated data according to Annex 2 of the German NTA.

Data / parameter	Conversion factor for lower concrete class ($20\text{ MPa} \le f_{ck} < 30\text{ MPa}$)	Conversion factor for embedment depth ($18\text{ mm} < L_E \le 33\text{ mm}$)
$F_{Rd,m}$	$\frac{f_{ck}}{30}$	–
$M_{Rd,S}$	$\frac{f_{ck}}{30}$	–
$M_{Rd,K}$	$\left(\frac{f_{ck}}{30}\right)^{2/3}$	$\frac{L_E - 3}{15}$
$H_{Rd,K}$	$\left(\frac{f_{ck}}{30}\right)^{2/3}$	–
k_{QK}	–	$\frac{15}{L_E - 3}$
k_{BM}	–	$1.1 - \frac{L_E}{180}$

Table 6. Conversion factors for different concrete classes / embedment according to Annex 2 of the German NTA.

Note

f_{ck} in MPa

L_E in cm

Water transport solutions

Build sustainable and durable maritime port and waterway infrastructures with our steel solutions. Quay walls made with steel sheet piles allow up to **20% faster construction and 15% lower cost*** when compared with alternative materials.
Steel is also the material of choice for breakwaters, dolphins, locks and canals.
The lifetime return on investment of ports built with ArcelorMittal AZ® steel sheet piles exceeds by 8%* the financial result brought by concrete solutions. **AMLoCor® steel grades are up to 5 times more corrosion-resistant** than standard steel grades, allowing optimised designs with service life of up to 100 years.
A specific Environmental Product Declaration based on comprehensive Life Cycle Analyses is available for ArcelorMittal steel sheet piles and EcoSheetPile™ Plus made of 100% recycled steel and with 100% renewable electricity. With the intrinsic ductility of steel, sheet piling solutions in conjunction with modern performance-based design methods help design and optimise safe ports in seismic areas.

* Results from a study by Tractebel, Belgium (2019).

Water based transport is essential to our global economy

Ship lock on river Main at Eddersheim, Germany

Hazard protection solutions

Dykes, flood and erosion protection barriers made with steel sheet piles are one of the most efficient ways of protecting against floods and rising sea levels.
A new design method for reinforcements and upgrades of existing flood protection systems using steel sheet piles leads to **up to 40% savings***.
Requiring little equipment and manpower, **steel sheet piles can be quickly installed** with guaranteed quality, even in remote locations.
AZ®-800, the widest sheet piles on the market, allow up to 14% less installation time. Dixeran® declutching detectors ensure against the loss of integrity of a sheet pile wall. Sealing systems such as AKILA® improve the imperviousness of the structures.

* Recent study by multi-disciplinary research team in the Netherlands (POV Macrostability, 2020).

Safeguarding our communities from natural disasters

Flood protection barrier protecting the city of St-Pierre de Gaubert, France

Mobility infrastructure solutions

Composite bridges with steel sheet pile abutments have **up to 10% shorter construction time and up to 15% less economic impact** on the community throughout their service life*. The use of steel sheet piles as load-bearing impervious permanent retaining walls in underground car parks maximizes the available surface inside the building.
Permanent steel sheet pile walls in underground car parks of 2 to 3 levels are **up to 50% more cost-effective**** than walls built with alternative materials, with significantly shorter execution time.
Silent and low vibration installation techniques minimise disruption in urban settings. **Steel sheet piles can be reused several times and are recyclable**, reducing the global environmental impact of projects.

* Study by Karlsruher Institut für Technologie (KIT), Germany (2019).
** Study by Royal Haskoning DHV, the Netherlands (2019).

Efficient and reliable mobility infrastructures make your journey smoother and safer

Underground car park with permanent steel sheet pile walls at Hopmarkt shopping center, Aalst, Belgium

Environmental protection solutions

Steel sheet piles are used as temporary and permanent retaining walls for landfill conversion, polluted soil remediation, riverbed cleaning operations and pollution containment.
Sealing systems such as AKILA® ensure the retaining walls are impervious, while suitable for contact with groundwater.
Enclosures retaining contaminated soils can be created even faster with the **unique 800 mm wide AZ®-800** steel sheet piles.
ArcelorMittal EcoSheetPile™ Plus has a much lower carbon footprint than other steel sheet piles*. This product range is the ideal solution to reduce the environmental impact of all retaining walls.

* Environmental Product Declaration for EcoSheetPile™ Plus (2021), based on a life-cycle analysis with "cradle-to-gate with options" methodology.

When faced with pollution risks, containment is vital

Fish pass at Sauveterre hydroelectric dam on river Rhône, France, allowing the restoration of the migration path of several fish and wildlife species. © Juan Robert

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Edition 9.2022

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09.2022_Knife edge support_Schneidenlagerung