ArcelorMittal

Piling Handbook

9th edition

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Foreword

ArcelorMittal is the world's largest producer of hot-rolled steel sheet piles. ArcelorMittal - Sheet Piling is in charge of the sales, marketing and promotion of hot rolled steel sheet piles, cold formed sheet piles, bearing piles and foundation solutions produced by following ArcelorMittal mills:

- hot rolled sheet piles: Belval and Differdange in Luxembourg as well as Dabrowa in Poland;
- cold formed sheet piles: "Palfroid" in Messempré, France;

Additionally, ArcelorMittal - Sheet Piling can supply steel tubes and any accessory required for a complete foundation solution package, including anchorage material, walers, fabricated piles, coated piles, driving caps, etc.

ArcelorMittal Belval is the world's largest rolling mill of hot rolled steel sheet piles and has been playing a leading role in the development of piling technology for over 100 years.

The first steel sheet piles were rolled in 1911: the "Ransome" and "Terre Rouge" piles. Since then the production program of ArcelorMittal's mill in Belval has undergone constant improvement and development to include U- with widths of up to 750 mm (AU) and Z-piles up to 800 mm wide (AZ-800).

ArcelorMittal Differdange produces the biggest HZM sections to form the most competitive high modulus combined wall system.

U-type piles produced by ArcelorMittal's mill in Dabrowa, Poland, are also marketed through ArcelorMittal - Sheet Piling.

ArcelorMittal's piling series are especially suitable for building reliable structures rapidly and cost-effectively. They are characterised by excellent section modulus to weight ratios and high moments of inertia. Steel sheet piles are used worldwide for the construction of quay walls and breakwaters in harbours, locks, and for bank reinforcement on rivers and canals. Other applications are temporary cofferdams in land and in water, permanent bridge abutments, retaining walls for underpasses or underground car parks, impervious containment walls, etc.

The Technical Department offers comprehensive services throughout the world with customised support to all the parties involved in the design, specification and installation of sheet and bearing piles, e.g. consulting engineers, architects, regional authorities, contractors, academics and their students.

Services provided free of charge by ArcelorMittal's in-house design and support teams:

- preliminary designs of complete solutions including anchorage systems and lifetime calculations;
- project optimizations offered to end-users to provide the most competitive piling package;

- elaboration of detailed project layouts and supply chains;
- assistance and recommendations on pile installation methods and driving equipment;
- promotion of “green sheet piles”, including Life Cycle Assessment.

In addition to offering these most comprehensive services for steel sheet piling solutions, ArcelorMittal issues the Piling Handbook. This Piling Handbook is intended to assist design engineers in their daily work and act as a reference book for the more experienced engineers.

The 9th edition of the Handbook includes substantial updates and contains all the new sections available in January 2016. This handbook reflects the dynamism of the foundations industry and is evidence of ArcelorMittal’s commitment to customer support. ArcelorMittal mission is to develop excellent working partnerships with its customers in order to consolidate its leadership in sheet piling technology, and remain the preferred supplier in the marketplace. We sincerely trust that you will find this Handbook a valuable and most useful document, and we look forward to working together with you on many successful projects around the world.

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In particular the authors would like to mention Andrew Bond and Robin Dawson, who have given us the benefit of their considerable experience.

Glossary

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Chapter 12	Quay wall, Brunsbüttel, Germany
Chapter 13	Retaining wall, Bern, Switzerland
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1.1. Introduction

Steel sheet piling is used in many types of temporary works and permanent structures. The sections are designed to provide the maximum strength and durability at the lowest possible weight consistent with good driving qualities. The design of the section interlocks facilitates pitching as well as driving and results in a continuous wall with a series of closely fitting joints. A comprehensive range of sections in both Z and U forms with a wide range of sizes and weights is available in various different grades of steel. The existing variety of steel sheet piles enables the most economic choice to be made to suit the nature and requirements of any given contract. For applications where corrosion is an issue, sections with minimum thickness can be delivered to maximise the effective life of the structure. The usual requirements for minimum overall thickness of 10 mm, 12 mm or ½ inch can be met. Corner and junction piles are available to suit all requirements.

1.2. Typical uses

Ports and harbours

Steel sheet piling is a tried and tested material to construct quay walls speedily and economically. Steel sheet piles can be designed to cater for heavy vertical loads and large bending moments.

River control structures and flood defence

Steel sheet piling has traditionally been used for the support and protection of river banks, lock and sluice construction, and flood protection. Ease of use, length of life and the ability to be driven through water make piles the obvious choice.

Pumping stations

Historically used as temporary support for the construction of pumping stations, sheet piling can be easily designed as the permanent structure with substantial savings in time and cost. Although pumping stations tend to be rectangular, circular construction should be considered as advantages can be gained from the resulting open structure.

Bridge abutments

Abutments formed from sheet piles are most cost effective in situations when a piled foundation is required to support the bridge or where speed of construction is critical. Sheet piling can act as both foundation and abutment and can be driven in a single operation, requiring a minimum of space and time for construction.

Road widening retaining walls

Key requirements in road widening include minimised land take and speed of construction – particularly in lane rental situations. Steel sheet piling provides these and eliminates the need for soil excavation and disposal.

Basements

Sheet piling is an ideal material for constructing basement walls as it requires minimal construction width. Its properties are fully utilised in both the temporary and permanent cases and it offers significant cost and programme savings. Sheet piles can also support vertical loads from the structure above.

Underground car parks

One specific form of basement where steel sheet piling has been found to be particularly effective is for the creation of underground car parks. The fact that steel sheet piles can be driven tight against the boundaries of the site and the wall itself has minimum thickness means that the area available for cars is maximised and the cost per bay is minimised.

Containment barriers

Sealed sheet piling is an effective means for the containment of contaminated land. A range of proprietary sealants is available to suit particular conditions where extremely low permeability is required.

Load bearing foundations

Steel sheet piling can be combined with special corner profiles to form small diameter closed boxes which are ideally suited for the construction of load bearing foundations. Developed for use as a support system for motorway sign gantries, the concept has also been used to create foundation piles for bridges.

Temporary works

For construction projects where a supported excavation is required, steel sheet piling should be the first choice. The fundamental properties of strength and ease of use - which steel offers - are fully utilised in temporary works. The ability to extract and re-use sheet piles makes them an effective design solution. However, significant cost reductions and programme savings can be achieved by designing the temporary sheet pile structure as the permanent works.

1.3. Steel qualities

1.3.1. Steel grades and availabilities

Hot rolled steel piling is supplied according to EN 10248 Part 1 and ArcelorMittal mill specifications with the grade designations detailed in Table 1.1.

Steel grade EN 10248	Min. yield strength R_{eH}	Min. tensile strength R_m	Min. elongation L_o=5.65 $\sqrt{S_o}$	Chemical composition (% max)					
	MPa	MPa	%	C	Mn	Si	P	S	N
S 240 GP	**240**	340	26	0.25	–	–	0.055	0.055	0.011
S 270 GP	**270**	410	24	0.27	–	–	0.055	0.055	0.011
S 320 GP	**320**	440	23	0.27	1.70	0.60	0.055	0.055	0.011
S 355 GP	**355**	480	22	0.27	1.70	0.60	0.055	0.055	0.011
S 390 GP	**390**	490	20	0.27	1.70	0.60	0.050	0.050	0.011
S 430 GP	**430**	510	19	0.27	1.70	0.60	0.050	0.050	0.011
S 460 AP[1)]	**460**	550	17	0.27	1.70	0.60	0.050	0.050	0.011

AMLoCor®	R_{eH}	R_m	L_o=5.65 $\sqrt{S_o}$								
	MPa	MPa	%	C	Mn	Si	P	S	N	Cr	Al
Blue 320	**320**	440	23	0.27	1.70	0.60	0.05	0.05	0.011	1.50	0.65
Blue 355	**355**	480	22	0.27	1.70	0.60	0.05	0.05	0.011	1.50	0.65
Blue 390	**390**	490	20	0.27	1.70	0.60	0.05	0.05	0.011	1.50	0.65

Table 1.1. Steel grades and qualities for sheet piles.

All the sections can be delivered in steel grades according to EN 10248-1. Special steel grades like **S 460 AP**, American **ASTM A 572** steel grades, steels with improved corrosion resistance like **AMLoCor** and **ASTM A 690**, or steels with copper addition in accordance with EN 10248 Part 1 Chapter 10.4 can be supplied on request. A modified steel grade A 690 with higher yield strength is also available upon request.

Galvanisation has an influence on the required chemical composition of the steel and must therefore be specified in the purchase orders.

We strongly recommend informing the supplier of all surface treatment to be applied to the product when placing orders.

Steel grades complying with other standards are also available at ArceloMittal Sheet Piling, see Table 1.2.

Europe	EN 10248	S 270 GP	S 320 GP	S 355 GP	S 390 GP	S 430 GP	S 460 AP[1)]
USA	ASTM	A 328	-	A 572 Gr. 50; A 690	A 572 Gr. 55	A 572 Gr. 60	A 572 Gr. 65
Canada	CSA	Gr. 260 W	Gr. 300 W	Gr. 350 W	Gr. 400 W	-	-
Japan	JIS	SY 295	-	-	SY 390	-	-

Table 1.2. Standard availabilities: product range per steel grade.

1) ArcelorMittal mill specification.

1.3.2. AMLoCor® - New corrosion resistant steel grade for marine applications

AMLoCor® is a **new "low corrosion" steel grade** that will revolutionize the design of port structures in the future.

The **key advantage of AMLoCor® is a significant reduction of the corrosion rates in the "Low Water Zone" (LWZ) and in the "Permanent Immersion Zone" (PIZ)**, which is normally the location of the maximum bending moments, and consequently highest steel stresses. This new steel grade is the solution to address the major concern of designers and port authorities: **durability of marine structures** like quay walls, breakwaters, jetties.

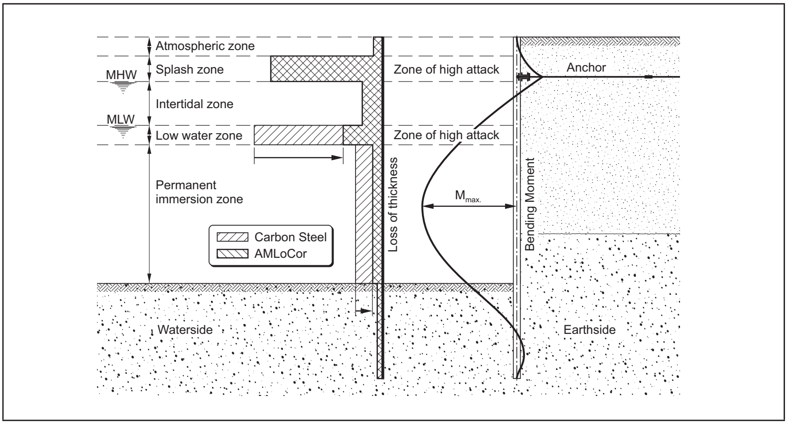


Fig. 1.1. Typical loss of steel thickness in a marine environment: regular carbon steel vs. AMLoCor®.

Eurocode 3 - Part 5 contains reference tables with typical corrosion rates valid for standard carbon steel in northern European countries. In-situ tests have proven that **the loss of steel thickness of AMLoCor is reduced by a factor 3 (PIZ) to 5 (LWZ) compared to standard structural steel** in the critical zones.

AMLoCor leads to considerable savings in steel weight compared to the unprotected carbon steel piling solution, as soon as loss of steel thickness due to corrosion in the immersion zone is significant. Cathodic protection or coatings can be used to increase the service life of the sheet pile structure. However, **AMLoCor® will in many cases yield the most cost-effective solution in the long-term.** AMLoCor is compatible with cathodic protection and coatings.

In addition AMLoCor protects steel from "ALWC" (Accelerated Low Water Corrosion) which is related to biological activity enhancing degradation of steel in the low water zone.

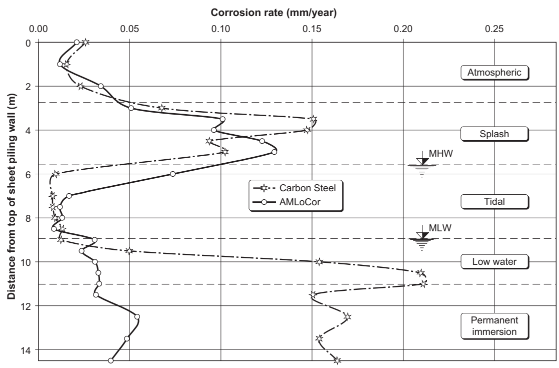


Fig. 1.2. Corrosion rates of carbon steel and AMLocCor® for a typical quay wall.

The mechanical properties of AMLoCor steel are fully equivalent to standard piling grades, so that structural resistance can be determined according to all relevant design codes used for steel sheet piling structures, like EN 1993-5: 2007 in European countries.

For the availability of sections in AMLoCor steel grades Blue 320, Blue 355 and Blue 390 (with yield strength of 320 MPa, 355 MPa and 390 MPa), it is referred to the general catalogue of ArcelorMittal Sheet Piling respectively please contact our technical department.

A driving test was performed in very compact soil in Denmark. Sheet piles in S 355 GP and AMLoCor Blue 355 were driven into very hard soils with some boulders. The sheet piles were monitored during driving, then pulled out and inspected. This test has demonstrated that the behaviour of AMLoCor sheet piles is as good as regular carbon steel sheet piles.

For more detailed information (e.g. on welding) please check our **brochure "AMLoCor®"**, part 1 to 3.

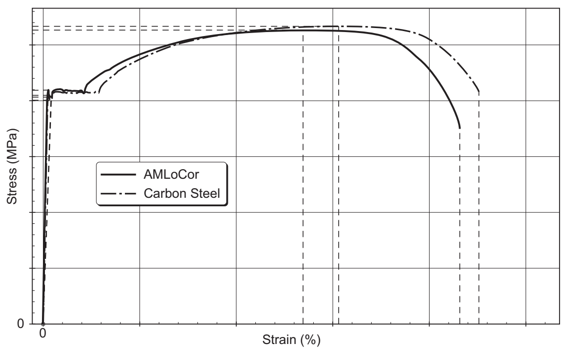
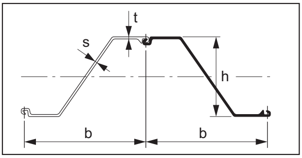


Fig. 1.3. Typical stress-strain diagram of carbon steel & AMLoCor®.

1.4. Z profile piles

1.4.1. Dimensions and properties



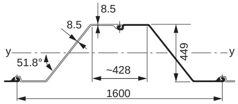
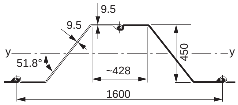
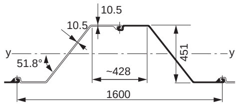
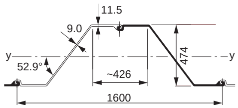
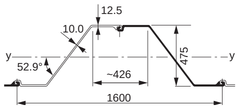
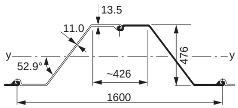
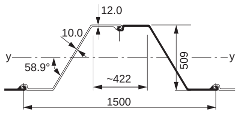
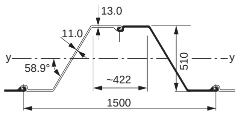
Section	Width	Height	Thickness		Sectional area	Mass		Moment of inertia	Elastic section modulus	Static moment	Plastic section modulus	Class[1)]						
	b mm	h mm	t mm	s mm	cm²/m	single pile kg/m	wall kg/m²	cm⁴/m	cm³/m	cm³/m	cm³/m	S 240 GP	S 270 GP	S 320 GP	S 355 GP	S 390 GP	S 430 GP	S 460 AP
AZ-800																		
AZ 18-800	800	449	8.5	8.5	129	80.7	**101**	41320	**1840**	1065	2135	3	3	3	3	3	4	4
AZ 20-800	800	450	9.5	9.5	141	88.6	**111**	45050	**2000**	1165	2330	3	3	3	3	3	3	3
AZ 22-800	800	451	10.5	10.5	153	96.4	**120**	48790	**2165**	1260	2525	2	2	3	3	3	3	3
AZ 23-800	800	474	11.5	9.0	151	94.6	**118**	55260	**2330**	1340	2680	2	2	2	3	3	3	3
AZ 25-800	800	475	12.5	10.0	163	102.6	**128**	59410	**2500**	1445	2890	2	2	2	2	2	3	3
AZ 27-800	800	476	13.5	11.0	176	110.5	**138**	63570	**2670**	1550	3100	2	2	2	2	2	2	2
AZ-750																		
AZ 28-750	750	509	12.0	10.0	171	100.8	**134**	71540	**2810**	1620	3245	2	2	2	2	3	3	3
AZ 30-750	750	510	13.0	11.0	185	108.8	**145**	76670	**3005**	1740	3485	2	2	2	2	2	2	3
AZ 32-750	750	511	14.0	12.0	198	116.7	**156**	81800	**3200**	1860	3720	2	2	2	2	2	2	2
AZ-700 and AZ-770																		
AZ 12-770	770	344	8.5	8.5	120	72.6	**94**	21430	**1245**	740	1480	2	2	3	3	3	3	3
AZ 13-770	770	344	9.0	9.0	126	76.1	**99**	22360	**1300**	775	1546	2	2	3	3	3	3	3
AZ 14-770	770	345	9.5	9.5	132	79.5	**103**	23300	**1355**	805	1611	2	2	2	2	3	3	3
AZ 14-770-10/10	770	345	10.0	10.0	137	82.9	**108**	24240	**1405**	840	1677	2	2	2	2	2	3	3
AZ 12-700	700	314	8.5	8.5	123	67.7	**97**	18880	**1205**	710	1415	2	2	3	3	3	3	3
AZ 13-700	700	315	9.5	9.5	135	74.0	**106**	20540	**1305**	770	1540	2	2	2	3	3	3	3
AZ 13-700-10/10	700	316	10.0	10.0	140	77.2	**110**	21370	**1355**	800	1600	2	2	2	2	3	3	3
AZ 14-700	700	316	10.5	10.5	146	80.3	**115**	22190	**1405**	835	1665	2	2	2	2	2	3	3
AZ 17-700	700	420	8.5	8.5	133	73.1	**104**	36230	**1730**	1015	2027	2	2	3	3	3	3	3
AZ 18-700	700	420	9.0	9.0	139	76.5	**109**	37800	**1800**	1060	2116	2	2	3	3	3	3	3
AZ 19-700	700	421	9.5	9.5	146	80.0	**114**	39380	**1870**	1105	2206	2	2	2	3	3	3	3
AZ 20-700	700	421	10.0	10.0	152	83.5	**119**	40960	**1945**	1150	2296	2	2	2	2	2	3	3
AZ 24-700	700	459	11.2	11.2	174	95.7	**137**	55820	**2430**	1435	2867	2	2	2	2	2	2	3
AZ 26-700	700	460	12.2	12.2	187	102.9	**147**	59720	**2600**	1535	3070	2	2	2	2	2	2	2
AZ 28-700	700	461	13.2	13.2	200	110.0	**157**	63620	**2760**	1635	3273	2	2	2	2	2	2	2
AZ 24-700N	700	459	12.5	9.0	163	89.7	**128**	55890	**2435**	1405	2810	2	2	2	2	2	2	2
AZ 26-700N	700	460	13.5	10.0	176	96.9	**138**	59790	**2600**	1510	3015	2	2	2	2	2	2	2
AZ 28-700N	700	461	14.5	11.0	189	104.1	**149**	63700	**2765**	1610	3220	2	2	2	2	2	2	2
AZ 36-700N	700	499	15.0	11.2	216	118.6	**169**	89610	**3590**	2055	4110	2	2	2	2	2	2	2
AZ 38-700N	700	500	16.0	12.2	230	126.4	**181**	94840	**3795**	2180	4360	2	2	2	2	2	2	2
AZ 40-700N	700	501	17.0	13.2	244	134.2	**192**	100080	**3995**	2305	4605	2	2	2	2	2	2	2
AZ 42-700N	700	499	18.0	14.0	259	142.1	**203**	104930	**4205**	2425	4855	2	2	2	2	2	2	2
AZ 44-700N	700	500	19.0	15.0	273	149.9	**214**	110150	**4405**	2550	5105	2	2	2	2	2	2	2
AZ 46-700N	700	501	20.0	16.0	287	157.7	**225**	115370	**4605**	2675	5350	2	2	2	2	2	2	2

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[1] S 240 GP	S 270 GP	S 320 GP	S 355 GP	S 390 GP	S 430 GP	S 460 AP
AZ 48-700	700	503	22.0	15.0	288	158.5	**226**	119650	**4755**	2745	5490	2	2	2	2	2	2	2
AZ 50-700	700	504	23.0	16.0	303	166.3	**238**	124890	**4955**	2870	5735	2	2	2	2	2	2	2
AZ 52-700	700	505	24.0	17.0	317	174.1	**249**	130140	**5155**	2990	5985	2	2	2	2	2	2	2
AZ																		
AZ 18[2]	630	380	9.5	9.5	150	74.4	**118**	34200	**1800**	1050	2104	2	2	2	3	3	3	3
AZ 18-10/10	630	381	10.0	10.0	157	77.8	**123**	35540	**1870**	1095	2189	2	2	2	2	3	3	3
AZ 26[2]	630	427	13.0	12.2	198	97.8	**155**	55510	**2600**	1530	3059	2	2	2	2	2	2	2
AZ 46	580	481	18.0	14.0	291	132.6	**229**	110450	**4595**	2650	5295	2	2	2	2	2	2	2
AZ 48	580	482	19.0	15.0	307	139.6	**241**	115670	**4800**	2775	5553	2	2	2	2	2	2	2
AZ 50	580	483	20.0	16.0	322	146.7	**253**	121060	**5015**	2910	5816	2	2	2	2	2	2	2

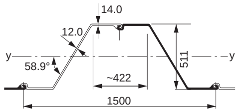
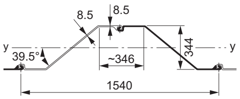
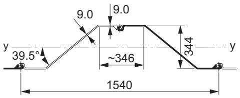
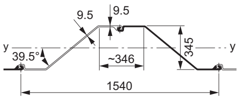
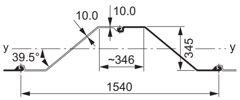
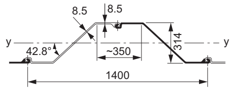
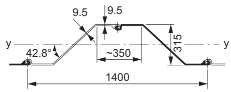
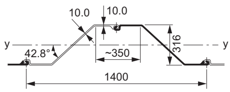
Table 1.3. Dimensions and properties of Z sections.

*[1] Classification according to EN 1993-5. Class 1 is obtained by verification of the rotation capacity for a class-2 cross-section.
A set of tables with all the data required for design in accordance with EN 1993-5 is available from our Technical Department.
Steel grade S 460 AP following specifications of the mill is available on request.*

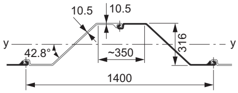
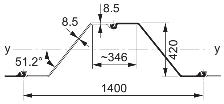
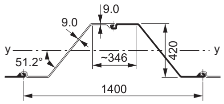
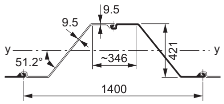
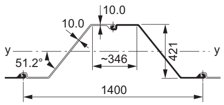
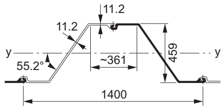
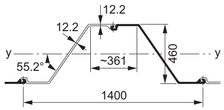
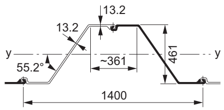
[2] AZ sections can be rolled-up or down by 0.5 mm and 1.0 mm on request.

Section	S = Single pile D = Double pile	Sectional area	Mass	Moment of inertia	Elastic section modulus	Radius of gyration	Coating area[1)]
		cm²	kg/m	cm⁴	cm³	cm	m²/m
AZ-800							
AZ 18-800							
	Per S	102.9	**80.7**	33055	**1470**	17.93	1.04
	Per D	205.7	**161.5**	66110	**2945**	17.93	2.08
	Per m of wall	128.6	**100.9**	41320	**1840**	17.93	1.30
AZ 20-800							
	Per S	112.8	**88.6**	36040	**1600**	17.87	1.04
	Per D	225.6	**177.1**	72070	**3205**	17.87	2.08
	Per m of wall	141.0	**110.7**	45050	**2000**	17.87	1.30
AZ 22-800							
	Per S	122.8	**96.4**	39035	**1730**	17.83	1.04
	Per D	245.6	**192.8**	78070	**3460**	17.83	2.08
	Per m of wall	153.5	**120.5**	48790	**2165**	17.83	1.30
AZ 23-800							
	Per S	120.5	**94.6**	44200	**1865**	19.15	1.06
	Per D	241.0	**189.2**	88410	**3730**	19.15	2.11
	Per m of wall	150.6	**118.2**	55260	**2330**	19.15	1.32
AZ 25-800							
	Per S	130.6	**102.6**	47530	**2000**	19.07	1.06
	Per D	261.3	**205.1**	95060	**4005**	19.07	2.11
	Per m of wall	163.3	**128.2**	59410	**2500**	19.07	1.32
AZ 27-800							
	Per S	140.8	**110.5**	50860	**2135**	19.01	1.06
	Per D	281.6	**221.0**	101720	**4275**	19.01	2.11
	Per m of wall	176.0	**138.1**	63570	**2670**	19.01	1.32
AZ-750							
AZ 28-750							
	Per S	128.4	**100.8**	53650	**2110**	20.44	1.06
	Per D	256.8	**201.6**	107310	**4215**	20.44	2.11
	Per m of wall	171.2	**134.4**	71540	**2810**	20.44	1.41
AZ 30-750							
	Per S	138.5	**108.8**	57500	**2255**	20.37	1.06
	Per D	277.1	**217.5**	115000	**4510**	20.37	2.11
	Per m of wall	184.7	**145.0**	76670	**3005**	20.37	1.41

[1)] *One side, excluding inside of interlocks.*

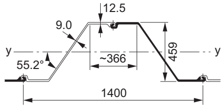
Section	S = Single pile D = Double pile	Sectional area	Mass	Moment of inertia	Elastic section modulus	Radius of gyration	Coating area[1)]
		cm²	kg/m	cm⁴	cm³	cm	m²/m
AZ 32-750							
	Per S	148.7	**116.7**	61350	**2400**	20.31	1.06
	Per D	297.4	**233.5**	122710	**4805**	20.31	2.11
	Per m of wall	198.3	**155.6**	81800	**3200**	20.31	1.41
AZ-700 and AZ-770							
AZ 12-770							
	Per S	92.5	**72.6**	16500	**960**	13.36	0.93
	Per D	185.0	**145.2**	33000	**1920**	13.36	1.85
	Per m of wall	120.1	**94.3**	21430	**1245**	13.36	1.20
AZ 13-770							
	Per S	96.9	**76.1**	17220	**1000**	13.33	0.93
	Per D	193.8	**152.1**	34440	**2000**	13.33	1.85
	Per m of wall	125.8	**98.8**	22360	**1300**	13.33	1.20
AZ 14-770							
	Per S	101.3	**79.5**	17940	**1040**	13.31	0.93
	Per D	202.6	**159.0**	35890	**2085**	13.31	1.85
	Per m of wall	131.5	**103.2**	23300	**1355**	13.31	1.20
AZ 14-770-10/10							
	Per S	105.6	**82.9**	18670	**1085**	13.30	0.93
	Per D	211.2	**165.8**	37330	**2165**	13.30	1.85
	Per m of wall	137.2	**107.7**	24240	**1405**	13.30	1.20
AZ 12-700							
	Per S	86.2	**67.7**	13220	**840**	12.38	0.86
	Per D	172.5	**135.4**	26440	**1685**	12.38	1.71
	Per m of wall	123.2	**96.7**	18880	**1205**	12.38	1.22
AZ 13-700							
	Per S	94.3	**74.0**	14370	**910**	12.35	0.86
	Per D	188.5	**148.0**	28750	**1825**	12.35	1.71
	Per m of wall	134.7	**105.7**	20540	**1305**	12.35	1.22
AZ 13-700-10/10							
	Per S	98.3	**77.2**	14960	**945**	12.33	0.86
	Per D	196.6	**154.3**	29910	**1895**	12.33	1.71
	Per m of wall	140.4	**110.2**	21370	**1355**	12.33	1.22

[1)] *One side, excluding inside of interlocks.*

Section	S = Single pile D = Double pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
AZ 14-700	Per S	102.3	**80.3**	15530	**980**	12.32	0.86
	Per D	204.6	**160.6**	31060	**1965**	12.32	1.71
	Per m of wall	146.1	**114.7**	22190	**1405**	12.32	1.22
AZ 17-700	Per S	93.1	**73.1**	25360	**1210**	16.50	0.93
	Per D	186.2	**146.2**	50720	**2420**	16.50	1.86
	Per m of wall	133.0	**104.4**	36230	**1730**	16.50	1.33
AZ 18-700	Per S	97.5	**76.5**	26460	**1260**	16.47	0.93
	Per D	194.9	**153.0**	52920	**2520**	16.47	1.86
	Per m of wall	139.2	**109.3**	37800	**1800**	16.47	1.33
AZ 19-700	Per S	101.9	**80.0**	27560	**1310**	16.44	0.93
	Per D	203.8	**160.0**	55130	**2620**	16.44	1.86
	Per m of wall	145.6	**114.3**	39380	**1870**	16.44	1.33
AZ 20-700	Per S	106.4	**83.5**	28670	**1360**	16.42	0.93
	Per D	212.8	**167.0**	57340	**2725**	16.42	1.86
	Per m of wall	152.0	**119.3**	40960	**1945**	16.42	1.33
AZ 24-700	Per S	121.9	**95.7**	39080	**1700**	17.90	0.97
	Per D	243.8	**191.4**	78150	**3405**	17.90	1.93
	Per m of wall	174.1	**136.7**	55820	**2430**	17.90	1.38
AZ 26-700	Per S	131.0	**102.9**	41800	**1815**	17.86	0.97
	Per D	262.1	**205.7**	83610	**3635**	17.86	1.93
	Per m of wall	187.2	**146.9**	59720	**2600**	17.86	1.38
AZ 28-700	Per S	140.2	**110.0**	44530	**1930**	17.83	0.97
	Per D	280.3	**220.1**	89070	**3865**	17.83	1.93
	Per m of wall	200.2	**157.2**	63620	**2760**	17.83	1.38

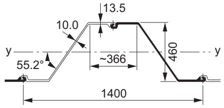
1) One side, excluding inside of interlocks.

AZ 24-700N



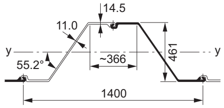
Section	S = Single pile D = Double pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
AZ 24-700N	Per S	114.3	**89.7**	39120	**1705**	18.50	0.96
	Per D	228.6	**179.5**	78240	**3410**	18.50	1.92
	Per m of wall	163.3	**128.2**	55890	**2435**	18.50	1.37

AZ 26-700N



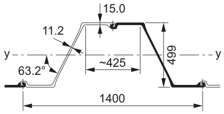
Section	S = Single pile D = Double pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
AZ 26-700N	Per S	123.5	**96.9**	41850	**1820**	18.41	0.96
	Per D	247.0	**193.9**	83710	**3640**	18.41	1.92
	Per m of wall	176.4	**138.5**	59790	**2600**	18.41	1.37

AZ 28-700N



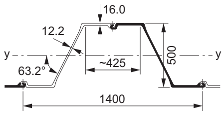
Section	S = Single pile D = Double pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
AZ 28-700N	Per S	132.6	**104.1**	44590	**1935**	18.33	0.96
	Per D	265.3	**208.2**	89170	**3870**	18.33	1.92
	Per m of wall	189.5	**148.7**	63700	**2765**	18.33	1.37

AZ 36-700N



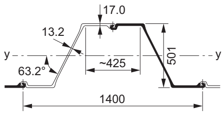
Section	S = Single pile D = Double pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
AZ 36-700N	Per S	151.1	**118.6**	62730	**2510**	20.37	1.03
	Per D	302.2	**237.3**	125450	**5030**	20.37	2.05
	Per m of wall	215.9	**169.5**	89610	**3590**	20.37	1.47

AZ 38-700N



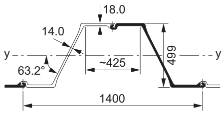
Section	S = Single pile D = Double pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
AZ 38-700N	Per S	161.0	**126.4**	66390	**2655**	20.31	1.03
	Per D	322.0	**252.8**	132780	**5310**	20.31	2.05
	Per m of wall	230.0	**180.6**	94840	**3795**	20.31	1.47

AZ 40-700N



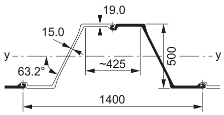
Section	S = Single pile D = Double pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
AZ 40-700N	Per S	170.9	**134.2**	70060	**2795**	20.25	1.03
	Per D	341.9	**268.4**	140110	**5595**	20.25	2.05
	Per m of wall	244.2	**191.7**	100080	**3995**	20.25	1.47

AZ 42-700N



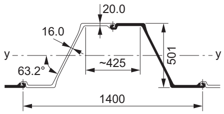
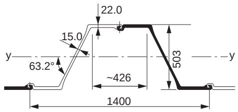
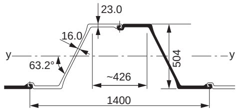
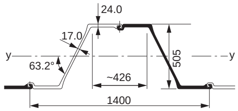
Section	S = Single pile D = Double pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
AZ 42-700N	Per S	181.1	**142.1**	73450	**2945**	20.14	1.03
	Per D	362.1	**284.3**	146900	**5890**	20.14	2.06
	Per m of wall	258.7	**203.1**	104930	**4205**	20.14	1.47

AZ 44-700N



Section	S = Single pile D = Double pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
AZ 44-700N	Per S	191.0	**149.9**	77100	**3085**	20.09	1.03
	Per D	382.0	**299.8**	154210	**6170**	20.09	2.06
	Per m of wall	272.8	**214.2**	110150	**4405**	20.09	1.47

[1)] *One side, excluding inside of interlocks.*

Section	S = Single pile D = Double pile	Sectional area	Mass	Moment of inertia	Elastic section modulus	Radius of gyration	Coating area[1)]
		cm^2	kg/m	cm^4	cm^3	cm	m^2/m
AZ 46-700N 	Per S	200.9	**157.7**	80760	**3220**	20.05	1.03
	Per D	401.8	**315.4**	161520	**6450**	20.05	2.06
	Per m of wall	287.0	**225.3**	115370	**4605**	20.05	1.47
AZ 48-700 	Per S	201.9	**158.5**	83760	**3330**	20.37	1.02
	Per D	403.8	**317.0**	167510	**6660**	20.37	2.04
	Per m of wall	288.4	**226.4**	119650	**4755**	20.37	1.46
AZ 50-700 	Per S	211.8	**166.3**	87430	**3470**	20.32	1.02
	Per D	423.6	**332.5**	174850	**6940**	20.32	2.04
	Per m of wall	302.6	**237.5**	124890	**4955**	20.32	1.46
AZ 52-700 	Per S	221.7	**174.1**	91100	**3610**	20.27	1.02
	Per D	443.5	**348.1**	182200	**7215**	20.27	2.04
	Per m of wall	316.8	**248.7**	130140	**5155**	20.27	1.46

1) One side, excluding inside of interlocks.

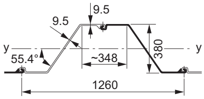
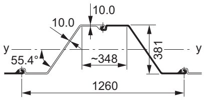
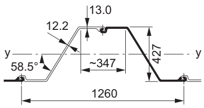
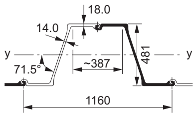
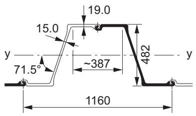
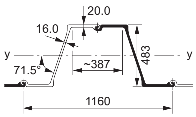
Section	S = Single pile D = Double pile	Sectional area	Mass	Moment of inertia	Elastic section modulus	Radius of gyration	Coating area[1)]
		cm²	kg/m	cm⁴	cm³	cm	m²/m
AZ							
AZ 18							
	Per S	94.8	**74.4**	21540	**1135**	15.07	0.86
	Per D	189.6	**148.8**	43080	**2270**	15.07	1.71
	Per m of wall	150.4	**118.1**	34200	**1800**	15.07	1.35
AZ 18-10/10							
	Per S	99.1	**77.8**	22390	**1175**	15.04	0.86
	Per D	198.1	**155.5**	44790	**2355**	15.04	1.71
	Per m of wall	157.2	**123.4**	35540	**1870**	15.04	1.35
AZ 26							
	Per S	124.6	**97.8**	34970	**1640**	16.75	0.90
	Per D	249.2	**195.6**	69940	**3280**	16.75	1.78
	Per m of wall	197.8	**155.2**	55510	**2600**	16.75	1.41
AZ 46							
	Per S	168.9	**132.6**	64060	**2665**	19.48	0.95
	Per D	337.8	**265.2**	128120	**5330**	19.48	1.89
	Per m of wall	291.2	**228.6**	110450	**4595**	19.48	1.63
AZ 48							
	Per S	177.8	**139.6**	67090	**2785**	19.43	0.95
	Per D	355.6	**279.2**	134180	**5570**	19.43	1.89
	Per m of wall	306.5	**240.6**	115670	**4800**	19.43	1.63
AZ 50							
	Per S	186.9	**146.7**	70215	**2910**	19.38	0.95
	Per D	373.8	**293.4**	140430	**5815**	19.38	1.89
	Per m of wall	322.2	**252.9**	121060	**5015**	19.38	1.63

Table 1.4. Technical information of Z sections.

[1)] One side, excluding inside of interlocks.

1.4.2. Interlocking in pairs

AZ® piles are normally supplied in pairs which saves time in handling and pitching. They can however, be supplied singly by prior arrangement but the purchaser must be warned that the bending strength of single AZ piles, especially the lighter ones, is very low and damage by plastic deformation under self-weight can easily occur during handling and driving.

1.4.3. Crimping and welding of the interlocks

Crimping or welding of AZ® piles is not necessary to guarantee the strength of the piled wall, but can be of benefit during handling and driving.

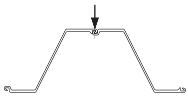
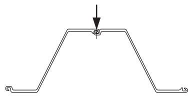
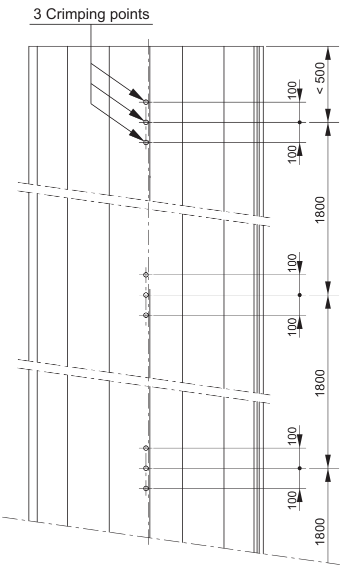
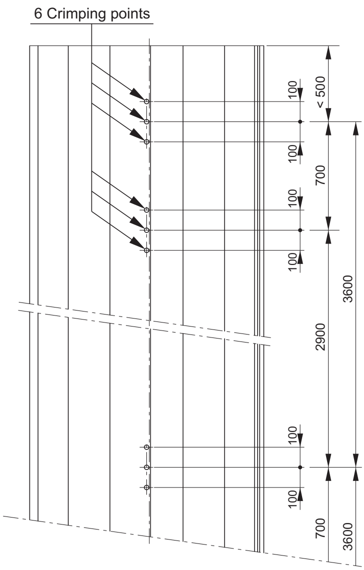
AZ® section standard crimping 	**AZ® section standard crimping** 
Pile length < 6 m: 3 crimping points per 1.8 m = 1.7 crimping points per m[1)]	Pile length ≥ 6 m: 6 crimping points per 3.6 m = 1.7 crimping points per m[1)]
	

Fig. 1.4. Standard crimping of AZ sections.

1) Amount and layout of crimping points may differ at both ends. Special crimping on request.

1.4.4. Pile form

Piles can be supplied as illustrated.

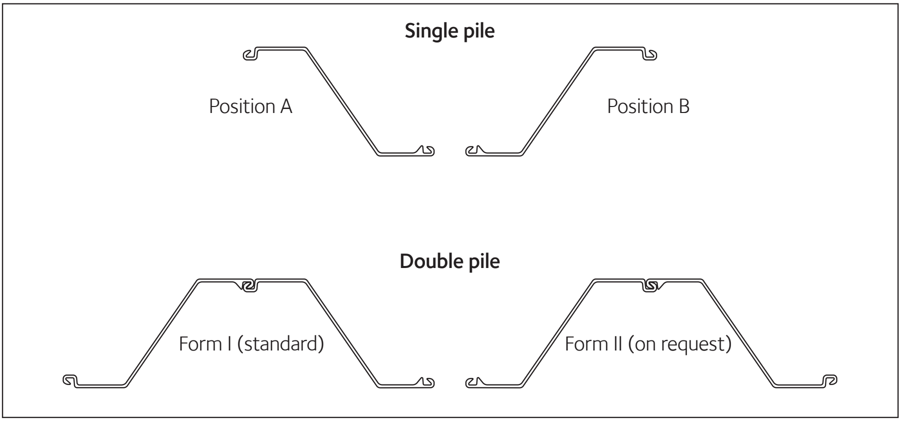


Fig. 1.5. Delivery forms of AZ sections.

Double piles will be supplied as Form I unless specified differently at the time of order.

1.4.5. Circular construction

Steel sheet piling can be driven to form a complete circle without the need for corner piles. AZ® piles have a maximum angle of deviation of 5°.

Table 1.5. gives the approximate minimum diameters of circular cofferdam which can be constructed using various sheet pile sections. The diameters are only intended to be for guidance as the actual interlock deviation achieved will be a function of the pile length, the pile section, the penetration required. Smaller diameters can be achieved by introducing bent corner piles, but larger diameters will result if pairs of piles that have been crimped or welded are used.

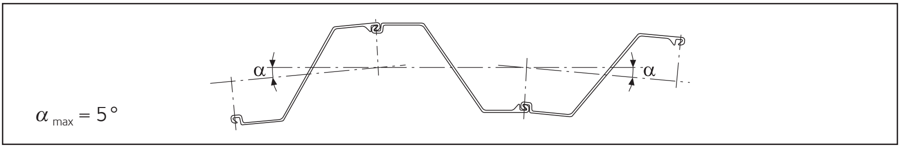


Fig. 1.6. Angle of deviation in interlocks.

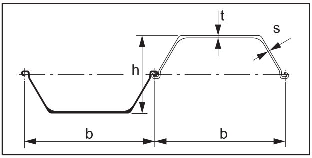
Section	**Minimum number of single piles used** Angle $\alpha = 5°$	**Approx. min diameter to internal face of wall** *m*
AZ 12, AZ 13, AZ 13-10/10, AZ 14	72	15.1
AZ 17, AZ 18, AZ 18-10/10, AZ 19	72	14.1
AZ 25, AZ 26, AZ 28	72	14.0
AZ 46, AZ 48, AZ 50	72	12.8
AZ 12-770, AZ 13-770, AZ 14-770, AZ 14-770-10/10	72	17.3
AZ 17-700, AZ 18-700, AZ 19-700, AZ 20-700	72	15.6
AZ 24-700, AZ 26-700, AZ 28-700	72	15.6
AZ 36-700N, AZ 38-700N, AZ 40-700N	72	15.5
AZ 42-700N, AZ 44-700N, AZ 46-700N	72	15.5
AZ 48-700, AZ 50-700, AZ 52-700	72	15.5
AZ 18-800, AZ 20-800, AZ 22-800, AZ 23-800, AZ 25-800, AZ 27-800	72	17.9
AZ 28-750, AZ 30-750, AZ 32-750	72	16.7

Table 1.5. Approximate min. diameter of circular pits with AZ-section.

Contact our technical representatives to obtain data for situations where plated box piles, double box piles or HZ®-M systems are to be used.

1.5. U profile piles

1.5.1. Dimensions and properties



Section	Width	Height	Thickness		Sectional area	Mass		Moment of inertia	Elastic section modulus	Static moment	Plastic section modulus	Class[1)]						
	b mm	h mm	t mm	s mm	cm²/m	single pile kg/m	wall kg/m²	cm⁴/m	cm³/m	cm³/m	cm³/m	S 240 GP	S 270 GP	S 320 GP	S 355 GP	S 390 GP	S 430 GP	S 460 AP
AU™ sections																		
AU 14	750	408	10.0	8.3	132	77.9	**104**	28680	**1405**	820	1663	2	2	3	3	3	3	3
AU 16	750	411	11.5	9.3	147	86.3	**115**	32850	**1600**	935	1891	2	2	2	2	2	3	3
AU 18	750	441	10.5	9.1	150	88.5	**118**	39300	**1780**	1030	2082	2	3	3	3	3	3	3
AU 20	750	444	12.0	10.0	165	96.9	**129**	44440	**2000**	1155	2339	2	2	2	3	3	3	3
AU 23	750	447	13.0	9.5	173	102.1	**136**	50700	**2270**	1285	2600	2	2	2	3	3	3	3
AU 25	750	450	14.5	10.2	188	110.4	**147**	56240	**2500**	1420	2866	2	2	2	2	2	3	3
PU® sections																		
PU 12	600	360	9.8	9.0	140	66.1	**110**	21600	**1200**	715	1457	2	2	2	2	2	2	3
PU 12-10/10	600	360	10.0	10.0	148	69.6	**116**	22580	**1255**	755	1535	2	2	2	2	2	2	2
PU 18^{-1}	600	430	10.2	8.4	154	72.6	**121**	35950	**1670**	980	1988	2	2	2	2	2	3	3
PU 18	600	430	11.2	9.0	163	76.9	**128**	38650	**1800**	1055	2134	2	2	2	2	2	2	2
PU 18^{+1}	600	430	12.2	9.5	172	81.1	**135**	41320	**1920**	1125	2280	2	2	2	2	2	2	2
PU 22^{-1}	600	450	11.1	9.0	174	81.9	**137**	46380	**2060**	1195	2422	2	2	2	2	2	3	3
PU 22	600	450	12.1	9.5	183	86.1	**144**	49460	**2200**	1275	2580	2	2	2	2	2	2	2
PU 22^{+1}	600	450	13.1	10.0	192	90.4	**151**	52510	**2335**	1355	2735	2	2	2	2	2	2	2
PU 28^{-1}	600	452	14.2	9.7	207	97.4	**162**	60580	**2680**	1525	3087	2	2	2	2	2	2	2
PU 28	600	454	15.2	10.1	216	101.8	**170**	64460	**2840**	1620	3269	2	2	2	2	2	2	2
PU 28^{+1}	600	456	16.2	10.5	226	106.2	**177**	68380	**3000**	1710	3450	2	2	2	2	2	2	2
PU 32^{-1}	600	452	18.5	10.6	233	109.9	**183**	69210	**3065**	1745	3525	2	2	2	2	2	2	2
PU 32	600	452	19.5	11.0	242	114.1	**190**	72320	**3200**	1825	3687	2	2	2	2	2	2	2
PU 32^{-1}	600	452	20.5	11.4	251	118.4	**197**	75410	**3340**	1905	3845	2	2	2	2	2	2	2

Table 1.6. Dimensions and properties of AU and PU sections.

The moment of inertia and section moduli values given assume correct shear transfer across the interlock.

[1)] Classification according to EN 1993-5. Class 1 is obtained by verification of the rotation capacity for a class 2 cross-section.

A set of tables with all the data required for design in accordance with EN 1993-5 is available from our Technical Department.

All PU® sections can be rolled-up or -down by 0.5 mm and 1.0 mm. Other sections on request.

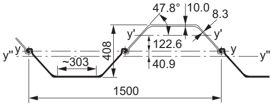
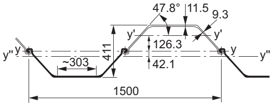
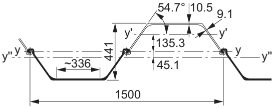
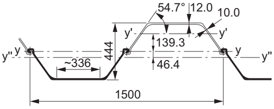
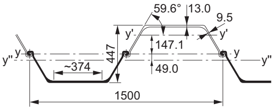
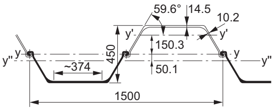
Section	Width	Height	Thickness		Sectional area	Mass		Moment of inertia	Elastic section modulus	Static moment	Plastic section modulus	Class[1)]						
	b mm	*h* mm	*t* mm	*s* mm	cm²/m	single pile kg/m	wall kg/m²	cm⁴/m	cm³/m	cm³/m	cm³/m	S 240 GP	S 270 GP	S 320 GP	S 355 GP	S 390 GP	S 430 GP	S 460 AP
GU® sections																		
GU 6N	600	309	6.0	6.0	89	41.9	**70**	9670	**625**	375	765	3	3	3	4	4	4	4
GU 7N	600	310	6.5	6.4	94	44.1	**74**	10450	**675**	400	825	3	3	3	3	3	4	4
GU 7S	600	311	7.2	6.9	100	46.3	**77**	11540	**740**	440	900	2	2	3	3	3	3	3
GU 7HWS	600	312	7.3	6.9	101	47.4	**79**	11620	**745**	445	910	2	2	3	3	3	3	3
GU 8N	600	312	7.5	7.1	103	48.5	**81**	12010	**770**	460	935	2	2	3	3	3	3	3
GU 8S	600	313	8.0	7.5	108	50.8	**85**	12800	**820**	490	995	2	2	2	3	3	3	3
GU 13N	600	418	9.0	7.4	127	59.9	**100**	26590	**1270**	755	1535	2	2	2	2	2	3	3
GU 14N	600	420	10.0	8.0	136	64.3	**107**	29410	**1400**	830	1685	2	2	2	2	2	2	2
GU 15N	600	422	11.0	8.6	146	68.7	**115**	32260	**1530**	910	1840	2	2	2	2	2	2	2
GU 16N	600	430	10.2	8.4	154	72.6	**121**	35950	**1670**	980	1988	2	2	2	2	2	3	3
GU 18N	600	430	11.2	9.0	163	76.9	**128**	38650	**1800**	1055	2134	2	2	2	2	2	2	2
GU 20N	600	430	12.2	9.5	172	81.1	**135**	41320	**1920**	1125	2280	2	2	2	2	2	2	2
GU 21N	600	450	11.1	9.0	174	81.9	**137**	46380	**2060**	1195	2422	2	2	2	2	2	3	3
GU 22N	600	450	12.1	9.5	183	86.1	**144**	49460	**2200**	1275	2580	2	2	2	2	2	2	2
GU 23N	600	450	13.1	10.0	192	90.4	**151**	52510	**2335**	1355	2735	2	2	2	2	2	2	2
GU 27N	600	452	14.2	9.7	207	97.4	**162**	60580	**2680**	1525	3087	2	2	2	2	2	2	2
GU 28N	600	454	15.2	10.1	216	101.8	**170**	64460	**2840**	1620	3269	2	2	2	2	2	2	2
GU 30N	600	456	16.2	10.5	226	106.2	**177**	68380	**3000**	1710	3450	2	2	2	2	2	2	2
GU 31N	600	452	18.5	10.6	233	109.9	**183**	69210	**3065**	1745	3525	2	2	2	2	2	2	2
GU 32N	600	452	19.5	11.0	242	114.1	**190**	72320	**3200**	1825	3687	2	2	2	2	2	2	2
GU 33N	600	452	20.5	11.4	251	118.4	**197**	75410	**3340**	1905	3845	2	2	2	2	2	2	2
GU 16-400	400	290	12.7	9.4	197	62.0	**155**	22580	**1560**	885	1815	2	2	2	2	2	2	-
GU 18-400	400	292	15.0	9.7	221	69.3	**173**	26090	**1785**	1015	2080	2	2	2	2	2	2	-

Table 1.7. Dimensions and properties of GU sections.

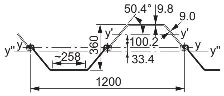
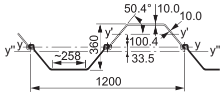
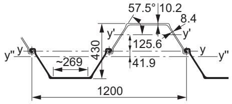
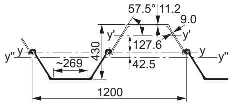
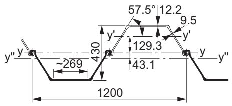
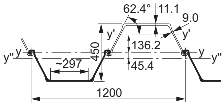
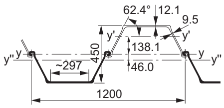
The moment of inertia and section moduli values given assume correct shear transfer across the interlock.

[1)] Classification according to EN 1993-5. Class 1 is obtained by verification of the rotation capacity for a class 2 cross-section.

A set of tables with all the data required for design in accordance with EN 1993-5 is available from our Technical Department.

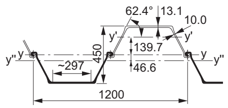
Section	S = Single pile D = Double pile T = Triple pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
AU™ sections							
AU 14							
	Per S	99.2	**77.9**	6590	**457**	8.15	0.96
	Per D	198.5	**155.8**	43020	**2110**	14.73	1.91
	Per T	297.7	**233.7**	59550	**2435**	14.15	2.86
	Per m of wall	132.3	**103.8**	28680	**1405**	14.73	1.27
AU 16							
	Per S	109.9	**86.3**	7110	**481**	8.04	0.96
	Per D	219.7	**172.5**	49280	**2400**	14.98	1.91
	Per T	329.6	**258.7**	68080	**2750**	14.37	2.86
	Per m of wall	146.5	**115.0**	32850	**1600**	14.98	1.27
AU 18							
	Per S	112.7	**88.5**	8760	**554**	8.82	1.01
	Per D	225.5	**177.0**	58950	**2670**	16.17	2.00
	Per T	338.2	**265.5**	81520	**3065**	15.53	2.99
	Per m of wall	150.3	**118.0**	39300	**1780**	16.17	1.33
AU 20							
	Per S	123.4	**96.9**	9380	**579**	8.72	1.01
	Per D	246.9	**193.8**	66660	**3000**	16.43	2.00
	Per T	370.3	**290.7**	92010	**3425**	15.76	2.99
	Per m of wall	164.6	**129.2**	44440	**2000**	16.43	1.33
AU 23							
	Per S	130.1	**102.1**	9830	**579**	8.69	1.03
	Per D	260.1	**204.2**	76050	**3405**	17.10	2.04
	Per T	390.2	**306.3**	104680	**3840**	16.38	3.05
	Per m of wall	173.4	**136.1**	50700	**2270**	17.10	1.36
AU 25							
	Per S	140.6	**110.4**	10390	**601**	8.60	1.03
	Per D	281.3	**220.8**	84370	**3750**	17.32	2.04
	Per T	422.0	**331.3**	115950	**4215**	16.58	3.05
	Per m of wall	187.5	**147.2**	56240	**2500**	17.32	1.36

[1)] *One side, excluding inside of interlocks.*

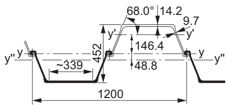
Section	S = Single pile D = Double pile T = Triple pile	Sectional area	Mass	Moment of inertia	Elastic section modulus	Radius of gyration	Coating area[1)]
		cm²	kg/m	cm⁴	cm³	cm	m²/m
PU® sections							
PU 12							
	Per S	84.2	**66.1**	4500	**370**	7.31	0.80
	Per D	168.4	**132.2**	25920	**1440**	12.41	1.59
	Per T	252.6	**198.3**	36060	**1690**	11.95	2.38
	Per m of wall	140.0	**110.1**	21600	**1200**	12.41	1.32
PU 12-10/10							
	Per S	88.7	**69.6**	4600	**377**	7.20	0.80
	Per D	177.3	**139.2**	27100	**1505**	12.36	1.59
	Per T	266.0	**208.8**	37670	**1765**	11.90	2.38
	Per m of wall	147.8	**116.0**	22580	**1255**	12.36	1.32
PU 18⁻¹							
	Per S	92.5	**72.6**	6960	**475**	8.67	0.87
	Per D	185.0	**145.2**	43140	**2005**	15.30	1.72
	Per T	277.5	**217.8**	59840	**2330**	14.69	2.58
	Per m of wall	154.2	**121.0**	35950	**1670**	15.30	1.43
PU 18							
	Per S	98.0	**76.9**	7220	**485**	8.58	0.87
	Per D	196.0	**153.8**	46380	**2160**	15.38	1.72
	Per T	294.0	**230.7**	64240	**2495**	14.78	2.58
	Per m of wall	163.3	**128.2**	38650	**1800**	15.38	1.43
PU 18⁺¹							
	Per S	103.4	**81.1**	7480	**495**	8.51	0.87
	Per D	206.8	**162.3**	49580	**2305**	15.49	1.72
	Per T	310.2	**243.5**	68600	**2655**	14.87	2.58
	Per m of wall	172.3	**135.2**	41320	**1920**	15.49	1.43
PU 22⁻¹							
	Per S	104.3	**81.9**	8460	**535**	9.01	0.90
	Per D	208.7	**163.8**	55650	**2475**	16.33	1.79
	Per T	313.0	**245.7**	77020	**2850**	15.69	2.68
	Per m of wall	173.9	**136.5**	46380	**2060**	16.33	1.49
PU 22							
	Per S	109.7	**86.1**	8740	**546**	8.93	0.90
	Per D	219.5	**172.3**	59360	**2640**	16.45	1.79
	Per T	329.2	**258.4**	82060	**3025**	15.79	2.68
	Per m of wall	182.9	**143.6**	49460	**2200**	16.45	1.49

[1)] *One side, excluding inside of interlocks.*

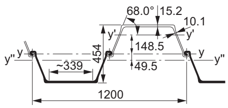
Section	S = Single pile D = Double pile T = Triple pile	Sectional area	Mass	Moment of inertia	Elastic section modulus	Radius of gyration	Coating area[1)]
		cm^2	kg/m	cm^4	cm^3	cm	m^2/m
PU 22^{+1}							



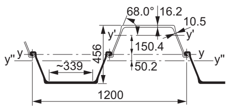
	Per S	115.2	**90.4**	9020	**555**	8.85	0.90
	Per D	230.4	**180.9**	63010	**2800**	16.54	1.79
	Per T	345.6	**271.3**	87020	**3205**	15.87	2.68
	Per m of wall	192.0	**150.7**	52510	**2335**	16.54	1.49
PU 28^{-1}							



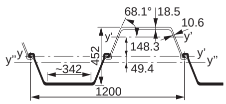
	Per S	124.1	**97.4**	9740	**576**	8.86	0.93
	Per D	248.2	**194.8**	72700	**3215**	17.12	1.85
	Per T	372.3	**292.2**	100170	**3645**	16.40	2.77
	Per m of wall	206.8	**162.3**	60580	**2680**	17.12	1.54
PU 28							



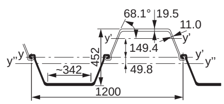
	Per S	129.7	**101.8**	10070	**589**	8.81	0.93
	Per D	259.4	**203.6**	77350	**3405**	17.27	1.85
	Per T	389.0	**305.4**	106490	**3850**	16.55	2.77
	Per m of wall	216.1	**169.6**	64460	**2840**	17.27	1.54
PU 28^{+1}							



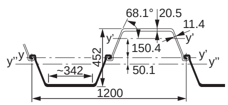
	Per S	135.3	**106.2**	10400	**600**	8.77	0.93
	Per D	270.7	**212.5**	82060	**3600**	17.41	1.85
	Per T	406.0	**318.7**	112870	**4060**	16.67	2.77
	Per m of wall	225.6	**177.1**	68380	**3000**	17.41	1.54
PU 32^{-1}							



	Per S	140.0	**109.9**	10740	**625**	8.76	0.92
	Per D	280.0	**219.8**	83050	**3675**	17.22	1.83
	Per T	420.0	**329.7**	114310	**4150**	16.50	2.74
	Per m of wall	233.3	**183.2**	69210	**3065**	17.22	1.52
PU 32							

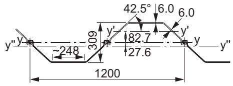
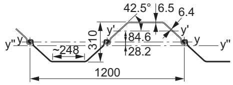
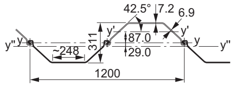
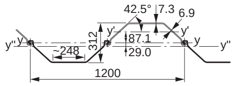
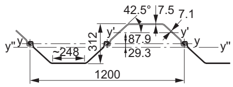
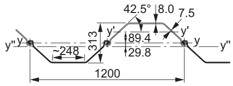
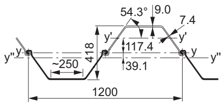


	Per S	145.4	**114.1**	10950	**633**	8.68	0.92
	Per D	290.8	**228.3**	86790	**3840**	17.28	1.83
	Per T	436.2	**342.4**	119370	**4330**	16.54	2.74
	Per m of wall	242.3	**190.2**	72320	**3200**	17.28	1.52
PU 32^{+1}							

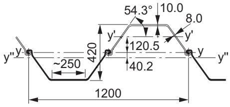
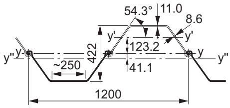
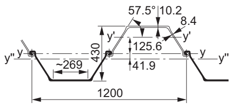
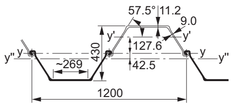
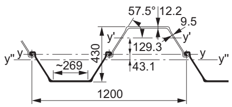
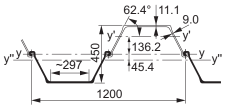
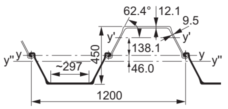


	Per S	150.8	**118.4**	11150	**640**	8.60	0.92
	Per D	301.6	**236.8**	90490	**4005**	17.32	1.83
	Per T	452.4	**355.2**	124370	**4505**	16.58	2.74
	Per m of wall	251.3	**197.3**	75410	**3340**	17.32	1.52

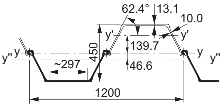
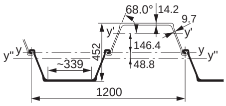
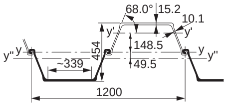
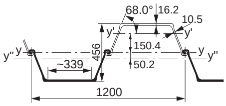
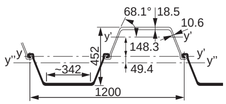
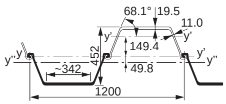
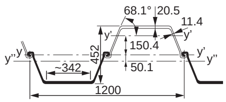
1) One side, excluding inside of interlocks.

Section	S = Single pile D = Double pile T = Triple pile	Sectional area	Mass	Moment of inertia	Elastic section modulus	Radius of gyration	Coating area[1)]
		cm²	kg/m	cm⁴	cm³	cm	m²/m
GU® sections							
GU 6N	Per S	53.4	**41.9**	2160	**215**	6.36	0.76
	Per D	106.8	**83.8**	11610	**750**	10.43	1.51
	Per T	160.2	**125.7**	16200	**890**	10.06	2.26
	Per m of wall	89.0	**69.9**	9670	**625**	10.43	1.26
GU 7N	Per S	56.2	**44.1**	2250	**220**	6.33	0.76
	Per D	112.4	**88.2**	12540	**810**	10.56	1.51
	Per T	168.6	**132.4**	17470	**955**	10.18	2.26
	Per m of wall	93.7	**73.5**	10450	**675**	10.56	1.26
GU 7S	Per S	60.2	**46.3**	2370	**225**	6.28	0.76
	Per D	120.3	**92.5**	13850	**890**	10.73	1.51
	Per T	180.5	**138.8**	19260	**1045**	10.33	2.26
	Per m of wall	100.3	**77.1**	11540	**740**	10.73	1.26
GU 7HWS	Per S	60.4	**47.4**	2380	**225**	6.28	0.76
	Per D	120.9	**94.9**	13940	**895**	10.74	1.51
	Per T	181.3	**142.3**	19390	**1050**	10.34	2.26
	Per m of wall	100.7	**79.1**	11620	**745**	10.74	1.26
GU 8N	Per S	61.8	**48.5**	2420	**225**	6.26	0.76
	Per D	123.7	**97.1**	14420	**925**	10.80	1.51
	Per T	185.5	**145.6**	20030	**1080**	10.39	2.26
	Per m of wall	103.1	**80.9**	12010	**770**	10.80	1.26
GU 8S	Per S	64.7	**50.8**	2510	**230**	6.23	0.76
	Per D	129.3	**101.5**	15360	**980**	10.90	1.51
	Per T	194.0	**152.3**	21320	**1145**	10.48	2.26
	Per m of wall	107.8	**84.6**	12800	**820**	10.90	1.26
GU 13N	Per S	76.3	**59.9**	5440	**395**	8.44	0.85
	Per D	152.6	**119.8**	31900	**1525**	14.46	1.69
	Per T	228.9	**179.7**	44350	**1785**	13.92	2.53
	Per m of wall	127.2	**99.8**	26590	**1270**	14.46	1.41

1) One side, excluding inside of interlocks.

Section	S = Single pile D = Double pile T = Triple pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
GU 14N 	Per S	81.9	**64.3**	5750	**410**	8.38	0.85
	Per D	163.8	**128.6**	35290	**1680**	14.68	1.69
	Per T	245.6	**192.8**	48970	**1955**	14.12	2.53
	Per m of wall	136.5	**107.1**	29410	**1400**	14.68	1.41
GU 15N 	Per S	87.5	**68.7**	6070	**425**	8.33	0.85
	Per D	175.1	**137.4**	38710	**1835**	14.87	1.69
	Per T	262.6	**206.2**	53640	**2130**	14.29	2.53
	Per m of wall	145.9	**114.5**	32260	**1530**	14.87	1.41
GU 16N 	Per S	92.5	**72.6**	6960	**475**	8.67	0.87
	Per D	185.0	**145.2**	43140	**2005**	15.30	1.72
	Per T	277.5	**217.8**	59840	**2330**	14.69	2.58
	Per m of wall	154.2	**121.0**	35950	**1670**	15.30	1.43
GU 18N 	Per S	98.0	**76.9**	7220	**485**	8.58	0.87
	Per D	196.0	**153.8**	46380	**2160**	15.38	1.72
	Per T	294.0	**230.7**	64240	**2495**	14.78	2.58
	Per m of wall	163.3	**128.2**	38650	**1800**	15.38	1.43
GU 20N 	Per S	103.4	**81.1**	7480	**495**	8.51	0.87
	Per D	206.8	**162.3**	49580	**2305**	15.49	1.72
	Per T	310.2	**243.5**	68600	**2655**	14.87	2.58
	Per m of wall	172.3	**135.2**	41320	**1920**	15.49	1.43
GU 21N 	Per S	104.3	**81.9**	8460	**535**	9.01	0.90
	Per D	208.7	**163.8**	55650	**2475**	16.33	1.79
	Per T	313.0	**245.7**	77020	**2850**	15.69	2.68
	Per m of wall	173.9	**136.5**	46380	**2060**	16.33	1.49
GU 22N 	Per S	109.7	**86.1**	8740	**546**	8.93	0.90
	Per D	219.5	**172.3**	59360	**2640**	16.45	1.79
	Per T	329.2	**258.4**	82060	**3025**	15.79	2.68
	Per m of wall	182.9	**143.6**	49460	**2200**	16.45	1.49

1) One side, excluding inside of interlocks.

Section	S = Single pile D = Double pile T = Triple pile	Sectional area	Mass	Moment of inertia	Elastic section modulus	Radius of gyration	Coating area[1)]
		cm²	kg/m	cm⁴	cm³	cm	m²/m
GU 23N 	Per S	115.2	**90.4**	9020	**555**	8.85	0.90
	Per D	230.4	**180.9**	63010	**2800**	16.54	1.79
	Per T	345.6	**271.3**	87020	**3205**	15.87	2.68
	Per m of wall	192.0	**150.7**	52510	**2335**	16.54	1.49
GU 27N 	Per S	124.1	**97.4**	9740	**576**	8.86	0.93
	Per D	248.2	**194.8**	72700	**3215**	17.12	1.85
	Per T	372.3	**292.2**	100170	**3645**	16.40	2.77
	Per m of wall	206.8	**162.3**	60580	**2680**	17.12	1.54
GU 28N 	Per S	129.7	**101.8**	10070	**589**	8.81	0.93
	Per D	259.4	**203.6**	77350	**3405**	17.27	1.85
	Per T	389.0	**305.4**	106490	**3850**	16.55	2.77
	Per m of wall	216.1	**169.6**	64460	**2840**	17.27	1.54
GU 30N 	Per S	135.3	**106.2**	10400	**600**	8.77	0.93
	Per D	270.7	**212.5**	82060	**3600**	17.41	1.85
	Per T	406.0	**318.7**	112870	**4060**	16.67	2.77
	Per m of wall	225.6	**177.1**	68380	**3000**	17.41	1.54
GU 31N 	Per S	140.0	**109.9**	10740	**625**	8.76	0.92
	Per D	280.0	**219.8**	83050	**3675**	17.22	1.83
	Per T	420.0	**329.7**	114310	**4150**	16.50	2.74
	Per m of wall	233.3	**183.2**	69210	**3065**	17.22	1.52
GU 32N 	Per S	145.4	**114.1**	10950	**633**	8.68	0.92
	Per D	290.8	**228.3**	86790	**3840**	17.28	1.83
	Per T	436.2	**342.3**	119370	**4330**	16.54	2.74
	Per m of wall	242.3	**190.2**	72320	**3200**	17.28	1.52
GU 33N 	Per S	150.8	**118.4**	11150	**640**	8.60	0.92
	Per D	301.6	**236.8**	90490	**4005**	17.32	1.83
	Per T	452.4	**355.2**	124370	**4505**	16.58	2.74
	Per m of wall	251.3	**197.3**	75410	**3340**	17.32	1.52

1) One side, excluding inside of interlocks.

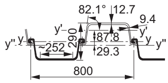
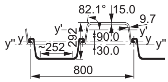
Section	S = Single pile D = Double pile T = Triple pile	Sectional area cm²	Mass kg/m	Moment of inertia cm⁴	Elastic section modulus cm³	Radius of gyration cm	Coating area[1)] m²/m
GU 16-400							
	Per S	78.9	**62.0**	2950	**265**	6.11	0.65
	Per D	157.9	**123.9**	18060	**1245**	10.70	1.28
	Per T	236.8	**185.9**	25060	**1440**	10.29	1.92
	Per m of wall	197.3	**154.9**	22580	**1560**	10.70	1.60
GU 18-400							
	Per S	88.3	**69.3**	3290	**290**	6.10	0.65
	Per D	176.7	**138.7**	20870	**1430**	10.87	1.28
	Per T	265.0	**208.0**	28920	**1645**	10.45	1.92
	Per m of wall	220.8	**173.3**	26090	**1785**	10.87	1.60

Table 1.8. Technical information of U sections.

[1)] One side, excluding inside of interlocks.

1.5.2. Interlocking in pairs

U piles can be supplied as single piles and double piles. U piles supplied interlocked in pairs minimise the number of handling and pitching operations on site.

It should be noted however that when interlocked in pairs, the resulting shape is asymmetric requiring care when stacking.

When U piles are interlocked prior to delivery in pairs there are two possible orientations when viewed from the end of the pile with the lifting hole as illustrated in Fig. 1.8. The orientation can be reversed by burning lifting holes at the bottom of the pile and picking it up using the revised holes.

The proportion of full section modulus of U-piles developed in different circumstances is accounted for by the factors in EN 1993 part 5 and are illustrated in the Design Chapters.

1.5.3. Crimping and welding of the interlocks

Pairs of piles can be crimped or welded together if required. Normally 3 to 4 crimps per metre are requested but other configurations can be accommodated with prior agreement. Each crimp is applied to provide an allowable shear resistance of 75 kN with less than 5 mm movement.

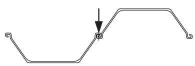
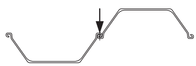
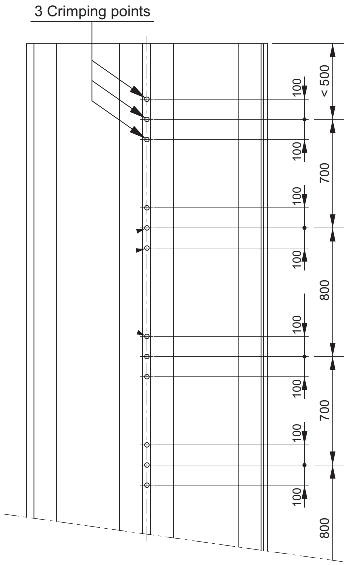
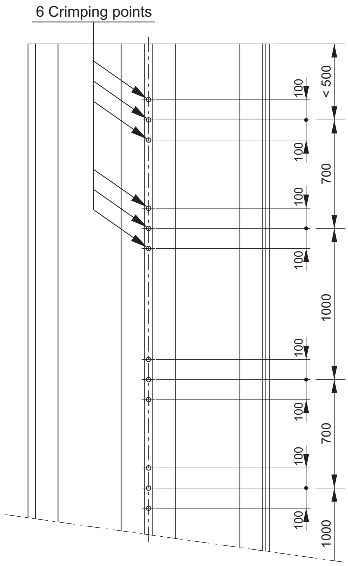
AU™ section standard crimping 	**PU®/GU® section standard crimping** 
3 crimping points per 0.75 m = 4 crimping points per m[1)]	6 crimping points per 1.7 m = 3.5 crimping points per m[1)]
	

Fig. 1.7. Standard crimping of AU, PU/GU section.

1) Amount and layout of crimping points may differ at both ends. Special crimping on request.

1.5.4. Pile form

Piles can be supplied as illustrated.

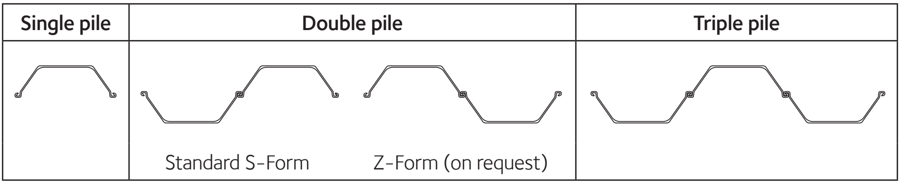


Fig. 1.8. Delivery forms of U sections.

1.5.5. Circular construction

Steel sheet piling can be driven to form a complete circle without the need for corner piles. The maximum angle of deviation for AU™, PU® and GU® sections is 5° for single piles.

The following table gives the approximate minimum diameters of circular cofferdam which can be constructed using various sheet pile sections. The diameters are only intended to be for guidance as the actual interlock deviation achieved will be a function of the pile length, the pile section, the penetration required. Smaller diameters can be achieved by introducing bent corner piles, but larger diameters will result from using pairs of piles that have been crimped or welded.

Section	Minimum number of single piles used Angle $\alpha = 5°$	Approx. min diameter to internal face of wall *m*
AU 14, AU 16, AU 17	72	16.8
AU 18, AU 20, AU 21	72	16.8
AU 23, AU 25, AU 26	72	16.8
PU 12, PU 12-10/10	72	13.4
PU 18^{-1}, PU 18, PU 18^{+1}	72	13.3
PU 22^{-1}, PU 22, PU 22^{+1}	72	13.3
PU 28^{-1}, PU 28, PU 28^{+1}	72	13.3
PU 32	72	13.3
GU 6N, GU 7N, GU 7S, GU 8N, GU 8S	72	13.4
GU 12-500, GU 13-500, GU 15-500	72	11.1
GU 16-400, GU 18-400	72	8.9

Table 1.9. Approximate min. diameter of circular construction.

1.6. Straight web piles

1.6.1. Dimensions and properties for AS 500® straight web piles

AS 500 straight web sheet piles are designed to form closed cylindrical structures retaining a soil fill. The stability of the cells consisting of a steel envelope and an internal body of soil is guaranteed by their own weight. Straight web sheet piles are mostly used on projects where rock layers are close to ground level or where anchoring would be difficult or impossible. Straight web sheet pile structures are made of circular cells or diaphragm cells, depending on the site characteristics or the particular requirements of the project. The forces developing in these sheet pile sections are essentially horizontal tensile forces requiring an interlock strength corresponding to the horizontal force in the web of the pile. AS 500 interlocks comply with EN 10248. **Please refer to our brochure "AS 500® straight web steel sheet piles - design & execution manual" for further details.**

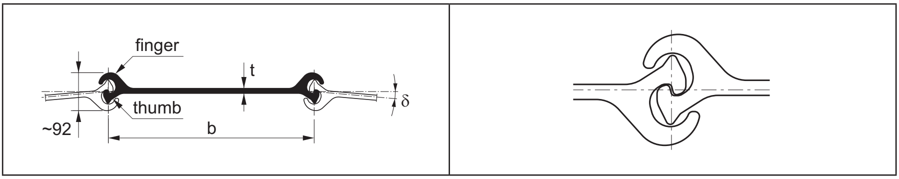


Fig. 1.9. Dimensions and properties for AS 500® straight web piles.

Section	Nominal width 1)	Web thickness	Deviation angle 2)	Perimeter	Steel section	Mass	Mass per m² of wall	Moment of inertia	Section modulus	Coating area 3)
				(single pile)				(single pile)		
	b	t	δ							
	mm	mm	°	cm	cm²	kg/m	kg/m²	cm⁴	cm³	m²/m
AS 500 - 9.5	500	9.5	4.5	138	81.3	63.8	128	168	46	0.58
AS 500 - 11.0	500	11.0	4.5	139	89.4	70.2	140	186	49	0.58
AS 500 - 12.0	500	12.0	4.5	139	94.6	74.3	149	196	51	0.58
AS 500 - 12.5	500	12.5	4.5	139	97.2	76.3	153	201	51	0.58
AS 500 - 12.7	500	12.7	4.5	139	98.2	77.1	154	204	51	0.58
AS 500 - 13.0 4)	500	13.0	4.5	140	100.6	79.0	158	213	54	0.58

Table 1.10. AS 500 sheet piles.

Note: All straight web sections interlock with each other.

1) The effective width to be taken into account for design purposes (layout) is 503 mm for all AS 500 sheet piles.

2) Max. deviation angle 4.0° for pile lengths > 20 m.

3) One side, excluding inside of interlocks.

4) Please contact ArcelorMittal Sheet Piling for further information.

Interlock Strength

The interlock complies with EN 10248. In Table 1.11., maximum guaranteed interlock resistances $R_{k,s}$ are listed. Verification of the sheet piles should consider the resistance of the interlocks and the web.

Section	$R_{k,s}$ [kN/m]
AS 500 - 9.5	3000
AS 500 - 11.0	3500
AS 500 - 12.0	5000
AS 500 - 12.5	5500
AS 500 - 12.7	5500
AS 500 - 13.0	6000

Table 1.11. Maximum guaranteed resistances of the interlocks.
Note: For the related steel grade to the values in Table 1.11. please contact ArcelorMittal Sheet Piling.

Junction piles

In general junction piles are assembled by welding in accordance with EN 12063. The connecting angle θ should be in the range from 30° to 45°.

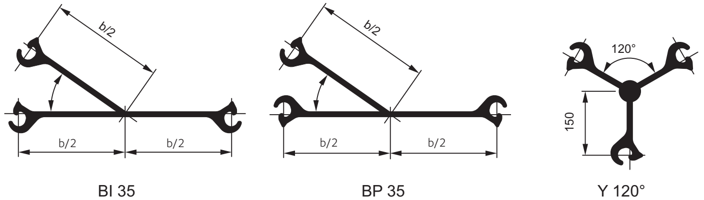
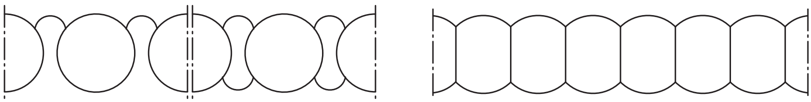


Fig. 1.10. Junction piles.

Types of cell



Circular cells with 35° junction piles and one or two connecting arcs.

Diaphragm cells with 120° junction piles.

Fig. 1.11. Type of cells.

Bent piles

If deviation angles exceeding the values given in Table 1.10. have to be attained, piles pre-bent in the mill may be used. Generally, β should be limited to 12°.

Fig. 1.12. AS 500 bent piles.

1.6.2. Box piles

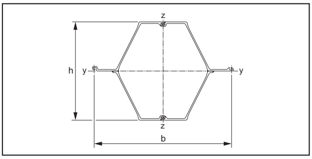
Welded box piles are fabricated from conventional hot rolled sheet piles. Welding details are available on request at ArcelorMittal Sheetpiling.

Box piles, formed from four single AZ sections or a pair of U sections, can be conveniently introduced into a line of sheet piling at any point where heavy loads are to be applied. They can be used to resist vertical and horizontal forces and can generally be positioned in the wall such that its appearance is unaffected.

Boxes may also be used as individual bearing piles for foundations or in open jetty and dolphin construction. Their large radius of gyration makes them particularly suitable for situations where construction involves long lengths of pile with little or no lateral support.

In general, box piles are driven open ended. Soil displacement and ground heave is normally eliminated since the soil enters the open end of the pile during initial penetration and forms an effective plug as the toe depth increases. Box piles can be driven into all normal soils, very compact ground and soft rocks.

CAZ box piles are formed by welding together two pairs of interlocked and intermittently welded AZ sheet piles.

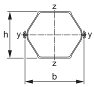


Section	Width	Height	Perimeter	Sectional area	Total section	Mass[1]	Moment of inertia		Elastic section modulus		Min. radius of gyration	Coating area[2]
	b mm	*h* mm	cm	cm^2	cm^2	kg/m	*y-y* cm^4	*z-z* cm^4	*y-y* cm^3	*z-z* cm^3	cm	m^2/m
CAZ-800 box piles												
CAZ 18-800	1600	898	438	363	7340	**285**	339470	650340	**7535**	**7915**	30.6	4.16
CAZ 20-800	1600	900	438	400	7372	**314**	372430	713410	**8250**	**8690**	30.5	4.16
CAZ 22-800	1600	902	439	436	7404	**342**	405710	776690	**8965**	**9465**	30.5	4.16
CAZ 23-800	1600	948	445	423	7764	**332**	447370	756450	**9405**	**9170**	32.5	4.24
CAZ 25-800	1600	950	446	460	7796	**361**	484690	820800	**10170**	**9990**	32.5	4.24
CAZ 27-800	1600	952	446	497	7829	**390**	522220	885310	**10930**	**10750**	32.4	4.24
CAZ-750 box piles												
CAZ 28-750	1500	1018	445	453	7829	**356**	547100	702950	**10715**	**9080**	34.8	4.23
CAZ 30-750	1500	1020	446	490	7861	**385**	590180	758880	**11535**	**9840**	34.7	4.23
CAZ 32-750	1500	1022	446	527	7892	**414**	633500	815060	**12360**	**10535**	34.7	4.23
CAZ-700 and CAZ-770 box piles												
CAZ 12-770	1540	687	389	328	5431	**257**	175060	557990	**5075**	**6985**	23.1	3.67
CAZ 13-770	1540	688	389	344	5446	**270**	183440	584640	**5310**	**7320**	23.1	3.67
CAZ 14-770	1540	689	390	360	5461	**283**	191840	611300	**5545**	**7655**	23.1	3.67
CAZ 14-770 -10/10	1540	690	390	376	5476	**295**	200280	637960	**5780**	**7995**	23.1	3.67

1) *The mass of the welds is not taken into account.*
2) *Outside surface, excluding inside of interlocks.*

Section	Width	Height	Perimeter	Sectional area	Total section	Mass[1)]	Moment of inertia		Elastic section modulus		Min. radius of gyration	Coating area[2)]
	b mm	*h* mm	cm	cm^2	cm^2	kg/m	*y-y* cm^4	*z-z* cm^4	*y-y* cm^3	*z-z* cm^3	cm	m^2/m
CAZ 12-700	1400	628	360	303	4524	**238**	137770	421600	**4365**	**5785**	21.3	3.39
CAZ 13-700	1400	630	361	332	4552	**261**	150890	461210	**4765**	**6335**	21.3	3.39
CAZ 13-700-10/10	1400	631	361	347	4565	**272**	157530	481090	**4965**	**6610**	21.3	3.39
CAZ 14-700	1400	632	361	362	4579	**284**	164130	500820	**5165**	**6885**	21.3	3.39
CAZ 17-700	1400	839	391	330	6015	**259**	265280	457950	**6300**	**6285**	28.3	3.69
CAZ 18-700	1400	840	391	347	6029	**272**	277840	479790	**6590**	**6590**	28.3	3.69
CAZ 20-700	1400	842	392	379	6058	**297**	303090	523460	**7170**	**7195**	28.3	3.69
CAZ 24-700	1400	918	407	436	6616	**342**	412960	596900	**8965**	**8260**	30.8	3.85
CAZ 26-700	1400	920	407	469	6645	**368**	444300	641850	**9625**	**8900**	30.8	3.85
CAZ 28-700	1400	922	408	503	6674	**395**	475810	686880	**10285**	**9510**	30.8	3.85
CAZ 24-700N	1400	918	407	401	6596	**315**	397130	550030	**8620**	**7655**	31.5	3.85
CAZ 26-700N	1400	920	407	434	6625	**341**	428490	594860	**9280**	**8280**	31.4	3.85
CAZ 28-700N	1400	922	408	468	6654	**367**	460020	639700	**9940**	**8905**	31.4	3.85
CAZ 36-700N	1400	998	434	534	7215	**419**	627000	710770	**12525**	**9895**	34.3	4.12
CAZ 38-700N	1400	1000	435	570	7245	**447**	667900	757530	**13315**	**10550**	34.2	4.12
CAZ 40-700N	1400	1002	436	606	7275	**476**	709010	804300	**14105**	**11205**	34.2	4.12
CAZ 42-700N	1400	998	433	646	7267	**507**	744440	855860	**14870**	**11915**	34.0	4.11
CAZ 44-700N	1400	1000	434	682	7298	**535**	785620	902800	**15660**	**12570**	33.9	4.11
CAZ 46-700N	1400	1002	434	718	7328	**564**	827030	949760	**16455**	**13225**	33.9	4.11
CAZ 48-700	1400	1006	435	710	7346	**558**	845530	931330	**16745**	**12965**	34.5	4.13
CAZ 50-700	1400	1008	435	746	7376	**586**	887420	977550	**17540**	**13620**	34.5	4.13
CAZ 52-700	1400	1010	436	782	7406	**614**	929550	1023800	**18335**	**14255**	34.5	4.13
CAZ box piles												
CAZ 18	1260	760	361	333	4925	**261**	222930	365500	**5840**	**5560**	25.9	3.41
CAZ 26	1260	854	377	440	5566	**346**	366820	480410	**8555**	**7385**	28.9	3.57
CAZ 46	1160	962	401	595	5831	**467**	645940	527590	**13380**	**8825**	32.9	3.81
CAZ 48	1160	964	402	628	5858	**493**	681190	556070	**14080**	**9300**	32.9	3.81
CAZ 50	1160	966	402	661	5884	**519**	716620	584560	**14780**	**9780**	32.9	3.81

Table 1.12. Dimensions and properties of CAZ box piles.

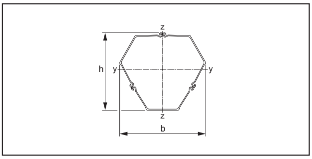


Section	Width	Height	Perimeter	Sectional area	Total section	Mass[1)]	Moment of inertia		Elastic section modulus		Min. radius of gyration	Coating area[2)]
	b	*h*					*y-y*	*z-z*	*y-y*	*z-z*		
	mm	mm	cm	cm^2	cm^2	kg/m	cm^4	cm^4	cm^3	cm^3	cm	m^2/m
CAU double box piles												
CAU 14-2	750	451	230	198	2598	**155.8**	54400	121490	**2415**	**3095**	16.6	2.04
CAU 16-2	750	454	231	220	2620	**172.5**	62240	130380	**2745**	**3325**	16.8	2.04
CAU 18-2	750	486	239	225	2888	**177.0**	73770	142380	**3035**	**3625**	18.1	2.14
CAU 20-2	750	489	240	247	2910	**193.8**	83370	151220	**3405**	**3850**	18.4	2.14
CAU 23-2	750	492	244	260	3013	**204.2**	94540	157900	**3845**	**4020**	19.1	2.19
CAU 25-2	750	495	245	281	3034	**220.8**	104810	166600	**4235**	**4240**	19.3	2.19
CU double box piles												
CU 12-2	600	403	198	168	1850	**132.2**	34000	70000	**1685**	**2205**	14.2	1.72
CU 12 -10/10-2	600	403	198	177	1850	**139.2**	35580	73460	**1765**	**2315**	14.2	1.72
CU 18-2	600	473	212	196	2184	**153.8**	58020	78300	**2455**	**2470**	17.2	1.86
CU 22-2	600	494	220	219	2347	**172.3**	73740	88960	**2985**	**2800**	18.3	1.94
CU 28-2	600	499	226	259	2468	**203.6**	96000	103560	**3850**	**3260**	19.2	2.00
CU 32-2	600	499	223	291	2461	**228.3**	108800	109200	**4360**	**3435**	19.3	1.97
CGU double box piles												
CGU 7N-2	600	348	187	112	1596	**88.2**	16510	48530	**950**	**1535**	12.1	1.62
CGU 7S-2	600	349	188	120	1604	**92.5**	18210	50630	**1045**	**1605**	12.3	1.62
CGU 14N-2	600	461	205	164	2079	**128.6**	44070	65550	**1910**	**2075**	16.4	1.79
CGU 18N-2	600	473	212	196	2184	**153.8**	58020	78300	**2455**	**2470**	17.2	1.86
CGU 22N-2	600	494	220	219	2347	**172.3**	73740	88960	**2985**	**2800**	18.3	1.94
CGU 28N-2	600	499	226	259	2468	**203.6**	96000	103560	**3850**	**3260**	19.2	2.00
CGU 32N-2	600	499	223	291	2461	**228.3**	108800	109200	**4360**	**3435**	19.3	1.97
CGU 16-400	400	336	169	158	1170	**123.9**	25270	31900	**1505**	**1465**	12.7	1.40

Table 1.13. Dimensions and properties of CAU, CU and CGU double box piles.

1) The mass of the welds is not taken into account.

2) Outside surface, excluding inside of interlocks.

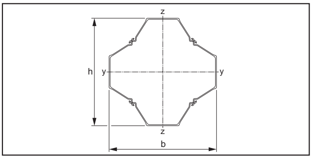


Section	Width	Height	Perimeter	Sectional area	Total section	Mass[1)]	Moment of inertia	Elastic section modulus		Min. radius of gyration	Coating area[2)]
	b	*h*					*y-y* / *z-z*	*y-y*	*z-z*		
	mm	mm	cm	cm^2	cm^2	kg/m	cm^4	cm^3	cm^3	cm	m^2/m
CAU triple box piles											
CAU 14-3	957	908	341	298	6454	**233.7**	300330	**6510**	**6275**	31.7	3.03
CAU 16-3	960	910	342	330	6486	**258.7**	333640	**7235**	**6955**	31.8	3.03
CAU 18-3	1009	927	355	338	6886	**265.5**	363690	**7825**	**7205**	32.8	3.17
CAU 20-3	1012	928	356	370	6919	**290.7**	399780	**8570**	**7900**	32.9	3.17
CAU 23-3	1036	930	361	390	7073	**306.3**	431940	**9235**	**8340**	33.3	3.24
CAU 25-3	1038	931	364	422	7106	**331.3**	469030	**9995**	**9035**	33.3	3.24
CU triple box piles											
CU 12-3	800	755	293	253	4431	**198.3**	173100	**4555**	**4325**	26.2	2.54
CU 12-10/10-3	800	755	293	266	4432	**208.8**	182100	**4790**	**4555**	26.2	2.54
CU 18-3	877	790	315	294	4931	**230.7**	227330	**5475**	**5185**	27.8	2.76
CU 22-3	912	801	326	329	5174	**258.4**	268440	**6310**	**5890**	28.6	2.87
CU 28-3	938	817	336	389	5356	**305.4**	330290	**7720**	**7040**	29.1	2.96
CU 32-3	926	809	331	436	5345	**342.4**	367400	**8585**	**7935**	29.0	2.92
CGU triple box piles											
CGU 14N-3	844	781	305	246	4763	**192.8**	182730	**4475**	**4330**	27.3	2.65
CGU 18N-3	877	790	315	294	4931	**230.7**	227330	**5475**	**5185**	27.8	2.76
CGU 22N-3	912	801	326	329	5174	**258.4**	268440	**6310**	**5890**	28.6	2.87
CGU 28N-3	938	817	336	389	5356	**305.4**	330290	**7720**	**7040**	29.1	2.96
CGU 32N-3	926	809	331	436	5345	**342.4**	367400	**8585**	**7935**	29.0	2.92

Table 1.14. Dimensions and properties of CAU, CU and CGU triple box piles.

1) The mass of the welds is not taken into account.

2) Outside surface, excluding inside of interlocks.



Section	Width	Height	Perimeter	Sectional area	Total section	Mass[1)]	Moment of inertia		Elastic section modulus		Min. radius of gyration	Coating area[2)]
	b mm	*h* mm	cm	cm^2	cm^2	kg/m	*y-y* cm^4	*z-z* cm^4	*y-y* cm^3	*z-z* cm^3	cm	m^2/m
CAU quadruple box piles												
CAU 14-4	1222	1222	453	397	11150	311.6	692030		11325		41.7	4.02
CAU 16-4	1225	1225	454	440	11193	345.0	770370		12575		41.8	4.02
CAU 18-4	1258	1258	471	451	11728	354.0	826550		13140		42.8	4.20
CAU 20-4	1261	1261	472	494	11771	387.6	910010		14430		42.9	4.20
CAU 23-4	1263	1263	481	520	11977	408.4	979870		15510		43.4	4.30
CAU 25-4	1266	1266	482	563	12020	441.6	1064910		16820		43.5	4.30
CU quadruple box piles												
CU 12-4	1025	1025	388	337	7565	264.4	394000		7690		34.2	3.36
CU 12-10/10-4	1025	1025	388	355	7565	278.4	414830		8095		34.2	3.36
CU 18-4	1095	1095	417	392	8231	307.6	507240		9270		36.0	3.65
CU 22-4	1115	1115	432	439	8556	344.6	593030		10635		36.8	3.80
CU 28-4	1120	1120	445	519	8799	407.2	725730		12955		37.4	3.93
CU 32-4	1120	1120	440	582	8782	456.6	811100		14480		37.3	3.87
CGU quadruple box piles												
CGU 14N-4	1081	1081	404	328	7997	257.1	409870		7585		35.4	3.51
CGU 18N-4	1095	1095	417	392	8231	307.6	507240		9270		36.0	3.65
CGU 22N-4	1115	1115	432	439	8556	344.6	593030		10635		36.8	3.80
CGU 28N-4	1120	1120	445	519	8799	407.2	725730		12955		37.4	3.93
CGU 32N-4	1120	1120	440	582	8782	456.6	811100		14480		37.3	3.87

Table 1.15. Dimensions and properties of CAU, CU and CGU quadruple box piles.

1) The mass of the welds is not taken into account.
2) Outside surface, excluding inside of interlocks.

1.7. Combined wall systems

The equivalent elastic section modulus W_{sys} per linear metre of combined wall is based on the assumption that the deflections of king piles and intermediary steel sheet piles are the same, leading to the following formulas:

$$I_{sys} = \frac{I_{king\ pile} + I_{ssp}}{b_{sys}}$$

$$W_{sys} = \frac{W_{king\ pile}}{b_{sys}} \times \left(\frac{I_{king\ pile} + I_{ssp}}{I_{king\ pile}} \right)$$

I_{sys}	[cm^4/m]	:	Moment of inertia of combined wall
W_{sys}	[cm^3/m]	:	Elastic section modulus of combined wall
$I_{king\ pile}$	[cm^4]	:	Moment of inertia of king pile
I_{ssp}	[cm^4]	:	Moment of inertia of intermediary sheet pile
$W_{king\ pile}$	[cm^3]	:	Elastic section modulus of king pile
b_{sys}	[m]	:	System width

1.7.1. HZ®/AZ® combined wall system

The HZ®-M wall is a combined wall system involving HZ®-M king piles as the main structural support elements and AZ® sheet piles as the infill members with special connectors to join the parts together.

The tables in this chapter give dimensions and properties for the component parts.

The outstanding feature of the HZ®/AZ® combined wall system is the extensive range of possible combinations using the entire AZ sheet pile offer, including the latest wide AZ-800 range, as well as all rolled-up and rolled-down AZ sections. **Please refer to our brochure "The HZ®-M Steel Wall System" for detailed information on the entire HZ®/AZ® range.**

Section (Sol. 102)	Dimensions							Sectional area	Mass	Moment of inertia	Elastic section modulus	Coating area	Connectors
	h mm	h_1 mm	*b* mm	t_{max} mm	*t* mm	*s* mm	*r* mm	cm²	kg/m	*y-y* cm⁴	*y-y* cm³	m²/m	
HZ 680M LT	631.8	599.9	460	29.0	16.9	14.0	20	257.8	**202.4**	177370	**5840**	3.05	A
HZ 880M A	831.3	803.4	458	29.0	18.9	13.0	20	292.4	**229.5**	351350	**8650**	3.44	A
HZ 880M B	831.3	807.4	460	29.0	20.9	15.0	20	324.7	**254.9**	386810	**9480**	3.45	A
HZ 880M C	831.3	811.4	460	29.0	22.9	15.0	20	339.2	**266.3**	410830	**10025**	3.45	A
HZ 1080M A	1075.3	1047.4	454	29.0	19.6	16.0	35	371.1	**291.3**	696340	**13185**	3.87	A
HZ 1080M B	1075.3	1053.4	454	29.0	22.6	16.0	35	394.1	**309.4**	760600	**14315**	3.87	A
HZ 1080M C	1075.3	1059.4	456	29.0	25.7	18.0	35	436.1	**342.4**	839020	**15715**	3.87	A
HZ 1080M D	1075.3	1067.4	457	30.7	29.7	19.0	35	470.1	**369.0**	915420	**17025**	3.87	A
HZ 1180M A	1075.4	-	458	34.7	31.0	20.0	35	497.3	**390.4**	973040	**17970**	3.88	A
HZ 1180M B	1079.4	-	458	36.7	33.0	20.0	35	514.5	**403.9**	1022780	**18785**	3.89	A
HZ 1180M C	1083.4	-	459	38.7	35.0	21.0	35	543.6	**426.8**	1086840	**19895**	3.90	B
HZ 1180M D	1087.4	-	460	40.7	37.0	22.0	35	570.5	**447.8**	1144400	**20795**	3.91	B
Connectors													
RH 16	61.8		68.2			12.2		20.1	**15.8**	83	**25**		A
RZD 16	61.8		80.5					20.7	**16.2**	57	**18**		A
RZU 16	61.8		80.5					20.4	**16.0**	68	**18**		A
RH 20	67.3		79.2			14.2		25.2	**19.8**	122	**33**		B
RZD 18	67.3		85.0					23.0	**18.0**	78	**22**		B
RZU 18	67.3		85.0					22.6	**17.8**	92	**22**		B

Table 1.16. Dimensions and properties of HZ®-M system, solution 102.

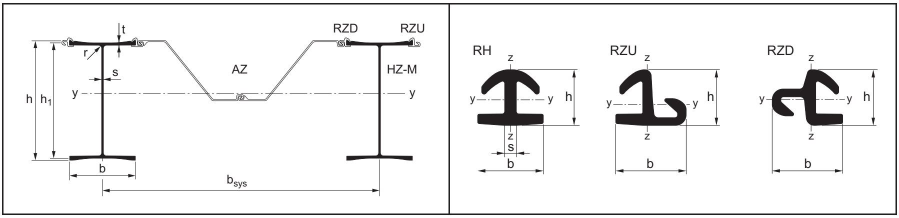


Fig. 1.13. Solution 12 and interlock types of HZ®/AZ® system.

The outstanding feature of this form of wall is the range of options that can be created by combining different beams, sheet piles and connectors.

For example the combination of a single beam and sheet pile with connectors to join everything together (solution 12) can be modified by adding additional "connectors" to the rear flange of the beam at the level of highest bending moment applied (solution 14) or by adopting two beams for every pair of sheet piles (solution 24 respectively 26).

Table 1.7. gives an indication of what properties can be generated for particular combinations of components. A maximum Section modulus of 46500 cm³/m can be achieved.

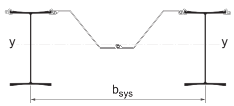
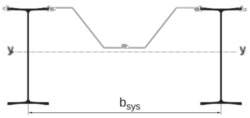
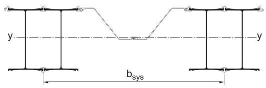
	Section	Sectional area	Momentof inertia	Elastic[1)] section modulus	Elastic[2)] section modulus	Mass[3)]		Coating area[4)]
						$Mass_{100}$	$Mass_{60}$	Water side
		cm^2/m	cm^4/m	cm^3/m	cm^3/m	kg/m^2	kg/m^2	m^2/m
Combination HZ...M - 12 / AZ 18-700	**HZ 680M LT**	255.9	136490	**4035**	4580	**201**	162	2.48
	HZ 880M A	274.1	240500	**5380**	6160	**215**	177	2.48
	**HZ 880M B**	290.6	259000	**5820**	6560	**228**	190	2.48
	HZ 880M C	298.1	271570	**6100**	6850	**234**	196	2.48
	HZ 1080M A	315.6	443030	**7745**	8690	**248**	209	2.47
	HZ 1080M B	327.6	476790	**8340**	9295	**257**	219	2.47
	HZ 1080M C	349.0	517420	**9065**	10010	**274**	235	2.48
	HZ 1080M D	366.5	557070	**9735**	10720	**288**	249	2.48
b_{sys} = 1.927 m	**HZ 1180M A**	380.4	586870	**10220**	11255	**299**	260	2.48
	HZ 1180M B	389.3	613030	**10680**	11705	**306**	267	2.48
	HZ 1180M C	406.5	651410	**11275**	12410	**319**	280	2.49
	HZ 1180M D	420.2	681600	**11830**	12895	**330**	291	2.50
Combination HZ...M - 12 / AZ 25-800	**HZ 680M LT**	263.0	143460	**4245**	4810	**206**	162	2.73
	HZ 880M A	279.5	237700	**5315**	6085	**219**	175	2.73
	**HZ 880M B**	294.4	254470	**5720**	6445	**231**	187	2.74
	HZ 880M C	301.2	265850	**5970**	6705	**236**	192	2.74
	HZ 1080M A	317.1	421160	**7365**	8260	**249**	204	2.73
	HZ 1080M B	327.9	451740	**7900**	8810	**257**	213	2.73
	HZ 1080M C	347.4	488570	**8560**	9455	**273**	228	2.73
	HZ 1080M D	363.2	524500	**9165**	10095	**285**	240	2.73
b_{sys} = 2.127 m	**HZ 1180M A**	375.8	551520	**9605**	10575	**295**	250	2.73
	HZ 1180M B	383.9	575220	**10020**	10980	**301**	257	2.74
	HZ 1180M C	399.5	610010	**10555**	11625	**314**	268	2.75
	HZ 1180M D	411.9	637390	**11065**	12060	**323**	278	2.75
Combination HZ...M - 24 / AZ 18-700	**HZ 680M LT**	326.8	197110	**6145**	5515	**257**	226	3.00
	HZ 880M A	356.2	363720	**8525**	7885	**280**	249	3.00
	**HZ 880M B**	382.2	392360	**9200**	8550	**300**	269	3.01
	HZ 880M C	394.3	412400	**9645**	9005	**309**	279	3.01
	HZ 1080M A	423.2	688290	**12515**	11775	**332**	301	2.99
	HZ 1080M B	442.2	741310	**13440**	12715	**347**	316	2.99
	HZ 1080M C	476.5	805720	**14585**	13870	**374**	343	3.00
	HZ 1080M D	504.4	868900	**15660**	15000	**396**	365	3.00
b_{sys} = 2.398 m	**HZ 1180M A**	526.7	916220	**16425**	15845	**413**	383	3.00
	HZ 1180M B	540.0	955000	**17075**	16535	**424**	393	3.00
	HZ 1180M C	569.5	1022790	**18200**	17595	**447**	416	3.02
	HZ 1180M D	589.3	1064090	**18895**	18330	**463**	431	3.03

Table 1.17. HZ®/AZ® combination (sample).

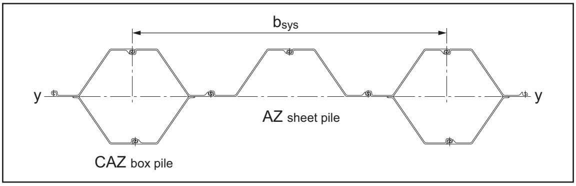
1) Referring outside of HZ-flange.

2) Referring outside of RH/RZ.

3) $L_{RH} = L_{HZ}$; $L_{RZU} = L_{RZD} = L_{AZ}$; $Mass_{100}$: L_{AZ} = 100 % L_{HZ} ; $Mass_{60}$: L_{AZ} = 60 % L_{HZ}

4) Excluding inside of interlocks, per system width.

1.7.2. Combined walls with Z-type sections

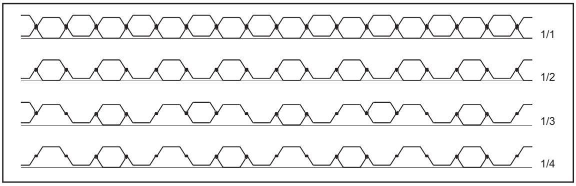


Combination	System width	Mass$_{100}$ 1)	Mass$_{60}$ 1)	Moment of inertia	Elastic section modulus
	b_{sys}			I_{sys}	W_{sys}
	mm	kg/m²	kg/m²	cm⁴/m	cm³/m
AZ-800					
CAZ 20-800 / AZ 13-770	3140	**148**	129	129580	**2870**
CAZ 20-800 / AZ 18-700	3000	**156**	135	141780	**3140**
CAZ 20-800 / AZ 20-800	3200	**153**	131	138910	**3075**
CAZ 25-800 / AZ 13-770	3140	**163**	144	165330	**3470**
CAZ 25-800 / AZ 18-700	3000	**171**	151	179200	**3760**
CAZ 25-800 / AZ 20-800	3200	**168**	146	173990	**3650**
AZ-750					
CAZ 30-750 / AZ 13-770	3040	**177**	157	205470	**4015**
CAZ 30-750 / AZ 18-700	2900	**185**	164	221760	**4335**
CAZ 30-750 / AZ 20-800	3100	**181**	158	213630	**4175**
AZ-700 and AZ-770					
CAZ 13-770 / AZ 13-770	3080	**137**	117	70740	**2045**
CAZ 13-700 / AZ 13-700	2800	**146**	125	64160	**2025**
CAZ 18-700 / AZ 13-770	2940	**144**	124	106220	**2520**
CAZ 18-700 / AZ 13-700	2800	**150**	129	109500	**2595**
CAZ 18-700 / AZ 18-700	2800	**152**	130	118130	**2800**
CAZ 26-700 / AZ 13-770	2940	**177**	156	162840	**3530**
CAZ 26-700 / AZ 13-700	2800	**185**	163	168950	**3660**
CAZ 26-700 / AZ 18-700	2800	**186**	164	177580	**3845**
CAZ 26-700N / AZ 13-770	2940	**168**	147	157460	**3410**
CAZ 26-700N / AZ 13-700	2800	**175**	154	163300	**3535**
CAZ 26-700N / AZ 18-700	2800	**176**	155	171930	**3725**
CAZ 38-700N / AZ 13-770	2940	**204**	183	238890	**4760**
CAZ 38-700N / AZ 13-700	2800	**213**	192	248800	**4960**
CAZ 38-700N / AZ 18-700	2800	**214**	193	257440	**5130**
CAZ 44-700N / AZ 13-770	2940	**234**	213	278930	**5560**
CAZ 44-700N / AZ 13-700	2800	**244**	223	290850	**5800**
CAZ 44-700N / AZ 18-700	2800	**246**	224	299480	**5970**
CAZ 50-700 / AZ 13-770	2940	**251**	230	313560	**6200**
CAZ 50-700 / AZ 18-700	2800	**264**	242	335840	**6640**
CAZ 50-700 / AZ 20-800	3000	**254**	231	319830	**6320**
AZ					
CAZ 18 / AZ 18	2520	**163**	139	105560	**2765**
CAZ 26 / AZ 18	2520	**196**	173	162660	**3795**
CAZ 48 / AZ 18	2420	**265**	241	299290	**6190**

Table 1.18. Combined walls with Z-type sections.

1) *$Mass_{100}$: L_{AZ} = 100% $L_{box\ pile}$; $Mass_{60}$: L_{AZ} = 60% $L_{box\ pile}$*

1.7.3. Combined walls with U-type sections



Section	1 / 1			1 / 2			1 / 3			1 / 4		
	Mass	Moment of inertia	Elastic section modulus	Mass	Moment of inertia	Elastic section modulus	Mass	Moment of inertia	Elastic section modulus	Mass	Moment of inertia	Elastic section modulus
	kg/m²	cm⁴/m	cm³/m	kg/m²	cm⁴/m	cm³/m	kg/m²	cm⁴/m	cm³/m	kg/m²	cm⁴/m	cm³/m
CAU box piles / AU™ sheet piles												
AU 14	**208**	72530	**3220**	**156**	40660	**1805**	**139**	43300	**1920**	**130**	37980	**1550**
AU 16	**230**	82990	**3660**	**173**	46230	**2035**	**153**	49560	**2185**	**144**	43440	**1755**
AU 18	**236**	98360	**4045**	**177**	55020	**2260**	**157**	58990	**2425**	**148**	51760	**1950**
AU 20	**258**	111160	**4545**	**194**	61830	**2525**	**172**	66680	**2725**	**162**	58460	**2180**
AU 23	**272**	126050	**5125**	**204**	69580	**2830**	**182**	75820	**3080**	**170**	66410	**2435**
AU 25	**294**	139750	**5645**	**221**	76800	**3105**	**196**	84080	**3395**	**184**	73590	**2675**
CU box piles / PU® sheet piles												
PU 12	**220**	56670	**2810**	**165**	32080	**1590**	**147**	33290	**1650**	**138**	29190	**1370**
PU 12-10/10	**232**	59300	**2945**	**174**	33480	**1660**	**155**	34820	**1730**	**145**	30520	**1430**
PU 18	**256**	96700	**4090**	**192**	54370	**2300**	**171**	58000	**2450**	**160**	50940	**1980**
PU 22	**287**	122900	**4975**	**215**	68730	**2785**	**192**	73940	**2995**	**180**	64920	**2395**
PU 28	**339**	160000	**6415**	**255**	88390	**3545**	**226**	96310	**3860**	**212**	84370	**3050**
PU 32	**381**	181330	**7270**	**285**	99790	**4000**	**254**	108660	**4355**	**238**	95070	**3445**
CGU box piles / GU® sheet piles												
GU 7N	**147**	27520	**1585**	**110**	15630	**900**	**98**	16140	**930**	**92**	14160	**775**
GU 7S	**154**	30350	**1740**	**116**	17150	**985**	**103**	17810	**1020**	**96**	15610	**845**
GU 14N	**214**	73440	**3185**	**161**	41520	**1800**	**143**	44090	**1915**	**134**	38760	**1550**
GU 18N	**256**	96700	**4090**	**192**	54370	**2300**	**171**	58000	**2450**	**160**	50940	**1980**
GU 22N	**287**	122900	**4975**	**215**	68730	**2785**	**192**	73940	**2995**	**180**	64920	**2395**
GU 28N	**339**	160000	**6415**	**255**	88390	**3545**	**226**	96310	**3860**	**212**	84370	**3050**
GU 32N	**381**	181330	**7270**	**285**	99790	**4000**	**254**	108660	**4355**	**238**	95070	**3445**
GU 16-400	**310**	63180	**3760**	**232**	35270	**2100**	**207**	36110	**2150**	**194**	31460	**1805**

Table 1.19. Combined walls with U-type sections.

Note: 1/1: only box piles
1/2: 1 box pile + 1 single pile
1/3: 1 box pile + 1 double pile
1/4: 1 box pile + 1 triple pile

1.7.4. Load bearing foundations

The development of rolled corner sections has enabled a new generation of bearing pile to be created. By interlocking a number of sheet piles with the same number of Omega bars, a closed tube results which can be driven into the ground sequentially. Using equipment that installs piles without noise and vibration, the ability to drive a closed section pile by pile means that load bearing foundations made of steel can be installed at sensitive sites and in urban areas where impact driven piles would not be tolerated.

In addition to the reduction in environmental disturbance offered by this system, the foundation is effectively load tested as it is installed and can be loaded immediately. Furthermore, the opportunity exists to extract the piles once the useful life of the structure is passed in a reversal of the installation process.

Table 1.20. gives the dimensions and properties for foundations created using 4, 5 and 6 sheet pile/omega 18 combinations and ultimate load capacities for both S 270 GP and S 355 GP steel grades. The capacity of the foundation in geotechnical terms will need to be assessed for the particular site location.

The effective radius of the pile (used for calculating torsional resistance) is the value given in the column headed “Max. boundary distance”.

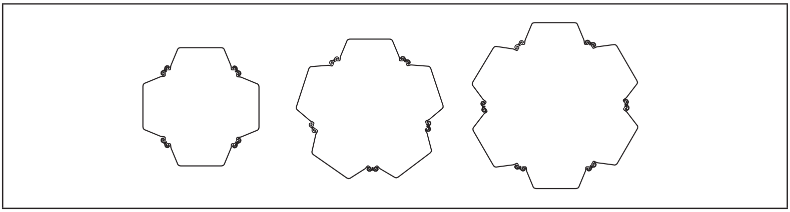


Fig. 1.14. Load bearing foundations using sheet piles.

Section		Steel area	Perimeter	Moment of inertia	Radius of gyration	Max. boundary distance	Elastic section modulus	Ultimate axial capacity		Coating area[1)]
								S 270 GP	S 355 GP	
		cm²	mm	cm⁴	mm	mm	cm³	kN	kN	m²
AU 16	4	531.2	4750	970430	427	632.7	**15340**	14342	18858	4.50
	5	664.1	5950	1826530	524	784.5	**23285**	17931	23575	5.62
	6	796.8	7160	3062970	620	928.9	**32975**	21514	28287	6.75
AU 20	4	585.5	4920	1114760	436	649.6	**17160**	15809	20784	4.67
	5	731.8	6170	2083840	534	801.3	**26005**	19759	25981	5.84
	6	878.2	7410	3476950	629	945.7	**36765**	23711	31177	7.01
AU 25	4	654.3	5020	1278190	442	652.4	**19595**	17666	23226	4.77
	5	817.8	6290	2383790	540	804.1	**29645**	22081	29033	5.96
	6	981.4	7560	3968200	636	948.5	**41835**	26498	34840	7.15
PU 12	4	428.5	4090	525260	350	522.6	**10050**	11570	15213	3.84
	5	535.7	5130	983560	429	655.9	**14995**	14464	19016	4.80
	6	642.8	6170	1645660	506	773.6	**21275**	17356	22819	5.76
PU 18	4	483.6	4380	645550	365	567.4	**11380**	13057	17167	4.13
	5	604.5	5490	1194290	444	690.9	**17285**	16322	21458	5.16
	6	725.4	6600	1979340	522	808.6	**24480**	19586	25750	6.19
PU 22	4	530.7	4520	735850	372	577.4	**12745**	14329	18840	4.27
	5	663.4	5670	1353710	452	700.9	**19315**	17912	23552	5.34
	6	796.1	6820	2233930	530	818.6	**27290**	21495	28260	6.41
PU 28	4	610.6	4620	875810	380	579.3	**15120**	16485	21675	4.28
	5	763.2	5770	1605200	460	702.9	**22840**	20605	27095	5.37
	6	915.9	6930	2640140	540	820.6	**32175**	24730	32515	6.47
PU 32	4	673.4	4580	964470	378	578.4	**16675**	18182	23905	4.33
	5	841.8	5740	1771180	459	701.9	**25235**	22729	29883	5.41
	6	1010.1	6900	2915900	537	819.6	**35575**	27273	35857	6.49
GU 13-500	4	403.9	3810	373590	304	472.3	**7910**	10906	14340	3.56
	5	504.9	4740	693400	371	590.3	**11750**	13633	17924	4.41
	6	605.9	5680	1153520	436	677.0	**17040**	16359	21509	5.27
GU 16-400	4	409.7	3520	271530	257	397.3	**6835**	11061	14544	3.27
	5	512.1	4380	499730	312	501.3	**9970**	13827	18179	4.05
	6	614.5	5240	826250	367	565.4	**14615**	16592	21815	4.83

Table 1.20. Dimensions and properties for foundations using sheet pile / omega 18 combinations.

1) One side, excluding inside of interlocks.

1.7.5. Jagged wall

AZ® jagged wall: AZ® sections threaded in reverse may form arrangements for special applications. The jagged wall arrangement represents a very economical solution for sealing screens (reduced height, reliable thickness, low driving resistance).

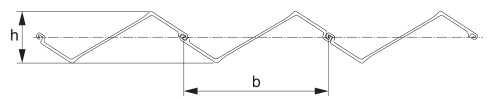


Fig. 1.15. AZ® Jagged wall.

Section	Width b mm	Height h mm	Sectional area cm²/m	Mass kg/m²	Moment of inertia cm⁴/m	Elastic section modulus cm³/m	Coating area[1)] m²/m²
AZ-800							
AZ 18-800	897	242	115	**90**	4780	**395**	1.16
AZ 20-800	897	243	126	**99**	5340	**440**	1.16
AZ 22-800	897	244	137	**107**	5900	**485**	1.16
AZ 23-800	907	255	133	**104**	6070	**475**	1.17
AZ 25-800	907	257	144	**113**	6670	**520**	1.17
AZ 27-800	907	258	155	**122**	7260	**565**	1.17
AZ-750							
AZ 28-750	881	278	146	**114**	7970	**575**	1.20
AZ 30-750	881	280	157	**123**	8700	**620**	1.20
AZ 32-750	881	281	169	**132**	9420	**670**	1.20
AZ-700 and AZ-770							
AZ 12-770	826	181	112	**88**	2330	**255**	1.12
AZ 13-770	826	182	117	**92**	2460	**270**	1.12
AZ 14-770	826	182	123	**96**	2600	**285**	1.12
AZ 14-770-10/10	826	183	128	**100**	2730	**300**	1.12
AZ 12-700	751	182	115	**90**	2410	**265**	1.13
AZ 13-700	751	183	126	**99**	2690	**295**	1.13
AZ 13-700-10/10	751	183	131	**103**	2830	**310**	1.13
AZ 14-700	751	184	136	**107**	2970	**325**	1.13
AZ 17-700	795	212	117	**92**	3690	**330**	1.16
AZ 18-700	795	212	123	**96**	3910	**350**	1.16
AZ 19-700	795	213	128	**101**	4120	**365**	1.16
AZ 20-700	795	214	134	**105**	4330	**385**	1.16
AZ 24-700	813	241	150	**118**	5970	**495**	1.19
AZ 26-700	813	242	161	**127**	6500	**535**	1.19
AZ 28-700	813	243	172	**135**	7030	**580**	1.19
AZ 24-700N	813	237	141	**110**	5580	**470**	1.19
AZ 26-700N	813	238	152	**119**	6100	**510**	1.19
AZ 28-700N	813	239	163	**128**	6630	**555**	1.19
AZ 36-700N	834	296	181	**142**	11900	**805**	1.23
AZ 38-700N	834	298	193	**152**	12710	**855**	1.23
AZ 40-700N	834	299	205	**161**	13530	**905**	1.23
AZ 42-700N	834	300	217	**170**	14650	**975**	1.24
AZ 44-700N	834	301	229	**180**	15460	**1025**	1.24
AZ 46-700N	834	302	241	**189**	16280	**1075**	1.24
AZ 48-700	836	303	241	**190**	16290	**1075**	1.23
AZ 50-700	836	303	253	**199**	17100	**1130**	1.23
AZ 52-700	836	305	265	**208**	17900	**1175**	1.23
AZ							
AZ 18	714	225	133	**104**	4280	**380**	1.19
AZ 18-10/10	714	225	139	**109**	4500	**400**	1.19
AZ 26	736	238	169	**133**	6590	**555**	1.21
AZ 46	725	308	233	**183**	16550	**1070**	1.30
AZ 48	725	310	245	**193**	17450	**1125**	1.30
AZ 50	725	312	258	**202**	18370	**1180**	1.30

Table 1.21. Dimensions and properties of AZ® Jagged wall.

[1)] One side, excluding inside of interlocks.

An arrangement of U-sheet piles forming a jagged wall offers economic solutions where high inertia and section modulus are needed. The final choice of section has to include drivability criteria. The statical values given below assume the solidarisation of the driving element, i.e. double pile. The OMEGA 18 section is normally threaded and welded at the mill, either by tack weld (no contribution to the section modulus of the jagged wall) or by an appropriately designed weld (full contribution to the section modulus). For walls with an anchorage or strut system, stiffeners have to be provided at the support levels.

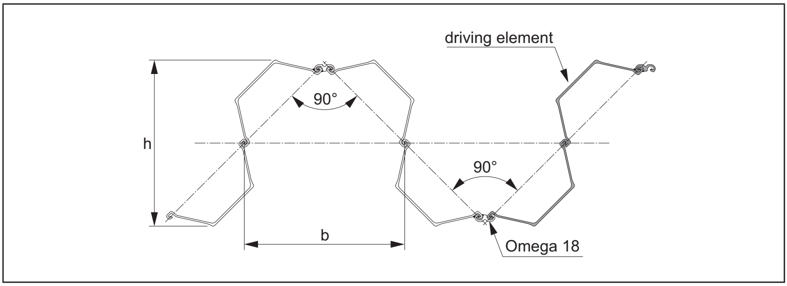


Fig. 1.16. U Jagged wall.

Section	Width	Height	Mass	Moment of inertia[1)]		Elastic section modulus[1)]		Static moment	
	b	*h*		without Omega 18	with Omega 18	without Omega 18	with Omega 18	without Omega 18	with Omega 18
	mm	mm	kg/m²	cm⁴/m	cm⁴/m	cm³/m	cm³/m	cm³/m	cm³/m
AU™ jagged wall									
AU 14	1135	1115	**153**	275830	334350	**5075**	**5995**	6160	7250
AU 16	1135	1115	**168**	307000	365520	**5650**	**6555**	6870	7960
AU 18	1135	1136	**172**	329320	387840	**5795**	**6825**	7180	8270
AU 20	1135	1139	**187**	362510	421030	**6365**	**7395**	7920	9005
AU 23	1135	1171	**196**	390650	449160	**6675**	**7675**	8470	9560
AU 25	1135	1173	**211**	424510	483020	**7240**	**8235**	9215	10300
PU® jagged wall									
PU 12	923	903	**163**	189000	229900	**4275**	**5090**	5175	6245
PU 12-10/10	923	903	**170**	198850	245250	**4495**	**5430**	5450	6525
PU 18	923	955	**186**	244340	290750	**5120**	**6090**	6430	7500
PU 22	923	993	**206**	285880	332290	**5760**	**6690**	7380	8450
PU 28	923	1028	**240**	349710	396110	**6805**	**7710**	8925	10000
PU 32	923	1011	**267**	389300	432400	**7705**	**8560**	10025	11095
GU® jagged wall									
GU 14N	923	920	**159**	198710	245140	**4320**	**5330**	5285	6360
GU 18N	923	955	**186**	244340	290750	**5120**	**6090**	6430	7500
GU 22N	923	993	**206**	285880	332290	**5760**	**6690**	7380	8450
GU 28N	923	1028	**240**	349710	396110	**6805**	**7710**	8925	10000
GU 32N	923	1011	**267**	389300	432400	**7705**	**8560**	10025	11095

Table 1.22. Dimensions and properties of U Jagged wall.

[1)] *The moment of inertia and elastic section moduli assume correct shear force transfer across the interlock on the neutral axis.*

1.8. Product tolerances

Hot rolled sheet piling products are supplied to EN 10248 Part 2 unless an alternative standard (e.g. ASTM) is specified.

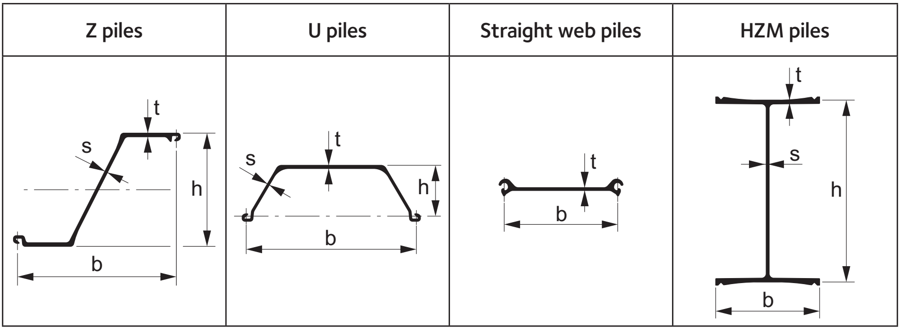


Fig. 1.17. Hot rolled steel sheet piles shapes.

Tolerances	AZ®	AU™, PU®, GU®	AS 500®	HZ®-M
Mass[1]	± 5%	± 5%	± 5%	±5 %
Length *(L)*	± 200 mm	± 200 mm	± 200 mm	± 200 mm
Height *(h)* [2]	*h* ≥ 300 mm: ± 7 mm	*h* ≤ 200 mm: ± 4 mm *h* > 200 mm: ± 5 mm	-	*h* ≥ 500 mm: ± 7 mm
Thicknesses *(t,s)*	*t, s* ≤ 8.5 mm: ± 0.5 mm *t, s* > 8.5 mm: ± 6%	*t, s* ≤ 8.5 mm: ± 0.5 mm *t, s* > 8.5 mm: ± 6%	*t* > 8.5 mm: ± 6%	*t, s* ≤ 12.5 mm: −1.0 mm / +2.0 mm *t, s* > 12.5 mm: −1.5 mm / +2.5 mm
Width single pile *(b)*	± 2% *b*	± 2% *b*	± 2% *b*	± 2% *b*
Width double pile *(2b)*	± 3% (2*b*)	± 3% (2*b*)	± 3% (2*b*)	± 3% (2*b*)
Straightness *(q)*	≤ 0.2% *L*	≤ 0.2% *L*	≤ 0.2% *L*	≤ 0.2% *L*
Ends out of square	± 2% *b*	± 2% *b*	± 2% *b*	± 2% *b*

Table 1.23. Tolerances of hot rolled steel sheet piles.

[1] *From the mass of the total delivery.*
[2] *Of single pile.*

1.9. Section profiles

Drawings of all the pile sections available from ArcelorMittal are located at the following website: sheetpiling.arcelormittal.com.

Sheet pile sections are subject to periodic review and minor changes to the profile may result. It is, therefore, recommended that users visit the ArcelorMittal Sheet Piling website to ensure that they are using the latest pile profiles.

1.10. Maximum and Minimum lengths

Standard hot rolled steel sheet piling can be manufactured in lengths up to 31 m, the HZ-M sheet piles are available up to 33 m long. However particular care will be required when handling long lengths of the lighter sections.

Should piles be needed which are longer than the maximum rolling length, splicing to create the required length may be carried out at the mill or on site.

When short piles are to be supplied direct from the mill it may be advantageous to order them in multiples of the required length and in excess of 6 m long with cutting to length being carried out on site.

When considering piles at either end of the length range, we recommend that contact is made with one of our representatives to discuss availability.
Table 1.24. summarizes the maximum rolling lengths of the different sections.

Section	**AZ**	**AU, PU**	**GU**[1)]	**AS 500**	**HZ-M**	**RH/RZ**	**Ω 18**	**C9/C14**	**Δ 13**
Length [m]	31	31	28	31	33	24	16	18	17

Table 1.24. Maximum rolling length of sheet piles and connectors.

[1)] *2016.*

1.11. Interlocking options

AZ, AU, PU and GU sheet piles feature Larssen interlocks in accordance with EN 10248. AZ, AU and PU can be interlocked together.

The theoretical interlock swing of ArcelorMittal's Larssen interlock is 5°.

1.12. Handling holes

Sheet pile sections are normally supplied without handling holes. If requested, they can be provided with handling holes in the centerline of the section. The standard handling hole dimensions are given in Table 1.25.

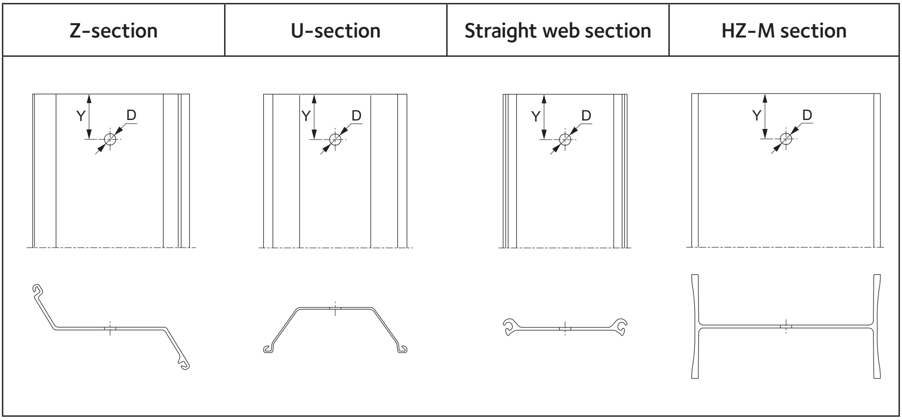


Fig. 1.18. Handling holes of hot rolled sections.

Diameter D [mm]	**40**	**40**	**50**	**50**	**63.5**	**40**
Distance Y [mm]	75	300	200	250	230	150

Diameter D [in]	**2.5**
Distance Y [in]	9

Table 1.25. Handling holes of hot rolled sections.

1.13. Plating to increase section modulus

When increased section modulus or inertia is required to cater for high bending moments over part of the pile length, it may be economic to attach appropriately sized plates to the pans of the piles to locally enhance the engineering properties of the section.

However this option is mostly considered when the pile is at the top of the range.

1.14. Plating to enhance durability

Plates can be attached to Z and U piles to provide increased durability to parts of the pile where corrosion activity may be high. This may be the case where the piles are to be installed in a facility where increased corrosion is expected. The economics of providing additional sacrificial steel instead of a heavier pile section will depend upon individual conditions but when the high corrosion effect is only expected over a short length of the pile, the plating option will very often prove to be the more cost effective solution.

1.15. Corners and junctions

Figure 1.19. illustrates the comprehensive range of hot rolled special sections that is available for use with ArcelorMittal hot rolled sheet piles to create corners and junctions (except for old GU sections). The special section is attached to the main sheet pile by welding in accordance with EN 12063 and is set back from the top of the pile by 200 mm to facilitate driving.

Corner profiles can also be formed by

- bending single rolled sections for changes in direction up to 25°;
- combining two single bent piles for angles up to 50°;
- cutting the piles and welding them together in the required orientation.

A comprehensive range of junction piles can be formed by welding a C9 hot rolled section onto the main sheet pile at the appropriate location and angle.

One advantage that the special connector has over the more traditional fabricated corner or junction section is that once a fabricated pile is formed it cannot easily be changed. In the case of temporary works, the rolled corners or junctions can be tacked in place before driving and burned off after extraction to leave a serviceable pile section and a junction or corner for use elsewhere. In the case of the Omega 18 and Delta 13 profiles, the angle is variable and enables corners to be formed at angles other than 90°.

Technical assistance is available on request to ascertain what is required for a particular project.

Drawings of the various rolled profiles may be downloaded from the following website: sheetpiling.arcelormittal.com.

Please note that:

- generally bent corners will be supplied as single piles;
- corner sections (C9, C14, Delta13, Omerga 18) are not compatible with old GU sections. Contact our technical department for alternative solutions.

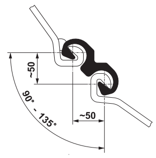
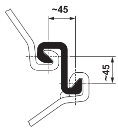
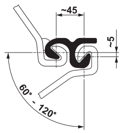
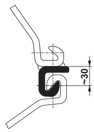
OMEGA 18 Mass ~ 18.0 kg/m	C 14 Mass ~ 14.4 kg/m	DELTA 13 Mass ~ 13.1 kg/m	C 9 Mass ~ 9.3 kg/m
			

Fig. 1.19. Corner sections.

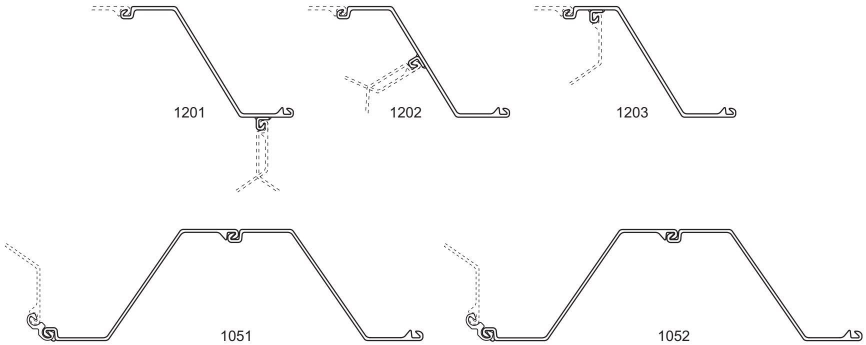


Fig. 1.20. Fabricated piles, corner and junction piles.

1.16. Junction piles

Junction piles that join circular cells and intermediary arcs can be provided. Bent piles are pre-bent at the mill. If the deviation angle exceeds 4.5° (4.0° if L > 20 m), bent piles can be used to set up structures with small radii.

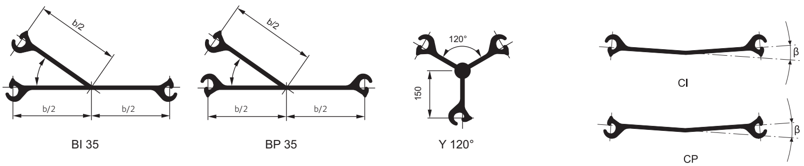


Fig. 1.21. Forms of junction piles.

1.17. Stacking of sheet piles

When stacking piles on site it is recommended that they are placed on timber or steel spacers – to allow straps or chains to be placed around the bundles – and on a level surface, to prevent the piles being distorted. The spacers should be placed at regular intervals up to 6 m apart along the length of the piles and it is recommended that the overhang is limited to 3 m. It is recommended that pile bundles are stacked not more than 4 high to prevent excessive loads on the bottom tier.

Bundles should ideally be staggered in plan - as illustrated in Fig. 1.23. – to provide stability. See EN 12063 for more information.

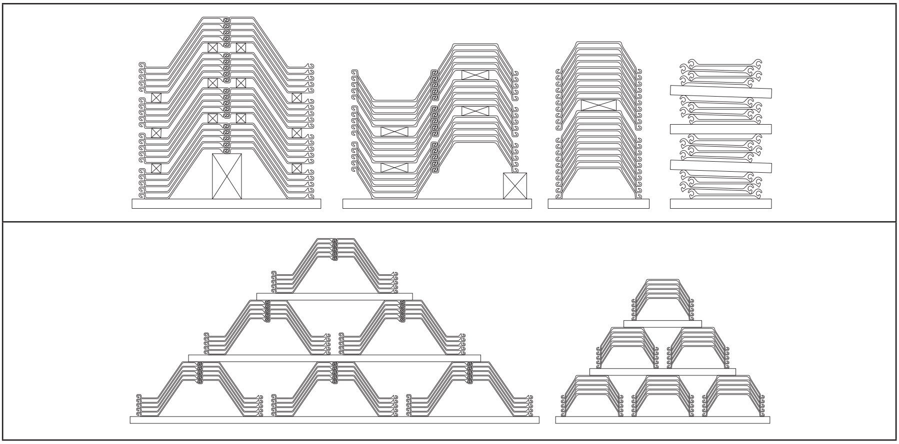


Fig. 1.22. Storage of uncoated sheet piles.

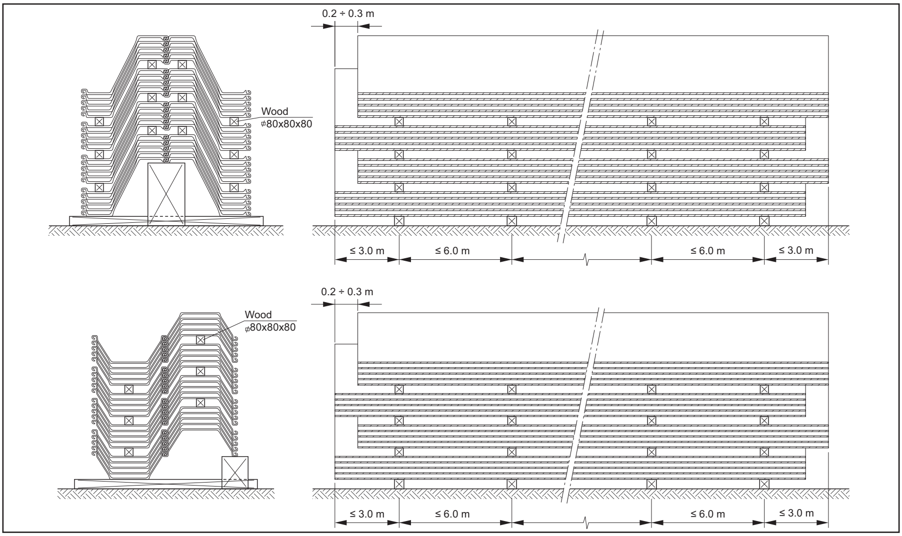


Fig. 1.23. Stacking of sheet pile - Section and longitudinal view.

1.18. Cold formed sheet piles

Cold formed sheet piles increase the range of sections available to designers particularly at the lower end of the section modulus range. Manufactured in accordance with European standards EN 10249, cold formed sections are complementary to the range of hot rolled sheet piles.

Cold formed sheet piles are normally used in the structural protection of river banks from erosion and collapse. They are recommended for retaining walls of medium height.

For detailed information it is referred to the special brochures for cold formed sheet piles from ArcelorMittal Sheet Piling.

1.18.1. Steel qualities

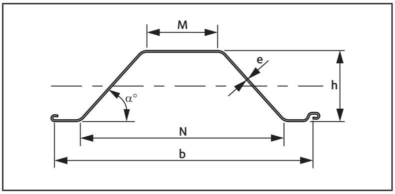
PAZ, PAU and PAL sections, as well as trench sheets are available in the steel grades according to EN 10249-1:

Steel grade EN 10249-1 [1)]	**Min. yield strength** R_{eH}	**Min. tensile strength** R_m	**Min. elongation** $L_o=5.65\sqrt{S_o}$
S 235 JRC	**235**	360 - 510	26
S 275 JRC	**275**	410 - 560	23
S 355 J0C	**355**	470 - 630	22

Table 1.26. Steel grades for cold formed sheet piles.

1) Mechanical properties according to EN 10025-2: 2004. Other steel grades available on request.

1.18.2. Omega sections

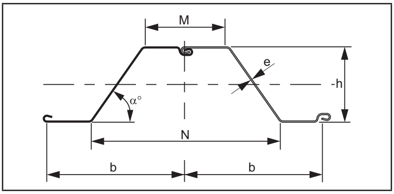


Section	Thickness[1]	Width	Height	Angle	Additional dimensions		Mass		Moment of inertia	Elastic section modulus	Static moment	Sectional area	Coating area[2]
	e mm	*b* mm	*h* mm	*α* °	*M* mm	*N* mm	single pile kg/m	wall kg/m²	cm⁴/m	cm³/m	cm³/m	cm²/m	m²/m
PAL 3030	3.0	660	89	41	260	466	19.4	**29.4**	500	**112**	65	37.5	0.80
PAL 3040	4.0	660	90	41	260	466	25.8	**39.2**	666	**147**	85	49.9	0.80
PAL 3050	5.0	660	91	41	260	466	32.2	**48.8**	831	**181**	105	62.2	0.80
PAL 3130	3.0	711	125	79	350	419	23.5	**33.1**	1244	**199**	110	42.2	0.97
PAL 3140	4.0	711	126	79	350	419	31.3	**44.0**	1655	**261**	145	56.1	0.97
PAL 3150	5.0	711	127	79	350	419	39.0	**54.9**	2063	**322**	180	70.0	0.97
PAL 3260	6.0	700	149	61	299	471	46.2	**66.0**	3096	**413**	245	84.1	0.92
PAL 3270	7.0	700	150	61	299	471	53.2	**76.0**	3604	**479**	285	96.8	0.92
PAL 3280	8.0	700	151	61	299	471	61.6	**88.0**	4109	**545**	325	112.1	0.92
PAL 3290	9.0	700	152	61	299	471	70.0	**100.0**	4611	**605**	365	127.4	0.92
PAU 2240	4.0	921	252	48	252	725	39.0	**42.3**	5101	**404**	240	53.9	1.22
PAU 2250	5.0	921	253	48	252	725	48.7	**52.8**	6363	**504**	300	67.3	1.22
PAU 2260	6.0	921	254	48	252	725	58.3	**63.3**	7620	**600**	360	80.7	1.22
PAU 2440	4.0	813	293	60	252	615	39.0	**48.0**	7897	**537**	320	61.1	1.22
PAU 2450	5.0	813	294	60	252	615	48.7	**59.9**	9858	**669**	395	76.3	1.22
PAU 2460	6.0	813	295	60	252	615	58.3	**71.8**	11813	**801**	475	91.4	1.22
PAU 2760	6.0	804	295	60	252	615	60.4	**75.1**	12059	**803**	495	95.7	1.16
PAU 2770	7.0	804	296	60	252	615	70.4	**87.5**	14030	**934**	575	114.4	1.16
PAU 2780	8.0	804	297	60	252	615	80.3	**99.8**	15995	**1063**	655	127.1	1.16

Table 1.27. Dimensions and properties of PAL and PAU sections.

1) Other thicknesses on request.
2) Single pile one side, excluding inside of interlocks.

1.18.3. PAZ sections



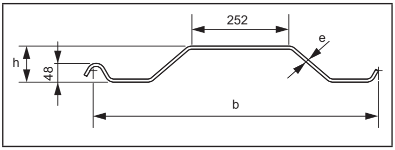
Section	Thickness[1]	Width	Height	Angle	Additional dimensions		Mass		Moment of inertia	Elastic section modulus	Static moment	Sectional area	Coating area[2]
	e mm	*b* mm	*h* mm	*α* °	*M* mm	*N* mm	single pile kg/m	wall kg/m^2	cm^4/m	cm^3/m	cm^3/m	cm^2/m	m^2/m
PAZ 4350	5.0	770	213	34	465	1078	38.2	**49.6**	4770	**448**	255	63.2	0.91
PAZ 4360	6.0	770	214	34	465	1078	45.8	**59.4**	5720	**534**	310	75.1	0.91
PAZ 4370	7.0	770	215	34	465	1078	53.3	**69.2**	6660	**619**	360	88.2	0.91
PAZ 4450	5.0	725	269	45	444	988	37.7	**52.0**	8240	**612**	350	66.2	0.91
PAZ 4460	6.0	725	270	45	444	988	45.1	**62.2**	9890	**730**	415	79.3	0.91
PAL 4470	7.0	725	271	45	444	988	52.4	**72.3**	11535	**846**	485	92.1	0.91
PAZ 4550	5.0	676	312	55	444	890	37.7	**55.8**	12065	**772**	435	71.0	0.91
PAZ 4560	6.0	676	313	55	444	890	45.1	**66.7**	14444	**922**	520	85.0	0.91
PAZ 4570	7.0	676	314	55	444	890	52.4	**77.5**	16815	**1069**	610	98.8	0.91
PAZ 4650	5.0	621	347	65	438	778	37.7	**60.7**	16318	**940**	530	77.3	0.91
PAZ 4660	6.0	621	348	65	438	778	45.1	**72.6**	19544	**1122**	635	92.5	0.91
PAZ 4670	7.0	621	349	65	438	778	52.4	**84.4**	22756	**1302**	740	107.5	0.91
PAZ 5360	6.0	857	300	37	453	1245	54.3	**63.3**	11502	**766**	450	80.7	1.04
PAZ 5370	7.0	857	301	37	453	1245	63.2	**73.7**	13376	**888**	520	93.9	1.04
PAZ 5380	8.0	857	302	37	453	1245	72.1	**84.0**	15249	**1009**	595	107.1	1.04
PAZ 5390	9.0	857	303	37	453	1245	81.0	**94.4**	17123	**1131**	665	120.3	1.04
PAZ 5460	6.0	807	351	45	442	1149	53.9	**66.8**	16989	**968**	560	85.1	1.04
PAZ 5470	7.0	807	352	45	442	1149	62.6	**77.6**	19774	**1123**	655	98.9	1.04
PAZ 5480	8.0	807	353	45	442	1149	71.4	**88.4**	22546	**1277**	745	112.7	1.04
PAZ 5490	9.0	807	354	45	442	1149	80.2	**99.3**	25318	**1431**	835	126.5	1.04
PAZ 54100	10.0	808	355	45	442	1149	89.2	**110.3**	27850	**1570**	920	140.5	1.04
PAZ 5560	6.0	743	407	55	438	1020	53.9	**72.5**	25074	**1233**	710	92.4	1.04
PAZ 5570	7.0	743	408	55	438	1020	62.6	**84.3**	29179	**1432**	825	107.4	1.04
PAZ 5580	8.0	744	409	55	438	1020	71.4	**96.0**	33263	**1628**	940	122.3	1.04
PAZ 5590	9.0	744	410	55	438	1020	80.2	**107.8**	37387	**1825**	1060	137.3	1.04
PAZ 55100	10.0	745	410	55	438	1020	89.2	**119.8**	41060	**2000**	1165	152.6	1.04
PAZ 5660	6.0	671	451	65	434	875	53.9	**80.3**	34340	**1525**	875	102.3	1.04
PAZ 5670	7.0	671	452	65	434	874	62.6	**93.3**	39954	**1770**	1020	118.9	1.04
PAZ 5680	8.0	672	453	65	434	874	71.4	**106.3**	45537	**2013**	1160	135.4	1.04
PAZ 5690	9.0	672	454	65	434	874	80.2	**119.3**	51180	**2259**	1300	151.9	1.04
PAZ 56100	10.0	673	455	65	434	874	89.2	**132.5**	56200	**2470**	1435	168.8	1.04

Table 1.28. Dimensions and properties of PAZ sections.

[1] Other thicknesses on request.

[2] One side. excluding inside of interlocks.

1.18.4. Trench sheet sections



Section	Thickness[1]	Width	Height	Mass		Moment of inertia	Elastic section modulus	Static moment	Sectional area	Coating area[2]
	e mm	*b* mm	*h* mm	single pile kg/m	wall kg/m²	*I* cm⁴/m	*W* cm³/m	cm³/m	cm²/m	m²/m
RC 8600	6.0	742	92	40.9	**55.1**	896	**194**	116	70.2	0.87
RC 8700	7.0	742	93	47.6	**64.2**	1045	**224**	135	81.8	0.87
RC 8800	8.0	742	94	54.2	**73.0**	1194	**254**	154	93.0	0.87

Table 1.29. Dimesnions and properties of RC sections.

[1] Other thicknesses on request.
[2] One side, excluding inside of interlocks.

1.18.5. Threading options

PAZ sheet piles are usually delivered threaded in pairs and welded at regular intervals using 150 mm runs of weld, the amount is dependent on the length of the sheet piles. Table 1.30. shows which piles can be interlocked together.

Series		**PAL**			**PAU**			**PAZ**							
		30	31	32	22	24	27	43	44	45	46	53	54	55	56
PAL	30	√	√												
	31	√	√												
	32			√			√								
PAU	22				√	√									
	24				√	√									
	27			√			√					√	√	√	√
PAZ	43							√	√	√	√				
	44							√	√	√	√				
	45							√	√	√	√				
	46							√	√	√	√				
	53						√					√	√	√	√
	54						√					√	√	√	√
	55						√					√	√	√	√
	56						√					√	√	√	√

Table 1.30. Threading options.

1.18.6. Sheet pile assembly

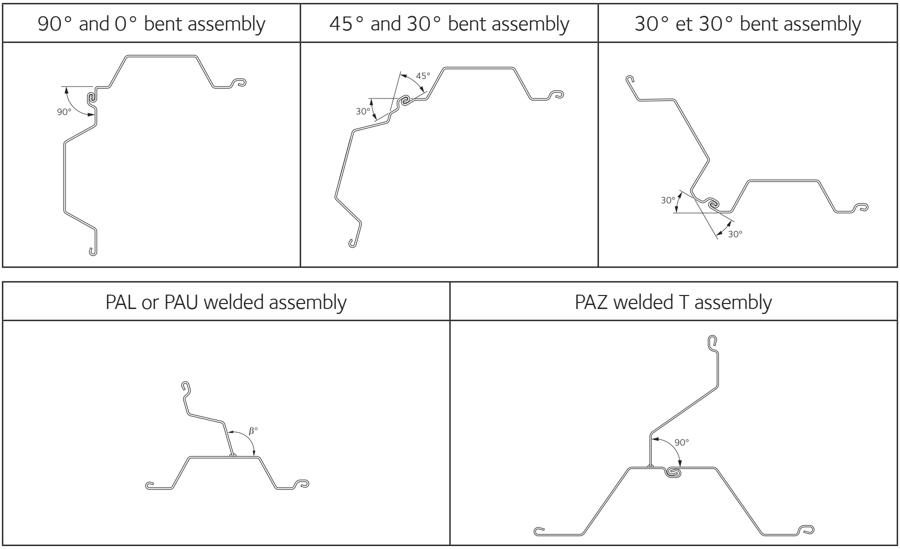


Fig. 1.24. Sheet pile assembly.

1.18.7. Thicknesses available for each range

Series		Steel grade		
		S 235 JRC	S 275 JRC	S 355 J0C
PAL	30	5.0	5.0	5.0
	31	5.0	5.0	5.0
	32	9.0	9.0	8.0
PAU	22	6.0	6.0	6.0
	24	6.0	6.0	6.0
	27	8.0	8.0	7.0
PAZ	43	7.0	7.0	7.0
	44	7.0	7.0	7.0
	45	7.0	7.0	7.0
	46	7.0	7.0	7.0
	53	9.0	9.0	8.0
	54	10.0	9.0	8.0
	55	10.0	9.0	8.0
	56	10.0	9.0	8.0

Table 1.31. Maximum allowable thickness per type of sheet pile and grade of steel.

1.18.8. Handling holes

The sheet pile sections can be provided with the standard handling holes given in Table 1.32. Other dimensions are available on request.

	Diameter *D* mm	**Distance** *Y* mm
PAL 30-31	40	150
PAL 32	45	150
PAU	45	200
PAZ	50	200

Table 1.32. Handling holes of cold formed sheet piles.

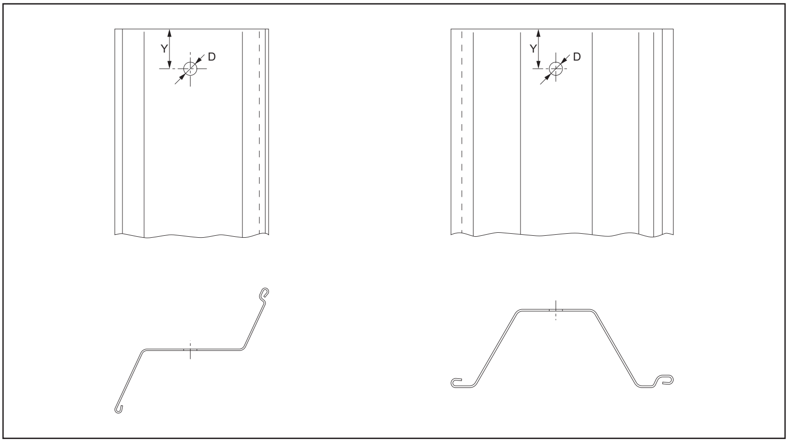


Fig. 1.25. Handling holes for cold formed sheet piles.

1.18.9. Tolerances

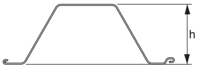
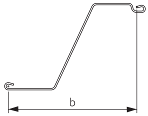
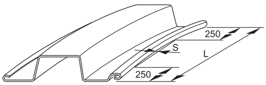
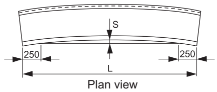
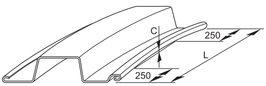
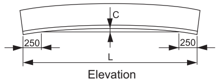
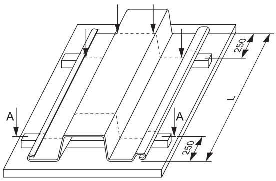
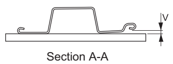
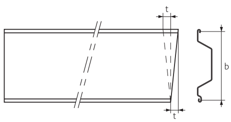
Characteristics	Figures	Nominale size	Tolerances
Height Height *h*		$h \leq 200$ mm	± 4 mm
		$200 < h \leq 300$	± 6 mm
		$300 < h \leq 400$	± 8 mm
		$400 < h$	± 10 mm
Width Width *b*		Single sheet piles	± 2% *b*
		Double sheet piles	± 3% *b*
Wall thickness Thickness *e* The tolerances on the wall thickness of the profiles shall comply with the requirements of table 3 of EN 10051, for a nominal width of strip and sheet over 1800 mm.		$e = 3.00$ mm	± 0.26 mm
		$3.00 < e \leq 4.00$	± 0.27 mm
		$4.00 < e \leq 5.00$	± 0.29 mm
		$5.00 < e \leq 6.00$	± 0.31 mm
		$6.00 < e \leq 8.00$	± 0.35 mm
		$8.00 < e \leq 10.00$	± 0.40 mm
Bending Bow-height *S*			0.25% *L*
Curving Bow-height *C*			0.25% *L*
Twisting Dimension *V*			±2% *L* or 100 mm max.
Length Length *L* Normal tolerance[1)]			± 50 mm
Squareness of ends Out-of-squareness *t* of end cuts			± 2% *b*
Mass Difference between the total actual mass and the total calcutated mass delivered[1)]			± 7%

Table 1.33. Tolerances in accordance with EN 10249 - Part 2.

[1)] Reduced tolerances are available on request.

ArcelorMittal

2 | Sealants

Chapter 2 - Sealants

Contents

2.1. Introduction

The ability of retaining walls to prevent or restrict the passage of ground water is of great importance in many applications e.g. in basements, underground car parks, underground tanks, temporary cofferdams and containment barriers.

A sealed sheet pile wall provides a safe, economic solution in any situation where control of groundwater is an issue, for instance to minimise the risk of settlement of adjacent property and keeping excavations dry. The water tightness of sheet pile interlocks almost invariably improves with time but a sealant will provide a means by which the flow/passage of water can be controlled immediately.

All construction projects are unique with ground conditions, water tightness requirements and installation methods varying from site to site; therefore the sealant system adopted must be designed accordingly.

The integrity of a sealant system in use will depend upon it's suitability with respect to the method of pile installation adopted and the ground conditions. Sealants are available to make driving easier and systems are also available to protect the sealants when driving the piles into gravels and difficult ground.

2.2. Basements

The use of permanent sheet piling for the walls of basement structures has, until recently, been considered on relatively few occasions partly because the interlocks were assumed to be a potential leakage point. If a steel basement was built, the interlocks would be seal-welded following installation to give a fully watertight wall. With narrow piles, this would involve a substantial amount of welding on site but following development of wider piles, the amount of sealing to be carried out reduced considerably making sealed basement walls a much more attractive option. The development of new forms of sealant and improved installation techniques means that sealed substructures can now be created using non-welded piles.

Table 2.1. is extracted from BS 8102: 2009, "The code of practice for protection of below ground structures against water from the ground" and indicates the performance level required for the range of possible basement grades. These are all achievable using steel sheet pile walls and appropriate interlock sealing systems.

Basement Grade	Basement usage	Performance level
1	Car parking; plant rooms (excluding electrical equipment); workshops	Some seepage and damp patches tolerable
2	Workshops & plant rooms requiring drier environment; retail storage areas	No water penetration but moisture vapour tolerable
3	Ventilated residential & working areas including offices, restaurants etc.; leisure centres	Dry environment
4	Archives and stores requiring controlled environment	Totally dry environment

Table 2.1. Extract from BS 8102 indicating basement performance levels.

2.3. Containment barriers

Sealed sheet pile cut off walls can be used to prevent leachate from contaminated ground or refuse and disposal sites from leaking through the ground into uncontaminated areas.

Traditionally these barriers to horizontal movement have been made using clay bunds or cement bentonite walls. These traditional methods take up large areas of ground, are generally formed away from the edge of the site and are prone to leakage. It should be noted that concrete and slurry wall systems are porous in the long term and, as they are both relatively brittle materials, their ability to retain water will diminish if cracks appear as a result of movement or exposure to loading fluctuations.

Sheet pile barriers offer a sealed solution on a much smaller footprint and the barrier can be placed at the site boundaries to maximise the ground area contained. Sheet piles used in basements or as the foundations at the perimeter of a building can also be sealed to prevent gases and leachate from redeveloped brown field sites from entering the building.

Steel sheet piles can also be removed at a later date and reused or recycled.

2.4. Demountable foundations

With the rapidly changing use of buildings and structures, designers are required to take into consideration the demolition and removal of the building at the end of its life. For a truly sustainable design, this requirement should also include the foundations.

All steel pile foundations and retaining systems, including most sealed pile walls, can be extracted and either reused or recycled. This has the advantage that the site will be free of obstructions and in a much better state to be redeveloped. Hence steel sheet piling offers sustainability of product and sustainability of site.

2.5. Site or factory application

It has been shown, by performance testing the various sealant products, that best results are obtained by thoroughly preparing the interlocks prior to the application of the sealant. This has the effect of removing any mill scale or other deleterious materials from the interlock and producing a steel surface that the sealant can properly adhere to.

Not only is it difficult to clean and prepare the interlocks to the required standard on the construction site but weather conditions, temperature and humidity or the presence of surface moisture may be detrimental to the bond between the sealant and the steel. Interlock preparation to new piles will ensure good adhesion of the sealant to the steel, reducing the risk of damage when driving the piles and loss of performance in service.

Once they have cured, most sealant products are inert and therefore a non-hazard. But handling the constituents requires care as this operation introduces the possibility of exposure to potentially hazardous substances and may involve working with hot fluids. By applying sealants in the workshop, rather than on a construction site, the handling of these materials can be controlled by stringent safety standards. The work is confined to experienced personnel operating in a controlled environment and third parties are not subjected to unnecessary risk.

Once the sealants have cured, the COSSH requirements and safety risks reduce dramatically so it is possible to carry out a risk assessment for sealant application in the workshop that is complete as the operation is carried out in a fixed and controlled environment. This does not occur on the construction site where conditions will vary with location.

2.6. How the sealants work

Sealant systems are designed to stop water penetrating the interlocking joints in sheet pile walls and consequently the actual performance of a sealant system will be a function of the interlock geometry and type of sealant applied.

Sealants generally operate in several ways; those that create a compression seal between adjacent parts of the interlock, those that displace to fill the voids and those that swell in contact with water and fill the gap between the interlocks (hydrophilic sealants). Compression and waterswelding sealants will generally resist greater water pressures than displacement sealants but, as indicated above, preparation of the steel surface is essential to performance.

The sealants that are soft in texture when applied to the interlock will generally perform as displacement seals when piles are interlocked together, as the material can be squeezed into the voids in the interlocks preventing water flow. However these sealants are usually supplied unprotected and performance can be affected when driving in gravelly soil or by jetting.

Sealants that are firmer in texture will tend to be squashed during installation and form a compression seal when piles are interlocked together. They are generally more durable than displacement sealants from both the design and installation points of view. Hydrophilic sealants have a relatively low volume during the installation phase but swell up following contact with water to fill the voids in the interlocks. The swelling action can occur if the sealant is wetted accidentally by spraying or in heavy rain so that hydrophilic sealants must be protected until driving operations start.

In addition to the materials that are applied before driving, it is also possible to seal weld the interlocks after installation. Further information is given in Chapter 2.12.

2.7. Installation techniques

One of the best ways to minimise the risk of water ingress through a sheet pile wall is to reduce the number of interlocks. Where installation conditions allow, steel sheet piles seal-welded together into multiple units e.g. pairs or triples, should be driven in that form.

It has been found during site trials that pitch and drive methods to install sheet piles are usually more practical than panel driving when using pre-applied sealants. Traditionally, panel driving rather than pitch and drive techniques, have been recommended to improve the accuracy with which sheet piles are installed. However, the need to work above ground level can make sealed sheet piles more difficult to pitch and sequential driving may disturb the sealants more. Dependant upon the type, more sealant may be extruded before the piles have been fully driven when panel driving.

Significant technological advances in sheet pile installation equipment have facilitated Pitch & Drive methods. Telescopic leader rigs and silent pressing machines have revolutionised pile installation and, in the right conditions, it is now possible to install piles accurately using this technique.

To ensure good joint integrity it is important to control the alignment of the piles in both the horizontal and vertical planes but excessive corrective actions can damage the sealants. If it is necessary to remove a pile then suitable repairs should be carried out to the sealant before reuse. If repair is not practical, withdrawn piles should be replaced by new ones.

The heat generated by vibro driving may cause some of the sealants to decompose or burn. It is essential not to overdrive sealed sheet piles with a vibratory hammer. If hard driving or refusal is encountered it is recommended that vibro driving ceases at once. The pile should then be driven to level with an impact hammer.

It has been found that some sealants reduce friction in the interlocks and make driving easier, but some compression seals can increase the interlock friction making pitching more difficult. This will not normally be a problem for telescopic rigs provided that the mast is of adequate size to enable the piles to be pitched easily.

It is essential that any application of heat to interlocks containing sealant, for example for cutting or welding, should only take place in well-ventilated areas.

Inhalation of smoke and vapours could be harmful and should be prevented. It is the contractor's responsibility to carry out adequate risk assessment procedures for any site operations that involve handling sealant substances, welding, cutting or trimming of piles and carrying out repairs.

When trimming piles containing sealants using oxy acetylene equipment, suitable fire extinguishing equipment and breathing apparatus should be available.

2.8. Location of the sealants

Hydrophilic sealants should always be applied in the trailing interlock to avoid premature swelling. Parts of a sheet pile, that will be below excavation level in service, cannot be economically sealed after installation and, if required, the sealant system should be applied before driving. Displacement or compression sealants should be applied to the leading interlock when it is necessary to seal the lower part of the pile.

If only the upper part of the pile requires sealant, a sealant system suitable for application to the trailing interlock should be specified. However it should not be forgotten that any exposed lengths of sheet pile can be seal-welded after driving to achieve the required watertightness.

The sealant system may be curtailed above the bottom of the pile if penetration into an impermeable strata is required and sealant is not necessary over that part of the pile.

Piles should always be specified and ordered long enough to allow for trimming, in order that the piles and sealed lengths are driven to the required depth. Please note that contractors and designers should specify the distance from the top of the piles to the start of the sealant if trimming with oxy-acetylene equipment is foreseeable.

2.9. Design life

The life expectancy of a sealant system is a function of the sealant adopted, the quality of workmanship at the installation stage and the nature of the conditions in which it is to operate.

Where seal welds are used in permanent works an allowance for corrosion loss through the life of the structure must be included within the weld size specification.

2.10. Chemical durability

After installation, the durability of the various sealant options in the presence of a number of chemicals can be summarised as in Table 2.2.
Please contact our technical department for the resistance of sealants to other substances.

Chemical	Hot applied bituminous product	Compressible sealant product	Hydrophilic sealant product
pH 3.5 to pH 11.5	Excellent	Excellent	Excellent
Seawater	Excellent	Excellent	Excellent
Mineral oil	Low	Medium	Low
Petrol	Very low	Medium	Low to medium
Crude oil	Very low	Medium	Low

Table 2.2. Chemical durability.

2.11. Permeability

The level of permeability achieved by an unsealed sheet pile wall will depend on the soil conditions, the pile section chosen, the water head and the quality of the installation. For this reason it is impossible to predict the permeability of an unsealed wall with any degree of accuracy. However, when sealants have been applied to the interlocks, many of the variables are no longer relevant and the permeability of the wall and sealant system as a whole may be assessed.

It is imperative for a wall to be watertight that the sheet piles must interlock correctly at corners and junctions. De-clutching caused by faulty installation practice has to be prevented.

A special de-clutching detector, Dixeran, has been developed by ArcelorMittal to confirm that pile interlock has been successfully threaded over the full length. The detector is attached to the leading interlock and gives a signal when the toe of the next pile reaches the same toe level.

Sealed and welded sheet pile walls should be impermeable if the sealant system is performing adequately and, as a sheet pile wall is very resistant to structural loading, movements occurring after the construction phase, that are sufficient to cause a seal to displace, are not expected in the normal course of events.

As Darcy's law for discharge through homogenous structures is not applicable to leakage phenomenon through sheet pile interlocks, a new concept of "joint resistance" has been developed by GeoDelft (Deltares).

$$q(z) = \rho \times \Delta p(z) / \gamma_w$$

$q(z)$	water discharge [$m^3/s/m$];
ρ	inverse joint resistance [m/s];
$\Delta p(z)$	pressure drop at level z [kPa];
γ_w	unit weight of water [kN/m^3].

Table 2.3. gives an indication of the relative permeability values for a number of sealant options. The average inverse joint resistance ρ_m was determined according to EN 12063.

Sealing system	**ρ [10^{-10} m/s]**			**Application of the system**	**Cost ratio[1)]**
	100 kPa	200 kPa	300 kPa		
No sealant	> 1000	-	-	-	0
Beltan® Plus	< 600	not recommended	-	easy	1.0
Arcoseal™	< 600	not recommended	-	easy	1.2
ROXAN® Plus System	0.5	0.5	-	with care	1.8
AKILA® System	0.3	0.3	0.5	with care	2.1
Welded interlock	0	0	0	2)	5.0

Table 2.3. Permeability of different sealants.

1) Cost ratio = $\dfrac{\textit{Cost of sealing system}}{\textit{Cost of bituminous Beltan Plus}}$

2) After excavation for the interlock to be threaded on jobsite.

2.12. Horizontal sealing

In addition to sealing the walls of an underground structure, it is also necessary to prevent water flow through the joint between the walls and floor. As with many construction activities, attention to detail and workmanship will ensure that the joint remains watertight, but the picture below illustrates a simple waterstop arrangement that can be formed by welding a plate to the piles before it is cast into the base slab. In this example, a hydrophilic strip has been attached to the plate to further enhance the performance of this water barrier.

Fig. 2.1. Welded horizontal plates with hydrophilic strip.

When designing the horizontal joint it is suggested that consideration is given to welding the slab reinforcement to the piles to prevent the concrete shrinking away as it cures thereby creating a crack.

2.13. Vertical Sealing sytems

2.13.1. Rational analysis of impervious steel sheet pile walls

Until the end of the 1980's, there was no consistent methodology available for the assessment of the seepage resistance of steel sheet pile (SSP) walls. The lack of such a methodology could conceivably lead to uneconomic design, especially in cases where the seepage resistance was substantially larger than the specific design requirements.

In collaboration with Deltares in The Netherlands (Delft Geotechnics), ArcelorMittal carried out an exhaustive research project about the permeability of steel sheet pile interlocks.

The main aim of the project was to determine the rate of seepage through SSP walls for various interlock filler materials, as well as for empty interlocks.

Two key areas of research were addressed:

- setting up a consistent theory to describe the leakage behaviour through single interlocks;
- in situ experimentation on SSP walls.

The research results are deployed to enable the designer to make a rational assessment of the rate of seepage for a specific case. A range of possibilities is discussed: highly permeable unfilled interlocks, filled interlocks for medium permeability and completely impervious welded interlocks.

The cost involved in each case can be balanced against the seepage resistance requirements and the most appropriate solution will present itself on the basis of the analysis (see Table 2.3.).

The concept of interlock resistance

The steel sheet piles themselves are completely impervious and therefore the only possible route for a fluid to cross the wall is through the interlocks. For porous medium like slurry walls, the seepage can be treated with Darcy's law with a suitably chosen coefficient of permeability *K*:

$$v = K \times i$$

where (v) is the so-called filtration rate and (i) represents the hydraulic gradient:

$$i = \left(\Delta\rho / \gamma_w\right) / s$$

In a horizontal plane, it is defined as the ratio of the difference in pressure height ($\Delta p/\gamma$) and the length of the filtration path (s), see [iv].

The type of flow (pipe, potential,...) through an interlock is quite complex and difficult to determine, but most likely it will not be a porous media type of flow. Hence, Darcy's law cannot be used for the local seepage. To accommodate this difficulty, researchers at Deltares introduced the concept of "interlock resistance".

A straight forward approach is to assume that the discharge is proportional to the pressure drop:

$q(z)$ proportional to $\Delta p\,(z)$

The proportionality coefficient is denoted by ρ:

$$q(z) = \rho \times \Delta p\,(z) / \gamma_w \quad \text{(eq. 1)}$$

with:

$q(z)$	discharge per unit of interlock length at level z [m^3/s/m];
$\Delta p(z)$	pressure drop at level z [kPa];
ρ	inverse interlock resistance [m/s];
γ_w	unit weight of water [kN/m^3].

Note that above formula is not based on a Darcy type of flow. All interlock properties are encased in ρ and this parameter is determined from in situ tests.

In situ measurements

In order to allow the design engineer to make practical use of equation (eq. 1) Deltares and ArcelorMittal carried out field tests on a large number of filler materials. The results of these tests yielded values for ρ.

To expose the filler material to extreme site conditions, steel sheet piles have been driven with a vibratory driver and an impact hammer. Each filler material has been applied in several interlocks.

The discharge through each interlock was measured as a function of the applied pressure drop using a special test device. The time dependent behaviour was monitored by taking readings at specific time intervals.

Table 2.3. shows the relevant criteria for selecting a watertightening system for an SSP wall and the range of values obtained from the tests for different types of filler materials. The results of the empty interlocks are also shown. It is most important to note that the ρ - values obtained for empty interlocks strongly depend on the soil properties, the variations being very large. The test results generally confirm that the hypothesis which leads up to (eq. 1) is well-founded, at least for a certain pressure range.

This testing programme clearly demonstrates that the use of filler products in the interlocks of a SSP wall considerably reduces the seepage rate.

Besides, field tests have proven that the sealing material applied inside the interlocks is confined inside the interlocks, even after installation by vibratory hammer, provided the specific installation procedure elaborated by ArcelorMittal is strictly enforced.

Practical use of the concept:

Example:

(Fig. 2.5.) shows a building pit in which the water table has been lowered about 5 m. The toe of the SSP wall penetrates into a layer that is assumed to be impervious. This assumption allows neglecting the flow around the toe.
The resulting hydrostatic pressure diagram is easily drawn:

$$max\,(\Delta p) = \gamma_w \times H$$

The total discharge through one interlock is obtained:

$$Q_1 = \int_0^{H+h} q(z) \times dz = (\rho / \gamma_w) \times \int_0^{H+h} \Delta p(z) \times dz \quad \text{(eq. 2)}$$

With the pressure drop:

$$\Delta p(z) = \begin{cases} \gamma_w \times z & z \leq H \\ \gamma_w \times H & H \leq z \leq H+h \end{cases}$$

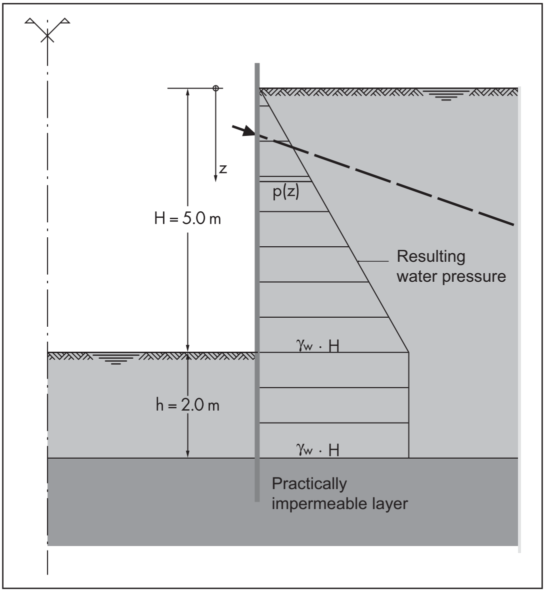


Fig. 2.2. Example for water pressure distribution along a wall.

Thus the integral in (eq. 2) yields the area in the pressure diagram and a result for Q_1 follows:

$$Q_1 = \rho \times H \times (0.5H + h)$$

The total number of interlocks in the SSP wall for the building pit is:

$$n = L / b$$

with:

L length of the perimeter of the building pit [m] ;

b system width of the pile [m].

The total discharge into the pit is:

$$Q = n \times Q_1 \quad \text{(eq. 3)}$$

(eq. 3) represents a safe approximation for the discharge, as certain aspects have been neglected, for example the influence of the flow pattern on the geometry of the water table.

Numerical example:

A building pit with a sheet pile wall made of AZ 18-700 ($b = 0.70$ m) has a perimeter $L = 161$ m.

The interlocks are sealed using the AKILA® system characterized by the ρ value

$$\rho = 0.3 \times 10^{-10} m / s$$

Geometrical data (Fig. 2.5.)

$$H = 5\ m\ and\ h = 2\ m$$

Amount of interlocks

$$n = 161 / 0.70 = 230$$

Discharge per interlock

$$Q_1 = 0.3 \times 10^{-10} \times 5.0 \times (0.5 \times 5.0 + 2.0)$$

$$Q_1 = 6.75 \times 10^{-10} m^3 / s$$

Total discharge into the pit

$$Q = 230 \times 6.75 \times 10^{-10} m^3 / s$$

$$Q = 1.552 \times 10^{-7} m^3 / s$$

$$Q = 0.56\ l / h$$

Note: The flow around the toe of the SSP wall was neglected. This is only correct, if the bottom layer is much more impervious than the wall. If this is not the case, then the water flow both, through and around the wall, needs to be considered. This is done with the aid of a 2D-seepage calculation program.

The imperviousness of steel sheet pile walls

For practical design purposes it is advisable to assess the degree of the required seepage resistance in order that a cost effective solution may be selected. Depending on the requirements, there are basically three possible solutions:

- in applications such as temporary retaining walls a moderate rate of seepage is often tolerable. An SSP wall made of piles with Larssen interlocks provides sufficient seepage resistance;
- in applications where a medium to high seepage resistance is required – such as cut-off walls for contaminated sites, retaining structures for bridge abutments and tunnels – double piles with a sealant applied in the workshop in the intermediate interlock is the best option. The free interlock is also filled with a filler material at the workshop and will be threaded on site. The lower end of the resistance range is adequately served by "Arcoseal™" or "Beltan® Plus" fillers, but it is noted that their use is limited to water pressures up to 100 kPa (10 m of water head). For high impervious requirements, as well as water pressures up to 200 kPa, filler like the "Roxan® Plus" system should be utilized. A wall designed in this way is between 100 to 1000 times more impervious than the simple sheet pile wall without filler. The new "AKILA®" system is efficient for water pressures up to 300 kPa;
- 100% watertightness may be obtained by welding every interlock. Double piles with a workshop seal-weld are used for the construction of the wall. The interlock threaded at the jobsite will be welded on site after excavation on the accessible portion.

AKILA®, Arcoseal™ , Beltan® Plus and Roxan® Plus are registered trademarks of ArcelorMittal.

2.13.2. Practical approach

In certain types of projects like underground car parks, tunnels, containing of waste, etc., the watertightness of the walls is an important criteria for the selection of the construction process.

Steel sheet piling, by definition the separation element between two different types of material, represent an ideal solution for the problem of watertight wall, provided it is possible to find:

- a method of calculating in a precise way the rate of flow through the interlocks;
- solutions to the practical problems which arise during the execution of watertight walls.

When we address the watertightness of steel sheet pile wall one ought to distinguish between two types of sealing:

- **vertical sealing**, which consists mainly of making the sheet piling interlocks watertight. According to the requested watertightness degree, several methods are possible:
  - products applied in the interlocks either before or after the piles are threaded, for average performance ($\rho = 6 \times 10^{-8}$ m/s) to high performance ($\rho = 0.3 \times 10^{-10}$ m/s);
  - welding of interlocks for 100% watertightness. Common interlocks of sheet piling supplied in pairs or triples are seal-welded in the workshop.
- **horizontal sealing**, which consists of the sealed junction between the steel sheet pile wall and a horizontal construction element connected to it (for example a concrete slab, a geotextile membrane, etc). In general two types of sealings are used:
  - sealing of the base slab, i.e. forming a watertight seal in zones which are often under water;
  - sealing the covering element of a cover slab.

Notes:

- When the project foresees a surface treatment of sheet piling by application of coating, it is essential to indicate it to ArcelorMittal's technical department. Indeed, the choice of the sealing system of interlocks depends not only on the watertightness degree requested by the project; to avoid any problems of adhesion, the system must also be compatible with the coating.

- To avoid rust stains on coated sheet piles, the gap on the backside of the interlock should be sealed on-site after installation.

- For practical reasons, it is common practice to leave a portion of the interlock at the top and tip of the pile unsealed. The sealer starts usually around 100 mm from the top of the sheet pile unless otherwise instructed in written by the customer.

2.13.3. Sealants with low permeability: Beltan® Plus and Arcoseal™

For applications with average performance requirements, two products are recommended:

Beltan® Plus (bitumen based) and Arcoseal™ (wax based), are heated up and then poured in a hot liquid phase inside the interlocks.

		Beltan® Plus	Arcoseal™
Application		Common and crimped interlocks	Common and crimped interlocks
Hydrostatic Pressure		≤ 100 kPa	≤ 100 kPa
ρ		600×10^{-10} m/s	600×10^{-10} m/s
Features	**Composition**	bitumen + polymer	mineral oil + paraffin wax
	Density at 20°C	1.00 g/cm³	0.87
	Softening Point	~72°C (DIN EN 1427)	~ 70 °C
	Colour	black	brown
Packaging		tins of 26 kg and 10 kg	barrels of 12 kg
Conditions of application	**Surface covered with standing water**	impossible	impossible
	Wet surface	to be avoided	impossible
	Surface temperature	- 10°C to + 70°C excellent	0°C to + 70°C excellent
Durability[2]	**Fresh water**	pH 3.5 ~ pH 11.5 excellent	pH 2 ~ pH 12 excellent
	Sea water	excellent	excellent
	Mineral oil	low	low to medium
	Petrol[1]	very low	low
	Crude oil[1]	very low	low
Consumption	**Leading interlock**	0.30 kg / metre	0.33 kg / metre
	Commom & crimped interlock	± 0.10 kg / metre	± 0.15 kg / metre

Table 2.4. Technical specifications of Beltan Plus and Arcoseal.

[1] Tested in laboratory on a pure solution.
[2] Durability for other chemical substances is available on request.

Note: Beltan® Plus and Arcoseal™ are certified by the "Hygiene-Institut des Ruhrgebiets" in Germany as suitable for use in contact with groundwater.

Application of Beltan® Plus and Arcoseal™ in the workshop

The application of this type of product in the workshop has to comply with following requirements:

- the interlocks must be dry and free of grease;
- the sheet piles must be laid out in a perfectly horizontal position;
- to achieve the required adherence, cleaning of the interlocks with compressed air, a steel wire brush or high-pressure water jet is necessary;
- to prevent the hot liquid product from flowing out of the ends of the sheet piles when the interlocks are filled, the ends must be clogged-up at the top and bottom by means of a mastic;
- the product is heated uniformly to a predefined temperature, and care must be taken not to overheat it;
- the product is stirred to give a homogeneous mixture;
- the interlocks are filled using an appropriate jug, taking into account the driving direction as well as the position in relation to hydrostatic pressure, the filled side of interlocks has to be driven in direct contact with hydrostatic pressure (water side);
- for single units, only the leading interlock will be filled;
- for paired units, the intermediate interlock must be crimped, and both, the intermediate as well as the leading interlock, will be filled.

Note: The sealing of intermediate interlocks of paired sheet piles (double piles, triple piles,...) is only achievable with "crimped interlocks". In order to achieve the resistance of the crimping points, the products must be applied after crimping. Beltan® Plus or Arcoseal ™ are not suitable to seal cantilever walls subjected to predominantly fluctuations loads.

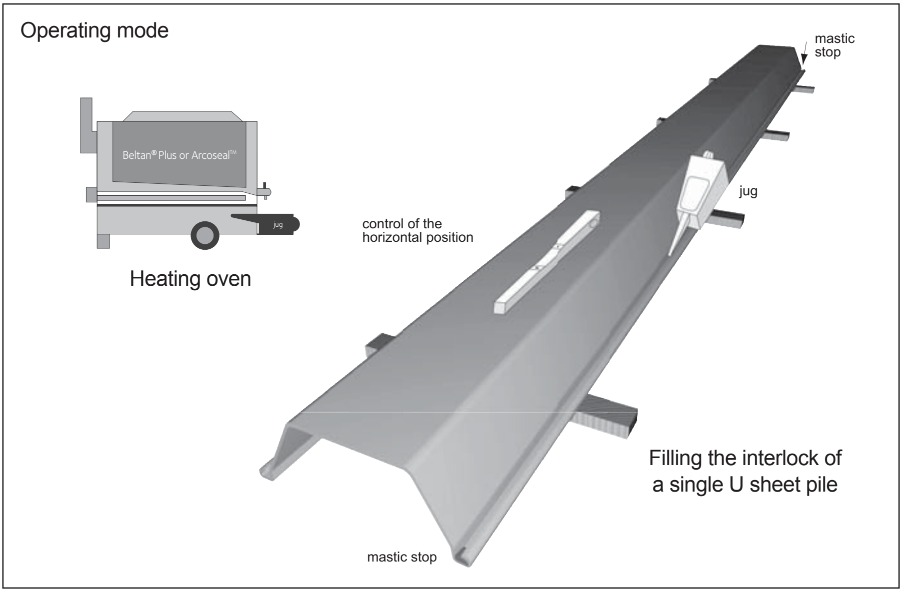


Fig. 2.3. Application of Beltan Plus.

Detail of application in single Z sheet piles

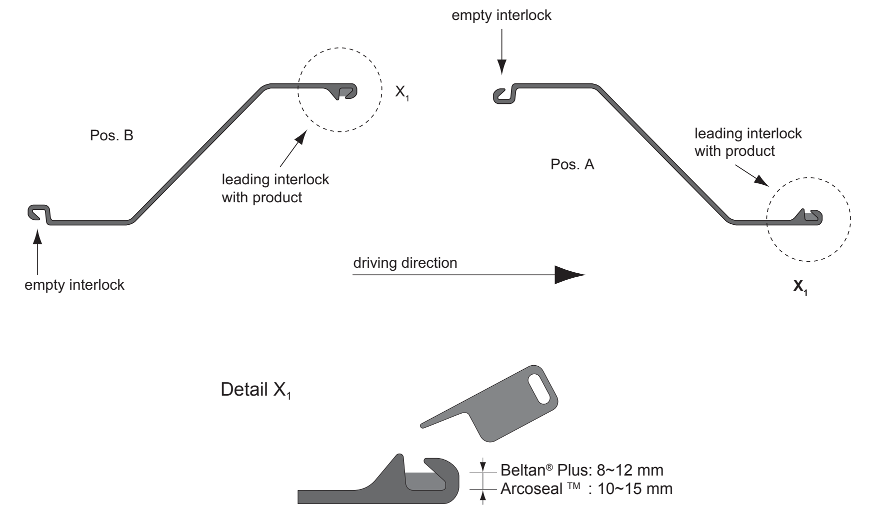


Fig. 2.4. Beltan Plus in single Z-Piles.

Detail of application in threaded Z sheet piles

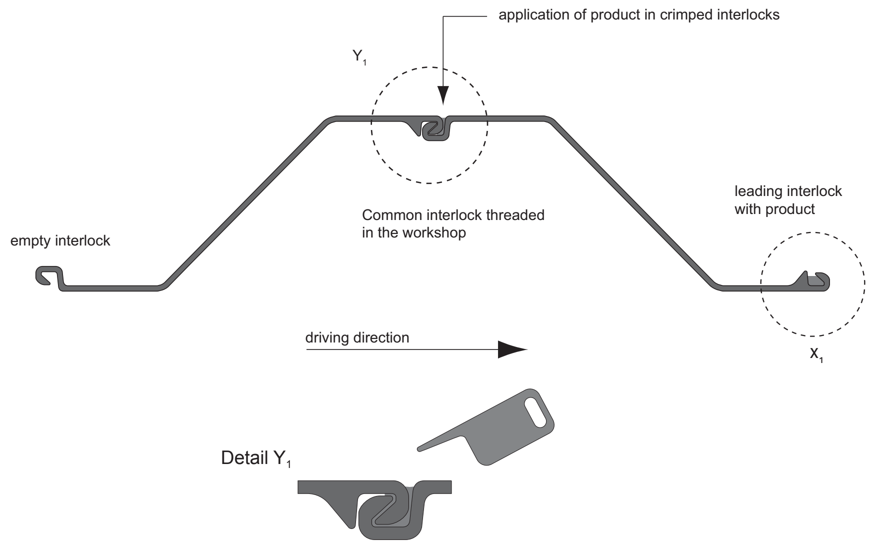


Fig. 2.5. Beltan Plus in double Z-Piles.

Application of Beltan® Plus and Arcoseal™ in situ

The application of Beltan® Plus or Arcoseal™ in situ is made in accordance with the same requirements as for the installation in the workshop.

In dry weather conditions, application in the open air may be acceptable.

During rainy weather, the application must be made under shelter.

Transport of the sealed sheet piles

If the sealing product has not yet solidified, the sheet piles must be transported horizontally with the openings of the treated interlocks turned upwards.

After the product has cooled down, the sheet piles must be protected from high temperatures (see note below) in order to prevent the product from flowing out of the interlock.

Note: Do not exceed the softening point of the product. For instance, it is recommended to avoid exposing sealed interlocks to direct sunlight during summertime.

Driving of the sealed sheet piles

Sheet piles which have been sealed using Beltan® Plus or Arcoseal™ are installed in a classic way, either by impact hammer, vibrator or by pressing.

As far as installation is concerned, it should be carried out as follows:

- the leading interlock must be the one provided with Beltan® Plus or Arcoseal™;
- when driving sealed sheet piles, care must be taken with guiding so as to prevent longitudinally or transversely out of plumb. The use of guides is essential to respect a maximum tolerance of 1% on the verticality;
- when sheet piling is simply installed without driving, it is possible that the sheet pile will not slide down to the required depth if there is an excess of the product in the interlock, or if the product has stiffened at low temperature. In such cases a driving equipment or any other means will be required on site to allow correct installation. Alternatively, the jammed intelock can be heated slowly and with due precautions by suitable means;
- when installing sheet piles in cold atmospheric conditions, a special mix of Beltan® Plus should be used;
- the installation of sealed sheet piles is not recommended with outside temperatures below -10°C (please contact us for more information).

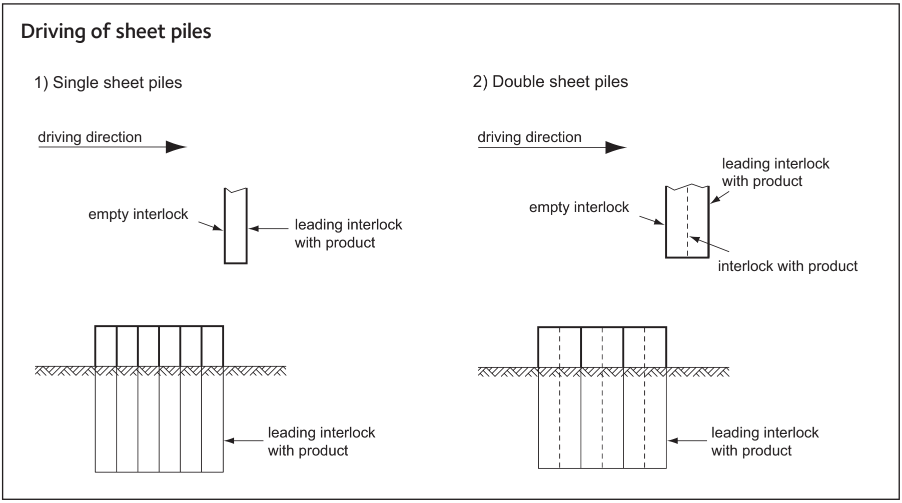


Fig. 2.6. Installation of sealed sheet piles.

2.13.4. Sealants with very low permeability: ROXAN® Plus and AKILA® system

For the applications with high performance requirements, it is advised to use the ROXAN® Plus system or the AKILA® system.

ROXAN® Plus system

This improved Roxan system consists of a water-swelling product Sikaswell®-S2 used in the trailing interlock and a Silane Modified Polymer MSP-2 used in the threaded and crimped interlock of double sheet piles. These two products are applied in the interlock without heating.

		ROXAN® Plus system	
Product		Sikaswell® S-2 +	MSP-2
Application		Trailing interlock	Common and crimped intelocks
Hydrostatic Pressure		≤ 200 kPa	
ρ		0.5 x 10^{-10} m/s	
Features	**Type**	Water-swelling polyurethane	Single component solvent free sealant
	Composition	polyurethane	MS polymer
	Density at 20°c	1.30 g/cm³	1.48 g/cm³
	Colour	Oxide red	Oxide red
Packaging		barrels of 30 kg	barrels of 25 kg
Conditions of application	**Surface covered with standing water**	impossible	impossible
	Wet surface	critical	critical
	Surface temperature	+5°C to 35°C (T°ambiant)	+5°C to +30°C (T° ambiant)
	Polymerization in rain	impossible	to be avoided
	Polymerization in UV light	excellent	excellent
Durability[2)]	**Fresh water**	pH 3.5 ~ pH 11.5 excellent	pH 3.5 ~ pH 11.5 excellent
	Sea water	excellent	excellent
	Mineral oil	low	medium
	Petrol[1)]	low to medium	medium
	Crude oil[1)]	low	medium
Expansion features	**Alternated cycle saturated in water / dry**	excellent	-
	Temperature range -10°C to +60°C	excellent	-
	In bentonite slurry	-	-
Consumption		± 0.15 kg / metre	± 0.35 kg / metre

Table 2.5. ROXAN® Plus system technical data.

[1)] Tested in laboratory on a pure solution.

[2)] Durability for other chemical substances is available on request.

Note: ROXAN® Plus is certified by the "Hygiene-Institut des Ruhrgebiets" in Germany as suitable for use in contact with groundwater.

Application of ROXAN® Plus system in the workshop

The application of the water-swelling product will always be made in the **trailing interlock** of single or threaded sheet piles, with the following requirements:

- the interlocks must be dry and free of grease;
- laying out the sheet piles in a perfectly horizontal position is not necessary, but recommended;
- to achieve the required adherence, cleaning of the interlocks with compressed air, a steel wire brush or high-pressure water jet is necessary;
- application of the product by extrusion and spreading the product using a special template (ArcelorMittal patent LU 88397) which distributes the product properly in the interlock.

Note: Spreading using the special template is essential to ensure the correct shape of the sealing.

The application of the MSP-2 product will always be made in the **intermediate interlock** of threaded and crimped sheet piles, with the following requirements:

- the interlocks must be dry and free of grease;
- the sheet piles must be laid out in a perfectly horizontal position;
- to achieve the required adherence, cleaning of the interlocks with compressed air, a steel wire brush or high-pressure water jet is necessary;
- to prevent the liquid product from flowing out of the ends of the sheet piles when the interlocks are filled, the ends must be clogged at the top and bottom using a mastic;
- the interlocks are filled using an appropriate jug, taking into account the driving direction as well as the position in relation to hydrostatic pressure.

Note: Standard crimping of the common interlocks of double piles is recommended. However, for combined walls like the HZ-M®/AZ® system, a special crimping pattern is advisable. Please contact our technical department for detailed information.

Detail of application in single Z sheet piles

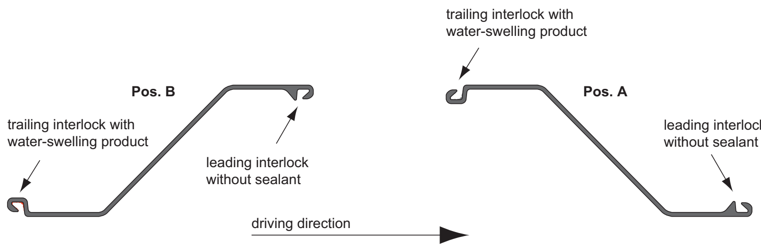


Fig. 2.7. ROXAN® Plus system in double Z-Piles.

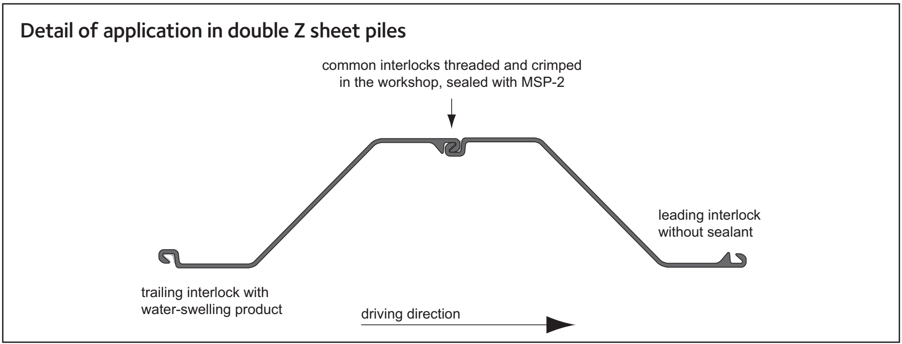


Fig. 2.8. ROXAN® Plus system in single Z-Piles.

Application of ROXAN® Plus system in situ

Application of the water-swelling product in situ is not advisable unless the work can be carried out under shelter. It must comply with the same requirements as for the application in the workshop (preferably under supervision of an experienced applicator).

Transport and storage of the sealed sheet piles

Sheet piles fitted with the water-swelling product must be transported so that the treated interlock with the waterswelling sealant never comes into contact with standing water (risk of expansion of the product after polymerization, and consequently loss of adhesion to steel). Care must be taken therefore to transport the piles with the **openings of the sealed free interlocks facing downwards**.

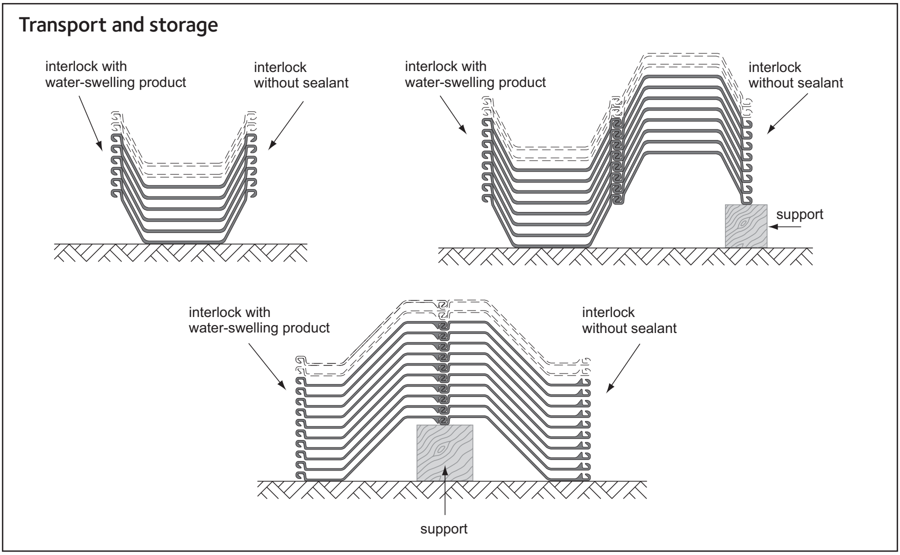


Fig. 2.9. Storage of sheet piles with water-swelling sealants.

Installation of the sealed sheet piles

Sheet piles with a water-swelling product are installed in a classic way, either by drop hammer, vibrator or by jacking.

As far as installation is concerned, it should be carried out as follows:

- care must be taken with guiding so as to prevent the piles from being longitudinally or transversely out of plumb. The use of guides is absolutely essential and installation must be carried out so that a tolerance of less than 1% on the verticality is respected;
- all the sealed sheet piling are delivered with the leading interlock chamfered on top and the trailing interlock (filled with water-swelling product) cut on toe (Fig. 2.14.). These two fabricated details allow the cleaning of the leading interlock of the sheet pile already driven by the engaging of the following one during the driving process. The purpose of this operation is to avoid damage to the water-swelling product;
- the water-swelling product must be lubricated before driving using “curd soap”. This product can be spread in the sealed interlock using a paintbrush or by any other means right before driving operations start;
- when sheet piling is simply placed in position without driving, it is possible that the piles will not slide down to the required depth because of the product. In such cases, a driving equipment must be provided on the site to allow correct installation;
- when piles are installed using a vibrator, care must be taken that the temperature in the interlocks never exceeds 130°C (risk of damaging the seal);
- during the installation process, the driving to final grade of each pile must be finished in less than two hours after the sealing product gets in touch with standing water (seawater, ground water, etc). Indeed, expansion of the sealing product would cause it to be torn off if driving is resumed after that period;
- the installation of sealed sheet piles is not recommended with outside temperatures below -10°C (please contact us for more information).

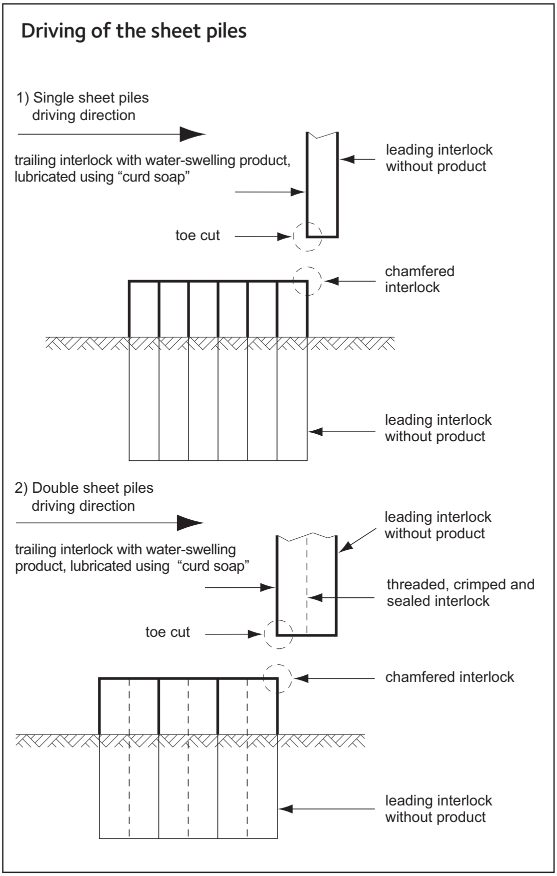


Fig. 2.10. Installation of piles with ROXAN® Plus system.

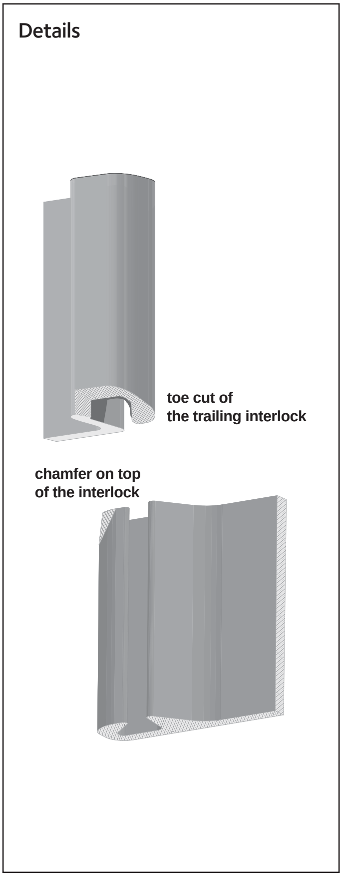


Fig. 2.11. Toe cut & chamfering.

AKILA® system

AKILA® is **an environmentally friendly high performance sealing system** for ArcelorMittal steel sheet piles. The system is based on three sealing "lips" mechanically extruded into the free interlocks using a product called MSP-1. The common interlock of double piles is sealed with a second product called MSP-2.

		AKILA® System	
Product		MSP-1 +	MSP-2
Application		Trailing interlock	Commom and crimped intelock
Hydrostatic Pressure ≤ 200 kPa		$\rho = 0.3 \times 10^{-10}$ m/s	
Hydrostatic Pressure ≤ 300 kPa		$\rho = 0.5 \times 10^{-10}$ m/s	
Features	**Type**	Single component solvent free compression sealant	Single component solvent free sealant
	Composition	MS polymer	MS polymer
	Density at 20°c	1.41 g/cm^3	1.48 g/cm^3
	Colour	Oxide red	Oxide red
Packaging		barrels of 25 kg	barrels of 25 kg
Conditions of application	**Surface covered with standing water**	impossible	impossible
	Wet surface	impossible	critical
	Surface temperature	+5°C to +35°C (T° ambiant)	+5°C to +30°C (T° ambiant)
	Polymerization in rain	to be avoided	to be avoided
	Polymerization in UV light	excellent	excellent
Durability[2)]	**Fresh water**	pH 3.5 ~ pH 11.5 excellent	pH 3.5 ~ pH 11.5 excellent
	Sea water	excellent	excellent
	Mineral oil	medium	medium
	Petrol[1)]	medium	medium
	Crude oil[1)]	medium	medium
Consumption		± 0.15 kg / metre	± 0.35 kg / metre

Table 2.6. AKILA® system technical data.

1) Tested in laboratory on a pure solution.
2) Durability for other chemical substances is available on request.

Note: AKILA® is certified by the "Hygiene-Institut des Ruhrgebiets" in Germany as suitable for use in contact with groundwater.

MSP-1 and MSP-2 belong to the family of silane modified polymers (MS-Polymers). Both products resist to humidity and weathering.
Their main characteristics are:

- single component elastic sealants;
- UV-stable;
- excellent adhesion to steel;
- resist to temperatures between -40°C and +90°C (up to 120°C for short periods);
- Shore A hardness after complete polymerization
  - 58 for MSP-1;
  - 44 for MSP-2 (after 14 days);
- durable in contact with freshwater, seawater, as well as various hydrocarbons, bases and acids (depending on concentration – a list is available on request).

MS-Polymers are solvent free and do not contain isocyanates. They can be considered as environmentally friendly products.

The leading interlocks have to be chamfered at the top (see Fig. 2.14.). Penetration of soil into the interlocks during driving should be prevented, for instance by inserting a bolt at the bottom of the interlock (bolt tack welded).

To improve the sliding of the interlocks, an environmentally friendly lubricant must be applied to the sealant in the interlocks prior to driving.
The layout and driving direction of the sheet pile wall shall be determined before ordering the sheet piles (delivery form of double piles, chamfering of interlocks, etc).

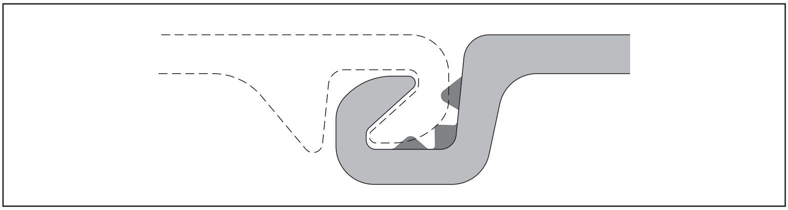


Fig. 2.12. MSP-1 Product extruded into trailing interlock.

A series of in-situ tests were carried out in stiff clays and in soft sandy soils. Single and crimped double sheet piles fitted out with the AKILA® system were driven into the ground using an impact hammer as well as a vibratory hammer.

In case of vibrodriving, sheet piles were driven continuously at a minimum rate of 20 seconds per meter. After installation, watertightness was tested at water pressures of 2 and 3 bar. The testing and the results were witnessed and certified by “Germanischer Lloyd”, an independent third party.

Application, transport and installation of the AKILA® system

Refer to the Roxan® Plus system, except that:

- the sealing products do not swell in contact with water, and hence, there are no restrictions concerning the installation time, nor the position of the sheet pile during handling and storage (once the products have cured);
- the leading interlocks have to be chamfered at the top (see Fig. 2.13.). Penetration of soil into the interlocks during driving should be prevented, for instance by inserting a bolt at the bottom of the interlock (bolt tack welded);
- to improve the sliding of the interlocks, an environmentally friendly lubricant must be applied to the sealant in the interlocks prior to driving. The lubricant can be supplied by ArcelorMittal on request;
- ambiant temperature during installation must be above 0 °C;
- in case of vibrodriving, sheet piles should be driven continuously at a minimum penetration rate of 3 meters per minute;
- we recommend prior consulation of ArcelorMittal's technical department in case the press-in method is to be be used.

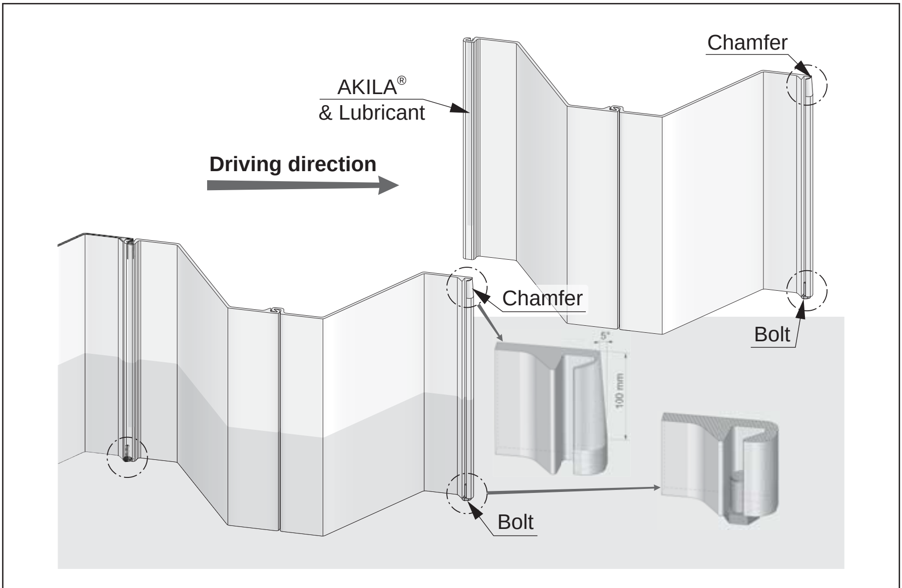


Fig. 2.13. Installation procedure in AKILA® system.

2.13.5. Seal-welding

Welding of the sheet pile interlocks is the most effective way of permanently sealing sheet pile interlocks. This is commonly carried out in basement construction where the exposed face of the piling is easily accessed and water tightness to Grade 2 or 3 as defined in BS8102 is required. However to achieve a quality weld it is necessary to clean the surface and carry out the welding in dry conditions.

The majority of electric arc welding processes are considered to be valid for sealing the interlocks of sheet piling threaded in the workshop or on the site.

To prevent problems linked to the quality of welding, it is advised to analyze beforehand the feasibility and the competitiveness of the welding process. It is necessary to remember that the competitiveness of a welding process rests on several factors, for example:

- deposition rate in kg/h;
- welding time, i.e. the time of arc per hour;
- efficiency of the welding product (the weight actually deposited per kg of product);
- preparation of the joint;
- welding position.

In a workshop, working conditions are well known and under control, but outside a workshop above criteria can be influenced by various factors and particular attention must be paid to the following points:

- accessibility of the pile;
- atmospheric conditions on the site;
- mechanical strength of the weld metal (thickness of seam and penetration to be observed);
- amount of moisture inside the interlocks;
- gap between the interlocks;
- aggressiveness of the environment acting on the welds.

A detailed analysis of the different welding options will determine the most suitable process for the encountered conditions.

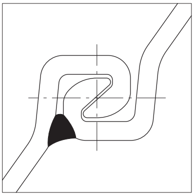


Fig. 2.14. Simple seal weld.

When the gap between adjacent interlocks is small enough, it is possible to create a seal by applying a simple fillet weld across the joint as illustrated above.

However, as sheet piling work is subject to on site tolerances, a range of practical options have been developed to cope with gaps of varying sizes.

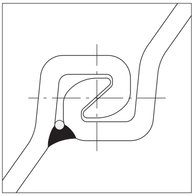


Fig. 2.15. Seal weld with additional filled.

Where the gap is too large to be bridged by a single pass, introduction of a small diameter bar can be effective with a weld run applied to either side of the joint to create the seal.

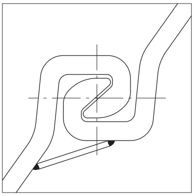


Fig. 2.16. Welded plate.

For wide gaps or where water is running though the interlock making the creation of an acceptable weld difficult to achieve by welding, a plate of sufficient width to suit the specific conditions across the joint, it is possible to create a vertical drain to channel any seepage away from the weld.

ArcelorMittal Sheet Piling is at your disposal to advise you on the choice of the best solution and process. Further welding procedures are available, especially for on-site solutions.

Possible ways of welding the interlocks of sheet piling

A distinction must be made between two ways of fitting together the interlocks of piles and two welding positions for them:

- in the case of piles being supplied to the site in double units, the common interlocks (threaded in the workshop) can be provided with seal-welding carried out at the factory or, possibly, on site before they are driven. This welding should be carried out in a horizontal position;
- interlocks threaded at the job-site can only be welded after the sheet piling has been installed, generally after excavation. This welding is carried out in a vertical position.

Note: If interlocks are to be welded on site after driving, a preliminary seal using a bituminous product is recommended. This sealing can be applied either in the factory or on site before driving, and prevents the interlock from becoming too humid which could cause serious problems during welding operations. In this case the positioning of the bituminous sealer must be as shown in Fig. 2.18. & Fig. 2.19., detail A, which prevents contact between the weld and the bituminous product! This requirement must be mentioned in the specification.

Choice of site welding process

The choice of possible processes is limited to the following systems:

a) Shielded metal arc welding (SMAW);

b) Gas shielded arc welding (GMAW);

c) Flux cored arc welding (FCAW).

2.13.6. Alternative solutions

It is possible to seal sheet pile walls by other processes than those described previously.

Composite wall with bentonite-cement

Composite walls combine the sealing qualities of bentonite with the mechanical strength of steel sheet piling. This system also allows work to be carried out at great depth and in difficult ground conditions. The disadvantage of the technique is the production of excavated material which is considered as polluted material.

Vertical pre-drilling on the axis of the leading interlock to be driven

A hole is drilled on the axis of the future leading interlock. Drilling diameter: between 300 and 450 mm. Distance between two holes: distance between the outer interlocks of the double pile. The extracted soil is replaced with bentonite. This method also assists driving and can be combined with a sealed interlock as described previously. The amount of excavated material is limited.

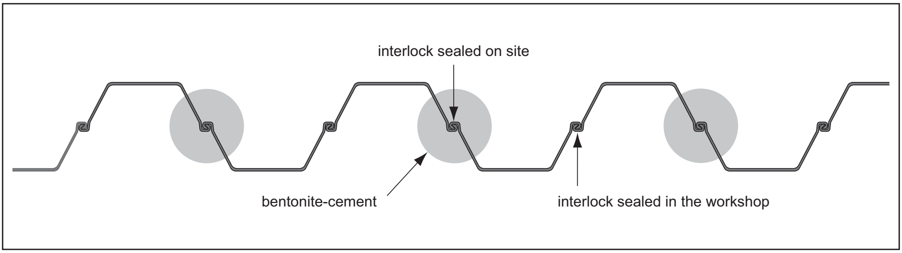


Fig. 2.17. Pre-drilling.

2.13.7. Repairing defects in the sealing of interlocks

When a driving incident damages a sealed interlock certain methods can be used as a repair.

The choice of the repair method depends on the following factors:

- type of sealing process (sealing product, welding, etc...);
- location of the sealing joint (see Fig. 2.18. and Fig. 2.19.);
- gap of interlocks (see Fig. 2.20.);
- level of humidity in the interlocks;
- accessibility.

In this chapter, methods for repairs above and below ground level are briefly explained.

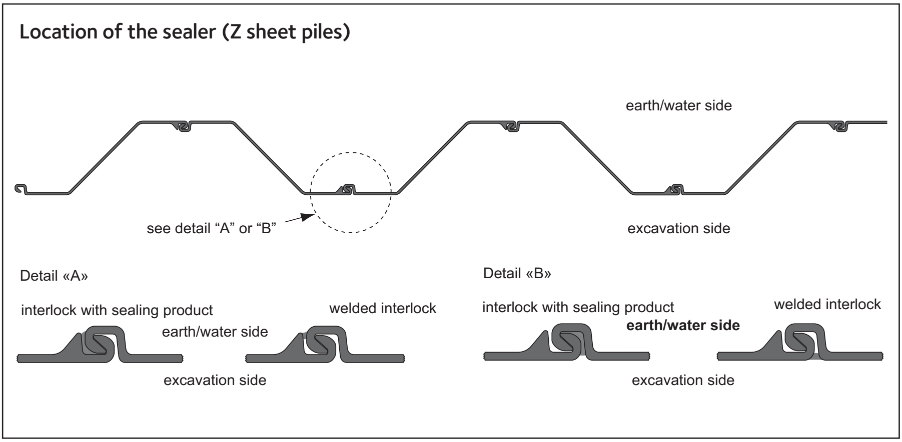


Fig. 2.18. Location of seals (Z-Piles).

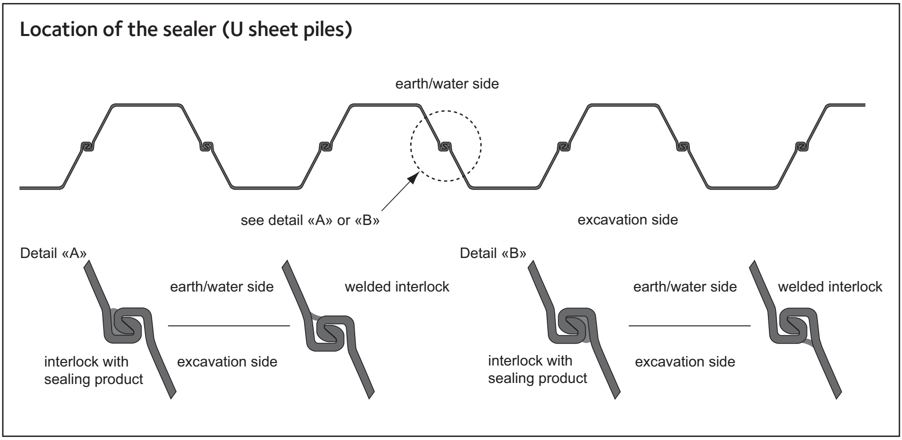


Fig. 2.19. Location of seals (U-Piles).

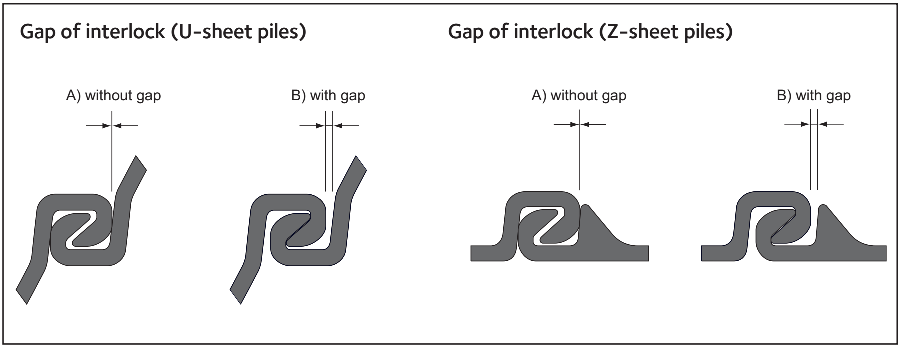


Fig. 2.20. Gap of interlock.

Repairs above ground level (interlock accessible on the excavation side)

Method 1

Application of a seal-weld along the interlock over the required height of the pile.

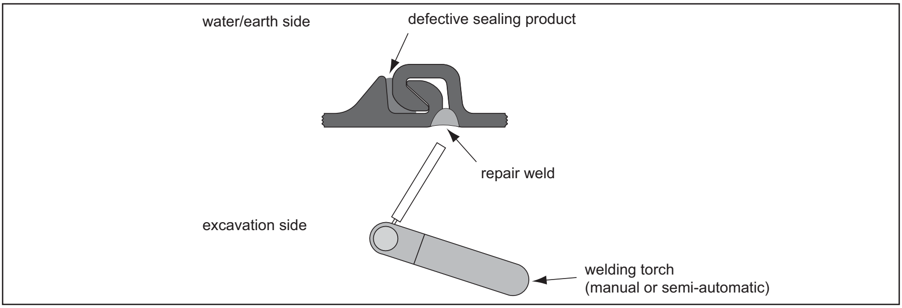


Fig. 2.21. Repair method 1 - repairs above ground level.

Method 2

Welding of a plate or an angle over the interlock over the required height of the pile.

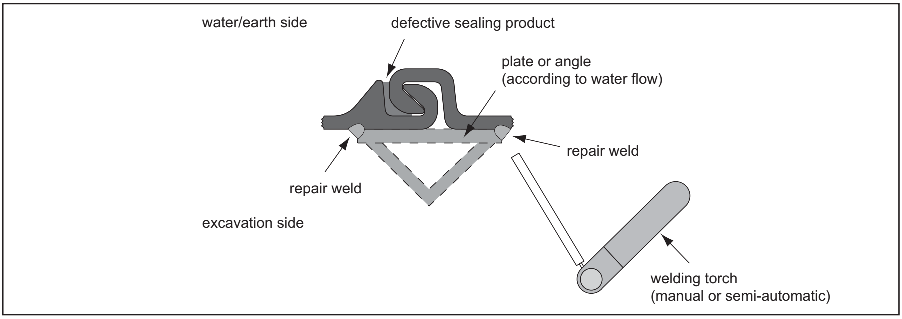


Fig. 2.22. Repair method 2 - repairs above ground level.

Method 3

Sealing by filling the gap between the interlocks with plastic sections, strips of water-swelling rubber or prelaminated timber laths over the required height of the pile.

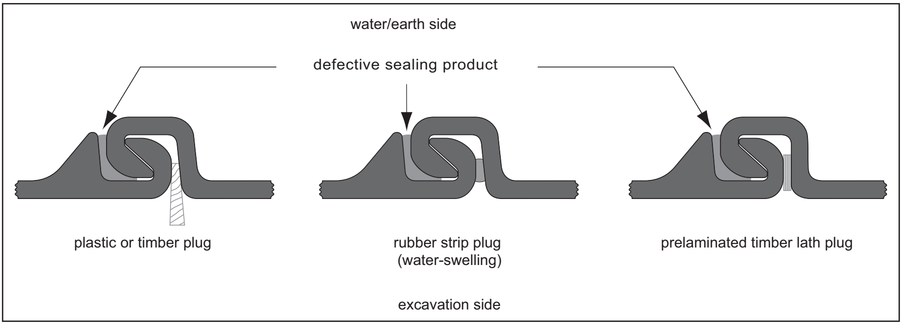


Fig. 2.23. Repair method 3 - repairs above ground level.

Repairs below ground level

Method 1

Excavation down the length of the interlock to be sealed and extension of the seal-weld or the interlock plug down to the necessary depth.

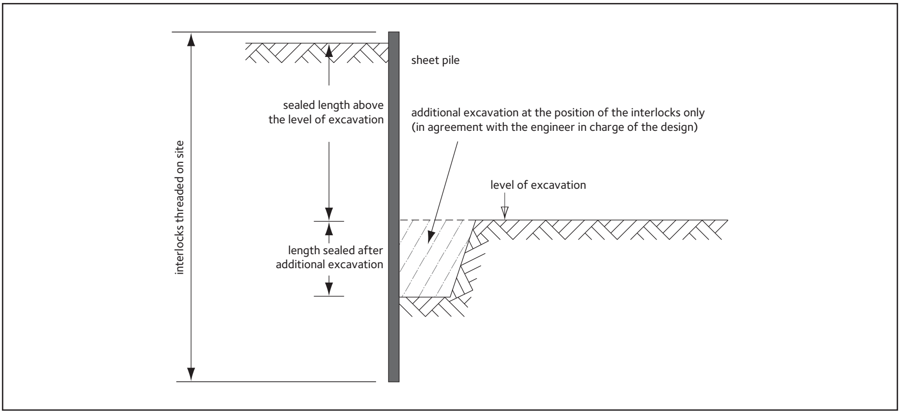


Fig. 2.24. Repair method 1 - repairs below ground level.

Method 2

Injection of a product (fast setting cement or bentonite) behind the wall along the interlock to be sealed.

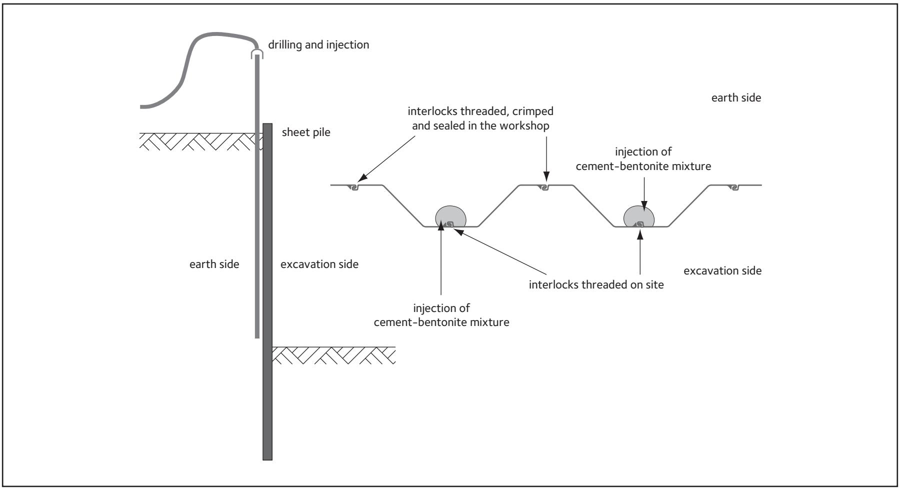


Fig. 2.25. Repair method 2 - repairs below ground level.

Method 3

In the event of more serious leaks, form a trench along the bottom of the excavation, install a drainage system and connect to a pumping system.

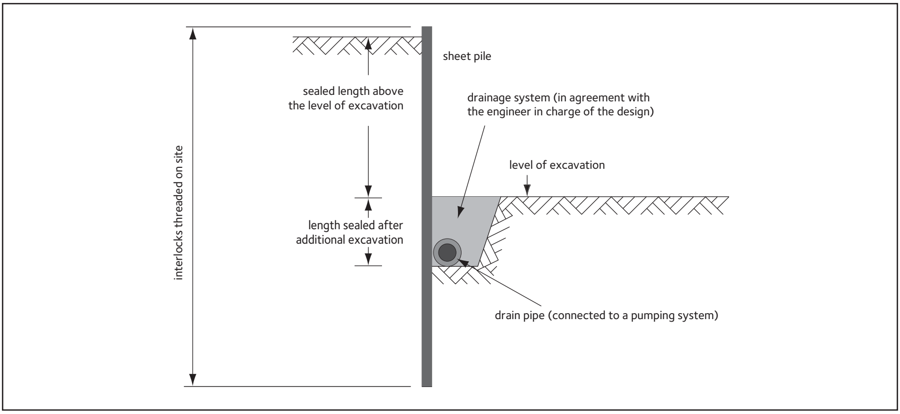


Fig. 2.26. Repair method 3 - repairs below ground level.

Repairs in water

In the event that it is required to create or to repair a seal on the water side, the solutions with a mixture of sawdust and fast setting cement poured from the water side might be suitable.

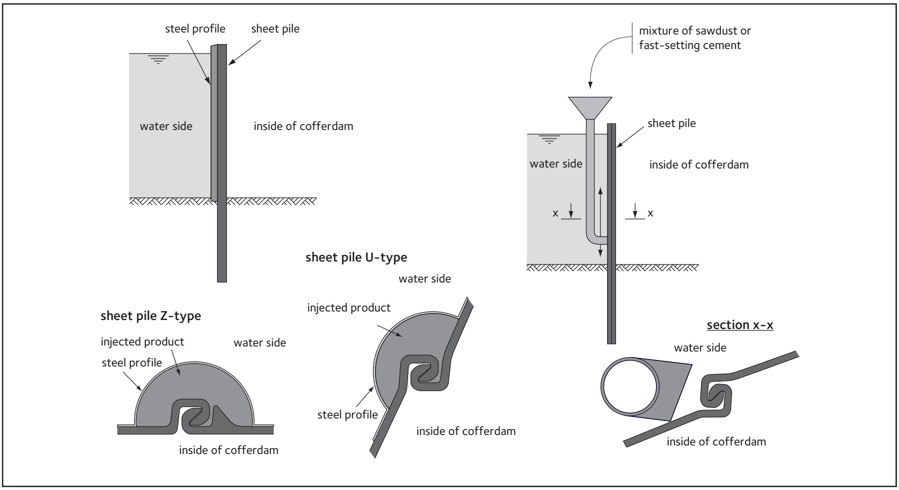


Fig. 2.27. Repairs in water.

Remarks:

It is important to note, that **all ρ values given in this document are characteristic values** (maximum values considered as "cautious estimates") which are results of in-situ tests. **For the determination of design values, a safety factor has to be carefully chosen** in order to balance the scattering of the test results and the imponderables inherent to the installation of the piles, the soil, local defects, etc. Please contact the Technical Department for guidelines on this matter.

References:

[i] Steel Sheet Pile Seepage Resistance, J.B. SELLMEIJER, Fourth International Landfill Symposium, Cagliari, Italy, 1993.

[ii] Joint Resistance of Steel Sheet Piles, Definition, J.B. SELLMEIJER, August 1993 (unpublished).

[iii] The Hydraulic Resistance of Steel Sheet Pile Joints, J.B. SELLMEIJER, J.P.A.E. COOLS, W.J. POST, J. DECKER, 1993 (publication ASCE).

[iv] EAU 2012, Recommandations of the Committee for Waterfront Structures, Harbours and Waterways, Berlin, 2012. (Ernst & Sohn).

3 | Environmental product declaration

Chapter 3 - Environmental product declaration

Contents

3.1. Environmental product declaration for steel sheet piling structures

This chapter is intended for understanding the environmental performance of steel sheet piling structures, used in the construction of quays and harbours, cofferdams, bridge abutments, retaining walls, foundation structures, etc.
The information given is based on the Life Cycle Assessment (LCA) study "Comparative environmental evaluation of retaining structures made of steel sheet piling or reinforced concrete" [i], which has been peer reviewed to be in compliance with the ISO standards 14040 [ii] and 14044 [iii] by "RDC Environment".

3.1.1. LCA

LCA (Life Cycle Assessment) is a set of techniques, based on ISO standards, used to account for the input and outputs of materials and energy, as well as the production of pollutants related to a product or service throughout its entire life-cycle. Three main phases can be identified: production (including provision of the raw materials and product manufactoring), usage and end-of-life.
In Fig. 3.1. the life-cycle including the phases with inputs and emissions are shown exemplarically for steel products.

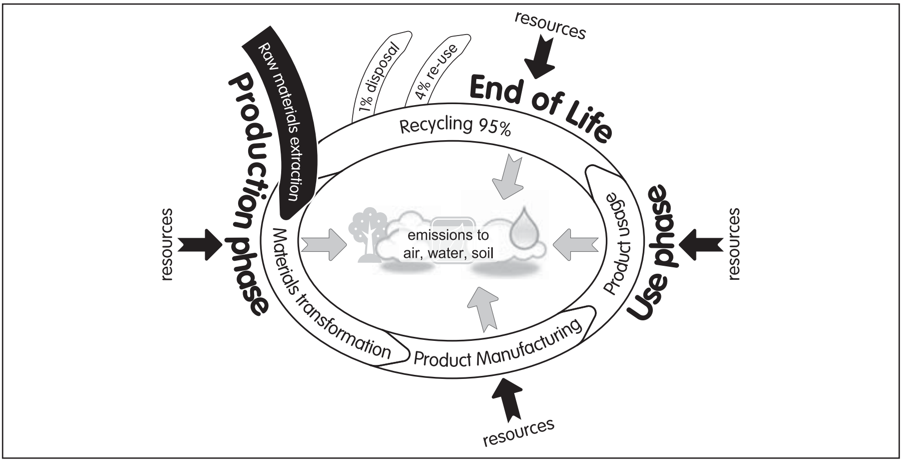


Fig. 3.1. Life-cycle of steel products.

3.1.2. The Functional Unit

The results of a LCA are related to the functional unit, which is used to describe the functions to be fullfilled by a product system. When comparing several products, it is necessary to consider an identical functional unit for the products.

3.1.3. Environmental indicators

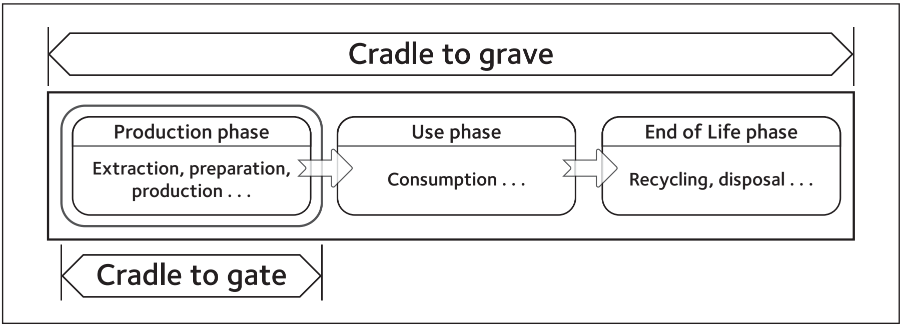
LCA practitioners usually assess some common environmental indicators. Generally, several substances contribute to a given environmental impact. For example, carbon dioxide (CO_2), methane (CH_4), nitrous oxide (N_2O) and some other components all contribute to global warming, and are therefore aggregated and expressed as “CO_2 - equivalent” to summarize the total contribution to this indicator. Acidification is expressed as sulphur dioxide (SO_2) equivalent, and so on for the other impacts.

The following environmental indicators have been focussed in the EPD for steel sheet piles:

- **Primary Energy Consumption** accounts for the total primary energy needed along all the stages. It is expressed in megajoules (MJ);
- **Global Warming Potential** represents the contribution of the product to the increase in atmospheric carbon, leading to global increase in temperature. It is expressed as CO_2-equivalent;
- **Acidification** occurs when the product contributes to the acidification of rain, provoking damages to vegetation and forests. It is expressed as SO_2-equivalent;
- **Ozone Formation At Low Altitude** is better known as summer smog and has consequences on breath diseases. It is expressed as C_2H_4 (Ethene)-equivalent;
- **Eutrophication** occurs when surface waters are artificially enriched by nutrients such as phosphated compounds, creating disturbances in the biological balance. Stated as PO_4 (Phosphate)-equivalent;
- **Water** consumption is calculated by examining the water intake minus outtake flows of the inventories. It is expressed as kg of water used.

3.2. Environmental burdens of steel products used for sheet piling structures

The official World Steel Association formula used to calculate the environmental burden E is explained hereafter. It is also available for some products in the European database ELCD (European Life Cycle Data).



$$E = E' - (RR - RC) \times LCI_{scrap} \quad \text{and} \quad LCI_{scrap} = y \times (X_{pr} - X_{re})$$

with

E' — "Cradle to gate" environmental burden due to the production phase (iron ore extraction, preparation, production...). Data used are mean values given by the World Steel Association (WSA), taking into account a mix of smelting process data and electrical arc furnace process data.
A scrap rate (scrap - RC) is included. It is determined for each of the 16 products treated by the World Steel Association;

RR — Recycling Rate at the end of life;

RC — Recycled Content = amount of scrap used to produce steel;

LCI_{scrap} — LCI given by World Steel Association, representing the environmental value of scrap. It represents how much environmental burden could be avoided by using scrap as raw material. For example, for «sections», 1.0 kg of scrap has a Global warming Potential of 1.613 kg CO_2 equivalent;

y — Efficiency of the electrical arc furnace in converting scrap into steel. For example following WSA LCI 2010 data, 1091 kg of scrap are necessary to produce 1000 kg of steel;

X_{pr} — LCI for primary steel production (slab production by blast furnace BOF with 100% iron ore input);

X_{re} — LCI for secondary steel production (slab production by electric arc furnace EAF with100% scrap input).

This formula allows to take into account the benefit of the end-of-life recycling and promotes the recycling of products at the end of their use phase.
The environmental burden of steel sheet piles and tierods are given in the Table 3.1., based on the LCI 2010 data given by the World Steel Association (WSA). Currently it is expected that only sections will be recycled.

Among the 16 life cycle inventories (LCI) of steel products provided by World Steel Association [iv], the following two LCIs are typically used for a LCA of steel sheet pile structures:

- **"sections" for sheet piles and wailings;**
- **"rebars" for tierods.**

		Sections			**Rebars**	
		Cradle to gate	Cradle to grave		Cradle to gate	Cradle to grave
	Unit [1)]	RR = 85%	85%	0%	70%	0%
Primary Energy Consumption	GJ	14.8	14.8	25.5	16.4	25.8
Global Warming Potential	*kg* CO2-eq	1143	1141	2513	1244	2300
Acidification	*kg* SO2-eq	3.21×10^{-3}	3.21×10^{-3}	5.45×10^{-3}	3.44×10^{-3}	5.87×10^{-3}
Ozone formation at low altitude	*kg* C2H4-eq	0.99×10^{-3}	0.99×10^{-3}	2.08×10^{-3}	1.09×10^{-3}	2.00×10^{-3}
Eutrophication	*kg* PO4-eq	0.28×10^{-3}	0.28×10^{-3}	0.44×10^{-3}	0.26×10^{-3}	0.42×10^{-3}
Water	*kg*	1332	1328	5398	13869	22994

Table 3.1. Environmental burden of sheet piles and tierods based on the LCI 2010 data given by the World Steel Association. European data used for "sections" and World data used for "rebars" [2)].

[1)] *Per tonne of steel produced.*
[2)] *RC = 84.9% (sections), RC = 69.8% (rebars). For example: GWP with LCI_{scrap} = 1.613 kg (sections), LCI_{scrap} = 1.512 kg (rebars).*

3.3. LCA for a 100 m retaining wall made from steel sheet piles

The **functional unit** selected is a 100 m retaining wall structure with a main wall (sheet pile) length of 9.9 m and an excavation depth of 6.0 m.

The following parameters have been considered:

- according to the LCA approach, all elements such as transportation, as well as installation and extraction of the sheet piles are taken into account;
- the main wall and the anchor walls are made of steel sections, while the tierods are made of steel rebars;
- usually wall and waling parts are recovered and therefore recycled, while tierods are generally not recovered due to technical reasons;

The excavation depth is a parameter imposed by the configuration of the construction site. During the use phase, impacts are negligible. Results are aggregated as the sum of the impacts of the production and end-of-life stage in Table 3.2.

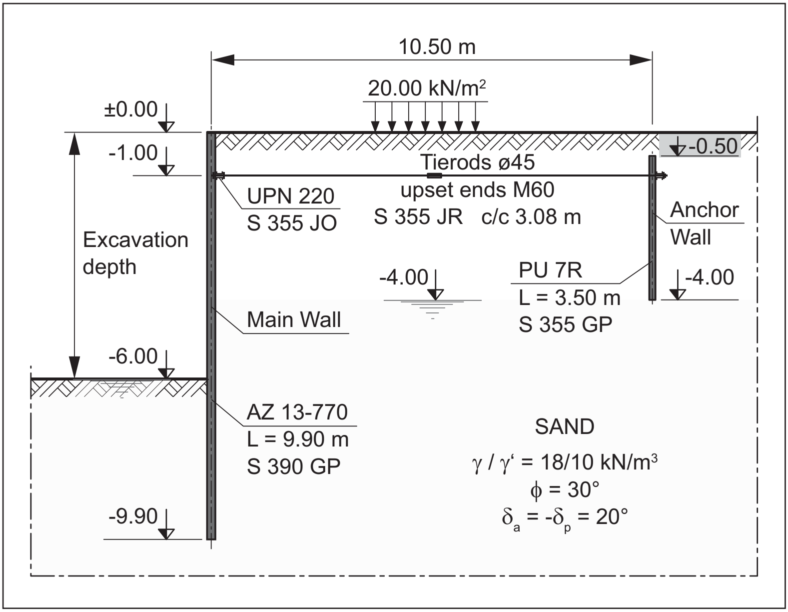


Fig. 3.2. Functional unit cross section: 100 m retaining wall structure.

	Unit	Production "cradle to gate"	Total "cradle to grave" No recycling (RR = 0 %)	Total "cradle to grave" $RR_{Sheet\ piles}$ = 85 % $RR_{Tierods}$ = 0 %
Primary Energy Consumption	GJ	2136	3635	2189
Global Warming Potential	*t* CO2-eq	163	354	167
Acidification	*t* SO2-eq	0.465	0.779	0.476
Ozone formation at low altitude	*t* C2H4-eq	0.139	0.277	0.143
Eutrophication	*t* PO4-eq	0.040	0.062	0.041
Water	*t*	253	842	289

Table 3.2. Environmental profile of the 100 m steel sheet piling structure (functional unit): 136 t of steel sheet piles and 4 t of tierods.

From the LCA, the following is to be concluded:

- the distribution of impacts is equivalent for all indicators; **steel production is the main contributor** (between 93% to 98% of the impacts);
- tierods represent between 3% and 11% of the impacts, while their weight proportion is only 3%;
- transportation, installation and extraction have a very low contribution to the global environmental impacts.

The steel sheet pile solution for 100 m retaining wall is a rapid, cost-effective, reliable and durable solution. The LCA demonstrates, it has also a relatively low environmental impact.

References:

[i] *Hettinger, A.L.; Bourdouxhe, M.P.; Schmitt, A. "Comparative Environmental evaluation of retaining structures made of steel sheet piling or reinforced concrete". ArcelorMittal, 2010.*

[ii] *ISO 14040. "Environmental management - Life cycle assessment - Principles and framework". 2006.*

[iii] *ISO 14044. "Environmental management - Life cycle assessment - Requirements and guidelines". 2006.*

[iv] *International Iron and Steel Institute. "World Steel Life Cycle Inventory – methodology report". November 2005.*

4 | Earth and water pressure

Chapter 4 - Earth and water pressure

Contents

Symbols (according to EN 1997 - Part 1)

General geotechnical notation		**Units**
c'	cohesion intercept in terms of effective stress	kN/m^2
c_u	undrained shear strength (total stress)	kN/m^2
K_a	coefficient of active earth pressure	-
K_{ac}	active pressure coefficient for cohesion	-
K_p	coefficient of passive earth pressure	-
K_{pc}	passive pressure coefficient for cohesion	-
q	surcharge pressure	kN/m^2
u	pore-water pressure	kN/m^2
z	vertical distance / depth	m
δ	structure-ground interface friction angle	degrees
γ	weight density of soil (aka bulk density)	kN/m^3
γ'	submerged weight density of soil	kN/m^3
γ_w	weight density of water	kN/m^3
φ'	effective angle of shearing resistance	degrees
φ'_d	design effective angle of shearing resistance	degrees
φ_{cv}	critical state angle of shearing resistance	degrees

Symbols

φ_p	peak angle of shearing resistance	degrees

4.1. Introduction

The assessment of soil stratification and assignment of appropriate engineering parameters is a fundamental part of the design process for an embedded retaining wall. The soil not only creates the forces attempting to destabilise the wall but also provides the means by which stability is achieved. So an understanding of the importance of soil in the design of retaining walls is paramount. Further, the selection of an appropriate pile section, installation method and equipment is itself a function of the properties of the ground.

Soil parameters for use in design calculations should be obtained, wherever possible, by sampling and testing material from the site. However, indicative parameter values are included in section 4.7. for use in preliminary calculations.

The amount and complexity of data needed to carry out retaining wall design is, to an extent, governed by the calculation method to be used. For example, if the analysis is to be carried out on the basis of limiting equilibrium, relatively simple soil data can be used to obtain a satisfactory answer; but if the problem is to be analysed using finite element techniques, the data required to adequately describe the behaviour of the soil is significantly more complex. Additional or more complicated soil data will involve greater site investigation cost and it is often the case that the client is not prepared to sanction greater expenditure at the investigation stage of a project. In many cases, however, the additional cost is easily recouped by avoiding false economies and conducting a more sophisticated analysis.

The precise and adequate determination of site conditions prior to the commencement of any form of civil engineering construction work is standard practice. Where piled foundations, cofferdams, retaining walls, etc. are to be driven, it is essential that as much information as possible be obtained regarding strata, ground water, tidal water, embankments, existing foundations, buried services and the like, in order to design the most suitable piling in terms of driveability, strength, stability, and economy.

Full use should be made of all available information, no matter how old, regarding previous investigation of the proposed site and its surroundings. Such information should be supplemented with data obtained from borehole sampling and testing, the number of boreholes depending upon the size, and nature of the site.

4.2. Ground Investigation Report (GIR)

For any project requiring an embedded sheet pile retaining wall structure supporting excavations below the water table, there should be available to the designer and contractor a comprehensive GIR report compliant with the requirements of Eurocode 7 - Part 2 [i]. For sheet-piled structures, the aspects and implications of information for the ground conditions relevant to pile driving and design and durability should be covered sufficiently.

According to Eurocode 7 - Part 2 the Ground Investigation Report must provide a factual account of all field and laboratory investigations, presented in accordance with the EN and/or ISO standards used in those investigations. The report documents the methods and procedures used - and results obtained - from desk studies, sampling, field tests, groundwater measurements, and laboratory tests. The factual account should include a description of the site and its topography, in particular: evidence of groundwater, areas of instability, difficulties during excavation, local experience in the area.

The contents of the GIR include:

Presentation

- a factual account of field and laboratory investigations;
- a description of the site conditions;
- documentation of methods, procedures, and results.

Evaluation

- results of field and laboratory investigations evaluated according to EN 1997-2;
- a review of the results;
- a description of the geometry of all strata;
- detailed descriptions of all strata;
- comments on irregularities.

Derived values

- correlations and their applicability.

Once the design is complete all the data, assumptions, interpretation, design, supervision, maintenance and monitoring information is produced in a Geotechnical Design Report (GDR).

4.3. Extent and depth of investigation

For piling work, the number of boreholes, or other form of investigation, should be adequate to establish the ground conditions along the length of the proposed piling and to ascertain the variability in those conditions. The centres between boreholes will vary from site to site but for retaining wall structures should generally be at intervals of 20 m to 200 m along the length of the wall. Closeness of position to the proposed pile line and spacing is particularly important for river walls and where glacial deposits with a high degree of variability prevail. For embedded sheet pile walls particular attention to ground levels and the position of the boreholes is important for the relevance of information for the designer.

The site investigation and borehole detailing and planning should follow guidelines and rules given in Eurocode 7 - Part 2 [i]. Annex B.3 provides outline guidance on the depth of investigation points for retaining structures and piles. The UK National Annex to Eurocode 7 - Part 2 [ii] makes this guidance mandatory. It is important that the depth of the boreholes should always extend beyond the anticipated lowest point of an embedded retaining wall or bearing pile.

4.4. Groundwater and seepage

Measurement of groundwater conditions, the level of the water table, and their variation with time is a vital part of any site investigation. The effect that water has on the engineering properties of soil should be clearly understood and carefully considered during the site investigation period. In addition to the tests on individual soil samples, the direction of seepage, upwards or downwards, should be determined before any decision is reached on the design of a sheet piled retaining wall together with a system incorporating reliable drainage.

4.5. Identification and classification of soil and rock

Eurocode 7 - Parts 1 and 2 - coupled with the associated geotechnical testing standards EN ISOs 14688 [iii] and 14689 [iv] – provide guidance for the description of soils and rocks. The requirements of these standards supersede those in British Standard BS 5930: 1999 [v] (which will be updated to reflect their content). Although the new standards are broadly similar to BS 5930, this section provides an overview of their new requirements.

4.5.1. Types of soils

1. coarse grained - cohesionless soils: granular materials such as sand, gravel, weathered rock, filling etc.;
2. fine grained - cohesive soils: clays and silts. Under certain conditions chalk and other similar materials can be treated as cohesive soils;
3. mixed soils: combinations of groups 1 and 2 such as sand with clay, or sand with silt;
4. rock.

4.5.2. Soil description

Soil description to EN ISO 14688 [iii] is outlined in the following.

The principal fraction of a composite soil should be indicated by a capital letter (e.g. Sa for sand or SAND) and the secondary fraction by lower-case letters (e.g. gr for gravelly). The shape of a soil's particle size distribution (or grading curve) is described by the terms multi-graded, medium-graded, even-graded, and gap-graded.

Example:

saGr = sandy gravel or (in the UK) sandy GRAVEL;

msaCl = medium sandy clay or (in the UK) medium sandy CLAY.

Description/abbreviation				Particle size d (mm)
Very coarse soil		Large boulder	*LBo*	> 630
		Boulder	*Bo*	200 - 630
		Cobble	*Co*	63 - 200
Coarse soil	**Gravel**	Coarse	*CGr*	20 - 63
		Medium	*MGr*	6.3 - 20
		Fine	*FGr*	2 - 6.3
	Sand	Coarse	*CSa*	0.63 - 2
		Medium	*MSa*	0.2 - 0.63
		Fine	*FSa*	0.063 - 0.2
Fine soil	**Silt**	Coarse	*CSi*	0.02 - 0.063
		Medium	*MSi*	0.0063 - 0.02
		Fine	*FSi*	0.002 - 0.0063
	Clay		*Cl*	< 0.002

Table 4.1. Soil description to EN ISO 14688.

4.5.3. Relative density of coarse soils

The relative density of a coarse soil is classified in EN ISO 14688-2 [iii] according to the value of its density index ID, defined as:

$$I_D = \frac{e_{\max} - e}{e_{\max} - e_{\min}}$$

with

e soil's void ratio;

e_{max} soil's maximum voids ratio;

e_{min} soil's minimum voids ratio.

The table 4.2. gives typical correlations for quartz sands between relative density, standard penetration test (SPT) blow count, cone resistance, and angle of shearing resistance derived from correlations given in Annexes D and F of
Eurocode 7 - Part 2:

Density index, I_D (%)	Relative density	SPT blow count (N_1) 60	Cone resistance q_c (MPa)	Angle of shearing resistance ϕ (°)
0 - 15	Very loose	0 - 3	<2.5	29 - 32
15 - 35	Loose	3 - 8	2.5 - 5.0	32 - 35
35 - 65	Medium dense	8 - 25	5.0 - 10.0	35 - 37
65 - 85	Dense	25 - 42	10.0 - 20.0	37 - 40
85 - 100	Very dense	42 - 58	> 20.0	40 - 42

Table 4.2. Typical correlations for quartz sands.

4.5.4. Consistency of fine or cohesive soils

The consistency of a fine soil is classified in EN ISO 14688-2 [iii] according to the value of its consistency index IC, defined as:

$$I_C = \frac{w_L - w}{w_L - w_P}$$

with

w soil's water content;

w_L soil's liquid limit;

w_P soil's plastic limit.

Consistency index I_C (%)	Consistency	Field description
0.00 - 0.25	Very soft	Exudes between fingers when squeezed in fist
0.25 - 0.50	Soft	Can be readily excavated with a spade and can be easily moulded by substantial pressure in the fingers
0.50 - 0.75	Firm	Can be excavated with a spade and can be remoulded by substantial pressure in the fingers
0.75 - 1.00	Stiff	Requires a pick or pneumatic spade for its removal and cannot be moulded with the fingers
> 1.0	Very stiff	Requires a pick or pneumatic spade for its removal and will be hard and brittle or very tough

Table 4.3. Consistency of fine soils.

4.5.5. Strength of fine or cohesive soils

The strength of a fine soil is classified in EN ISO 14688-2 [iii] according to the value of its undrained shear strength c_u measured in a field or laboratory strength test.

Undrained shear strength c_u (kPa)	Strength	Equivalent consistency
< 10	Extremely low	Very soft
10 - 20	Very low	Very soft
20 - 40	Low	Soft
40 - 75	Medium	Firm
75 - 150	High	Stiff
150 - 300	Very high	Very stiff
> 300	Extremely high	Hard

Table 4.4. Strength of fine soils.

When both the consistency and strength are measured, the soil might be described as, for example, a “stiff fissured high strength CLAY”. The soil’s consistency would be based on the field log and its strength on the results of subsequent laboratory tests.

4.5.6. Identification and classification of rock

Rock description according to EN ISO 14689-1 [iv] is based on the terms published by the International Society of Rock Mechanics [vi], in reference to those in BS 5930: 1999 [v].

Generally it is difficult for sheet piles or bearing piles to penetrate competent rock, as the driving forces cause too much damage to the piles. However, bearing piles are most efficient where it is possible for the piles to found in rock. It is possible to achieve penetration into weak rock with the choice of high yield steels up to 460 MPa for sheet piles and also by strengthening the toe of steel bearing piles with an appropriate pile shoe detail.

Where rock is encountered, it is important that high quality continuous core samples are obtained to allow detailed description and laboratory testing.

For further information about pile installation into rock, see Chapter 11.

4.6. Types of borehole sample and methods of testing

4.6.1. Field testing

Eurocode 7 - Part 2 provides guidance on the applicability of in situ tests for deriving soil parameters, as summarized below.

The most common in situ test used in the UK is the standard penetration test (SPT), but greater consideration should be given to using other tests – particularly the cone penetration test and pressuremeter test.

Field test and Abbreviation		Parameter	Applicability (High, Medium, Low)		
			Rock	Coarse soil	Fine soil
Standard penetration test	**SPT**	Soil type	–	*M*	*H*
		Extension of layers	–	*M*	*M*
		Particle size	–	*M*	*H*
		Water content	–	*M*	*M*
		Density	–	*M*	*M*
		Shear strength	–	*M*	*L*
		Compressibility	–	*M*	*M*
		Chemical tests	–	*M*	*M*
Cone penetration test with or without pore pressure measurement	**CPT CPTU**	Soil/rock type	*L*	*M*	*M*
		Extension of layers	–	*H*	*H*
		Density	–	*M*	*M*
		Shear strength	–	*M*	*H*
		Compressibility	–	*H*	*M*
		Groundwater level	–	*M*	*H*
		Pore water pressure	–	*M*	*M*
		Chemical tests	–	*L*	*H*
Plate loading test	**PLT**	Shear strength	*M*	*H*	*H*
		Compressibility	–	*H*	*H*
Pressuremeter		Soil/rock type	*L*	*L*	*L*
		Extension of layers	*L*	*L*	*L*
		Shear strength	–	*H*	*H*
		Compressibility	–	*H*	*H*
		Permeability	–	–	*L*
		Pore water pressure	–	–	*L*
Dynamic probing, light, medium, heavy, super heavy		Soil type	–	*L*	*L*
		Extension of layers	–	*H*	*M*
		Density	–	*M*	–
		Shear strength	–	*M*	*L*
		Compressibility	–	*M*	*M*

Table 4.5. Applicability of in situ tests for deriving soil parameters.

4.6.2. Geophysical methods

There are a range of geophysical methods which may provide information on subsurface ground conditions. This is particularly relevant for providing information concerning ground water, rock levels or location of physical buried obstructions that would cause difficulty to pile driving. The methods are particularly useful in: examining a large volume of ground both laterally and with depth, identifying voids or hard spots, marked changes in ground properties, and position of water tables. However, geophysics is not a substitute for an intrusive investigation as it does not provide samples for description and testing, cannot reliably identify changes in strata, and does not provide in situ ground properties apart from small strain stiffness.

4.6.3. Laboratory testing

Eurocode 7 – Part 2 provides guidance on the applicability of laboratory tests for determining common geotechnical parameters for soils, as summarized in the Table 4.6.

The selection of appropriate laboratory tests is essential to ensure that parameters used in design are representative of the ground at the site.

Laboratory test	**Parameter**	**Applicability (F = full, P = partial)**		
		Coarse *(Gr/Sa)*	Fine *(Si)*	Fine *(Cl)*
Bulk density determination	Bulk density	*F*	*F*	*F*
Oedometer	Oedometer modulus	*P*	*F*	*F*
	Compression index	*P*	*F*	*F*
	Coeff. of consolidation	–	*F*	*F*
Particle size analysis	Permeability	*F*	–	–
Direct simple shear	Undrained shear strength	–	*F*	*F*
Ring shear	Residual shear strength	*F*	*F*	*F*
Translational shear box	Drained shear strength	*F*	*F*	*F*
	Residual shear strength	*P*	*P*	*P*
	Undrained shear strength	–	*P*	*P*
Strength index	Undrained shear strength	*F*	*F*	*F*
Triaxial	Drained shear strength	*F*	*F*	*F*
	Undrained shear strength	–	*F*	*F*
	Young's modulus	*F*	*F*	*F*
	Shear modulus	*F*	*F*	*F*
	Oedometer modulus	*P*	–	–
	Compression index	*P*	–	–
	Coeff. of consolidation	–	*F*	*F*

Table 4.6. Applicability of laboratory tests for determining common geotechnical parameters for soils.

4.6.4. Chemical analysis

Influence of corrosion on the durability of steel needs to be thoroughly identified in the GIR. Although the chemical analysis of soil and leachates may be provided in the GIR - expert interpretation for the durability of steel may be required for the design of embedded steel piles in sites polluted by industrial waste. Also in this instance, so that the correct decisions for selection of appropriate sealants for watertightness performance and protective coatings can be taken by the designer for the durability required.

4.7. Geotechnical parameters

The following sub-sections discuss the selection of suitable parameters for use in the geotechnical design of embedded retaining walls and bearing piles.

Section 4.7.3 - 4.7.5 provide typical parameters for a variety of soil and rock types that may be used to guide preliminary design and to ensure that parameters obtained from ground investigations are within expected limits. These “typical” parameters provide a basis for initial decision making and are not a substitute for properly designed and analysed ground investigations.

4.7.1. Derived values

The derived value of a geotechnical parameter is defined in Eurocode 7 - Part 1 as the “value ... obtained by theory, correlation or empiricism from test results”. Test results may be converted into derived values by use of correlations (such as that between cone penetration resistance and angle of shearing resistance in sand), theoretical considerations (such as conversion of triaxial compression into plane strain strengths for clays), or through empirical rules (such as those between standard penetration test blow count and undrained strength for clays). The annexes to Eurocode 7 - Part 2 provide a number of suitable correlations to determine geotechnical parameters from in situ tests.

When available, test results may be supplemented by other relevant data, such as that from nearby sites (i.e. comparable experience) or from research studies of the materials encountered.

4.7.2. Characteristic values

The characteristic value of a material property is defined in the head Eurocode, EN 1990 [vii], as “[where a low value is unfavourable] the 5% fractile value; [where a high value is unfavourable] the 95% fractile value”.

Because of the inherent difficulties in selecting characteristic geotechnical parameters on the basis of EN 1990’s statistical definition, Eurocode 7 - Part 1 redefines the characteristic value as “a cautious estimate of the value affecting the occurrence of the limit state”. A cautious estimate is an approximate calculation or judgement that is careful to avoid problems or dangers.

Prior to the publication of Eurocode 7, the design of retaining walls in the UK was based on “representative” soil parameters, defined in BS 8002: 1994 [viii] as "conservative estimates... of the properties of the soil as it exists in situ... properly applicable to the part of the design for which it is intended".

In practice, the difference between BS 8002's representative value and Eurocode 7's cautious estimate is merely one of semantics. Similarly there is no practical difference between "moderately conservative", as defined in CIRIA C580 [ix], and a "cautious estimate". It is appropriate to select the characteristic value as a cautious estimate of the average value of the material strength (or other relevant material property) that governs the occurrence of the limit state.

It is recommended that the required information for sheet pile wall design should now be in accordance with the Eurocodes and Piling Handbook 9th edition.

4.7.3. Typical parameters for coarse soils

Table 4.7. gives typical characteristic conservative drained parameters for coarse (i.e. cohesionless) soils, for use in retaining wall design when site specific data is not available. The values are adapted from the EAU 2004 [xix] and other references. These recommended values may be used in case no further information is available. Otherwise the local values from the site investigation shall prevail.

Soil	Classification	Weight density γ (kN/m^3)		Effective cohesion	Angle of shearing resistance
		Dry	Saturated	c' (kPa)	φ'_{cv} (°)
Gravel	Loose	16	20	-	35
	Dense	19	22	-	37.5
Sand	Loose	16	20	-	30
	Dense	18	21	-	32.5
Fine sand	Loose	14	19	-	25
	Dense	18	21	-	27.5
Granular fill	Made ground	16	20	-	30 1)
	Gravel	19	21	-	35
	Sand	17	20	-	30
	Brick	16	19	-	30 1)
	Rock	19	22	-	35

Table 4.7. Typical drained parameters for coarse soils.

1) According to available material properties.

Eurocode 7 uses the term "constant volume angle of friction" and symbol φ'_{cv}, which are synonymous. In the Piling Handbook, the Eurocode 7 terms and symbol are used.

4.7.4. Typical parameters for fine soils

Table 4.8. gives typical characteristic drained parameters for fine (i.e. cohesive) soils, for use in retaining wall design when site specific data is not available. The values are adapted from EAU 2004 [xix].

Soil	Classification	Weight density γ (kN/m^3)		Effective cohesion	Undrained cohesion	Angle of shearing resistance
		Dry	Saturated	c' (kPa)	c_u(kPa)	φ'_{cv} (°)
Clay	Soft	16	18	0	20	20
	Firm	18	21	5	50	20
	Stiff/low pl.	19	22	7.5	100	20
	Stiff/interm. pl.	19	22	12	100	25
	Stiff/high pl.	19	22	15	80	22.5
Silt	Soft/low pl.	18	19	0	40	27.5
	Soft/interm. pl.	17	18	0	50	25
	Firm/low pl.	19	21	10	200	32.5
	Firm/interm. pl.	19	20	15	200	30
Organic	Clay	15	17	2	10	20
	Peat	11	14	1)	1)	1)
	Loam	17	20	7	100	22.5
	Rock	19	22	100	250	35

Table 4.8. Typical characteristic parameters for fine soils (drained and undrained).

1) The shear parameters of peat scatter in such a range that mean empirical values cannot be given.

Table 4.9. gives a correlation between plasticity index and the constant-volume angle of shearing resistance for fine soils, for use in retaining wall design.
This correlation is taken from [ix].

Plasticity index IP (%)	**Constant-volume angle of shearing resistance** ϕ_{cv} (°)
15	30
30	25
50	20
80	15

Table 4.9. Plasticity index for fine soils.

4.7.5. Typical parameters for rock

Table 4.10. gives typical characteristic drained parameters for soft rock masses, for use in retaining wall design when site specific data is not available. The values have been derived from values given in [ix], [x], and [xi].

Rock	**Classification**	**Weight density** γ (kN/m³)		**Effective cohesion**	**Angle of shearing resistance**		**Young's modulus**
		Dry	Saturated	c' (kPa)	φ'_p (°)	φ'_{cv} (°)	E' (MPa)
Chalk	Grade A	19	23	20	39	34	1500 - 3000
	Grade B	18	22	20	39	34	300 - 1500
	Grade C	18	22	20	39	34	120 - 900
	Grade Dm	14	19	0	31	30	6
	Grade Dc	14	19	0	33	30	75
Rock[1)]	Weak sandstone	19	22	0	42	36	50 - 1000
	Weak siltstone	19	22	0	35	30	100 - 1500
	Weak mudstone	19	22	0	28	25	20 - 500

Table 4.10. Typical characteristic drained parameters for soft rock masses.

[1)] The angles of friction for rocks represent the situation where the rock is broken into granular particles and has little relict fabric.

4.8. Information required for design of embedded sheet pile walls

Having determined the nature of the ground within the site from the Ground Investigation Report and ascertained the individual soil properties, it is desirable to release certain basic information to the piling designer to ensure the best possible arrangement in terms of strength and economy.

The minimum details should include the following:

- historical records covering the previous development of the site, particularly the location of old foundations and other buried structures;
- copies of relevant site drawings showing the projected retaining wall / site boundaries and proximity of waterways, buildings, roads and services;
- environmental restrictions – noise and vibration if relevant;
- surcharge and loadings temporary and permanent;
- serviceability limitations;
- durability or design life of structure;
- fire resistance requirements;
- sustainability issues for selection of materials;
- details of ground water levels, flooding and tidal range;
- clear brief on stage construction, design excavation levels or design bed or dredged levels and profile of submerged ground levels where relevant;

- design wave levels and berthing loads for Marine structures and information relevant to design to BS 6349-2 [xv], requirements for protection of steel or cathodic protection and maintenance preferences for instance;
- information pertaining to control of watertightness for sealed walls;
- requirements for impermeability performance or seepage for flood control.

4.9. Earth pressures calculation

4.9.1. Calculation of earth pressures

The pressure applied to a vertical wall, when the ground surfaces are horizontal can be calculated as follows

Active pressure $$p_a = \gamma \times z \times \tan^2(45 - \frac{\phi}{2}) - 2 \times c_u \times \tan(45 - \frac{\phi}{2})$$

Passive Pressure $$p_p = \gamma \times z \times \tan^2(45 + \frac{\phi}{2}) + 2 \times c_u \times \tan(45 + \frac{\phi}{2})$$

The terms $\tan^2(45 - \frac{\phi}{2})$ and $\tan^2(45 + \frac{\phi}{2})$

can be more conveniently referred to as K_a coefficient of active earth pressure and K_p coefficient of passive pressure respectively.

Hence $$p_a = K_a \gamma \times z - 2 \times c_u \times \sqrt{K_a}$$

and $$p_p = K_p \gamma \times z + 2 \times c_u \times \sqrt{K_p}$$

The above expressions however do not allow for the effects of friction and adhesion between the earth and the wall. They are based on extensions of the Rankine Equation (by the addition of cohesion) in [xvi].

Subsequent research has further developed these formulae to allow for the effects of wall friction, wall adhesion etc on the earth pressure coefficients. These are shown in paragraph 4.9.2.

Also the coefficients are subject to re-calculation for the effects of sloping ground as described in subsequent paragraph.

Note: The formulae in this chapter 4.9.1. represent the total stress condition. For effective stress the undrained shear strength parameter of the soil (c_u) is simply replaced by the effective cohesion value of the soil c'.

4.9.2. Limiting earth pressures from effective stress analysis

When limiting equilibrium conditions apply, the horizontal effective earth pressures that act on an embedded retaining wall under active and passive conditions (σ'_a and σ'_p), owing to the self-weight of the ground alone, are given by:

$$\sigma'_a = K_a(\sigma_v - u) - K_{ac}c'$$

$$\sigma'_p = K_p(\sigma_v - u) + K_{pc}c'$$

with

σ_v	vertical total stress (or overburden pressure) at depth z;
u	pore water pressure at the same depth (see below);
c'	soil's effective cohesion;
K_a, K_{ac}, K_p, K_{pc}	earth pressure coefficients.

The presence of a blanket surcharge q at ground surface increases the earth pressure acting on a vertical wall by an amount:

$$\Delta\sigma_a = K_a q$$

or

$$\Delta\sigma_p = K_p q$$

depending on whether the surcharge applies to the active or passive side of the wall.

From the expression given above, the horizontal total earth pressures under active and passive conditions (σ_a and σ_p) are derived as:

$$\sigma_a = K_a(\sigma_v - u + q) - K_{ac}c' + u$$

$$\sigma_p = K_p(\sigma_v - u + q) + K_{pc}c' + u$$

Values of the earth pressure coefficients K_a and K_p are summarized below (based on the method given in Annex C of Eurocode 7) [xvii].

φ (deg)	Active coefficient (K_a) for δ/φ =			Passive coefficient (K_p) for δ/φ =		
	0	1/2	2/3	0	-1/2	-2/3
0	1.000	1.000	1.000	1.000	1.000	1.000
15	0.589	0.544	0.534	1.698	1.939	1.995
20	0.490	0.445	0.434	2.040	2.477	2.582
25	0.406	0.363	0.353	2.464	3.222	3.413
30	0.333	0.294	0.285	3.000	4.288	4.633
35	0.271	0.237	0.229	3.690	5.879	6.510
40	0.217	0.189	0.182	4.599	8.378	9.573
45	0.172	0.149	0.143	5.828	12.567	14.954

Table 4.11. Limiting earth pressure coefficients for various angles of shearing resistance and wall friction.

Values of the earth pressure coefficients K_{ac} and K_{pc} can be obtained approximately from:

$$K_{ac} = 2\sqrt{K_a\left(1+\frac{a}{c'}\right)} \quad \text{and} \quad K_{pc} = 2\sqrt{K_p\left(1+\frac{a}{c'}\right)}$$

The UK National Annex to Eurocode 7 - Part 1 limits the values of K_{ac} and K_{pc} to:

$$K_{ac} \le 2.56\sqrt{K_a} \quad \text{and} \quad K_{pc} \le 2.56\sqrt{K_p}$$

Suitable values of δ/φ are discussed in Section 4.9.7.

4.9.3. Limiting earth pressures from total stress analysis

Total stress analysis of earth pressures acting on an embedded retaining wall involves the soil's undrained shear strength and neglects pore-water pressures. The analysis only applies to clay soils sheared at constant volume, i.e. in short-term design situations.

When limiting equilibrium conditions apply, the horizontal total earth pressures that act on an embedded retaining wall under active and passive conditions (σ_a and σ_p) are given by:

$$\sigma_a = (\sigma_v + q) - 2c_u\sqrt{1+\frac{a_u}{c_u}}$$

$$\sigma_p = (\sigma_v + q) + 2c_u\sqrt{1+\frac{a_u}{c_u}}$$

with

σ_v vertical total stress (or overburden pressure) at depth z;

q any blanket surcharge at ground surface;

c_u soil's undrained shear strength;

a_u any undrained adhesion between the wall and the ground.

Suitable values of a_u/c_u are discussed in Section 4.9.8.

4.9.4. Tension cracks

Determining the actual depth of tension cracks is complicated. Theoretically, tension can occur in cohesive soils under active conditions when the horizontal total earth pressure becomes negative ($\sigma_a < 0$), i.e. wherever:

$$\sigma_v + q < 2c_u$$

where the symbols are defined in section 4.9.3 above (and adhesion has been ignored). In uniform soil of weight density γ, this implies that a tension crack will form to a depth z_{tc} given by:

$$z_{tc} = \frac{2c_u - q}{\gamma}$$

as shown in the Fig. 4.1.

Because it would be unconservative to rely on tension acting between the ground and the wall, it is common practice to ignore the effects of tension within the depth of tension crack.

Traditional UK codes of practice, such as [xii] and [xiv], have recommended allowing for a "minimum equivalent fluid pressure" (MEFP) given by:

$$\sigma_{a,\min} = MEFP \approx \frac{\gamma_w z}{2}$$

where z is the depth below ground surface.

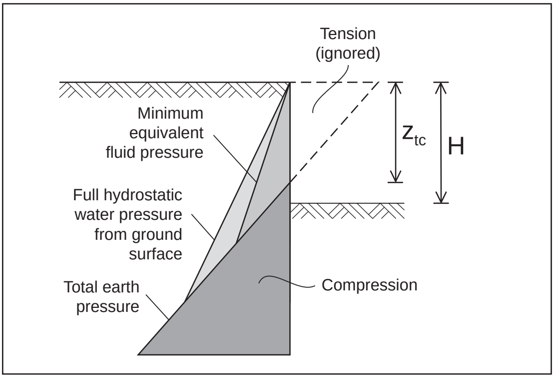


Fig. 4.1. Possible depth of tension cracks.

More recent UK guidance has suggested that, for cantilever walls and propped walls where water is able to enter the tension crack, the minimum active pressure on the wall $\sigma_{a,min}$ should be taken as full hydrostatic water pressure from ground surface, i.e.:

$$\sigma_{a,\min} = \gamma_w z$$

and, for propped walls where water cannot enter the tension crack:

$$\sigma_{a,\min} = MEFP \approx \frac{\gamma_w z}{2}$$

where z is the depth below ground surface and γ_w is the weight density of water.

The introduction of full hydrostatic water pressure into the theoretical tension crack results in very conservative designs, particularly when the soil's undrained strength is large (and hence z_{tc} is large). Less onerous (and perhaps more reasonable) designs would result from curtailing the minimum effective earth pressure at the bottom of the tension crack (i.e. at depth z_{tc}).

4.9.5. At-rest earth pressures

When at-rest conditions apply, the horizontal effective earth pressure (σ'_{h0}) that acts on an embedded retaining wall at a particular depth z, owing to the self-weight of the ground alone, is given by:

$$\sigma'_{h0} = K_0 \left(\sigma_v - u\right)$$

where σ_v is the total vertical stress (or overburden pressure) at the same depth, u is the pore water pressure at that depth, and K_0 is the at-rest earth pressure coefficient.

The presence of a uniform or blanket surcharge q at ground surface increases the horizontal earth pressure that acts on the wall by an amount:

$$\Delta\sigma_{h0} = K_0 q$$

From the expression given above, the at-rest horizontal total earth pressure σ_{h0} (including the effects of any surcharge) is then derived as:

$$\sigma_{h0} = K_0 \left(\sigma_v - u + q\right) + u$$

The at-rest earth pressure coefficient can be determined from the soil's angle of effective shearing resistance φ and its over-consolidation ratio (OCR, defined as the ratio of the maximum past overburden pressure to the current overburden pressure in the ground):

$$K_0 = \left(1 - \sin\varphi\right)\sqrt{OCR}$$

Values of the earth pressure coefficient K_0 are summarized below for different OCRs.

φ (deg)	Active coefficient (K_o) for OCR=					
	1	2	3	4	5	10
15	0.74	1.05	1.28	1.48	1.66	2.34
20	0.66	0.93	1.14	1.32	1.47	2.08
25	0.58	0.82	1.00	1.15	1.29	1.83
30	0.50	0.71	0.87	1.00	1.12	1.58
35	0.43	0.60	0.74	0.85	0.95	1.35
40	0.36	0.51	0.62	0.71	0.80	1.13
45	0.29	0.41	0.51	0.59	0.65	0.93

Table 4.12. At-rest earth pressure coefficients for various angles of shearing resistance and over-consolidation ratio.

4.9.6. Intermediate earth pressures

In some situations it is necessary to consider earth pressures that are developed due to compaction effects or repeated/cyclic loading. Complex soil structure interaction results, as for each load application further movements occur which may lead to an increase in earth pressures on the retained side of the wall. The horizontal effective earth pressure under active conditions (σ'_a) is then given by:

$$K_a\left(\sigma'_v + q\right) - K_{ac}c' \le \sigma'_a \le K_0\left(\sigma'_v + q\right)$$

with

σ'_v	vertical effective stress behind the wall;
q	magnitude of any blanket surcharge at ground surface;
c'	soil's effective cohesion;
K_a and K_0	limiting active and at-rest earth pressure coefficients defined in Sections 4.9.1 and 4.9.5, respectively.

A common approximation in this case is to assume:

$$\sigma'_a \approx \left(\frac{K_a + K_0}{2}\right) \times \left(\sigma'_v + q\right)$$

Approximate solutions are available to take into account compaction pressures [xiv]. More sophisticated analysis may be accomplished using numerical methods, which are outside the scope of the Piling Handbook.

4.9.7. Friction between the ground and wall

Design values of the angle of friction at the ground/wall interface δ_d should be limited to

$$\delta_d \leq k\varphi'_{cv,d}$$

where $\varphi'_{cv,d}$ is the soil's design constant-volume angle of effective shearing resistance and the constant $k = 2/3$ is recommended in Eurocode 7 - Part 1 [xviii] for sheet pile walls.

The value of interface friction is critical in the design of walls since it affects the shape of the potential failure mechanism, for both active and passive pressures - reducing the active earth pressure coefficient and increasing the passive coefficient. For a cautious approach designers may opt to select a lower value for the interface friction angle, δ. Note the resulting coefficients have different signs for sheet pile walls, active wall friction is conventionally positive and passive wall friction is negative for calculation purposes. Therefore the limiting value is $+2/3\ \varphi'_{cv,d}$ for active pressures and $-2/3\ \varphi'_{cv,d}$ for passive pressures.

Lower values of interface friction δ may be relevant for walls that are subject to significant vertical load. The importance of this factor is discussed in Chapter 6 when considering sheet piles in bearing.

Further, lower values of δ may be appropriate where jetting or pre-boring is necessary to ensure piles reach the required depth of penetration. Such techniques are often required in association with pile press systems and careful assessment is required to judge any necessary reduction in δ. For instance wall friction may be reduced when jetting or augering treatment of the ground is expected without dynamic methods involved in installation. If vibrodrivers and impact hammers are used to complete the driving then higher values for wall friction may apply.

It should be noted that restrictions on δ do not apply when assessing the shaft resistance of bearing piles (see Chapter 6).

The Piling Handbook 9th edition recommends the following approach for the designer:

- for normal sheet pile walls where the value of $\varphi'_{cv,d}$ is known:
  $\delta_a = +\ 2/3\ \varphi'_{cv,d}$ and $\delta_p = -\ 2/3\ \varphi'_{cv,d}$;
- where there is not sufficient information to establish the value of $\varphi'_{cv,d}$ or methods of installation are not certain then the designer may choose to adopt:
  $\delta_a = +\ 0.5\ \varphi'_d$ and $\delta_p = -\ 0.5\ \varphi'_d$;
- for sheet pile walls that may be subject to adverse soil movement such as settlement or for short anchor walls installed in fill then it is recommended:
  $\delta_a = \delta_p = 0$.

4.9.8. Adhesion between the ground and wall

Values of undrained wall adhesion a_u at the ground/wall interface are normally limited to:

$$a_u \leq \frac{c_u}{2}$$

Lower values of wall adhesion, however, may be relevant for walls installed in (soft) low strength clays.

The earth pressure coefficients K_{ac} and K_{pc} are then found from the following equation:

$$K_{ac} = K_{pc} = 2 \times \sqrt{(1+\frac{a_u}{c_{ud}})} \leq 2.56$$

a_u / c_u	$K_{ac} = K_{pc} = 2 \times \sqrt{(1+\frac{a_u}{c_{ud}})}$
0.00	2.00
0.25	2.24
0.50	2.45

Table 4.13. Values of the earth pressure coefficient K_{ac} , K_{pc} for different values of undrained wall adhesion.

Values of undrained wall adhesion are needed when determining limiting horizontal earth pressures in a total stress analysis.

4.9.9. Sloping ground surface

The design soil parameters should always be assumed to be based on horizontal ground profiles. However both active and passive parameters should be adjusted or re-calculated for sloping ground surfaces either behind or in front of the wall.

For the excavated or passive side it is normal to assume a worst credible horizontal level for design and ignore beneficial effects of soil above the design level. Design of walls with protected berms on the excavated side is treated differently.

For the active or retained side there are different possible approaches to adjust the design parameters.

Guidance for the procedure and calculation of coefficients for earth pressures for sloping ground with adjustment for wall friction are given in Eurocode 7 - Part 1 - Annex C [xviii].

4.9.10. Battered walls

The effect of batters up to 5° may be neglected.

Batters over 5° for sheet pile walls are unusual and may present difficulties with installation and directional changes so are not recommended.

4.9.11. Concentrated and linear surcharge

It is common in the UK to design embedded retaining walls to withstand a minimum 10 kPa uniform distributed load surcharge acting behind the wall, however, the following methods are recommended for assessing the additional horizontal earth pressures that bear on a sheet pile wall owing to the presence of a selection of surcharges with finite dimensions, as illustrated in the Fig. 4.2.

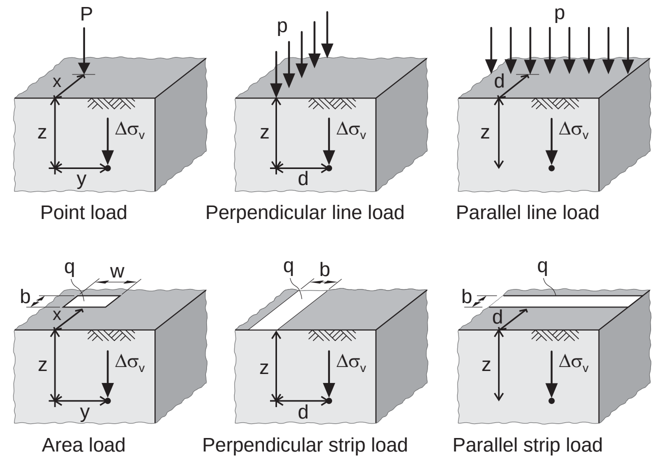


Fig. 4.2. Surcharge definition.

The increase in horizontal earth pressure $\Delta\sigma_h$ at the depth z below the point of application of the surcharge may be estimated from the equation:

$$\Delta\sigma_h = K_a \Delta\sigma_v$$

where $\Delta\sigma_v$ is the increase in vertical stress at the face of the wall at the same depth, owing to the presence of the surcharge; and K_a is the active effective earth pressure coefficient relevant for soil at that depth.

4.9.12. Point loads

The value of $\Delta\sigma_v$ beneath a point load may be estimated from elasticity theory, using [xiv]:

$$\Delta\sigma_v = \frac{3Pz^3}{2\pi\left(x^2 + y^2 + z^2\right)^{5/2}}$$

with

P magnitude of the point load;

x, y, z dimensions defined in the diagram above.

Example:

A point load P = 110 kN is applied at a distance x = 2 m back from the face of a sheet pile wall retaining H = 5 m of soil. The increase in vertical stress at formation level (at $z = H$) adjacent to the sheet pile is greatest directly in line with the point load (at y = 0 m), where it is calculated to be:

$$\Delta\sigma_v = \frac{3Pz^3}{2\pi\left(x^2+y^2+z^2\right)^{\frac{5}{2}}} = \frac{3\times110\times5^3}{2\pi\left(2^2+0^2+5^2\right)^{\frac{5}{2}}} = 1.4\,\text{kPa}$$

4.9.13. Line loads

The value of $\Delta\sigma_v$ beneath a line load may be estimated from elasticity theory, using [xiv]:

$$\Delta\sigma_v = \frac{2pz^3}{\pi\left(d^2+z^2\right)^2}$$

with

p magnitude of the line load;

d, z dimensions defined in the diagram above,

for loads both perpendicular and parallel to the wall.

Example:

A line load p = 55 kN/m is applied parallel to the sheet pile wall from the previous example, at a distance d = 2 m back from the face of the wall. The increase in vertical stress at formation level (z = 5 m) is calculated to be:

$$\Delta\sigma_v = \frac{2pz^3}{\pi\left(d^2+z^2\right)^2} = \frac{2\times55\times5^3}{\pi\left(2^2+5^2\right)^2} = 5.2\,\text{kPa}$$

4.9.14. Strip loads

The value of $\Delta\sigma_v$ beneath a strip load may be estimated from elasticity theory, using [xiv] applied to flexible walls:

$$\Delta\sigma_v = \frac{q}{\pi}\left(\alpha + \sin\alpha\cos\left[\alpha + 2\vartheta\right]\right)$$

$$\vartheta = \operatorname{atan}\left(\frac{d}{z}\right)$$

$$\alpha = \operatorname{atan}\left(\frac{d+b}{z}\right) - \vartheta$$

with

q magnitude of the strip load;

d, b, z defined in Fig. 4.2. for loads both perpendicular and parallel to the wall.

Note: α and ϑ should be entered into these equations in radians.

Example:

A strip load of width $b = 1$ m and magnitude $q = 55$ kPa is applied parallel to the sheet pile wall from the previous example, at a distance $d = 2$ m back from the wall face. The increase in vertical stress at formation level ($z = 5$ m) is calculated to be:

$$\vartheta = \operatorname{atan}\left(\frac{d}{z}\right) = \operatorname{atan}\left(\frac{2}{5}\right) = 21.8^\circ = 0.381\,\text{rad}$$

$$\alpha = \operatorname{atan}\left(\frac{d+b}{z}\right) - \vartheta = \operatorname{atan}\left(\frac{2+1}{5}\right) - \vartheta = 9.2^\circ = 0.16\,\text{rad}$$

$$\Delta\sigma_v = \frac{q}{\pi}\left(\alpha + \sin\alpha\cos\left[\alpha + 2\vartheta\right]\right)$$

$$= \frac{55}{\pi}\left(0.16 + \sin 9.2^\circ \cos\left[9.2^\circ + 2\times 21.8^\circ\right]\right) = 4.5\text{kPa}$$

4.9.15. Area loads

The value of $\Delta\sigma_v$ beneath an area load may be estimated from elasticity theory, by applying the principle of superposition, using [xiv]:

$$\Delta\sigma_v = f(x+b, y+w) + f(x,y) - f(x, y+w) - f(x+b, y)$$

$$f(m,n) = \frac{q}{2\pi}\left(\Omega + \lambda\left[\frac{1}{z^2+m^2} + \frac{1}{z^2+n^2}\right]\right)$$

$$\Omega = \operatorname{atan}\left(\frac{m\times n}{z\times\sqrt{z^2+m^2+n^2}}\right)$$

$$\lambda = \frac{z\times m\times n}{\sqrt{z^2+m^2+n^2}}$$

where q is the magnitude of the area load and the dimension z is defined the diagram above.

Note: Ω should be entered into these equations in radians.

Example:

An area load of breadth $b = 1.25$ m, width $w = 0.8$ m, and magnitude $q = 110$ kPa is applied to the sheet pile wall from the previous example, at a distance $x = 2$ m back from the wall face. The increase in vertical stress at formation level (z = 5 m) is greatest directly in line with the area load (at y = 0 m), where it is calculated to be:

$$\Delta\sigma_v = f\left(2+1.25, 0+0.8\right)+f\left(2, 0.8\right)$$
$$-f\left(2, 0+0.8\right)-f\left(2+1.25, 0\right)$$
$$\Delta\sigma_v = 4.0+0-2.9-0 = 1.1\text{kPa}$$

These are treated in a similar manner to superimposed loads except that allowance should be made for dissipation of the load at increasing depth.

There are various methods of allowing for this dissipation and the following is suggested by Krey when designing for cohesionless soils.

The maximum increase in horizontal total stress $\Delta\sigma_h$ is given by:

$$\sigma_{h,max} = \frac{4q \times \tan^2(45° - \varphi'/2)}{2 + (1 + \tan^2(45 - \varphi'/2))\ \ x/z}$$

with

q magnitude of surcharge

a $= x \times \tan\varphi'$

c $= x / \tan(45° - \varphi'/2)$

d $= z / \tan(45° - \varphi'/2)$

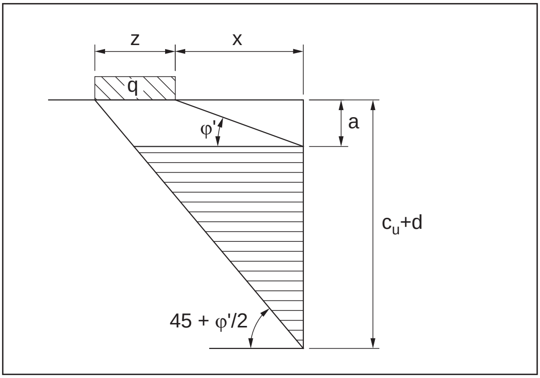


Fig. 4.3. Force distribution area lords.

4.10. Earth pressure calculation

In this section a set of design earth pressures for use in Serviceability Limit State checks have been produced for persistent and transient design situations. Earth pressures for the persistent situation are based on effective stresses and for the transient situation on mixed (total and effective) stresses.

The purpose of this example is to show how the pressure diagram is constructed from the calculation of overburden pressures. Please note that all partial factors are unity for the Serviceability Limit State calculation and are omitted for clarity. An assessment of the stress in the soil at any change of circumstance, i.e. stratum boundary, water level, formation/excavation level etc is carried out at for both sides of the wall.

Persistent SLS design situation (based on effective stress analysis):

Earth and water pressures: persistent SLS design situation
Density and strength of soils

Layer		Depth (m)	Characteristic values					
			γ_{unsat} (kN/m³)	γ_{sat} (kN/m³)	φ'_{peak} (deg)	φ'_{cv} (deg)	c' (kPa)	c_u (kPa)
Ia	Made ground	0.0 - 1.2	14.7	-	30	30	0	-
Ib		1.2 - 2.4	-	19.1				
II	Low strength clay	2.4 - 6.1	-	17.2	20	20	0	25
III	Sand and gravel	6.1 - 11.0	-	20.6	40	35	0	-
IV	Medium strength clay	11.0 - 16.5	-	18.6	25	20	2	65

Table 4.14. SLS design situation.

Earth and water pressures: persistent SLS design situation
Effective stress analysis - vertical stresses

Depth (m)	Layer	Vertical total stress σ_v (kPa)	Pore pressure u (kPa)	Vertical effective stress σ'_v (kPa)
Active side				
0.0	I	= surcharge = 10.0	= 0.0	10.0 – 0.0 = 10.0
-1.2		+ (14.7 x 1.2) = 27.6	= 0.0	27.6 – 0.0 = 27.6
-2.4		+ (19.1 x 1.2) = 50.6	+ (9.81 x 1.2) = 11.8	50.6 – 11.8 = 38.8
-2.4	II		as above	
-6.1		+ (17.2 x 3.7) = 114.2	+ (9.81 x 3.7) = 48.1	114.2 – 48.1 = 66.1
-6.1	III		as above	
-11.0		+ (20.6 x 4.9) = 215.1	+ (9.81 x 4.9) = 96.1	215.1 – 96.1 = 119.0
-11.0	IV		as above	
-15.0[1)]		+ (18.6 x 4.0) = 289.5	+ (9.81 x 4.0) = 135.3	289.5 – 135.3 = 154.2
Passive side				
-4.9	Water	= 0.0	= 0.0	0.0 – 0.0 = 0.0
-6.6		+ (9.81 x 1.7) = 16.7	+ (9.81 x 1.7) = 16.7	16.7 – 16.7 = 0.0
-6.6	III		as above	
-11.0		+ (20.6 x 4.4) = 107.3	+ (9.81 x 4.4) = 59.8	107.3 – 59.8 = 47.5
-11.0	IV		as above	
-15.0[1)]		+ (18.6 x 4.0) = 181.7	+ (9.81 x 4.0) = 99.0	181.7 – 99.0 = 82.7

Table 4.15. SLS design situation, vertical stresses.

[1)] *Toe of wall.*

Earth and water pressures: persistent SLS design situation
Earth pressure coefficients for effective stress analysis

Layer	Active conditions				Passive conditions			
	δ/φ_{pk}	K_a	a/c	K_{ac}	δ/φ_{pk}	K_p	a/c	K_{pc}
I	0[1)]	0.33	–	–	Above formation			
II	0[1)]	0.49	–	–				
III	0[1)]	0.22	–	–	0[1)]	4.60	–	–
IV	0.5[2)]	0.36	0.5	1.48[3)]	0.5[2)]	3.22	0.5	4.40[3)]

Table 4.16. SLS design situation, earth pressure coefficients.

– *Ignored because c' = 0*
1) *Set to zero because of pre-augering to -11 m*
2) *Calculated from $\delta/\varphi_{cv} = 2/3$ with $\varphi_{cv} = 20° \Rightarrow \delta = 13.3°$, hence with $\varphi_{peak} = 25° \Rightarrow \delta/\varphi_{pk} \approx 0.5$*
3) *Calculated from $K_{ac} = 2\sqrt{K_a\left(1+a/c\right)}$ and $K_{pc} = 2\sqrt{K_p\left(1+a/c\right)}$*

Earth and water pressures: persistent SLS design situation
Effective stress analysis - horizontal stresses

Depth (m)	Layer	Horizontal effective stress σ_h (kPa)	Pore pressure u (kPa)	Horizontal total stress σ'_h (kPa)
Active side				
0.0	I	0.33 x 10.0 = 3.3	0.0	3.3 + 0.0 = 3.3
-1.2	I	0.33 x 27.6 = 9.1	0.0	9.1 + 0.0 = 9.1
-2.4	I	0.33 x 38.8 = 12.8	11.8	12.8 + 11.8 = 24.6
-2.4	II	0.49 x 38.8 = 19.0	11.8	19.0 + 11.8 = 30.8
-6.1	II	0.49 x 66.1 = 32.4	48.1	32.4 + 48.1 = 80.5
-6.1	III	0.22 x 66.1 = 14.6	48.1	14.6 + 48.1 = 62.6
-11.0	III	0.22 x 119.0 = 26.2	96.1	26.2 + 96.1 = 122.3
-11.0	IV	0.36 x 119.0 – 1.48 x 2 = 39.9	96.1	39.9 + 96.1 = 136.0
-15.0	IV	0.36 x 154.2 – 1.48 x 2 = 52.6	135.3	52.6 + 135.3 = 187.9
Passive side				
-4.9	Water	0.0	0.0	0.0 + 0.0 = 0.0
-6.6	Water	0.0	16.7	0.0 + 16.7 = 16.7
-6.6	III	4.60 x 0.0 = 0.0	16.7	0.0 + 16.7 = 16.7
-11.0	III	4.60 x 47.5 = 218.5	59.8	218.5 + 59.8 = 278.3
-11.0	IV	3.22 x 47.5 + 4.40 x 2 = 161.7	59.8	161.7 + 59.8 = 221.5
-15.0	IV	3.22 x 82.7 + 4.40 x 2 = 275.0	99.0	275.0 + 99.0 = 374.0

Table 4.17. SLS design situation, horizontal stresses.

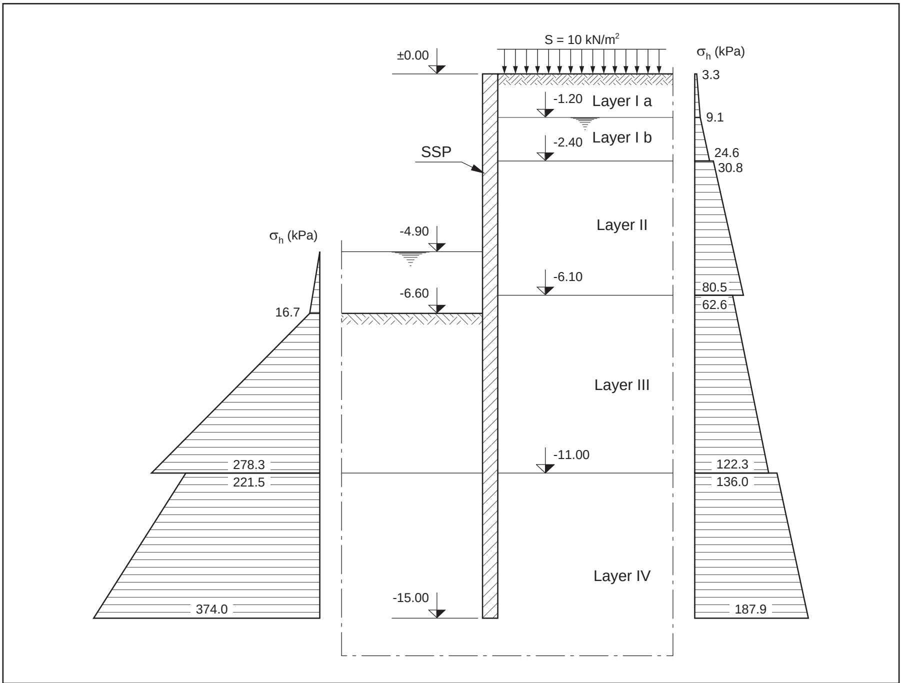


Fig. 4.4. Resulting pressure diagram.

Transient SLS design situation (based on mixed total and effective stress analysis):

Earth and water pressures: transient design situation
Earth pressure coefficients for mixed total and effective stress analysis

Layer	Active conditions				Passive conditions			
	δ/φ_{pk}	K_a	a/c	K_{ac}	δ/φ_{pk}	K_p	a/c	K_{pc}
I	0[1)]	0.33	–	–	Above formation			
II	[2)]	1.00	0[1)]	2.0[3)]				
III	0[1)]	0.22	–	–	0[1)]	4.60	–	–
IV	[2)]	1.00	0.5	2.45[3)]	[2)]	1.00	0.5	2.45[3)]

Table 4.18. Transient design situation, earth pressure coefficients.

– *Ignored because c' = 0*

1) *Set to zero because of pre-augering to -11 m*

2) *Ignored in total stress analysis*

3) *Calculated from* $K_{ac} = 2\sqrt{K_a\left(1+a/c\right)}$ *and* $K_{pc} = 2\sqrt{K_p\left(1+a/c\right)}$

Earth and water pressures: transient design situation
Mixed total and effective stress analysis - horizontal stresses

Depth (m)	Layer	Horizontal effective stress σ_h (kPa)	Pore pressure u (kPa)	Horizontal total stress σ'_h (kPa)
Active side				
0.0	I	0.33 x 10.0 = 3.3	0.0	3.3 + 0.0 = 3.3
-1.2		0.33 x 27.6 = 9.1	0.0	9.1 + 0.0 = 9.1
-2.4		0.33 x 38.8 = 12.8	11.8	12.8 + 11.8 = 24.6
-2.4	II	→	→	1.0 x 50.6 - 2.0 x 25 = 0.6
-6.1		→	→	1.0 x 114.2 - 2.0 x 25 = 64.2
-6.1	III	0.22 x 66.1 = 14.6	48.1	14.6 + 48.1 = 62.6
-11.0		0.22 x 119.0 = 26.2	96.1	26.2 + 96.1 = 122.3
-11.0	IV	→	→	1.0 x 215.1 - 2.45 x 65 = 55.9
-15.0		→	→	1.0 x 289.5 - 2.45 x 65 = 130.3
Passive side				
-4.9	Water	0.0	0.0	0.0 + 0.0 = 0.0
-6.6		0.0	16.7	0.0 + 16.7 = 16.7
-6.6	III	4.60 x 0.0 = 0.0	16.7	0.0 + 16.7 = 16.7
-11.0		4.60 x 47.5 = 218.5	59.8	218.5 + 59.8 = 278.3
-11.0	IV	→	→	1.0 x 107.3 + 2.45 x 65 = 266.6
-15.0		→	→	1.0 x 181.7 + 2.45 x 65 = 341.0

Table 4.19. Transient design situation, horizontal stresses.

References:
[i] EN 1997, Eurocode 7 - Geotechnical design, Part 2: Ground investigation and testing, CEN, Brussels, Belgium.
[ii] UK National Annex to BS EN 1997-2: 2007, British Standards Institution, London.
[iii] EN ISO 14688. Geotechnical investigation and testing - Identification and classification of soil. International Organization for Standardization, Switzerland.
[iv] EN ISO 14689. Geotechnical investigation and testing - Identification and classification of rock. International Organization for Standardization, Switzerland.
[v] BS 5930: 1999 + A2: 2010. Code of practice for site investigations. (Status: current, partially replaced) British Standards Institution, UK
[vi] International Society of Rock Mechanics.
[vii] EN 1990. Basis of structural design. CEN, Brussels, Belgium.
[viii] BS 8002: 1994. Code of practice for earth retaining structures. (Status: superseded, withdrawn). British Standards Institution, UK; status: currently under revision.
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[xviii] EN 1997, Eurocode 7 - Geotechnical design, Part 1: General rules. 2014.
[xix] EAU 2004, Recommendations of the Committee for Waterfront Structures, Harbour and Waterway, Berlin, 2004.

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5 | Design of steel sheet pile structures

Chapter 5 - Design of steel sheet pile structures

Contents

5.1. Introduction

The scope of this chapter is to cover the design of embedded sheet pile retaining walls and other types of structures that may use steel piling products. Design recommendations are based on Eurocode – particularly for steel sheet pile walls, namely, EC 7 [iv] and EC 3 - Part 5 [vi].

A sheet pile retaining wall has a significant portion of its structure embedded in the soil and a very complex soil/structure interaction exists as the soil not only loads the upper parts of the wall but also provides support to the embedded portion.

Current design methods for retaining walls do not provide a rigorous theoretical analysis due to the complexity of the problem. The methods that have been developed to overcome this, with the exception of finite element modelling techniques, introduce empirical or empirically based factors that enable an acceptable solution to the problem to be found. As a result, no theoretically correct solution can be achieved and a large number of different methods to solve this problem have been devised. Soil Structure Interaction (SSI) and Limit State equilibrium methods (LEM) involve different procedures for analysis and are the most widely used in conjunction with appropriate software at the time of writing.

The design of a retaining structure to Eurocode will involve different sets of calculations, one to determine the geometry of the structure to achieve equilibrium under the design conditions, the other to determine the structural requirements of the wall to resist bending moments and shear forces. Also Eurocode requires the structure itself to satisfy safety and stability criteria for various limit states.

For steel sheet pile walls durability and driveability in the ground conditions are an important feature of the design. EN 1990 [i] requires structures to be designed to sustain all likely actions and influences likely to occur during their execution and use and to remain fit for use. Structures must have adequate:

- structural resistance;
- serviceability;
- durability.

These basic requirements should be met by:

- choice of suitable materials;
- appropriate design and detailing;
- specifying control procedures for design, production, execution and use of the structure.

Designers should not overlook the possibility of global failure resulting from deep-seated slip failure of the soil and ensure that the proposed pile toe passes through the critical slip plane. Similarly, anchor walls should be located outside potential slip planes. Cellular and double wall gravity structures may require more complex geotechnical checks on internal soil failure mechanisms.

5.2. General design considerations

An earth retaining structure must be designed to perform adequately under two particular sets of conditions, those that can be regarded as the worst that could conceivably occur during the life of the structure and those that can be expected under normal service conditions. These design cases represent the ultimate and serviceability limit states respectively for the structure.

Ultimate limit states to be taken into account in design include instability of the structure as a whole including the soil mass, failure of the structure by bending or shear and excessive deformation of the wall or soil to the extent that adjacent structures or services are affected.

Where the mode of failure of the structure involves translation or rotation, as would be expected in the case of a retaining wall, the stable equilibrium of the wall relies on the mobilisation of shear stresses within the soil. Full mobilisation of soil shear strength results in limiting active and passive conditions and these can only act simultaneously on the structure at the point of collapse, the ultimate limit state.

Design for serviceability involves a consideration of the deformation of the structure and movement of the ground to ensure that acceptable limits are not exceeded.

The designer of a retaining wall must assess the design situations to which the wall could be subjected during its lifetime and apply these to the structure to analyse their effect.

The design situations should include the following where appropriate:

5.2.1. Applied loads and any combinations of loadings

Includes surcharges and externally applied loads on each side of the wall. The surcharge load acting on a wall will depend on its location and intended usage but unless specifically quantified, it is recommended that a surcharge is allowed for on the retained side of the wall to allow for the presence of plant or materials during construction.

Where very high levels of surcharge or concentrated loads occur, e.g. ports and harbours, it is often more economical to carry them on bearing piles which transfer them to a lower stratum where no lateral pressure is exerted on the retaining structure (see 5.8.4).

5.2.2. Geometry of the problem

The basic retained height to be used in calculations will be the difference in level between the highest anticipated ground level on the active side of the wall and the lowest level on the passive. An allowance for uncertainties of the ground surface level, including unexpected or unplanned excavation, during the life of the structure in front of the wall of 10% of the retained height of a cantilever or 10% of the distance below the lowest support in a supported wall up to a maximum of 0.5 m should be included in the ultimate limit state calculations (see 5.18). It should be noted that if excavation for pipes or cables in the passive zone is likely then the trench depth is considered to be part of the basic design excavation depth and should not be treated as unplanned excavation. Scour should also be considered in the design excavation depth, not in the allowance.

The additional allowance for uncertain excavation depth does not apply to serviceability calculations.

5.2.3. Material characteristics

In permanent structures, the long-term performance of steel must be considered and a heavier pile section may be needed to take into account installation and durability requirements.

5.2.4. Groundwater

Variations in ground water levels, due to dewatering, flooding or failure of drainage systems need to be taken into account in design. Consider the effects of providing weep-holes in the webs of sheet piles and drainage media to prevent the accumulation of ground water behind the wall; however these must be designed for maintenance or to prevent clogging by any fines transported in the flowing water. Details for the design of filter weep-holes for sheet piled structures are shown in the Recommendations of the Committee for Waterfront Structures Harbours and Waterways 2012 [xix].

5.2.5. Environment and installation

Impact driving of sheet piles into dense soils or, for instance, when using pitch and drive methods for silent vibrationless pressing it may necessitate the provision of a section larger than that needed to satisfy the structural requirements. Driveability should be considered at an early stage in the design process as the need to provide a minimum section for driving may lead to a more efficient support system and may also offset any additional thickness needed to achieve the desired life expectancy for the structure. Ground pre-treatment by pre-augering may also be considered as an option for improvement of driveability of lighter sections and satisfy environmental considerations by minimising noise and vibration.

5.3. Design philosophy

The design philosophy adopted in the Piling Handbook 9th Edition is that provided by the Structural Eurocodes. It is based on limit state principles, in which a distinction is made between ultimate and serviceability limit states.

Ultimate limit states are concerned with the safety of people and the structure. Examples of ultimate limit states include loss of equilibrium, excessive deformation, rupture, loss of stability, transformation of the structure into a mechanism, and fatigue.

Serviceability limit states are concerned with the functioning of the structure under normal use, the comfort of people, and the appearance of the construction works.

Limit state design involves verifying that relevant limit states are not exceeded in any specified design situation. Verifications are performed using structural and load models, the details of which are established from three basic variables: actions (i.e. loads), material properties, and geometrical data. Actions are classified according to their duration and combined in different proportions for each design situation.

The Structural Eurocodes that are relevant to the design of sheet pile walls and steel bearing piles are:

- EN 1990, Eurocode – Basis of structural design [i];
- EN 1991, Eurocode 1 – Actions on structures [ii];
- EN 1993, Eurocode 3 – Design of steel structures [iii];
- EN 1997, Eurocode 7 – Geotechnical design [iv].

Eurocode is based on Limit State Design concepts and it is recommended the reader has a basic knowledge of this and of soil mechanics to fully understand the design procedures in this chapter. However worked examples are provided in Piling Handbook 9th Edition for guidance purposes only.

The relevant concepts of the Structural Eurocodes are discussed but the reader is referred to both the Eurocodes themselves and other supporting guidance for a more detailed understanding.

5.3.1. Principles of limit state design

The design calculations prepared to demonstrate the ability of a retaining wall to perform adequately under the design conditions must be carried out with full knowledge of the purpose to which the structure is to be put. In all cases, it is essential to design for the collapse condition or Ultimate Limit State (ULS) and to assess the performance of the wall under normal operating conditions, the Serviceability Limit State (SLS).

Limit state design involves verifying that neither ultimate nor serviceability limit states are exceeded. Verification of either of these two categories of limit states may be omitted if sufficient information is available to prove it is satisfied by the other.

Ultimate limit states are concerned with the safety of people and the structure so that failure will not occur. Serviceability limit states are concerned with functioning and appearance of the structure. For sheet pile walls the concerns may focus on deflection and settlement issues. When a wall is dependent upon its support system for stability and where it is foreseen that accidental loading could cause damage or loss of part or all of that support system, the designer should be able to demonstrate that progressive collapse of the structure will not occur. An example of this is the effect that loss of a tie rod may have on a wall design.

For the designer limit states should be verified by calculation or load tests, an observational method, or a combination of these methods. For the design of sheet pile walls calculation is likely to be the most appropriate method. However, where the control of deformations is critical a combination of calculation and the observational method may be required.

Eurocode 7 - Part 1 [v] identifies five ultimate limit states for which different sets of partial factors are provided:

- failure or excessive deformation in the ground (GEO);
- internal failure or excessive deformation of the structure (STR);
- loss of static equilibrium (EQU);
- loss of equilibrium or excessive deformation due to uplift (UPL);
- hydraulic heave, piping, and erosion (HYD).

Eurocode 3 - Part 5 [vi] requires steel piles to be verified for the following ultimate limit states:

- failure in the ground (i.e. limit state GEO);
- structural failure (i.e. limit state STR);
- combination of failure in the ground and structure.

In addition, serviceability of the structure must be verified.

These various limit states are discussed in detail in the following sections of this chapter: limit state GEO in Section 5.4.1.; STR in 5.4.2. and 5.4.3.; UPL in 5.4.4.; HYD in 5.4.5.; EQU in 5.4.6.; and serviceability in 5.4.7.

5.3.2. Eurocode EC 7 - Design approach

During the development of Eurocode 7, it became clear that some countries (including the United Kingdom) wanted to adopt a load and material factor approach to the verification of strength, while others (e.g. Germany) preferred a load and resistance factor approach. To accommodate these differing wishes, a compromise was reached whereby each country could choose – through its National Annex – one (or more) of three Design Approaches that should be used within its jurisdiction.

In Design Approach 1, two separate calculations are required (Combinations 1 and 2), one with factors applied solely to actions and the other with factors applied mainly to material properties. This is the approach adopted in the UK through its National Annex.

In Design Approach 2, one calculation is required, with factors applied to effects of actions and resistances simultaneously.

In Design Approach 3, one calculation is required, with factors applied to geotechnical actions and material properties simultaneously.

The Piling Handbook 9th Edition covers Design Approach 1 only and does not cover Design Approaches 2 or 3. For details of these alternative approaches, the reader should refer to Eurocode 7 - Part 1 [v] itself or to other publications that cover these approaches.

5.4. The Design Limit States

5.4.1. Limit state GEO

For embedded sheet pile walls this entails the basic procedure for analysing the stability of the wall or structure.

Eurocode 7 - Part 1 [v] defines limit state GEO as "failure or excessive deformation of the ground, in which the strength of soil or rock is significant in providing resistance".

Verification of limit state GEO involves checking that design effects of actions do not exceed their corresponding design resistances. This is expressed by the inequality:

$$E_d \leq R_d$$

with

E_d design effects of actions;

R_d the corresponding design resistance.

To verify limit state GEO, it is only necessary to ensure that the inequality above is satisfied. Factoring of the values used to calculate E_d and R_d provides the necessary reliability (safety).

5.4.2. Limit state STR

For steel sheet pile walls this procedure entails checking the capacity of the designed structural sections to EC 3 – Part 5 design rules.

Eurocode 7 - Part 1 [v] defines limit state STR as 'internal failure or excessive deformation of the structure or structural elements … in which the strength of structural materials is significant in providing resistance'.

Verification of limit state STR involves checking that design effects of actions do not exceed their corresponding design resistances. This is expressed by the inequality:

$$E_d \leq R_d$$

with

E_d design effects of actions;

R_d the corresponding design resistance.

For steel sheet pile retaining walls and bearing piles, verification against structural failure involves guarding against:

- failure by bending and/or axial force;
- failure due to overall flexural bending (allowing for ground restraint);
- local failure where loads are applied (e.g. web crippling and buckling).

5.4.3. Limit state STR - Plastic design of steel sections

Eurocode 3 [iii] allows steel sections to be designed using plastic models. However, the resistance and rotation capacity of cross-sections is limited by its local buckling resistance.

Eurocode 3 identifies four classes of cross-section, as follows:

1. those that can form a plastic hinge with the rotation capacity required from plastic analysis without reduction of the resistance;
2. those that can develop their plastic bending resistance, but have limited rotation capacity because of local buckling;
3. those in which the stress in the extreme compression fibre of the steel member assuming an elastic distribution of stresses can reach the yield strength, but local buckling is liable to prevent development of the plastic bending resistance;
4. those in which local buckling will occur before the attainment of yield stress in one or more parts of the cross-section.

Fig. 5.1. illustrates the difference in bending moment capacity for the four classes at cross section.

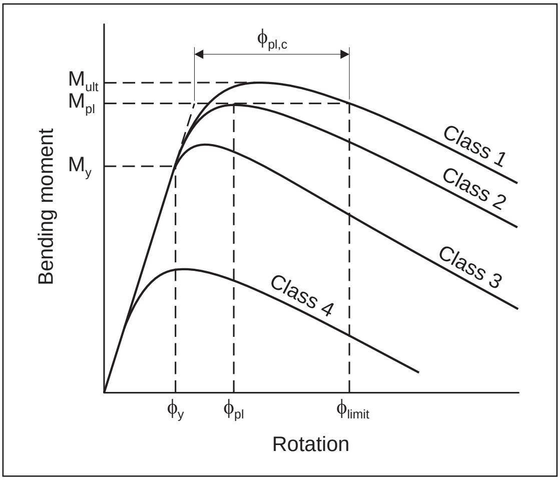

Fig. 5.1. Rotation - bending moment capacity diagram for the four cross-section classes [vii].

The Piling Handbook limits itself to the design of steel piles in Classes 2 and 3 only. For the design of sections in Classes 1 and 4, the reader should refer to Eurocode 3 - Parts 1-1 [ix] for buildings, 3 for bridges [x], and 5 [vi] for piling.

For Plastic Design of sheet pile walls it is necessary to prove sections are Class 1 by verification of their rotational capacity and take account of corrosion effects in the life of the structure. For sheet pile walls Class 1 verification may only be relevant in a short term design situation. It is usually more practical and may lead to a more economical solution to select a Class 2 section and take advantage of the use of plastic section properties of the sheet pile in an elastic analysis if the section remains Class 2 during the design life.

Note it is particularly important in Eurocode design to verify Class 4 sections for structural capacity where combinations of shear, axial loading bending and buckling effects arise taking into account reduction in section properties due to corrosion. For sheet pile design it is recommended that when choosing a pile the section is heavy enough, not only for driving and installation, but also remains Class 3 after the required corrosion loss. Identifying the position of the critical section for Class 4 sections in permanent sheet pile walls is not straight forward.

The system used by Eurocode 3 to classify sections takes account of the grade of steel used by way of the coefficient ε, defined as:

$$\varepsilon = \sqrt{\frac{235}{f_y}}$$

with

f_y yield strength in MPa (N/mm^2)

The table below gives values of ε for usual steel grades to EN 10248 or ArcelorMittal's internal mill specification for hot-rolled sheet piles.

Steel grade	Yield strength, f_y (N/mm^2)	ε
S 270 GP	270	0.93
S 320 GP	320	0.86
S 355 GP	355	0.81
S 390 GP	390	0.78
S 430 GP	430	0.74
S 460 AP[1)]	460	0.71

Table 5.1. Value of the coefficient ε for some common steel grades.

1) ArcelorMittal mill specification.

The rules for calculation of the section class are covered in 8.2.

5.4.4. Limit state UPL

This limit state should be checked when heave of the base of an excavation or the stability of a concrete plug is a potential failure mechanism.

Eurocode 7 - Part 1 [v] defines limit state UPL as "loss of equilibrium of the structure or ground due to uplift by water pressure or other vertical actions".

The details of this Limit State check procedure is outside the scope of the Piling Handbook 9th Edition.

5.4.5. Limit state HYD

Eurocode 7 - Part 1 [v] defines the limit state HYD as "hydraulic heave, internal erosion or piping in the ground caused by hydraulic gradients".

Piping is a particular form of limit state HYD and occurs when the pressure on the soil grains due to the upward flow of water is so large that the effective stress in the soil approaches zero. In this situation the soil has no shear strength and assumes a condition that can be considered as a quicksand, which will not support any vertical load. This is obviously a very dangerous situation for personnel operating in the excavation and will also lead to a significant reduction in passive resistance afforded to the embedded wall by the soil. In extreme cases this can lead to a complete loss of stability and failure of the embedded wall. The likelihood of piping for a given cross section should be assessed.

This limit state is of particular importance to be considered for the design of temporary cofferdams and retaining walls with significant water pressure and is covered in more detail in Chapter 9.

5.4.6. Limit state EQU

Eurocode 7 - Part 1 [v] defines the limit state EQU as "loss of equilibrium of the structure or the ground as a rigid body where the strength of the ground or the materials is insignificant".

Verification of limit state EQU involves checking that destabilizing effects of actions do not exceed the corresponding stabilizing effects, plus any resistance that enhances those stabilizing effects.

This Limit State usually applies for checking the stability of gravity structures e.g circular straight web cells or double walled structures. Also sliding failure is normally checked for anchored structures.

Checking of limit state EQU does not prevent from additional GEO verifications which is usually critical for internal sliding failure of cellular structures and anchored walls (straight or logarithmic spiral failure line).

5.4.7. Serviceability Limit States

The head Eurocode [i] defines serviceability limit states as “states that correspond to conditions beyond which specified service requirements for a structure or structural member are no longer met”.

Verification of serviceability involves checking that design effects of actions (e.g. settlements) do not exceed their corresponding design limiting values (i.e. limiting settlements). This is expressed by the inequality:

$$E_d \leq C_d$$

with

E_d design effects of actions;

C_d the limiting design value of the relevant serviceability criterion.

The following serviceability limit state design situations should be checked for all geotechnical structures:

- excessive settlement;
- excessive heave;
- unacceptable vibrations.

Eurocode 3 - Part 5 [vi] requires the following serviceability limit state (SLS) criteria to be taken into account:

- vertical and horizontal displacement limits, to suit the supported or directly connected structure (and, for retaining walls, adjacent structures);
- vibration limits, to suit structures directly connected to, or adjacent to, the foundation;
- deformations limits, to suit the retaining wall itself (for retaining walls).

The global analysis of sheet pile retaining walls and pile foundations under SLS conditions should be based on a linear elastic model of the structure. It should be demonstrated that no plastic deformations occur under serviceability loads.

All partial factors for serviceability limit states are 1.0.

The loadings considered should be those that the designer considers may apply under normal operational circumstances. Extreme or accidental events should be excluded. The ground surfaces (or unplanned excavation) rule is also excluded for serviceability checks.

This Limit State may be critical to a design when magnitude of deflections or settlement of the ground or structure may be an issue.

5.5. Partial factors

5.5.1. Partial factors – GEO

To safeguard against failure in the Ultimate Limit State Eurocode design rules apply Partial Factors on loads, materials and actions. The values of the partial factors may vary for different limit states and may be referred to in "sets" that apply to a Limit State for the relevant design and verification procedure.

The partial factors specified in the UK National Annex to Eurocode 7 - Part 1 [v] for the design of embedded retaining walls to Design Approach 1 are summarized in the table below.

Parameter			Partial factor	Combination 1	Combination 2
Actions	**Permanent**	Unfavourable	γ_G	1.35	1.00
		Favourable	$\gamma_{G,fav}$	1.00	1.00
	Variable	Unfavourable	γ_Q	1.50	1.30
		Favourable[1)]	-	0	0
Material properties	**Effective shearing resistance**		γ_φ	1.00	1.25
	Effective cohesion		γ_c	1.00	1.25
	Undrained shear strength		γ_{cu}	1.00	1.40
	Unconfined compressive strength		γ_{qu}	1.00	1.40
	Weight density		γ_γ	1.00	1.00
Earth resistance			γ_{Re}	1.00	1.00

Table 5.2. Partial factors for ultimate limit state GEO in persistent and transient design.

1) Favourable variable actions are ignored (hence factors are zero).
Partial factors for checking Serviceability Limit States are all 1.00.
Partial factors for GEO in accidental design situations are all 1.00.

5.5.2. Partial factors for steel materials – STR

The partial material factors specified for steel (sheet pile sections to EN 10248 and EN 10249) in Eurocode 3 - Part 1-1 [ix] for buildings and Part 2 [x] for bridges for ultimate limit states of structural failure are summarized in the table below.

Partial factor for resistance of		Buildings	Bridges
Cross-sections (whatever their class)	γ_{M0}	1.00	1.00
Members to instability assessed by member checks	γ_{M1}	1.00	1.10
Cross-sections in tension to fracture	γ_{M2}	1.25	1.25

Table 5.3. Partial material factors for steel in Ultimate Limit States. Structure failure in persistent and transient design situations.

Partial factors for steel for ultimate limit state STR in accidental design situations are all 1.0 All partial factors for steel materials are based on compliance with their respective materials standard documents for manufactured quality. For sheet piles hot-rolled EN 10248 - Parts 1 & 2 apply and for cold formed piles EN 10249 applies.

5.6. Earth pressure calculation - Limit State Design

Limit state design philosophy is now a generally accepted method used in the design of embedded retaining walls. The limit states to consider are the Ultimate Limit State (ULS) which represents the collapse of all or part of the retaining wall, and the Serviceability Limit State (SLS) which represents the state, short of collapse where the appearance or condition of the structure becomes unacceptable. The designer should also carry out a risk assessment in order to make a reasonable assessment of any accidental load cases, that may result in progressive failure of the wall. i.e. changes to the design, construction control procedures etc.

It is important that from the outset, the designer establishes the performance criteria of the wall, as this will assist in determining which limit state will govern the design.

It is generally recognised that the loading conditions under ULS are normally more severe than the SLS condition, however there are cases (for example in the design of urban basements) when SLS conditions (deflections, settlements etc) are just as critical as ensuring the structural integrity of the wall in the ULS condition.

The method of calculating design earth pressures using the limit state approach, involves reducing the soil strength parameters by an appropriate partial factor.

The factors used in this chapter correspond to those used in Design Approach 1 in accordance with EC 7 – Part 1 and the UK National Annex.

For soils and earth pressures the category of parameters applies to Materials for the design to the GEO and STR Limit States. The earth pressures are calculated from values derived from moderately conservative soil parameters called characteristic values as defined in EC 7.

There are two sets of parameters applicable to Design Approach 1 which needs 2 sets of calculations to be performed to establish the most onerous combination. These 2 cases are called Combination 1 and Combination 2. Combination 2 factors the soil strength but for combination 1 the applied loads are factored (differently) but not the soil strength which remains unfactored.

5.7. Establishing Design Values

5.7.1. Basic variables

The basic variables in a limit state design are:

- material properties, e.g. weight density, shear strength, modulus of elasticity, etc.;
- actions on the structure, e.g. permanent and variable loads, change in temperature, settlement, effective earth pressures, pore water pressures, etc.;
- geometrical dimensions, e.g. excavation level, groundwater levels, pile size, etc.

5.7.2. Material properties

5.7.2.1. Weight density of ground

Design values of the weight density of ground γ_d are obtained from characteristic values γ_k as follows:

$$\gamma_d = \frac{\gamma_k}{\gamma_\gamma}$$

where

γ_γ is a partial factor equal to 1.0.

Note that in this equation, γ_d and γ_k have units of kN/m^3 and γ_k is dimensionless.

If a range of weight densities have been measured, for example as part of a ground investigation for the site, then the value of γ_k should be selected as a cautious estimate of the operational weight density. Since the weight of retained ground produces a destabilizing action on the wall, an upper estimate of weight density is appropriate.

Example:

Layer	**Depth** z (m)	**Measured weight densities,** γ (kN/m^3)	**Characteristic weight density,** γ_k (kN/m^3)
A	0 - 2.5	18.2, 18.6	18.5
B	2.5 - 10	18.2, 19.5, 20.2, 21.3	20.3

Table 5.4. Weight densities - example.

5.7.2.2. Ground strength

Design values of ground strength are obtained from characteristic values by applying an appropriate partial factor:

$$c'_d = \frac{c'_k}{\gamma_c}, \; \varphi'_d = \operatorname{atan}\left(\frac{\tan\varphi'_k}{\gamma_\varphi}\right), \; c_{u,d} = \frac{c_{u,k}}{\gamma_{cu}}$$

where

c' soil's effective cohesion;

φ' soil's angle of effective shearing resistance;

c_u soil's undrained shear strength;

subscripts k and d denote characteristic and design values, respectively;

γ_c, γ_φ, and γ_{cu} are partial material factors specified in Eurocode 7 – Part 1 as well as its UK National Annex and are listed in Table 5.5.

Limit state			**Partial factor**		
			γ_c	γ_ϕ	γ_{cu}
Ultimate	**GEO/STR**	DA1-1 1)	1.00	1.00	1.00
		DA1-2 2)	1.25	1.25	1.40
	UPL	Unfavourable	1.25	1.25	1.40
	EQU		1.10	1.10	1.20
Serviceability			1.00	1.00	1.00

Table 5.5. Partial material factors from the UK NA to BS EN 1997-1.

1) Design approach 1, Combination 1;
2) Design approach 1, Combination 2

The following table allows design values of φ' and c_u to be determined from their characteristic values.

Angle of shearing resistance				**Undrained shear strength**			
ϕ_k	ϕ_d for γ_ϕ			$C_{u,k}$	$C_{u,d}$ for γ_{cu}		
	1.00	1.10	1.25		1.00	1.20	1.40
15	15	13.7	12.1	10	10	8.3	7.1
20	20	18.3	16.2	25	25	20.8	17.9
25	25	23.0	20.5	50	50	41.7	35.7
30	30	27.7	24.8	75	75	62.5	53.6
35	35	32.5	29.3	100	100	83.3	71.4
40	40	37.3	33.9	150	150	125.0	107.1
45	45	42.3	38.7	200	200	166.7	142.9

Table 5.6. Design values of shearing resistance and undrained shear strength for different partial factors.

5.8. Actions

The following sub-sections discuss permanent actions from stresses in the ground (see 5.8.1.), water pressures (5.8.2.) and also actions influenced by surcharge loads (5.8.3.).

The design value of an action F_d is given by:

$$F_d = \gamma_F F_{rep} = \gamma_F \psi F_k$$

where F_{rep} and F_k are the action's representative and characteristic values, respectively; γ_F is a partial factor; and ψ is a combination factor obtained from Eurocode 1 [ii].

For permanent actions (e.g. the self-weight of the structure and ground), the combination factor ψ equal 1.0 and the previous equation may be written:

$$G_d = \gamma_G G_{rep} = \gamma_G G_k$$

where the symbol G denotes a permanent action.

For variable actions (e.g. traffic, wind, and snow loading) in persistent and transient design situations, the combination factor ψ either equals 1.0 (for the leading variable action) or equals ψ_0 (for accompanying variable actions). The previous equation may be written:

$$Q_{d,1} = \gamma_Q Q_{rep,1} = \gamma_Q Q_{k,1}$$
$$Q_{d,i} = \gamma_Q Q_{rep,i} = \gamma_Q \psi_{0,i} Q_{k,i}$$

where

the symbol Q denotes a variable action;

the subscripts 1 and i denote leading and accompanying, respectively;

the value of ψ_0 is obtained from Eurocode 1.

Guidance for classification of actions and accidental loading and also combinations of actions with variable frequency for design situations is given in EN 1990 – 2002 and BS 6349 for marine structures.

Combinations of actions with complex representation including seismic actions values are outside the scope of the Piling Handbook.

The sets of partial factors for GEO/STR actions for retaining walls and piles are given in EC 7 Annex A and the National Annex (see Table 5.5.).

For Combination 1 it is normal to apply the Partial Factor 1.35 to the effect of the actions so in a typical analysis the variable unfavourable action is factored by 1.111 firstly (i.e 1.111 x 1.35 = 1.5) for the application of the partial factor on the effect of all the actions to be correct.

For Combination 2, the permanent action partial factors are 1.00, so it is not relevant to apply a factor to the effect of the actions. The partial factor on unfavorable, variable actions is 1.30 and applied to the characteristic value of the action directly. The complete set of partial factors for DA1 is given in Table 5.5.

5.8.1. Stresses in the ground

The total characteristic vertical stress $\sigma_{v,k}$ acting at a depth z below ground level owing to the self-weight of the ground alone is given by:

$$\sigma_{v,k} = \int_0^z \gamma_k dz$$

where γ_k is the soil's characteristic weight density. For layered ground, it is usual to calculate $\sigma_{v,k}$ from:

$$\sigma_{v,k} = \sum_i \gamma_{k,i} t_i$$

with

$\gamma_{k,i}$ characteristic weight density of the i-th layer (assumed uniform);

t_i i-th layer thickness.

Example:

Characteristic vertical stress in a layered soil.

Layer	Thickness t (m)	Depth z (m)	Weight density γ_k (kN/m^3)	Total characteristic vertical stress $\sigma_{v,k}$ (kPa)
A	2.5	0.0	18.5	0.0
		2.5		46.3
B	7.5	2.5	20.3	46.3
		10.0		198.5

Table 5.7. Characteristic vertical stress - example.

Note: For the calculation of vertical stress of soil for earth pressure on sheet pile retaining walls the weight of the soil is not factored.

The calculation of earth pressures for permanent actions are dependent on the selection of characteristic values of ground strength and the application of appropriate partial factors as explained in 5.7.2.2.

5.8.2. Water pressures

The consideration of water pressures and their effects is extremely significant in geotechnical design of embedded retaining walls. The characteristic values may not be certain and Eurocode interpretation of rules to ascertain design values may differ between the Design Approaches and National Annexes.

Water pressures should be considered as permanent actions in accordance with EC 7.

When hydrostatic groundwater conditions exist, the characteristic pore water pressure u_k acting at a depth z below ground level is given by:

$$u_k = \gamma_{w,k}\left(z - d_w\right)$$

with

$\gamma_{w,k}$ characteristic weight density of water (9.81 kN/m^3 or approx. 10 kN/m^3);

d_w depth of the water table below ground surface.

Example:

assuming water table at d_w = 5 m and $\gamma_{w,k}$ = 10 kN/m^3

Layer	Depth z (m)	Depth below water table $z - d_w$ (m)	Characteristic pore water pressure u_k (kPa)
A	0.0	-	0
	2.5	-	0
B	2.5	-	0
	5.0	0.0	0
	10.0	5.0	50

Table 5.8. Pore water pressure - example.

Eurocode 7 - Part 1 [v] requires that design water pressures are either derived from a cautious estimate of the characteristic water pressures by applying the appropriate partial factor or that a suitably conservative estimate of the water pressure together with an appropriate safety margin is used directly. It is recommended that water pressures are normally treated as permanent actions with appropriate partial factors as indicated in the figure below.

There is some debate whether it is more sensible to factor characteristic pore water pressures, given that the weight density of water is a relatively well known value. However, unsafe designs can result from treating characteristic water pressures as design values and EC 7 allows the effect of actions to have a partial factor applied.

The Piling Handbook 9th Edition recommends an approach that provides a balance between providing reliability and maintaining realism in the design is as follows. When partial factors $\gamma_G > 1.0$ are applied to effective earth pressures (e.g. in Design Approach 1, Combination 1), then pore water pressures should also be multiplied by $\gamma_G > 1.0$, but no safety margin should be applied to water levels. When partial factors $\gamma_G = 1.0$ are applied to effective earth pressures (e.g. in Design Approach 1, Combination 2), then pore water pressures should also be multiplied by $\gamma_G = 1.0$ after an appropriate safety margin Δh_w has been applied to water levels. By applying the margin Δh_w for Combination 2 then the water pressure is at the highest possible in the design life of the structure for the ULS case. For Combination 2 the highest possible ground water level could be at ground surface level (or above in the case of a flood wall) and this case should represent an event for a return period of not less than 50 years for a permanent structure.

The figure below illustrates the design water pressures that are obtained from this approach. The highest normal water level expected during the time-span of the design situation being verified is at a height hw above formation level. The highest possible water level during the same time-span is at height $h_w + \Delta h_w$ above formation.

For both combinations, the design water pressure
$u_d = \gamma_G \times u_k$ at depth $z \geq (H - h_w - \Delta h_w)$ is calculated as:

$u_d = \gamma_G \times \gamma_{(w,k)} \times (z-(H-h_w-\Delta h_w))$

where the values of γ_G and Δh_w are as summarized in the Table 5.9.

Combination	**Partial factor** γ_G	**Water level**	**Safety margin** Δh_w
1	1.35	Highest normal	0
2	1.00	Highest possible	>0

Table 5.9. Recommended treatment of ground water in Design Approach 1.

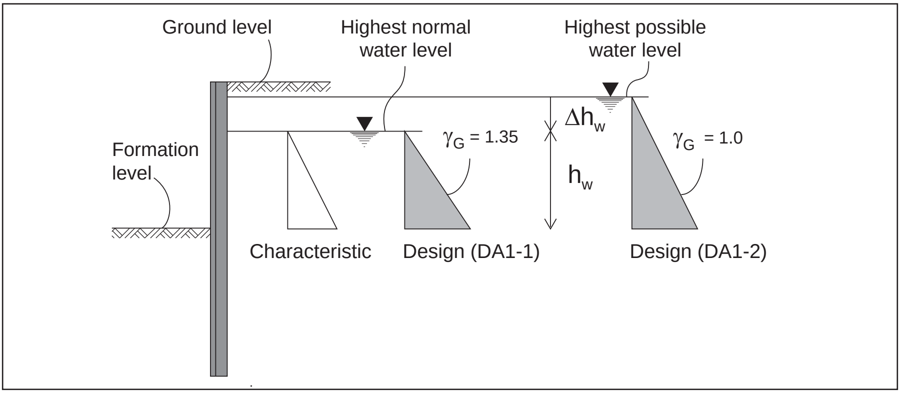

Fig. 5.2. ULS - Design water pressures

Thus for DA1 Combination 1 the effect of the permanent action of highest normal water pressure is treated by applying the partial factor 1.35.
Note: the variable unfavourable actions are firstly factored by 1.11 and the resultant variable action effect is factored by 1.35. This method is in accordance with 2.4.7.3.2(2) of EC 7 [v] and is described by Simpson et al 2011 [xxii] and referred to as DA1. Combination 2 of DA1 is unaffected and no further partial factor is applied to the effect of the actions. The partial factor on highest possible water pressure for DA1-2 , γ_G = 1.0.

5.8.2.1. Hydrostatatic pressure distribution and tidal variations

For marine projects the designer should refer to the recommendations in BS 6349 to assess the design hydrostatic pressure distribution. It should be noted that there is a difference between the design situation for no drainage behind the wall and the incorporation of a reliable drainage system with suitable fill behind the wall. Useful guidance for detailing reliable drainage systems can be found in EAU Recommendations for Waterfront Structures.

For variable water pressure distribution in the operating highest normal situation the differential is recommended to be factored in accordance with DA1-1 (permanent unfavourable action partial factor set A1) and for the extreme case highest possible is in accordance with DA1-2 where γ_G = 1.0 as per Table 5.2. above.

For double wall structures and embedded anchor walls a variable phreatic groundwater surface and unfavourable hydrostatic pressure on the back of the anchor wall requires consideration.

5.8.2.2. Differential head across the wall in drained conditions

Where there is an imbalance in water levels on either side of the wall water pressures may be derived from a flow net or other approximate methods.

A particularly useful simplification is to assume that any difference in head across a wall decreases linearly around the wall in drained conditions, as illustrated below, however other simplified assumptions may be considered.

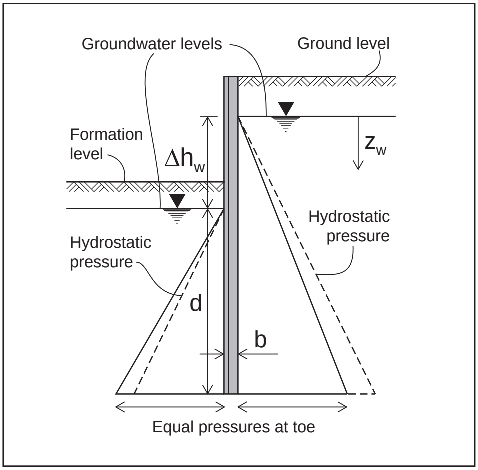

Fig. 5.3. Seepage around the toe.

For sheet pile walls the value of *b* is negligible.

For example, taking the bottom of the embedded sheet pile as datum, the total head h_{GWL} on the retained side of the wall at groundwater level is given by:

$$h_{GWL} = \Delta h_w + d$$

with

Δh_w difference in head between the water levels on the two sides of the wall;

d depth of the wall toe below the lower groundwater level.

The total head h at any depth z_w below the water level on the retained side of the wall is then given by:

$$h = h_{GWL} - \Delta h_w \left(\frac{z_w}{h_{GWL} + d + b} \right)$$

$$\approx \Delta h_w \left(1 - \frac{z_w}{\Delta h_w + 2d} \right) + d$$

At the toe of the wall, $z_w = \Delta h_w + d$ and the equation above reduces to:

$$h_{toe} = \frac{2d\left(\Delta h_w + d\right)}{\Delta h_w + 2d}$$

Alternatively water pressures can be calculated at varying depths using a flow net analysis method as described in Chapter 9 for design situations with cofferdams and for also checking HYD Limit state. Provided the hydraulic gradient is taken into account and the seepage is in steady state without destabilising the soils then the resulting increase in active pressure and decrease in passive pressure is taken into account.

In combined wall design situations it may be possible to equalise the water pressures at the toe of the intermediate piles in drained conditions in this way and make the adjustments in pressures at different levels for consideration in the geotechnical analysis.

However the steady state seepage phenomenon is usually not taken into account because the effect is also offset by an increase / decrease of the earth pressure leading to a more complex calculation. This may be analysed using FEM methods if strictly necessary for the design.

Also the designer should always consider driving the piles to a deeper less permeable strata rather than risk failure in the HYD limit state or underestimate the de-stabilising effect of piping failure mechanisms caused by excessive seepage beneath the toe of the piles.

5.8.3. Surcharge loads

Assessment of the relevant surcharge loading to be taken by the wall must consider the influence of nearby buildings, stockpiles, plant movements, etc. Particular attention should be given to repeated loading, e.g. from crane tracks behind quay walls, where the earth pressures induced against the wall may increase with each application of load.

It is common in the UK to design embedded retaining walls to withstand a minimum surcharge acting behind the wall. For example, a blanket surcharge of 10 kPa has traditionally been applied to walls retaining less than 3 m of soil [xii and xiii]. Highway structures have traditionally been designed for a blanket surcharge of 10-20 kPa, representing “HA” through to the heaviest “HB” loading [xiv]; and railways for a blanket surcharge of 30-50 kPa [xv]. However, Eurocode 7 - Part 1 [v] does not require a minimum surcharge to be assumed in design. Therefore earth pressures should be calculated in accordance with the methods described in Chapter 4 and surcharges applied where relevant with the appropriate partial factor for the action. Chapter 4.8.13. describes the methods to calculate various types of surcharge configurations.

5.8.4. Variable actions

Variable actions such as imposed loads on the wall or connecting structures from wind or waves or berthing forces should be taken into account with the appropriate partial factor (see Table 5.2.).

5.8.5. Other Actions

All other significant actions where relevant in a design situation need to be taken into consideration. These include effects due to extreme variations in temperature (rare in UK), collision forces (accidental or resulting from colliding mass) and seepage forces (see 5.8.2). Snow and ice loading, dynamic and seismic actions can be considered as accidental or variable but usually rare for retaining walls in the UK. These actions are outside the scope of the Piling Handbook where more complex representation is necessary for the analysis.

5.8.6. Combinations of actions

Particularly in the marine sector combinations of loading with different values and frequencies require to be assessed for assigning characteristic values and partial factors.

BS 6349 -2 2010 gives guidance and recommendations for combinations of operational surcharges for design of quay walls and harbour structures. Unfavourable action combinations and accidental loading actions are also needed to be taken into account for checking ULS cases. Annex (A) BS 6349-2:2010 gives recommended partial factors and combination factors.

5.8.7. Single source principle

Both favourable and unfavourable actions can be considered resulting from a single source. As such the same partial factor is applied to the sum of the actions or their effects. This EC 7 rule 24.2. (9) [v] is known as the single source principle. An example of this is the horizontal earth active and passive pressure components on a retaining wall.

5.9. Effects of actions

For sheet pile walls the effects of actions are calculated, for example, to determine the pile length, bending moments, shear forces and prop or anchor loads to verify the structural requirements for the wall and components.

For structures in Category 1 and 2 (ref Section 5.12.) a two dimensional analysis of pressure diagrams (see Chapter 4) are carried out for Combination 1 and 2 to check stability and at the point of equilibrium the shear forces and moments of forces are computed to realize the effects of actions. The structure is designed to resist the highest values of action effects in the Ultimate Limit State from the analysis of both combinations.

The effect of actions is expressed as a function of the actions, material properties and dimensions of the problem:

$$E_d = \gamma_E \, E\left\{F_d ; X_d ; a_d\right\}$$

with

E_d design effect of the actions;

γ_E partial factor on the effect of the action;

$E\{F_d, X_d, a_d\}$ effect of the action derived from the design actions, material properties and dimensions.

See Section 5.7.1.

5.10. Earth resistance

The resistance to the effects of design actions is a function of the actions, material properties and dimensions:

$$R_d = \frac{R\left\{F_d, X_d, a_d\right\}}{\gamma_R}$$

with

R_d design resistance;

γ_R partial factor on the resistance;

$R\{...\}$ resistance derived from the design actions F_d, design material properties X_d, and design dimensions a_d.

Earth resistance action effects are considered to determine resistance of prestressed anchorages in Design Approach 1 and also for resistance to vertical loading effects on piles in Design Approach 1 (see Chapter 6).

5.11. Deformations – Seviceability Limit State

Reliability is introduced into design against loss of serviceability by selecting suitable limiting values of displacement and then comparing expected deformations with these limiting values.

Partial factors for serviceability limit states are normally taken as 1.0. Hence the equation for verification of serviceability becomes:

$$E_d = E\left\{F_{rep}, X_k, a_{nom}\right\} \leq C_d$$

and no partial factors are introduced.

It is important to recognize that actions and material properties may vary during the structure's design life and hence serviceability limit states may need to be checked at various times. Thus it is appropriate to consider the deformations due to installation separately from those due to excavation in front of the wall and subsequently during the design life of the wall.

Of critical importance in verifying serviceability is appropriate selection of the limiting effects of actions. These must represent a realistic assessment of what is necessary for the long term performance of the structure, rather than overly conservative limits which simplify structural analysis.

The requirements for nearby structures are of particular relevance especially when they are sited within one to two pile lengths of the wall. Evidence from case histories presented in [xiii] suggests that both horizontal and vertical movements caused by wall installation are small beyond one-and-a-half times the pile length from the wall.

Movements caused by excavation in front of the wall depend on the level of lateral restraint. For high support stiffness (high propped wall, top-down construction), surface movements are of the order of 0.1-0.15% of the retained height; whereas, for low support stiffness (cantilever or low-stiffness temporary supports), they are of the order of 0.35-0.4% of the retained height. Movements are negligible at a distance of four times the retained height behind the wall.

The calculation of retaining wall deformations is complicated and there are no currently available tools that will do this reliably. Soil-spring and numerical models are useful in assessing redistribution of loads within a wall and propping system, but are not capable of reliably predicting deformations unless calibrated against measured performance of similar walls in similar ground conditions. This requirement is emphasised in Eurocode 7 - Part 1 [v] and in other supporting documentation [xiii].

For the assessment of deformation of sheet pile walls, substantial weight should be given to the evidence from case histories. For simple cantilever and singly propped walls, it is unlikely that a soil-spring model or numerical analysis will provide any greater accuracy in predicted deformations than those based on evidence from case histories. For these types of walls, limiting equilibrium analysis is generally adequate.

For multi-propped walls, limiting equilibrium methods are less suitable but may be used to provide an initial assessment of the required propping levels and bending moments in the piles to allow economic use of more sophisticated techniques.

Soil-spring and numerical methods are of particular use in refining designs where the assessment of the complex interaction between the retaining wall and nearby structures and/or infrastructure is essential to the economic execution of the project.

5.12. Types of structure - Geotechnical risk

In order to classify geotechnical risk, Eurocode 7 - Part 1[v] introduces three Geotechnical Categories, their design requirements, and the design procedure they imply. The Geotechnical Categories are defined in a series of Application Rules, not Principles, and hence alternative methods of assessing geotechnical risk could be used.

5.12.1. Geotechnical Category 1

Geotechnical Category 1 (GC1) includes small and relatively simple structures with negligible risk. The design requirements are negligible risk of instability or ground movements; ground conditions are “straightforward”; and there is no excavation below water table (or such excavation is “straightforward”). Routine design and construction (i.e. execution) methods may be used.

5.12.2. Geotechnical Category 2

Geotechnical Category 2 (GC2) includes conventional types of structure and foundation with no exceptional risk or difficult soil or loading conditions. Design requirements include quantitative geotechnical data and analysis to ensure fundamental requirements are satisfied. Routine field and laboratory testing and routine design and execution may be used.

Examples of structures in GC2 include:

- embedded sheet pile walls and other structures retaining or supporting soil or water;
- excavations;
- bridge piers and abutments;
- embankments and earthworks;
- ground anchors and other tie-back systems;
- breakwaters, quay walls and cellular structures.

Embedded sheet pile walls and steel bearing piles will typically form part of a GC2 structure and the guidance given in the Piling Handbook 9th Edition relates to the levels of investigation and procedures for analysis required for this geotechnical category.

5.12.3. Geotechnical Category 3

Geotechnical Category 3 (GC3) includes very large or unusual structures; structures or parts of structures not covered by GCs 1 or 2. Alternative provisions and rules to those in Eurocode 7 may be required to design these structures.

Examples of structures in GC3 include: structures involving abnormal risks or unusual or exceptionally difficult ground or loading conditions; structures in highly seismic areas; structures in areas of probable site instability or persistent ground movements that require separate investigation or special measures. Design and analysis of these types of structure may be outside the scope of the Piling Handbook for complete appraisal.

5.13. Types of embedded sheet pile walls

Embedded sheet pile retaining walls can be divided into cantilever or supported types. Cantilever walls are dependent solely upon penetration into the soil for their support and clearly fixity of the toe is required to achieve equilibrium of the forces acting on the structure. As fixity of the wall toe requires longer and, in many cases, heavier piles to achieve the necessary penetration into the soil, this type of wall can only be economic for relatively low retained heights. Variations in soil properties, retained height and water conditions along a wall can have significant effects on the alignment of a cantilever wall and care must be taken when designing them for permanent structures.

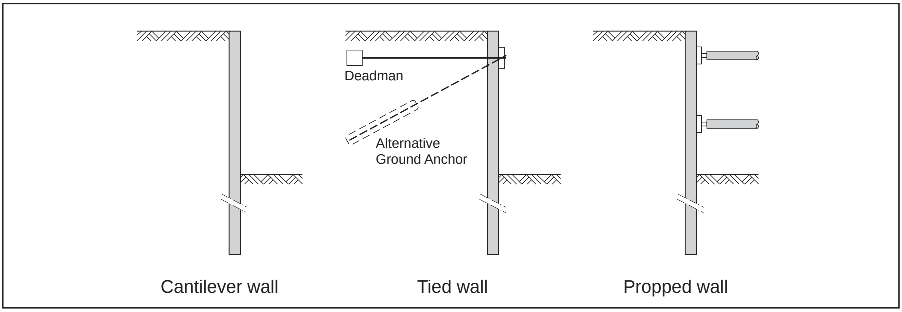

Fig. 5.4. Types of wall.

Supported walls, which can be either strutted or anchored, achieve stability by sharing the support to be provided between the soil and the supporting member or members. The provision of longitudinal walings transfers and distributes the soil loadings from the wall to ties or struts to the piles minimising variations in displacement along the structure.

The maximum retained height to which a cantilever wall can be considered to be effective will generally be governed by the acceptable deflection of the wall under load and the depth of penetration required for the pile to be driven with appropriate plant and equipment to achieve the minimum toe level. For standard sheet piles up to 5200 cm^3/m section modulus, 4 to 5 metres retained height may be achievable for cantilever design, however, U-piles will deflect more than Z piles. Propped walls supported by a single tie or prop will generally be cost effective up to a retained height in the order of 10 metres. Steel sheet pile walls with structural elements of greater section modulus than 5000 cm^3/may be required for highly loaded walls of significant retained height. These retaining walls are known as High Modulus Walls, which may be composed entirely of special HZ-M piles, steel tubes or box piles or a combination using either of these types as “primary elements”, with “secondary” infill piles, usually Z piles, and are known as Combi-walls.

Typically for deep cofferdams when more than one level of supports is used, wall stability becomes a function of the support stiffness and the conventional active/passive earth pressure distribution does not necessarily apply and these structures need to be subjected to more complex analysis.

5.14. Selection of design system

Modern computer software packages provide the engineer with the opportunity to carry out a simple Limit Equilibrium design (LEM), a more complex Soil Structure Interaction (SSI) calculation or a sophisticated Finite Element (FE) analysis. As the complexity of the analysis method rises, the amount and complexity of data also increases and the analysis method should therefore be selected to suit the sophistication of the structure and to ensure that any economies deriving from a more complex analysis can be realised.

When the structure is such that there will be little or no stress redistribution, as can be expected for a cantilever wall, limit equilibrium calculations and soil-structure interaction analyses are likely to give similar wall embedment depths and wall bending moments. For supported walls, where redistribution of stresses may be expected, a soil structure interaction analysis may provide a more economic design involving reduced bending moments and pile length but increased support loads compared to LEM analyses. LEM analyses may be used to calculate minimum pile length and SSI methods to calculate bending moment and prop loads. It is recommended to use more than one programme to compare results.

Advanced design systems such as FE Analysis are required for Category 3 structures. The Piling Handbook 9th Edition is mainly concerned with Category 1 and 2 retaining wall structures where LEM analyses and SSI methods are relevant for the designer to compute effects of actions for appropriate design situations.

When designing an earth retaining structure, the designer may choose to adopt either free or fixed earth conditions at the toe of the wall. The difference between these two conditions lies in the influence which the depth of embedment has on the deflected shape of the wall. For Ultimate Limit State design approach to Eurocode a Free Earth analysis is generally recommended for simply supported walls, for cantilever walls Fixed Earth conditions always apply.

According to EC 3 Part 5, 2.5.3.1. (1) [vi]–The analysis of the structure should be carried out using a suitable soil-structure model in accordance with EN 1997-1[v].

5.14.1. Soil-structure interaction or subgrade reaction analysis

Subgrade reaction theory idealizes the soil as a series of linear-elastic / perfectly-plastic springs. The forces on the wall and in any props or anchors supporting it are calculated from deformations along the wall. Iteration brings forces into equilibrium while keeping movements compatible with the elastic properties of the wall.

The springs' subgrade reaction coefficients k are estimated from field and laboratory measurements of soil stiffness (when available), otherwise from crude rules-of-thumb. The springs' load capacities are normally defined using limiting earth pressure coefficients (K_a for tension, K_p for compression).

To verify that an ultimate limit state is not exceeded, Eurocode 7 requires partial factors to be applied to actions, material properties, and resistances. Their values depend on which Design Approach is adopted. No partial factors are given in Eurocode 7 for stiffness and hence the design value of the springs' subgrade reaction coefficients should be identical to their characteristic values. However it is recommended that spring stiffnesses for ultimate limit state calculations are taken as 50% of their serviceability values (to account for the soil's greater compressibility at large strain). This can be achieved by dividing subgrade reaction coefficients k by a model factor $\gamma_{Rd} = 2.0$.

The application of partial factors to soil strength changes the values of the active and passive earth pressure coefficients used to define the ultimate resistance of the soil springs. As a consequence, the interaction between the ground and the structure will differ from that under serviceability loads, particularly if some of the springs reach their load capacity prematurely. The displacements obtained from ultimate limit state calculations using subgrade reaction models should be ignored, since they do not represent the true behaviour of the structure.

It is not straightforward to apply partial safety factors to actions or resistance when subgrade models are employed. The logic necessary to determine whether a particular component of earth pressure should be treated as a favourable action, an unfavourable action, or a resistance is extremely complicated – even if there was a universally agreed interpretation of the Eurocode. If part of the ground starts to unload, would that signal a switch from one interpretation of earth pressure to another? Unless the computer program has been specially written to include the relevant factors at the appropriate points in the calculation, then the only way to achieve their intended effect is to adjust input parameters instead. This could be attempted by increasing weight densities by 1.35. to simulate the application of γ_G. However, this is generally not a wise thing to do, since it may lead to unintended side-effects in other parts of the calculation.

Table 5.10. below summarizes one possible way of using a subgrade reaction model to verify embedded retaining walls for ultimate limit states, according to Eurocode 7 (UK approach).

Step		Partial factor	Combination 1	Combination 2
1	Multiply variable actions by ratio γ_Q/γ_G	γ_G	1.35	1.00
		γ_Q	1.50	1.30
		γ_Q / γ_G	1.11	1.30
2	Apply partial factors to soil strengths	$\gamma_\varphi = \gamma_c$	1.00	1.25
		$\gamma_{cu} = \gamma_{qu}$	1.00	1.40
		γ_γ	1.00	1.00
3	Perform soil structure interaction analysis			
4	Check ratio of restoring to overturning moment	γ_G	1.35	1.00
		γ_{Re}	1.00	1.00
		$\gamma_G \times \gamma_{Re}$	1.35	1.00
5	Apply partial factors to action effects	γ_G	1.35	1.00
Values shown on shaded background are used in the analysis				

Table 5.10. Steps for using a subgrade reaction model with Eurocode 7.

First, variable actions are “pre-factored” by the ratio $\gamma_Q/\gamma_G > 1$ so that subsequent parts of the calculation can treat them as permanent actions.

Second, soil strengths are factored down by $\gamma_M \geq 1$. The resulting design values of surcharge and material properties are entered into the computer program and the soil structure interaction analysis is performed (Step 3).

For cantilever and single-propped walls, toe embedment is then verified (in Step 4), by checking that the ratio of the restoring moment about the point of fixity M_R to the overturning moment M_O about the same point is at least equal to the product of γ_G (the partial factor on unfavourable actions) and γ_{Re} (the partial factor on passive resistance). If the wall passes this check, then design bending moments and shear forces in the wall (and design forces in any props or anchors) may be obtained from the calculated action effects by multiplying by γ_G.

5.14.2. FE Modelling

FE Modelling (Numerical Method) is more sophisticated than SSI modelling, and usually required for analysis of Category 3 Structures and Design situations. It can also be used for typical Category 2 structures but this is considered to be outside the scope of the Piling Handbook to detail. It is particularly useful for modelling complex double wall structures and multi-prop walls to analyse deformation and deflection criteria. There are different ways to set up the input to derive Eurocode design requirements in accordance with Design Approach 1.

5.15. Design situations

Note: The following design situations are represented by diagrams with simplified pressure diagrams applicable for typical LEM analysis.

5.15.1. Cantilever walls

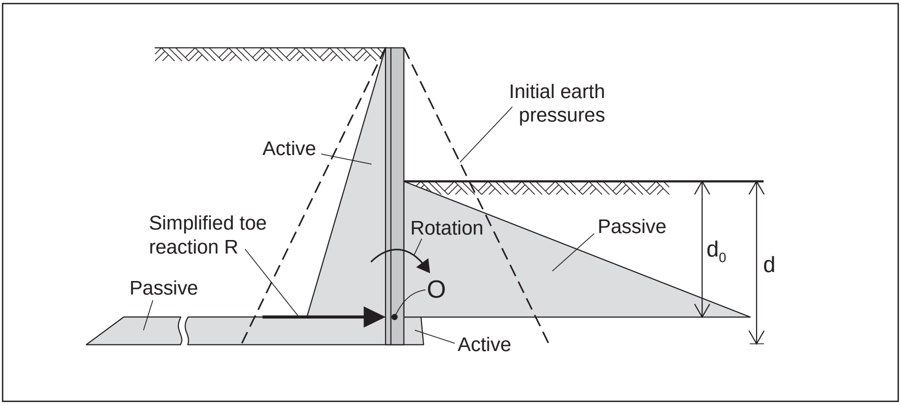

Fig. 5.5. LEM design model for cantilever walls.

The assumption of fixed earth conditions is fundamental to the design of a cantilever wall where all the support is provided by fixity in the soil. Increased embedment at the foot of the wall prevents both translation and rotation and fixity is assumed.

The stability of an embedded cantilever wall can be verified by assuming “fixed-earth” conditions, as illustrated below. The wall, which is assumed to rotate about the fixed point “O”, relies on the support of the ground to maintain horizontal and moment equilibrium.

Above the point of fixity, ground on the retained (left-hand) side of the wall goes into an active state and that on the restraining (right-hand) side into a passive state. The earth pressures bearing on the wall decrease from their initial, at-rest (K_0) values to active (K_a) values on the left and increase towards fully passive (K_p) on the right.

Below the point of fixity, ground on the retained side goes into a passive state and that on the restraining side into an active state. The earth pressures below *O* therefore increase towards fully passive values on the left and decrease to active values on the right.

The situation shown in the figure is often simplified by replacing the earth pressures below *O* with an equivalent reaction *R*. The depth of embedment (d_O) required to ensure moment equilibrium about the point of fixity is then increased by 20% to compensate for this assumption, i.e. $d = 1.2\ d_O$, but in certain cases, it is worth utilizing a more accurate formula to optimize the “overlength” of the sheet pile (see EAU 2012) [xxiii] .

For calculations of effects of actions to Section 5.8.1. the minimum pile length is calculated by taking moments of the active and passive pressures to establish point “0” at the point of equilibrium and applying 20% additional embedment depth below “0”. Maximum bending moments (positive and negative) are calculated at the points of zero shear.

5.15.2. Free earth support walls

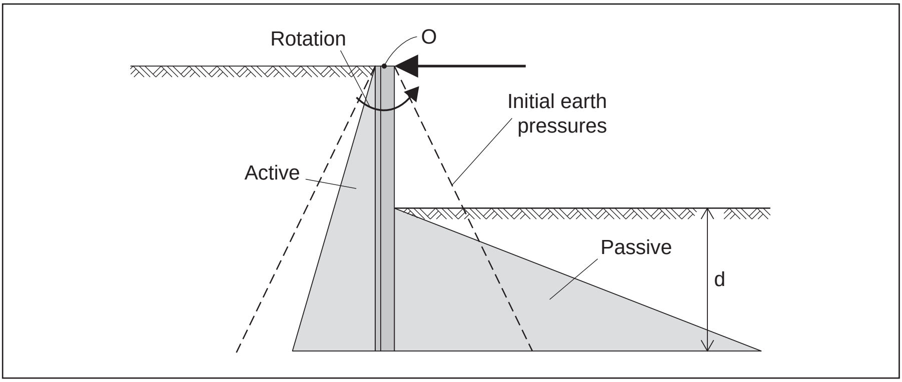

Fig. 5.6. LEM design model for free earth support walls.

A wall designed on free earth support principles can be considered as a simply supported vertical beam. The wall is embedded a sufficient distance into the soil to prevent translation, but is able to rotate at the toe, providing the wall with a pinned support at "O". A prop or tie near the top of the wall provides the other support. For a given set of conditions, the length of pile required is minimized, but the bending moments are higher than for a fixed earth support wall (see Chapter 5.15.3.).

For calculations of effects of actions to section 5.9. the minimum pile length is calculated by taking moments of the active and passive pressures about the support or anchor at point "0". The maximum shear force is calculated at the point of zero moment and the prop force is derived from the shear forces to balance the difference of the active and passive pressure forces at the depth of free earth support equilibrium.

Designers must be careful when selecting the design approach to adopt. For example, walls installed in soft cohesive soils, may not generate sufficient pressure to achieve fixity and in those soils it is recommended that free earth conditions are assumed.

5.15.3. Fixed earth support walls

A tied or single-propped wall designed on fixed earth principles acts as a propped vertical cantilever. Increased embedment at the foot of the wall prevents both translation and rotation and fixity is assumed. The tie or prop provides the upper support reaction. The effect of toe fixity is to create a fixed end moment in the wall, reducing the maximum bending moment for a given set of conditions but at the expense of increased pile length.

When a retaining wall is designed using the assumption of fixed earth support, provided that the wall is adequately propped and capable of resisting the applied bending moments and shear forces, no failure mechanism relevant to an overall stability check exists. However empirical methods have been developed to enable design calculations to be carried out.

It is important to note that when designing the pile length to free earth support in the ULS case then in reality in the SLS case a fixed or partially fixed condition may occur.

Fixed earth conditions may be appropriate where the embedment depth of the wall is taken deeper than that required to satisfy lateral stability, e.g. to provide an effective groundwater cut-off or adequate vertical load bearing capacity. However, where driving to the required depth may be problematic, assumption of free earth support conditions will minimise the driven length and ensure that the bending moment is not reduced by the fixity assumed. However for major quay walls where High Modulus or HZ® - AZ® walls are necessary a fixed earth support design situation may be considered for economic reasons. For this type of wall the King HZ-M piles are driven to the full length for fixed earth stability and the intermediate piles to a minimum depth of embedment required to retain the fill. In certain conditions water pressures may equalise at the toe of the intermediate piles. Slots may be designed to be cut in the webs of the sheets before driving to reduce water pressure where appropriate.

When designing a wall involving a significant retained height and multiple levels of support, the overall pile length will often be sufficient to allow the designer to adopt fixed earth conditions for the early excavation stages and take advantage of reduced bending moment requirements.

The design methods used to determine the pile length required for both free and fixed earth support conditions do not apply if the support is provided below the mid point of the retained height as the assumptions made in the analysis models will not be valid – analysis may in such cases be required to follow recommendations for low propped walls – see Section 5.15.6.

5.15.4. Singly propped or anchored walls

Steel sheet pile walls are advantageous in respect of capacity for significant retained height with a single level of prop or anchor to support the pile near the top. Using high steel grades up to S 430 GP the sections can be verified to resist high bending moments. ArcelorMittal can also supply sheet piles in S 460 AP.

5.15.4.1. Determination of prop or anchor load – Ultimate Limit State

In a similar manner to the design of the main wall, the anchorage or support system may be assessed on the basis of serviceability and ultimate limit states.

The recommended procedure for calculation of the design prop or anchor load for embedded sheet pile retaining wall analysis is as follows. A distinction will be made between LEM analyses and the other methods which utilise earth pressure redistribution in the analysis The retaining wall effects may be analysed by Soil Structure Interaction (SSI), FE or Limit Equilibrium methods (LEM). However the operative rules in EC 3 – Part 5 [vi] are as follows:

7.1.(1)P The effects of actions in anchors, walings, bracing and connections shall be determined from the structural analysis taking into account the interaction between the soil and the structure.

2.5.3.1. (1) The analysis of the structure should be carried out using a suitable soil-structure model in accordance with EN 1997-1 [v].

2.5.3.1. (2) Depending on the design situation, anchors may be modelled either as simple supports or as springs.

2.5.3.1. (3) If connections have a major influence on the distribution of internal forces and moments, they should be taken into account in the structural analysis.

The calculated ULS design prop or anchor load should be the greater of:

- the value obtained from the ULS case of the analysis. If different design situations are relevant then the higher load shall apply. For Design Approach 1 – both Combination 1 and Combination 2 are checked.
  Note: Combination 1 effectively replaces the old system of calculating the SLS check and factoring by 1.35.;
- the value obtained from the progressive failure check treated as an accidental Design situation in normal operating conditions. If more than one design situation is relevant the highest value shall apply. Note: the Design situation

conditions similar to the SLS limit state check apply e.g. Normal operating conditions and no unplanned excavation. A redundancy check applies for the failure of one prop or support anchor (see 5.15.4.2. and EC 7 Cl. 8.2. (1)) [v].

It is strongly recommended that the wall prop load action effects are calculated at least by SSI or subgrade reaction analysis methods and not just by LEM analyses alone. Prop or anchor load values obtained from LEM analysis should be increased by at least 30% to allow for soil pressure redistribution and arching effects not taken into account in such calculations. However this increase could be higher especially for single prop walls where undrained conditions are considered. In such cases SSI methods are strongly advised to derive the prop or anchor loads and the higher value used if total stress parameters are used. This should only be considered where short term design situations are appropriate. If there is any doubt on the period of exposure of the design situation in such a case then drained effective stress parameters should be used instead for the analysis.

It should also be noted that for the ULS check high levels of hydrostatic water pressure distribution apply, which will have a significant effect on the calculated prop / anchor load. This may have a greater effect than the water pressures under normal operating conditions that apply when checking for redundancy effects.

It is therefore important to check all cases to derive the design prop/anchor load because for Design Approach 1 Combination 1, Combination 2 and the progressive failure check combine different sets of parameters, geometry and model factors for deriving the calculated values.

5.15.4.2. Design of prop load to safeguard against progressive failure – Ultimate Limit State

In certain situations, progressive collapse of the structure may be a consequence of an extreme condition, i.e. failure of a tie rod or anchor and under such circumstances the designer should carry out a risk assessment. If necessary, the possibility of progressive failure should be avoided by changing the design or applying controls to the construction activities. However EC 7 Cl 8.3. (1) [v] states the consequences of the failure of any anchorage shall be taken into account as a design consideration. EC 3 Part 5 Cl. 7.3. (2) [vi] warns of the consequences of failure of a strut (or anchor) could lead to progressive failure.

Therefore the design situations for loss of a tie bar, anchor or prop require checking and appropriate structural support combinations incorporated in the design, such as robust walings to spread the loading to adjacent supports.

The effectiveness of discrete anchorages needs to be given careful consideration. The waling to the main wall will need to be checked to ensure that it will not collapse if the span between supports doubles following the loss of a tie rod or anchor. The ties or anchors on either side of the one that has failed will share the load from the missing tie or anchor which normally accounts for an increase in the design capacity of the anchor or prop by 50% in the typical design situation.

Dependent upon the magnitude of the loads involved, the resistance to be provided by each discrete anchorage may need to increase to resist the loading. If the tie rods are attached to a continuous anchorage, the total area of the anchorage will not change but the walings will need to be strong enough to provide the necessary support over a double span. If the anchor or prop level is close to the top of the piles a suitable reinforced concrete capping beam can be designed to spread the load over the span at prop level.

Note: Although the check for loss of a tie or strut is an extreme accidental design situation, and the accidental loading case applies, in checking the potential for progressive failure, calculations should be carried out, but with partial factors set to unity and without unplanned excavation. The resulting bending moments and support forces being treated as ultimate loads.

5.15.5. Calculation of prop / anchor load – summary table for analytic methods

Table 5.11. summarises design situations and applicable partial factors and geometry for the three checks to Design Approach 1 depending on whether SSI or LEM analyses are used to analyse a Category 2 structure.

Analytical method	**Earth & water pressure partial factors**			**Partial Factors on...**			
	Ground sufaces rule, see 5.18.	BS 6349 Water pressure, see 5.8.2.	Soils factored	Variable and surcharge load	Accidental loading	Action effects	Prop load model factor, see 5.15.4.1.
SSI							
ULS-Comb 1	Applies	1.00[1)]	1.00	1.11	n/a	1.35[1)]	1.00
ULS - Comb 2	Applies	1.00	>1.00	1.30	n/a	1.00	1.00
ULS-Accidental	n/a	Extreme x 1.00	1.00	1.00	Extreme x 1.00	1.00	1.50[2)]
LEM							
ULS-Comb 1	Applies	1.00[1)]	1.00	1.11	n/a	1.35[1)]	1.30[3)]
ULS - Comb 2	Applies	1.00	>1.00	1.30	n/a	1.00	1.30[3)]
ULS-Accidental	n/a	Extreme x 1.00	1.00	1.00	Extreme x 1.00	1.00	1.95[4)]

Table 5.11. Partial factors for calculation of prop or tie load - Summary.

Notes:

1) Effect of permanent action water pressure Comb 1 is factored by 1.35.

2) Includes additional 50% allowance for capacity of adjacent support if one tie or prop fails in Accidental situation to prevent progressive failure collapse.

3) Includes 30% additional allowance for effect of arching of the soil and re distribution of earth pressures in single level support walls.

4) Allows for both additional capacity if one prop or tie fails and also effect of arching of soil soil and re-distribution of earth pressures in single level support walls.

For every extreme loading, the design verifications have to be carried out separately (accidental load case).

5.15.6. Low propped walls - walls supported near formation level

Research at Imperial College, London has shown that the earth pressures acting on retaining walls that are restrained with a single level of supports at or near excavation level, are different to those assumed in conventional limit equilibrium calculations.

Conventional calculations assume that the mode of failure for a retaining structure supported at or near the top will be in the form of a forward rotation of the pile toe and the pressure distribution at failure is based on this assumption. The failure mode assumed for a low propped wall is that the pile will move away from the soil at the top in a similar manner to a cantilever and the pile will move back into the soil below the support level. This will result in the generation of passive pressures on the back of the wall and active pressures on the front.

To design a wall incorporating a low prop, there are two fundamental requirements that must be satisfied for the calculation method to be correct. Firstly, the prop must be sufficiently rigid to act as a pivot and prevent any forward movement of the wall and secondly, the sheet piles forming the wall must be capable of resisting the bending moments induced at the prop level to ensure that rotation of the pile occurs rather than buckling.

The design rules resulting from the Imperial College work suggest that the earth pressures below the support should be calculated assuming that active pressures apply at and above the prop position with full passive pressure at the toe of the pile; the change from one to the other being linear.

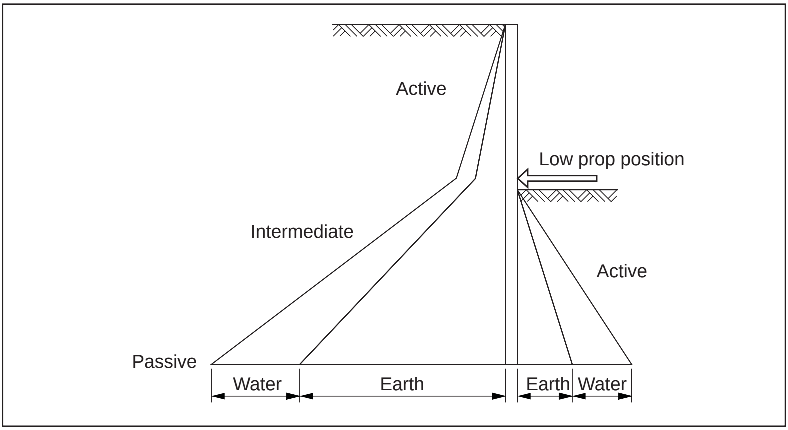

Fig. 5.7. LEM design model for low propped walls.

The support may be considered to be at low level if the depth to the support exceeds two thirds of the retained height of the excavation.

The operation of a low propped wall is very complex and it is recommended that the design of such a structure is carried out using soil structure interaction.

5.16. Multi prop walls – walls supported by more than one level of struts or ties

When more than one level of supports is used, wall stability becomes a function of the support stiffness and the conventional active/passive earth pressure distribution does not necessarily apply.

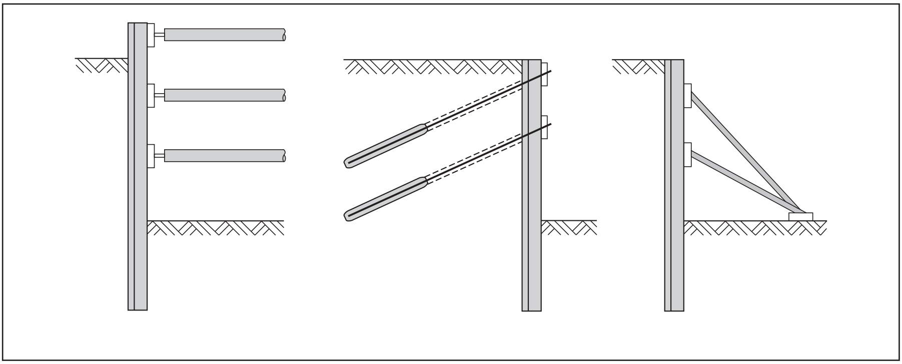

Fig. 5.8. Examples of multi prop walls.

The design of multi-propped walls generally requires the consideration of various construction stages. At each stage the design bending moments and shear forces in the sheet piles and prop loads (if a prop is installed at that stage) need to be determined. The length of the pile is usually determined by the required penetration below formation level for the final construction stage. Thus for all intermediate stages it is likely that there will be sufficient penetration to prevent instability.

The level or location of the props is critical to the economic design and performance of multi-propped walls. By careful selection of prop levels the maximum bending moment and shear forces in the wall may be controlled along with prop forces and deformations.

Care should be exercised in selecting the appropriate water levels to ensure that more severe conditions are unlikely to occur during the life time of the wall (see 5.8.2.).

5.16.1. Calculation methods for multi prop walls

Consideration of the wall as either fixed or free in terms of its mode of operation directly affects the bending moments, shear forces and support reactions acting on the wall.

For multi-propped walls the design of the first two stages (Stage 1 - excavation to first support level; Stage 2 – installation of prop and excavation to second support level) may be assessed from limit equilibrium fixed-earth support methods. As additional levels of support are added the problem becomes indeterminate and methods – such as the “distributed prop load”, “hinge” and “continuous beam” methods – may be adopted. These methods tend to be conservative. These are described below.

Where a more detailed understanding of the bending moments and forces in props are required it is recommended that subgrade reaction or numerical models be used. It is likely that such methods will indicate lower maximum bending moments, shear forces, and prop loads compared with the approximate methods above. However, it is often difficult to “calibrate” these more sophisticated models and for routine designs, particularly temporary works, it may be more acceptable to use the less rigorous methods.

When more than one level of support is provided to a wall the potential mode of failure is significantly different to that assumed for a wall with a single support provided that the supports are not close enough together to act as a single support. With multiple levels of support, the wall will not fail by rotation in the conventional manner – failure will be as a result of collapse of the support system or excessive bending of the piles. Consequently, provided that the wall and supports are sufficiently strong to resist the worst credible loading conditions, failure of the structure cannot occur.

5.16.2. Distributed prop loads

The distributed prop load (DPL) method for calculating prop loads for propped temporary excavations is based on the back analysis of field measurements of prop loads relating to 81 case histories, of which 60 are for flexible walls (steel sheet pile and king post) and 21 are for stiff walls (contiguous, secant, and diaphragm).

The case history data relate to excavations ranging in depth from 4 to 27 m, typically 5 to 15 m in soft and firm clays (soil class A), 10 to 15 m in stiff and very stiff clays (soil class B), and 10 to 20m in coarse-grained soils (soil class C) – see the table below for the definitions of the soils classes.

The DPL method should only be used for multi-propped walls of similar dimensions and constructed in similar soil types.

Distributed prop load diagrams for soil classes A to C are provided for flexible (F) and stiff (S) walls in the diagram 5.9. below.

Soil Class:

A normally and slightly overconsolidated clay soils (soft to firm clays);

B heavily overconsolidated clay soils (stiff and very stiff clays);

C coarse grained soils;

D mixed soils.

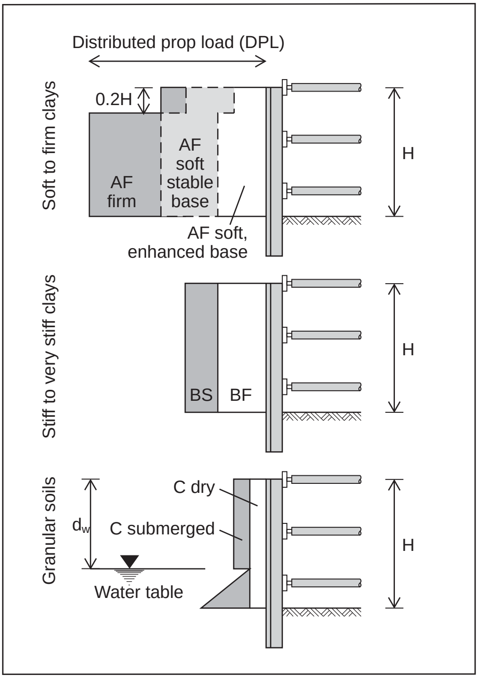

Fig. 5.9. Distributed prop loads for flexible (F) and stiff (S) walls.

The magnitude of the distributed prop load (DPL) in each case is summarized in the following table.

Class	Soil	Over retained height	DPL
AS	Same as AF for medium strength clay		
AF	Medium strength clay	Top 20%	$0.2\ \gamma H$
		Bottom 80%	$0.3\ \gamma H$
	Low strength clay with stable base	Top 20%	$0.5\ \gamma H$
		Bottom 80%	$0.65\ \gamma H$
	Low strength clay with enhanced base stability	Top 20%	$0.65\ \gamma H$
		Bottom 80%	$1.15\ \gamma H$
BS	High to very high strength clay	All	$0.5\ \gamma H$
BF	High to very high strength clay	All	$0.3\ \gamma H$
C	Granular soil, dry	All	$0.2\ (\gamma - \gamma_w)H$
	Granular soil, submerged	Above water	$0.2\ \gamma H$
		Below water	$0.2\ (\gamma - \gamma_w)\ H + \gamma_w\ (z - d_w)$

Table 5.12. Magnitude of distributed prop load for walls installed in different soil types.

Example:

Consider a sheet pile wall that is retaining $H = 5$ m of soft clay, which has characteristic weight density $\gamma_k = 19.5$ kN/m^3. A separate check of the excavation's base stability has shown it to be stable. The DPL over the top 20% of the retained height (i.e. to a depth of 1 m) is:

$$DPL = 0.5\gamma_k \mathrm{H} = 0.5 \times 19.5 \times 5 \approx 48.8 \text{ kPa}$$

while, over the bottom 80% of the retained height (i.e. from 1 m to 5 m depth), it is:

$$DPL = 0.65\gamma_k \mathrm{H} = 0.65 \times 19.5 \times 5 = 63.4 \text{ kPa}$$

Consider the same wall retaining sand, with characteristic weight density $\gamma_k =$ 18 kN/m^3, and water, with weight density $\gamma_w = 9.81$ kN/m^3, at a depth $d_w = 2$ m. The DPL above the water table (i.e. to a depth of 2 m) is:

$$DPL = 0.2\gamma_k \mathrm{H} = 0.2 \times 18 \times 5 = 18 \text{ kPa}$$

while, below the water table (i.e. from 2 m to 5 m depth), it is:

$$\begin{aligned} DPL &= 0.2\left(\gamma_k - \gamma_w\right)\mathrm{H} + \gamma_w\left(z - d_w\right) \\ &= 0.2 \times \left(18 - 9.81\right) \times 5 + 9.81 \times \left(z - 2\right) \\ &= 8.2 \text{ kPa at } z = 2 \text{ m} \\ &= 37.6 \text{ kPa at } z = 5 \text{ m} \end{aligned}$$

Individual prop loads are obtained by integrating the distributed prop load diagrams over the depth of influence of the prop being considered. The prop load P is given by:

$$P = s \times \int_{z_a}^{z_b} DPL \times dz$$

with

s prop's horizontal spacing (i.e. on plan);

z_a depth to a point midway between the current prop and the one above (as shown in the diagram below);

z_b depth to a point midway between the current prop and the one below;

DPL distributed prop load at depth z.

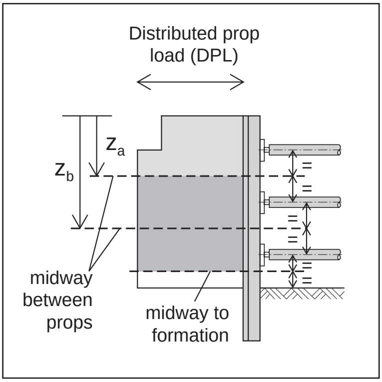

Fig. 5.10. Distributed prop load – example.

For the top prop only, the entire DPL envelope above that prop is included in the integration (i.e. the depth z_a = 0 m). For the bottom prop only, the envelope is curtailed halfway towards formation level.

Example (continued from previous example):

The previous sheet pile wall is supported by three levels of prop at depths d_1 = 1.0 m, d_2 = 2.5 m, and d_3 = 4.0 m. The props are spaced at s = 2.5 m horizontal spacing.

For the top prop, z_a = 0.0 m and:

$$z_b = \frac{d_1 + d_2}{2} = \frac{1.0 + 2.5}{2} = 1.75 \text{ m}$$

Hence the force carried by prop 1 is:

$$P_1 = s \times \int_{0m}^{1.75m} DPL_1 \times dz$$

$$= 2.5 \times (48.8 \times 1.0 + 63.4 \times 0.75) = 241 \text{kN}$$

For the second prop, z_a = 1.75 m and:

$$z_b = \frac{d_2 + d_3}{2} = \frac{2.5 + 4.0}{2} = 3.25 \text{ m}$$

Hence the force carried by prop 2 is:

$$P_2 = s \times \int_{1.75m}^{3.25m} DPL_2 \times dz$$

$$= 2.5 \times 63.4 \times (3.25 - 1.75) = 238 \text{ kN}$$

For the bottom prop, z_a = 3.25 m and:

$$z_b = \frac{d_3 + H}{2} = \frac{4.0 + 5.0}{2} = 4.5 \text{ m}$$

Hence the force carried by prop 3 is:

$$P_3 = s \times \int_{3.25m}^{4.5m} DPL_3 \times dz$$

$$= 2.5 \times 63.4 \times (4.5 - 3.25) = 198 \text{ kN}$$

The Distributed Prop Load method provides an estimate of characteristic prop loads. Design prop loads should be obtained by applying appropriate partial factors from Eurocode 7 - Part 1, see 5.15.5. [v].

5.16.3. Hinge method

When designing a wall involving a significant retained height and multiple levels of support, the overall pile length will often be sufficient to allow the designer to adopt fixed earth conditions for the early excavation stages and take advantage of reduced bending moment requirements.

This method allows the structure to be analysed at successive stages of construction and the assumption is made that a hinge occurs at each support position except the first. The spans between the supports are considered as simply supported beams loaded with earth and water pressures and the span between the lowest support and the excavation level is designed as a single propped wall with the appropriate earth and water pressures applied. Prop loads calculated using this method include the respective load from adjacent spans.

The analysis of structures using this method is carried out on a stage-by-stage basis with excavation being carried out to sufficient depth to enable the next level of support to be installed. It is therefore possible that the support loads and bending moments calculated for a given stage of excavation are exceeded by those from a previous stage and it is important that the highest values of calculated support force and bending moment are used for design purposes.

5.16.4. Continuous beam method

The wall is assumed to act as a vertical beam subjected to a pressure distribution with reactions at support points. The bottom of the beam is also assumed to be supported below excavation level by a soil reaction at the point at which the net active pressure on the wall falls below zero. Mobilised earth pressures are assumed to act on the wall, the magnitude of these pressures being dependent upon a factor governed by the permissible movements of the wall being designed. The minimum recommended mobilised earth pressure is however 1.3 times that resulting from the use of K_a to determine soil pressures on the wall.

Each support is modelled either as rigid or as a spring, depending on its compressibility. The displacement at a rigid support is zero, whereas in a spring it is proportional to the force carried by the spring.

The hypothetical soil support is modelled in one of three ways:

- if the net pressure (active minus passive pressure at a given depth) does not fall to zero anywhere along the wall, the hypothetical soil support is ignored and the embedded portion of the wall is treated as if it were a cantilever. This situation is likely to occur if there is only a short depth of embedment or the net pressures are particularly large. The applied load in this case is carried entirely by the props;
- if the net pressure does fall to zero along the length of the wall, the hypothetical soil support is considered as a rigid prop. This situation is likely to occur if there is a large depth of embedment or the net pressures are particularly small. The applied load in this case is shared by the props and the soil. The force carried by the soil is equal to the jump in shear force that occurs at the hypothetical soil support. Under this assumption it is essential to check that the force assumed to be provided by the hypothetical soil support is not greater than the available soil resistance below that support. If it is greater, the following method should be applied;
- if the net pressure falls to zero, but the available soil resistance below the point at which that occurs is less than that required by the rigid soil prop a finite soil reaction equal in magnitude to the available soil resistance should be adopted in subsequent calculations. This situation is likely to occur if there is a moderate depth of embedment. The applied load in this case is shared by the props and the soil. The force carried by the soil is equal to the change in shear force that occurs at the hypothetical soil support.

5.17. Bending moment reduction

The simplifying assumption made in design calculations concerning the linear increase in active and passive pressures in a material does not take into account the interaction between the soil and the structure. Studies have shown that this can have a significant effect on the distribution of earth pressures and consequent bending moments and shear forces on a structure.

The result of a redistribution of pressures is therefore a reduction in the maximum bending moment on a wall, but an increase in support reaction. Support loads calculated by a limit equilibrium analysis are generally lower than those resulting from soil structure interaction.

This reduction in calculated bending moments is a function of the soil type and the flexibility of the wall in comparison to the supported soil. When a supported, flexible wall deflects, a movement away from the soil occurs between the support position and the embedded portion of the wall. This effect often leads to a form of arching within the supported soil mass which allows the soil to maximise its own internal support capabilities effectively reducing the pressures applied to the wall. For a relatively flexible structure, such as an anchored sheet pile wall, the effect of wall deformation will be to increase the pressures acting above the anchor level, as the wall is moving back into the soil using the support as a pivot, and reduce the pressures on the wall below this level where the biggest deflections occur.

Redistribution should not be considered for cantilever walls or where the structure is likely to be subjected to vibrational or large impact forces that could destroy the soil “arch”. Similarly, if the support system is likely to yield or movement of the wall toe is expected, moment reduction should not be applied. Where stratified soils exist, moment reduction should be viewed with caution since soil arching is less likely to occur in soils of varying strength.

The beneficial effects of soil arching on wall bending moment are automatically taken into account in analysis packages based on soil-structure interaction.

5.18. Ground surfaces rule - unplanned excavation

For the Ultimate Limit State Eurocode 7 - Part 1, 9.3.2.2. [v] requires an allowance to be made for the uncertainty of the ground surface level throughout the design life of the structure , which includes the possibility of an unplanned excavation reducing formation level on the restraining side of a retaining wall (see figure below). N.B.: this unplanned excavation rule can only be safely ignored if the surface level on the supported side can be specified to be reliable through the construction and design life period of the structure.

For marine structures dredging tolerances and potential scour should be taken into account before the unplanned excavation rule is applied to check the ULS requirements.

For the Serviceability Limit State no additional allowance should be made for excavation below the formation level expected in normal circumstances. However, the expected formation level should take into account any temporary excavation for services, if these can be reasonably expected, and if appropriate any allowance for the excavation or dredging constructional tolerance.

5.18.1. Ground surface rule for cantilevered walls

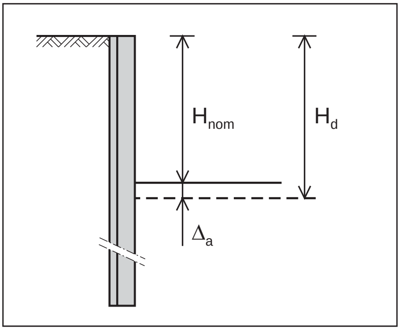

Fig. 5.11. Ground surface rule for cantilever walls.

The design retained height of the wall H_d is given by:

$H_d = H_{nom} + \Delta_a$

with

H_{nom} nominal retained height;

Δ_a allowance for ground surface rule.

When normal levels of site control are employed, verification of ultimate limit states should assume Δ_a is given by:

$\Delta_a = H_{nom} / 10 \leq 0.5$m

Retained height		With normal site control					
Nominal	H_{nom} (m)	1.0	2.0	3.0	4.0	5.0	> 5.0
Design	H_d (m)	1.1	2.2	3.3	4.4	5.5	H_{nom} + 0.5

Table 5.13. Ground surface rule – example for cantilever walls.

5.18.2. Ground surface rule for supported walls

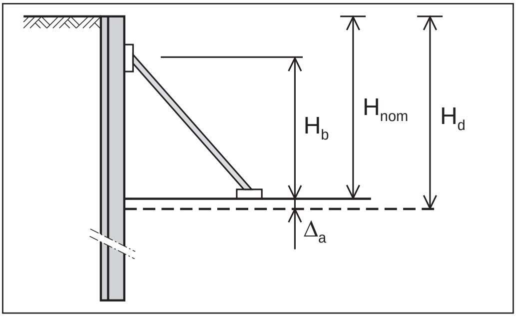

Fig. 5.12. Ground surface rule for supported walls.

The design retained height of the wall H_d is given by:

$H_d = H_{nom} + \Delta_a$

where H_{nom} is the nominal retained height and Δ_a is the appropriate allowance.

When normal levels of site control are employed, verification of ultimate limit states should assume Δ_a is given by:

$\Delta_a = H_b / 10 \leq 0.5$ m

where H_b is the nominal retained height below the bottom prop.

Eurocode 7 - Part 1 also warns that, where the surface level is particularly uncertain, larger values of Δ_a should be used. However, it also allows smaller values (including $\Delta_a = 0$) to be assumed when measures are put in place throughout the execution period to control the formation reliably.

The rules for unplanned excavations apply to ultimate limit states only and not to serviceability limit states. Anticipated excavations in front of the wall should be considered specifically – they are not, by definition, unplanned.

Planned excavations include French drains, pipe trenches, buried close-circuit television cables, etc.

These rules provide the designer with considerable flexibility in dealing with the risk of over-digging. A more economical design may be obtained by adopting $\Delta_a = 0.1$, but the risk involved must be controlled during construction – leading to a supervision requirement that must be specified in the Geotechnical Design Report and tolerances not allowed to exceed 0.1m. A designer who wants to minimize the need for supervision must guard against the effects of over-digging by adopting $\Delta_a = 10\%\ H_{nom}$ (limited to a maximum of 0.5 m as per the equation above).

The ULS design condition should include the additional allowance for unplanned excavation with surcharges taken into account in operational design situations.

If an allowance is to be made for softening of the passive soil in a total stress analysis it should be applied beneath the unplanned excavation level.

5.19. Softened Zone

Where soft cohesive soils are exposed at dredge or excavation level, it is advisable when calculating passive pressures to assume that the cohesion increases linearly from zero to the design cohesion value over a finite depth of passive soil.

An allowance in the design should also be made for softening of the soil on the restraining side of the wall for the duration of the temporary works, i.e. due to excavation disturbance and dissipation of pore water pressures at excavation level. The value of the undrained shear strength on the restraining side should be assumed to be zero at excavation level rising to c_u at a depth of L, given by:

- L = 0.5 m where there is no potential for groundwater recharge either at excavation level or within the soil;
- $L = \sqrt{12\,c_v t}$ where recharge occurs at excavation level but with no recharge within the soil (c_v is the coefficient of consolidation and t the time elapse).

5.20. Berms

Berms may be used to maintain stability of a cantilever or singly propped wall while excavation continues in the centre of a site or over short sections of the wall. They are also useful in reducing the lateral deformations associated with cantilever walls.

For limit equilibrium analysis it is recommended that the "raised effective formation level" approach be used as outlined in [xiii] and [xviii].

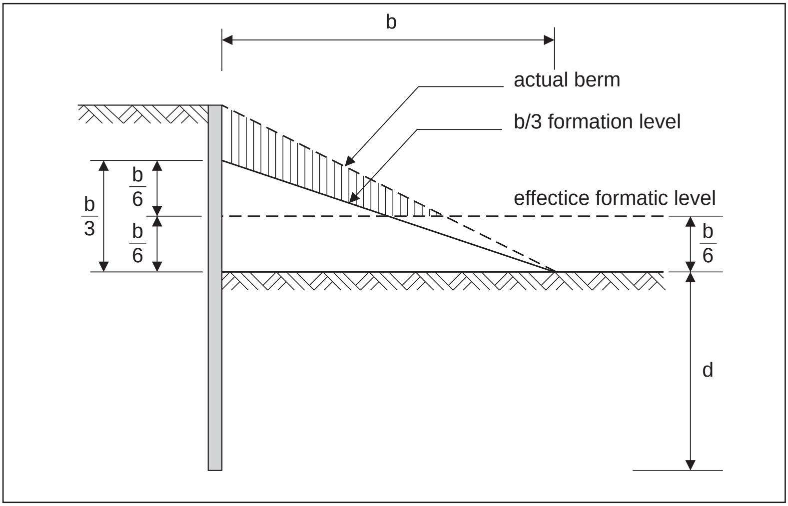

Fig. 5.13. (Temporary) support from berms.

The figure above indicates the approach. The effective formation level on the berm side of the wall is taken as b/6 above excavation level. Any portion of the actual berm above the effective formation level and a berm at 1 on 3 is treated as a surcharge applied at the revised formation level.

5.21. Selection of pile section

The absolute minimum sheet pile section required for the retaining wall is that obtained from consideration of the action effects, bending moments and shear forces derived by calculation for the particular case in question. However, it is also necessary to consider installation of the piles to the required design toe level in accordance with EC 7, 9.4.1.8. (P) [v] ("for sheet piling the need for a section stiff enough to be driven to the design penetration without loss of interlock") this is necessary for not only stability and structural integrity of the wall but also for vertical load carrying capacity if relevant. This also implies that the designed section to be adopted may be determined by the method of installation as hard driving conditions may require a heavier stiffer section to prevent buckling by impact or pressing during installation. This aspect is covered in more detail in the installation Chapter 13.

Furthermore, the requirements with respect to the effective life of the retaining wall will also need to be assessed. The effect of corrosion on the steel piles is to reduce the section strength and the design must ensure that the section selected will be able to provide sufficient sectional resistance for verification of all combined effects of actions at the end of the specified life span. In many instances the need for a heavy section or higher steel grade for driving automatically introduces some if not all of the additional strength needed for durability.

However for the most economical solution and also for environment considerations there is a need to consider ground pre-treatment to facilitate driving. In these circumstances it may be possible to install the minimum structurally acceptable section for instance if pre-augering followed by vibrodriving is the selected technique.

The procedure for verification of the pile section to Eurocode EC 3 - Part 5 [vi] is covered in Chapter 8.

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6 | Axially loaded steel piles

Chapter 6 - Axially loaded steel piles

Contents

6.1. Introduction

Steel sections can be used as bearing piles where soil and ground conditions preclude the use of shallow foundations. They transmit vertical loads from the structure through the upper soft layers to ground of adequate strength for support. Steel sheet piles can be used as simple bearing piles and have the added advantage that they can be designed as a retaining wall that carries vertical loads. The main advantages that steel piles have over alternative systems are as follows:

- steel piles have a very high load-carrying capacity which can be further enhanced, given suitable ground conditions, by the use of high yield strength steel. The option of using a higher grade steel is also useful when hard-driving conditions are anticipated or when carrying high compression loading down to bear on rock strata. Steel driven piles are particularly suitable for tension capacity and can be driven at varying angles to the vertical and horizontal to resist horizontal loading effects;
- steel bearing piles are extractable at the end of the life of the structure and therefore the opportunity for either re-use or recycling exists, resulting in an economic and environmental sustainable solution. The resulting site is enhanced in value since there are no old foundations that can obstruct or hinder future development;
- driven steel bearing piles are of the low-displacement type and therefore there is no spoil to dispose of, which is of particular benefit when piles are being installed into contaminated ground;
- steel sheet piles can carry high loads due to significant skin friction area for bridge abutment and basement walls and also when interlocked together as box piles for gantry bases using special omega connectors;
- the carrying capacity of steel piles can be tested simply on site during installation by driving using dynamic methods or alternatively when steel sheet piles are pressed skin friction can be assessed using instrumentation by pressing and withdrawal action;
- for marine structures Steel Combined Wall systems can carry high vertical loads (e.g. from crane rail).

6.2. Types of steel bearing pile

Main types of steel bearing piles:

- H-piles, see Chapter 6.2.1.;
- Sheet piles, see Chapter 6.2.2.;
- Box piles, see Chapter 6.2.3.;
- HZ®-M piles, see Chapter 6.2.4.;
- Tubular piles, see Chapter 6.2.5.

6.2.1. H-piles

H-piles are typically columns and bearing pile sections. Where piles are fully embedded, i.e. the whole length of the pile is below ground level, an H section pile is most suitable. This situation usually occurs when piles are used to support land-sited structures such as road and railway bridges and industrial buildings. As H-piles have a different section modulus in the Y and Z directions, it is important that the direction of principal horizontal force/moment is known.

The sections drive well as minimal displacement piles and perform well in tension and are particularly suitable for driving to a hard strata or bearing onto rock. Also H-piles are versatile to provide resistance to horizontal component forces when designed and driven as raker piles.

6.2.2. Sheet piles

As well as their use in the construction of earth-retaining structures, sheet piles also have the ability to carry significant axial load – a capability that has been utilised since their introduction at the start of the 20th century. The vertical carrying capacity of sheet piles has been particularly useful in maritime structures, where quay walls need to support cranes on the sheet piles in addition to the surcharges imposed on the ground behind a wall; and land-based structures, such as bridge abutments and basements below multi-storey buildings.

Sheet piles are useful for the construction of basement walls in buildings on restricted redevelopment sites. The narrow profile of the finished wall together with equipment that allows installation right up to the site boundary means that the usable space in the basement is maximised.

The foundation loads from the perimeter of the building frame can be applied directly to the sheet piling. The point loads from the building frame can be distributed to the entire sheet pile wall by means of a capping beam and these loads are then shed to the founding soil over the full length of the basement perimeter. If a steel frame is being used for the building, the anchor bolts for the columns can be cast into the reinforced concrete capping beam in readiness for frame erection. A major advantage from this form of construction is its potential for saving time on site, since, once installed, the steel piles can be loaded immediately. This sustainable method of construction is quicker than more traditional methods and has a major advantage over other solutions because the joints in the steel sheet piles can be seal welded (refer to Chapter 2) after driving and proven to be completely watertight by testing before the interior finishing is completed well before the water table rises. The installed piles, when painted, give an appropriate finish for basement car parks and various cost effective cladding systems are available for habitable basements.

For design of sheet piles in basements carrying vertical load it may be required to consider fire resistance effects on piles carrying the vertical load. Fire resistance can be provided by intumescent paint or cladding if required. For further guidance on fire resistance and basements refer to ArcelorMittal brochures – “Fire Resistance” and also “Underground Car Parks”.

6.2.3. Box piles

Box piles can be used as isolated bearing piles or used in combined systems with sheet piles.

Box sections are formed by welding together two or more sheet piles to form a single section and are sub-divided into the following types (see Chapter 1 for details):

- CAZ box piles;
- CU and CAU box piles.

Alternatively, instead of welding, the sheets may be interlocked together using omega special interlocks and the piles then can be independently driven and extracted using driving or pressing equipment to achieve a highly sustainable solution (see Chapter 11). This application has been successfully used on Highways schemes to provide sustainable foundations for gantries.

Box piles are most useful when part of the pile is exposed above ground level and when a straight face is required for fixing details, as in pier and jetty construction, or when hard-driving conditions are anticipated. They can also be incorporated into a plain sheet pile wall to increase its bending strength and stiffness and/or its ability to support axial loads. These sections possess a comparatively uniform radius of gyration about each axis, and hence provide excellent column properties, which is a particular advantage in these situations.

6.2.4. HZ®-M piles

For greater bearing and structural capacity than sheet piles and boxpiles the HZ-M system and series of profiles can be used. The HZ-M system is particular versatile for construction of heavily loaded quay walls where the primary elements are required to take vertical loading and the driveability is required for hard ground conditions.

HZ-M sheet pile systems are particularly suitable for supporting a conventional abutment or integral bridge abutments. The flexible nature of steel allows the abutments to move in response to the lateral loads which are transmitted to the abutment from the bridge beams due to thermal expansion, but remain robust enough to cope with the vertical loads and such lateral loads as braking forces and impacts.

6.2.5. Tubular piles

Steel tubular piles are manufactured from different sources of steel material. Spriral welded tubular piles to BS EN 10219 [xi] or American Standard API (See Chapter 1) are supplied by ArcelorMittal up to 3 m diameter and 25 mm wall thickness and in lengths in excess of 50 m. These provide high capacity piles for carrying vertical load and also by using them for fabricated primary elements in combi walls they are particularly suitable for marine structures carrying high combined loading for retained heights greater than 15 m.

6.3. Design situations for bearing piles

6.3.1. General

Pile foundations shall be designed for all potential limit states including overall stability of the pile-soil system. This is particularly relevant for retaining structures such as bridge abutments where sheet piles may provide a combined retaining and bearing pile function.

Typically pile foundations are used to support buildings and bridges, when the upper strata have insufficient bearing capacity to carry the loads or, more commonly, the settlement of a shallow footing exceeds the acceptable limit for the structure. For sheet pile walls, it is an efficient use of materials to consider the wall as both a retaining and load bearing structure.

The following should be considered when designing pile foundations:

- ground and ground-water conditions on the site, including the presence or possibility of obstructions in the ground;
- stresses generated in the pile during installation;
- effect of the method and sequence of pile installation on piles, which have already been installed and on adjacent structures or services;
- tolerances within which the pile can be installed reliably;
- deleterious effects of chemicals in the ground;
- possibility of connecting different ground-water regimes;
- handling and transportation of piles;
- effects of pile construction on neighbouring buildings.

The factors above should be considered in the light of:

- spacing of the piles in pile groups;
- displacement or vibration of adjacent structures due to pile installation;
- type of hammer or vibrator used;
- dynamic stresses in the pile during driving.

6.3.2. Design limit states

The scope of the Piling Handbook is limited to guidance for checking Ultimate Limit State carrying capacity of steel piles but the designer must allow design of the structure for consideration of all relevant limit states

The Serviceability Limit State , checking settlement and deflections is outside the scope of the Piling Handbook.

As discussed in Chapter 5, Eurocode 7 - Part 1 [i] identifies five Ultimate Limit States for which different sets of partial factors are provided:

- failure or excessive deformation in the ground (GEO);
- internal failure or excessive deformation of the structure (STR);
- loss of static equilibrium (EQU);
- loss of equilibrium or excessive deformation due to uplift (UPL);
- hydraulic heave, piping, and erosion (HYD).

Of these, limit states GEO and STR (discussed in Chapter 5) are most relevant to bearing piles. Limit states for piles may be verified by calculation (see Section 8) or by testing.

Verification of limit state GEO involves checking that design effects of actions do not exceed their corresponding design resistances. This is expressed in Eurocode 7 by the inequality:

$$E_d \leq R_d$$

where E_d = design effects of actions and R_d = the corresponding design resistance.

The derivation of the design effects of actions, E_d, is discussed in Section 6.4. and the derivation of the design resistance, R_d, in Chapters 6.5., 6.6., 6.7. and 6.8. Note the symbol E_d is replaced by F_d in Chapter 6.4.

In Design Approach 1 for pile foundations, two separate calculations are required, one with factors applied solely to actions and the other with factors applied mainly to resistances (and not to material properties, as is the case for general structures). This is the approach adopted in the UK through its National Annex [ii].

The Piling Handbook does not cover Design Approaches 2 or 3. For details of these approaches, the reader should refer to Eurocode 7 - Part 1 [i] itself.

The partial factors specified in the UK National Annex for the design of driven bearing piles to Design Approach 1 are summarized in Table 6.1.

Parameter			Partial factor	Combination 1		Combination 2
Actions	Permanent	Unfavourable	γ_G	1.35		1.00
		Favourable	$\gamma_{G,fav}$	1.00		1.00
	Variable	Unfavourable	γ_Q	1.50		1.00
		Favourable	$\gamma_{Q,fav}$	0		0
Material properties[1)]					M1	M2
	Effective shearing resistance		γ_φ	1.00	1.00	1.25
	Effective cohesion		γ_c	1.00	1.00	1.25
	Undrained shear strength		γ_{cu}	1.00	1.00	1.40
	Unconfined compressive strength		γ_{qu}	1.00	1.00	1.40
	Weight density		γ_γ	1.00	1.00	1.00
Resistance[2)]					w/o	w
	Base resistance		γ_b	1.00	1.70	1.50
	Shaft resistance in compression		γ_s	1.00	1.50	1.30
	Total resistance		γ_t	1.00	1.70	1.50
	Shaft resistance in tension		$\gamma_{s,t}$	1.00	2.00	1.70

Table 6.1. Partial factors for design of pile foundations for ultimate limit state GEO in persistent and transient design situations.

1) In combination 2, set M1 is used for calculating resistances of piles or anchors and set M2 for calculating unfavourable actions on piles owing e.g. to negative skin friction or transverse loading.

2) Without explicit verification of SLS, the larger resistance factors apply (column w/o); with explicit verification, the smaller values apply (column w).

Partial factors for GEO in accidental design situations are all 1.00.

Partial factors for checking Serviceability limit states are all 1.00.

6.4. Limit state design actions and effects

The following sub-sections discuss piles subject to compression loading (Section 6.4.1.), tension loading (6.4.2.) and transverse loading (6.4.3.). For simplicity only one variable action has been considered, where there is more than one variable action combination factors are required.

6.4.1. Piles subject to compression loading

For a pile foundation subject to compression, the design compressive action $F_{c,d}$ acting on the pile (including the self-weight of the pile) is given by:

$$F_{c,d} = \gamma_G \left(P_{G,K} + W_{G,K}\right) + \gamma_Q P_{Q,k}$$

where $P_{G,k}$ is the permanent characteristic action on the pile, $W_{G,k}$ is the characteristic weight of the pile, $P_{Q,k}$ is the characteristic variable action on the pile, γ_G and γ_Q are the partial factors on permanent and variable actions respectively.

Example:

Component of load		$F_{c,k}$ **(kN)**	**Combination 1**		**Combination 2**	
			Partial factor	$F_{c,d}$ (kN)	Partial factor	$F_{c,d}$ (kN)
Permanent	P_G	1200	1.35	1620	1.00	1200
Pile weight	W_G	27	1.35	36	1.00	27
Variable	P_Q	300	1.50	450	1.30	390
Total	F_c	1527		2106		1617

Table 6.2. Component of load - Compression loading.

6.4.2. Piles subject to tension loading

For a pile foundation subject to tension, the design tensile action $F_{t,d}$ acting on the pile minus the self-weight of the pile) is given by:

$$F_{t,d} = \gamma_G T_{G,k} - \gamma_{Gfav} W_{G,k} + \gamma_Q T_{Q,k}$$

Since it is conservative to do so, the pile's self-weight is often omitted from traditional calculations of pile pullout.

Example:

Component of load		$F_{c,k}$ **(kN)**	**Combination 1**		**Combination 2**	
			Partial factor	$F_{t,d}$ (kN)	Partial factor	$F_{t,d}$ (kN)
Permanent	T_G	−700	1.35	−945	1.00	−700
Pile weight	W_G	+27	1.00	+27	1.00	+27
Variable	T_Q	−100	1.50	−150	1.30	−130
Total	F_t	−773		−1068		−803

Table 6.3. Component of load - Tension loading.

6.4.3. Piles subject to transverse loading

For a pile foundation subject to transverse loading, the design transverse action $F_{tr,d}$ acting on the pile is given by:

$$F_{tr,d} = \gamma_G H_{G,k} + \gamma_Q H_{Q,k}$$

where $H_{G,k}$ is the characteristic permanent horizontal action and $H_{Q,k}$ is the characteristic variable horizontal action.

6.4.4. Resistance to compression

Design values of pile resistance to compression ($R_{c,d}$) are obtained from characteristic shaft and base resistances ($R_{s,k}$ and $R_{b,k}$ respectively) by dividing by the appropriate partial factors:

$$R_{c,d} = R_{s,d} + R_{b,d} = \frac{R_{s,k}}{\gamma_s} + \frac{R_{b,k}}{\gamma_b}$$

or, when the shaft and base resistances are not determined separately:

$$R_{c,d} = \frac{R_{c,k}}{\gamma_t}$$

The first equation is normally used when designing piles by calculation (i.e. on the basis of ground test results or ground parameters) and the second when the shaft and base components cannot be determined separately (for example, when designing piles using static load or dynamic impact tests).

Example:

Component		**Characteristic resistance** R_k (kN)	**Combination 1**		**Combination 2**	
			Partial factor	R_d (kN)	Partial factor[1)]	R_d (kN)
Shaft	R_s	300	1.0	300	1.5	200
Base	R_b	250	1.0	250	1.7	147
Total	R_t	550	-	550	-	347
Alternative	-	550	1.0	550	1.7	324

Table 6.4. Component - Resistance to compression.

[1)] Without explicit verification of SLS.

6.4.5. Resistance to tension

Design values of pile resistance to tension ($R_{t,d}$) are obtained from the characteristic shaft resistance to tension ($R_{t,k}$) by dividing by the appropriate partial factor:

$$R_{t,d} = \frac{R_{t,k}}{\gamma_{st}}$$

Example:

Component		**Characteristic resistance** R_k (kN)	**Combination 1**		**Combination 2**	
			Partial factor	R_d (kN)	Partial factor[1)]	R_d (kN)
Shaft	R_s	300	1.0	300	2.0	150

Table 6.5. Component - Resistance to tension

[1)] Without explicit verification of SLS.

6.4.6. Resistance to transverse loading

The horizontal resistance of a pile is a function of the ground strength, the strength of the pile, the length of the pile and the fixity of the pile head. “Short” piles are those where the horizontal resistance is governed by the ground strength alone, whereas “long” piles are those where the resistance is governed by both pile and ground strength.

No partial resistance factors are explicitly given in Eurocode 7 for horizontal or transverse loading. It would be logical to treat a laterally loaded pile in a similar manner to an embedded wall. Thus, the design resistance of short piles may be assessed for Design Approach 1 Combination 2 on the basis of design material properties using the partial material factors used for embedded retaining walls. For long piles, the design resistance may be obtained as for short piles but including factored material properties for the pile.

6.5. Calculation of vertical bearing pile resistance of the ground

The characteristic compressive resistance of a pile ($R_{c,k}$) may be determined from:

$$R_{c,k} = R_{s,k} + R_{b,k}$$

where $R_{s,k}$ is the pile’s characteristic shaft resistance and $R_{b,k}$ its characteristic base resistance.

The characteristic shaft resistance of a pile ($R_{s,k}$) may be determined from:

$$R_{s,k} = \frac{\sum_{i=1}^{n} A_{s,i} q_{s,i,k}}{\gamma_{Rd}}$$

where $A_{s,i}$ is the pile’s shaft area in layer i; $q_{s,i,k}$ is the characteristic unit shaft resistance in layer i; γ_{Rd} is a model factor; and the summation is performed for layers i = 1 to n.

It is essential for sheet pile walls that carry vertical load only the embedded length of the sheet pile wall below the formation or excavation level is considered to provide the necessary resistance (any benefit from soil friction above this level is ignored). For basement walls where a low level slab or prop supports the wall it is usual to consider both sides of the embedded portion of the sheet pile wall providing skin friction resistance. For embedded retaining walls carrying vertical loads where the embedded portion of the sheet pile is providing passive resistance to support the wall it is recommended to consider one side only below the formation level to provide skin friction resistance.

The characteristic base resistance of a pile ($R_{b,k}$) may be determined from:

$$R_{b,k} = \frac{A_b q_{b,k}}{\gamma_{Rd}}$$

where A_b is the gross cross-sectional area of the pile base; $q_{b,k}$ is the characteristic unit base resistance; and γ_{Rd} is a model factor.

Values of A_s and A_b that are suitable for designing sheet piles, H piles, and box piles subject to vertical loading depend on the degree of plugging of the section, see Section 6.7. However a cautious approach would be to ignore plugging effects when predicting carrying capacity.

According to the UK National Annex to Eurocode 7 - Part 1 [ii], the value of the model factor γ_{Rd} should be taken as 1.4., except when the calculated resistance has been verified by a load test (see Section 6.12.) taken to the calculated unfactored, ultimate load – in which case γ_{Rd} may be taken as 1.2. In design by calculation model factors are used however, in design by testing these are replaced by correlation factors, see Section 6.13.

Example:

The sum of the shaft resistance has been assessed as 1250 kN and the corresponding base resistance as 725 kN. It is not anticipated that the pile capacity will be verified from maintained load tests. Therefore the characteristic shaft resistance, $R_{s,k}$ = 1250/1.4 = 893 kN and the characteristic base resistance, $R_{b,k}$ = 725/1.4 = 518 kN.

To obtain the design resistance, R_d, further partial factors need to be applied as described in Section 6.4.

The principal difficulty is in assessing both q_s and q_b. It is accepted that there is no completely reliable method as both determining the operating parameters and the effects of installation make prediction complicated. Thus Eurocode 7 places greater emphasis on design based on static pile load tests, but this has severe drawbacks. For most situations design will be carried out using geotechnical calculations based on empirically derived and tested formulae.

The following paragraphs show methods of deriving the capacity for granular coarse grained soils; fine grained cohesive soils and for end bearing or penetration into rock.

6.5.1. Capacity resistance - Granular soils

Granular soils include gravels, sands and silts. For these soils the tendency for excess pore pressures to develop during driving and subsequent loading is small and thus effective stress methods of analysis are most appropriate.

Approaches that are based on in situ tests are of particular interest as they avoid the need to sample the ground and hence disturb it. This is particularly the case when assessing the capacity of piles in granular soils where it is virtually impossible to obtain undisturbed samples for laboratory testing.

6.5.2. Cone penetration test (CPT)

Possibly the most suitable method for determining the skin friction and end bearing of driven piles is that based on cone penetration test (CPT) results. However, this method is less reliable in dense gravels, marls and other hard soils as it is difficult for the cone to penetrate such materials.

The shaft and base resistances of piles installed in granular soils may be determined from the sleeve friction (f_s) and cone resistance (q_c) measured in cone penetration tests, as follows:

$$q_{s,k} = f_s \quad \text{and} \quad q_{b,k} = \frac{q_{c,1} + q_{c,2} + 2q_{c,3}}{4}$$

where

$q_{c,1}$ = the average cone resistance over two pile diameters below the pile base;

$q_{c,2}$ = minimum cone resistance over two pile diameters below the pile base and;

$q_{c,3}$ = average of minimum values of cone resistance over eight pile diameters above the pile base. This method was originally published by Fleming and Thorburn [iii].

Example:

The following readings were obtained from a cone penetration test:

$q_{c,1}$ = 4.65MPa;

$q_{c,2}$ = 3.86MPa;

$q_{c,3}$ = 3.27MPa.

Hence the characteristic base resistance is estimated as:

$$q_{b,k} = \frac{q_{c,1} + q_{c,2} + 2q_{c,3}}{4} = \frac{4.65 + 3.86 + 2 \times 3.27}{4} = 3.76 \text{ MPa}$$

6.5.3. Standard penetration test (SPT)

The shaft and base resistances of piles installed in granular soils may be determined from the blow count (N) measured in standard penetration tests, as follows:

$$q_{s,k} = 2N_s(\text{kPa}) \quad \text{and} \quad q_{b,k} = 400N_b(\text{kPa})$$

where N_s is the average blow count (per 300 mm penetration) over the embedded length of the pile and N_b is the predicted blow count (per 300 mm penetration) at the level of the pile base, determined from:

$$N_b = \frac{N_1 + N_2}{2}$$

where N_1 is the smallest blow count measured over two effective diameters below toe level and N_2 is the average blow count measured over ten effective diameters below the pile toe.

When the standard penetration test is conduced below the water table in fine sands and silts, the measured blow count should be reduced when its value exceeds 15, as follows:

$$N = 15 + \left(\frac{N_m - 15}{2} \right)$$

where N_m is the measured blow count. This method was originally published by Terzaghi and Peck [iv].

Example:

The following readings were obtained from a standard penetration test:
N_1 = 26 and N_2 = 34.

Hence the characteristic base resistance is estimated as:

$$q_{b,k} = 400\ N_b = 400 \times \left(\frac{N_1 + N_2}{2} \right) = 400 \times \left(\frac{26 + 34}{2} \right)$$

$$= 12 \text{ MPa}$$

6.6. Bearing capacity theory

6.6.1. Piles installed in granular soils

The shaft and base resistances of piles installed in granular soils may be determined from bearing capacity theory, as follows:

$$q_{s,k} = \left(\frac{N_q}{50} \right) \overline{\sigma'_{v,s}} \tan \varphi_{cv,k} \quad \text{and} \quad q_{b,k} = N_q \sigma'_{v,b}$$

where N_q is a bearing capacity factor based on the soil's characteristic angle of shearing resistance (φ'_k); $\sigma'_{v,s}$ is the vertical effective stress along the pile shaft; $\varphi_{cv,k}$ is the soil's characteristic constant volume (aka critical state) angle of shearing resistance; and $\sigma'_{v,b}$ is the vertical effective stress at the pile base. The bar over $\sigma'_{v,s}$ signifies the average value along the pile shaft is taken. This method was published by Fleming et al. [v].

In the equation above, the term $\varphi_{cv,k}$ is taken directly as the angle of interface friction between the pile and the soil. Although it does so for the design of retaining walls, Eurocode 7 - Part 1 does not specify a limit for the value of interface friction in the design of bearing piles.

Values of N_q can be obtained from various bearing capacity theories, that due to Berezantzev [vi] being very popular in the UK.

Angle of shearing resistance, φ'	Pile slenderness ratio, L/D				
	5	10	15	20	25
25.0	11.1	8.98	7.75	6.97	6.14
27.5	17.3	14.3	12.7	11.8	10.7
30.0	27.0	22.9	20.9	19.8	18.4
32.5	42.3	36.8	34.3	33.0	31.2
35.0	66.3	59.0	56.0	54.7	52.4
37.5	104	94.0	90.6	89.4	86.3
40.0	160	148	144	144	139

Table 6.6. Values of N_q derived from Berezantzev [vi].

The soil's angle of shearing resistance φ' can be assessed from the results of standard or cone penetration tests. Alternatively, it may be measured in the laboratory from direct shear tests or tri-axial tests.

Example:

The effective stress at the base of the pile $\sigma'_{v,b}$=156 kPa; the angle of shearing resistance of the bearing soil is 35°; the pile is 0.5 m in diameter and penetrates 5.0 m into the bearing stratum. Thus L/D = 10 and, from the Table 6.6., N_q = 59.

The characteristic base resistance of the pile is then estimated as:

$$q_{b,k} = N_q \sigma'_{v,b} = 59 \times 156 = 9.2 \text{ MPa}$$

The average effective stress, over the 5.0 m penetration, is $\sigma'_{v,s}$ = 131 kPa. The constant volume angle of shearing resistance $\varphi_{cv,k}$ = 30°.

The characteristic shaft resistance of the pile is then estimated as:

$$q_{s,k} = \frac{N_q}{50} \times \overline{\sigma'_{v,s}} \times \tan \varphi_{cv,k} = \frac{59}{50} \times 131 \times \tan 30° = 89.2 \text{ kPa}$$

6.6.2. Piles installed in fine or cohesive soils

Cohesive soils include clays and some fine silts. For these soils excess pore pressures are generated during installation and loading. The values of these excess pore pressures are difficult to ascertain and thus it is convenient to use the soil's undrained shear strength for calculation of resistance.

The ultimate capacity of a bearing pile in cohesive soils is a function of the undrained shear strength of the soil and its area in contact with the pile.

The shaft and base resistances of piles installed in fine soils may be determined from bearing capacity theory, as follows:

$$q_{s,k} = \alpha \overline{c_{u,s,k}} \quad \text{and} \quad q_{b,k} = N_c c_{u,b,k} \approx 9 c_{u,b,k}$$

where α is an adhesion factor; N_c is a bearing capacity factor; $c_{u,s,k}$ is the soil's average characteristic undrained shear strength along the pile shaft; and $c_{u,b,k}$ is the soil's characteristic undrained shear strength at the pile base. The bar over $c_{u,s,k}$ signifies the average value along the pile shaft is taken.

Values of α can be obtained from various sources, those due to Randolph and Murphy [vii] are becoming popular in the UK.

Angle of shearing resistance, φ'	c_u/σ'_v						
	0.25	0.3	0.4	0.5	0.75	1	2
17.5	0.84	0.76	0.66	0.59	0.48	0.42	0.35
20	0.89	0.82	0.71	0.63	0.52	0.45	0.38
22.5	0.95	0.87	0.75	0.67	0.55	0.47	0.40
25	1.00	0.91	0.79	0.71	0.58	0.50	0.42
27.5	1.00	0.96	0.83	0.74	0.61	0.52	0.44
30	1.00	1.00	0.87	0.77	0.63	0.55	0.46

Table 6.7. Values of α according to Randolph and Murphy [vii].

(c_u/σ'_v) is the ratio of undrained strength to effective overburden strength and may be assessed from laboratory tests or in situ measurements.

Example:

The average value of c_u/σ'_v over the shaft of a pile in clay has been determined from a series of laboratory tests as equal to 0.4; the characteristic angle of shearing resistance of the soil is 22.5°. From the Table 6.7., α = 0.75.
The average characteristic undrained strength along the shaft of the pile is $c_{u,s,k}$ = 120 kPa.

The characteristic shaft resistance of the pile is estimated as:

$$q_{s,k} = \alpha \times c_{u,s,k} = 0.75 \times 120 = 90\,\text{kPa}$$

Alternatively, the shaft resistance of piles installed in fine soils may be determined from effective stress theory, as follows:

$$q_{s,k} = \beta \times \overline{\sigma'_{v,s}} = K_s \times \tan\delta_k \times \overline{\sigma'_{v,s}}$$

where β is analogous to α; $\sigma'_{v,s}$ is the vertical effective stress along the pile shaft; K_s is an earth pressure coefficient for the shaft; and δ_k is the characteristic angle of interface friction. The bar over $\sigma'_{v,s}$ signifies the average value along the pile shaft is taken.

For normally consolidated clays, values of β vary between 0.22 and 0.28. However, for overconsolidated clays, installation effects and other factors make the prediction of β more difficult. This, coupled with the difficulties of predicting the pore pressure at any stage in the design life of the pile, makes the use of effective stress methods less popular.

6.7. End bearing and plugging effects

6.7.1. H-piles & box piles

As steel H-piles and open-ended box piles are driven into the ground so there is tendency for the section to become "plugged" with soil. This changes the effective cross-sectional area, A_b, for the purposes of assessing end bearing and the piles surface area, A_s, for assessing shaft resistance.

This situation arises where the soil does not shear at the pile/soil interface but away from the pile and a plug of soil forms at the base which is drawn down with the pile as it is driven. The various conditions are shown in Fig. 6.1. (typically for H piles).

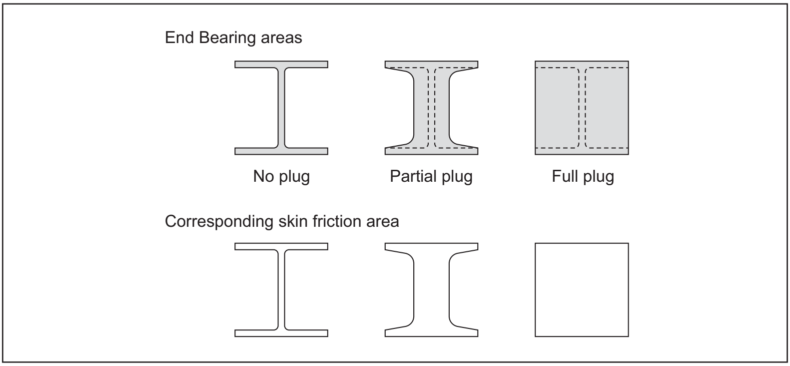

Fig. 6.1. Plugging effects and skin friction of N-Piles.

The methods suggested by Jardine et al. [viii] for assessing the potential for a plug to form and how this should be taken into account when calculating base and shaft resistance may be used or the following simple method adopted.

The shaft friction area (A_s) may be calculated assuming that no plug forms but when assessing the end bearing area (A_b), full plugging is assumed but a reduction factor of 0.5 for clay soils and 0.75 for sands is then introduced.

Example:

An HP 305 x 110 kg/m steel H pile is to be driven 12 m into medium strength clay. The pile's cross-section dimensions are h = 308 mm, b = 311 mm, t_w =15.3 mm, and t_f = 15.4 mm.

The shaft area of the pile is calculated as:

$$\begin{aligned} A_s &= (4b - 2t_w + 2h) \times L \\ &= (4 \times 0.311 - 2 \times 0.0153 + 2 \times 0.308) \times 12 \\ &= 21.95 \text{ m}^2 \end{aligned}$$

and the base area as:

$$A_b = \frac{(b \times h)}{2} = \frac{(0.311 \times 0.308)}{2} = 479 \text{ cm}^2$$

6.7.2. Sheet Piles

The carrying capacity for standard sheet piles is mainly based on shaft friction.

In case of sheet pile retaining wall, the skin area on the active earth pressure side is not used, see 2012's EAU-Recommendations [xii] (Chapter 8.2.5.6.6) . If the chosen pile length allows a deeper embedment than the theoretical base, this additional skin area on the active side may be adopted.

If wall friction angle is applied with $\delta_a = 0°$, full skin area can be applied.

Additional capacity can sometimes be utilised from end bearing.

The effective toe area for a continuous standard sheet pile wall is given in Fig. 6.2. For values of base area (sectional area) and skin area (surface area), refer to Chapter 1.

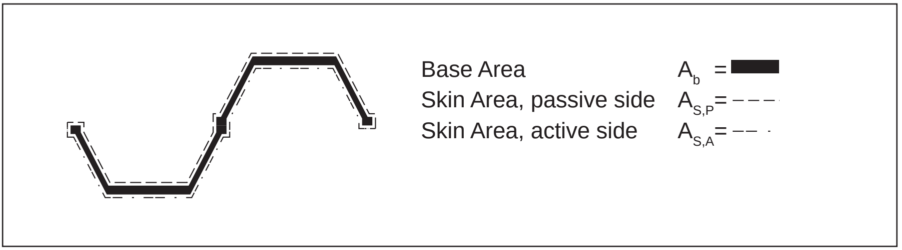

Fig. 6.2. Effective sheet pile area.

Plugging effects are possible, see EAU 2012 [xii] (Ch. 8.2.5.6.7), with U- and Z-sheet pile sections. Its formation depends on several parameters, such as section geometry, soil conditions, embedment ratio (depth/section width) as well as installation method.

An alternative design approach can be taken from French National application standard NF P 94-262 [xiii] for the implementation of Eurocode 7.

Depending on soil conditions, only shaft friction is considered, or a combination of shaft friction and end bearing, including eventual plugging effect according to given sheet pile geometry.

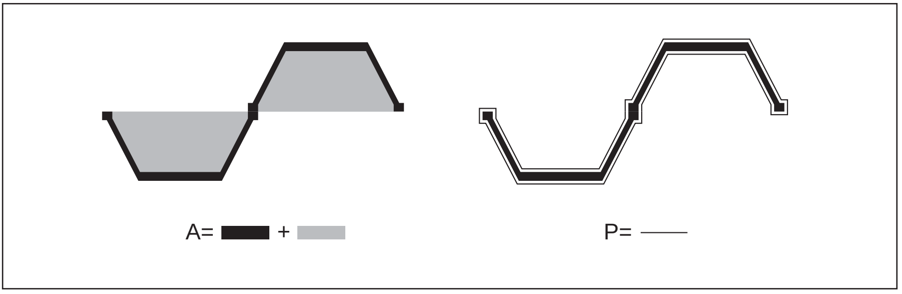

Fig. 6.3. Effective sheet pile area according to NF P94-262 [xiii].

6.8. Pile capacity from end bearing in rock

It is imperative that the pile section is designed using high steel grades or with pile shoes to accommodate dynamic forces required to penetrate or bear directly onto rock. The driving forces can be expected to be at least twice the structural capacity of the pile.

When rock or another suitably competent layer exists, steel piles can transmit the loads from the structure to the foundation in end bearing alone.

The ability of the rock on which the pile is founded to withstand the foundation loads must be determined by establishing the compressive strength of the strata (MPa) from site investigation.

Steel piles in high yield steel strength S430 MPa may be driven successfully without too much damage into weathered or weak rock up to 30 MPa compressive strength. Techniques are explained in Chapter 11.

For carrying high loads bearing piles and tubular primary piles in combined walls may require toe reinforcement to enable penetration into weak rock to prevent damage detrimental to the carrying capacity.

Dynamic testing is a key method to assess carrying capacity for testing steel piles driven into rock.

6.9. Design by testing

Steel Piles are particularly useful for trial driving and testing. They are removeable and re-useable by extraction if necessary so it is possible to allow for using this method at preliminary design stage to check options and suitability for a project using sections readily available from stock.

Design by testing involves using the results of static load, dynamic impact, pile driving formulae or ground tests to define the total pile resistance.

Except where ground tests profiles are used this approach only works where trial piles are installed and the results of tests on these piles are used to design the working piles. Traditionally, on smaller contracts, pile testing is avoided by using a large factor of safety on the calculated capacity. Where tests are performed on working piles these approaches only ensure (or otherwise) that the working piles meet the requirements of the standard.

For most contracts, design by static load testing is generally impractical as there is insufficient lead time between the main piling works and the test programme. Preliminary tests are rarely performed on piles with similar diameters and lengths, making it difficult to derive a sensible mean test result. In many cases, the ultimate load from a test is obtained by extrapolation of the load-displacement curve, adding further to the uncertainty in any calculated mean.

When designing piles by testing, the characteristic resistance of a pile ($R_{c,k}$) may be determined from the smaller of:

$$R_{c,k} = Min\left\{\frac{(R_{c,m})_{mean}}{\xi_1}, \frac{(R_{c,m})_{min}}{\xi_2}\right\}$$

where $(R_{c,m})_{mean}$ is the mean value of the pile's resistance measured in a number of tests; $(R_{c,m})_{min}$ is the minimum value measured in those tests; and ξ_1 and ξ_2 are correlation factors applied to these mean and minimum values, respectively.

In design by testing correlation factors are used instead of the model factor presented in Chapter 6.4. To obtain the design resistance, R_d, further partial factors need to be applied as described in Chapter 6.13.

6.10. Pile groups

Where piles are installed in groups to support a structure, the performance of the group is dependent upon the layout of the piles and may not equate to the sum of the theoretical performance of individual piles in the group.

A general rule is that the centre to centre spacing of the pile should not be less than four times the maximum lateral dimension of the pile section. However a check of the settlement of the overall group should be made.

For box piles comprising individual piles connected by special connectors such as omega bars, where the pile joints are not fully welded , the resistance of the box piles shall be calculated on the basis of the section properties of the individual elements, as a conservative approach. Alternatively; reduction factors applied in accordance with National Annex of Eurocode 3 - Part 5 may be applied to take into account partial shear transfer at the unwelded interlocks.

6.11. Negative skin friction or downdrag

As well as carrying loads from the structure, piles can be subject to actions that arise from movement of the ground in which they are installed. This phenomenon is known as “downdrag” when it involves the ground consolidating, thus bringing additional downwards force onto the pile shaft. Ground movements in other directions (e.g. upwards or horizontal) can induce heaving, stretching, or other displacement of the pile.

Such displacements may be treated in one of two ways: either as an indirect action in a soil-structure interaction analysis; or as an equivalent direct action, calculated separately as an upper bound value.

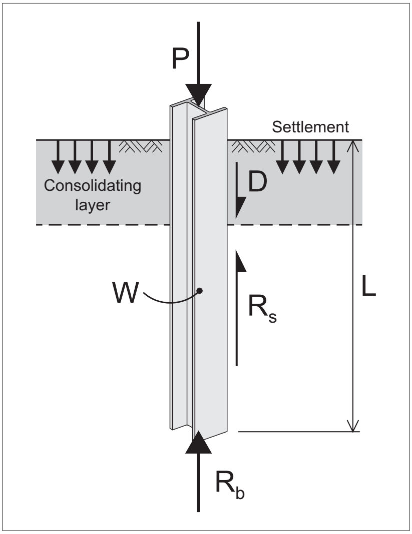

Fig. 6.4. Negative skin friction.

Consider a pile installed through a superficial layer as shown in Fig. 6.4. Consolidation of the layer (owing, for example, to fill being placed upon it) will occur after the pile has been installed, resulting in additional loading being applied to the pile.

Soil-structure interaction analysis allows the "neutral" depth (where the settlement of the consolidating matches that of the pile under load) to be determined, albeit approximately. In many situations, the effort involved in this type of analysis is outweighed by uncertainties in obtaining suitable ground parameters for use in the analysis.

It is more usual to account for downdrag by inclusion of an appropriate upper-bound action. The characteristic vertical compressive action $F_{c,k}$ applied to the pile is then:

$$F_{c,k} = P_{G,k} + W_{G,k} + D_{G,k}$$

where $P_{G,k}$ and $W_{G,k}$ are as defined above and $D_{G,k}$ is the characteristic downdrag acting on the pile (a permanent action). Note that when downdrag is included in this equation, any variable actions may be ignored (hence the absence of P_{Qk}).

Typically, the consolidating layer is cohesive and downdrag is calculated from:

$$D_{G,k} = \alpha \times c_{u,k} \times A_{s,D}$$

where α = an appropriate adhesion factor, $c_{u,k}$ = the clay's characteristic undrained strength, and $A_{s,D}$ = the surface area of the pile shaft in the consolidating layer. In selecting values for α and $c_{u,k}$, it is important to choose upper values so as to maximize the value of $D_{G,k}$.

Example:

An end bearing tubular steel pile penetrates through recently placed fill overlying low strength clay, $c_{u,k}$ = 25 kPa and into dense gravel from which the resistance to vertical actions is obtained. The thickness of the consolidating clay layer is 5 m, the diameter of the pile is 500 mm, and α = 1.0 is assumed. The downdrag action, ignoring any contribution from the overlying fill, is then:

$$D_{G,k} = \alpha \times c_{u,k} \times A_{s,D} = 1.0 \times 25 \times (\pi \times 0.5 \times 5) = 196 \text{ kN}$$

Although Eurocode 7 suggests downdrag should be considered in ultimate limit states, strictly speaking it is only relevant to serviceability limit states. Downdrag results in additional settlement of piles, which needs to be compared with the limiting total and differential settlements defined for the structure. In rare cases when piles are mainly end-bearing, downdrag may result in excessive compressive loads in the piles, leading to end-bearing failure in the ground or structural failure of the pile.

6.12. Set up effects

The properties of the soil immediately adjacent to a driven pile are changed by the process of forcing the pile into the ground, giving rise to a phenomenon called set up.

Set up is the time interval during which the soil recovers its properties after the driving process has ceased. In other words the load capacity of an individual pile will increase with time after the pile has been driven. In granular soils this can be almost immediate but in clays this can take days, or months for some high plasticity clays.

In granular soils this change can be in the form of liquefaction caused by a local increase in pore water pressure due to the displacement by the pile. In clays it can be due to the remoulding of the clay in association with changes in pore water pressures.

The load capacity of the piles should be verified by testing and if sufficient time for set up to occur is not available before the pile is loaded then its effects should be taken into account in the design. The important point to remember is that in clay soils the capacity of the piles will tend to improve over time.

6.13. Testing the load capacity of steel bearing piles

The main categories of tests that are commonly used today to determine the load capacity of steel bearing piles are as follows. All testing should follow requirements of Eurocode 7 - Part 1 [i].

6.13.1. Static testing

Existing are e.g. Maintained Load Test and Constant Rate of Penetration Test.

Both these tests use similar apparatus and in both cases the test load is applied by hydraulic jack(s) with kentledge or tension piles/soil anchors providing a reaction.

Modern pile pressing systems provide this information as part of the installation process. The amount of force required to install the pile can be used to gauge the likely capacity of the pile.

Eurocode 7 - Part 1 distinguishes between static load tests carried out on piles that form part of the permanent works (“working piles”) and on piles installed, before the design is finalized, specifically for the purpose of testing (“trial piles” – or what, in the UK, are commonly termed “preliminary piles”). Trial piles must be installed in the same manner and founded in the same stratum as the working piles.

The disadvantage of static testing is relative cost and time.

6.13.2. Ground tests

This approach is to be used where the pile’s characteristic resistance may be derived directly from in situ tests and where it is likely that this can be achieved for several locations across the site. Typically this may be done from cone penetration tests, dilatometer tests, pressuremeter tests or standard penetration tests. Typical methods for assessing resistance from the cone penetration and standard penetration tests are given in Sections 6.5.2. and 6.5.3 .

When designing piles on the basis of ground test results, the characteristic resistance of a pile ($R_{c,k}$) should be determined from the smaller of:

$$R_{c,k} = \left(R_{b,k} + R_{s,k}\right) = \frac{R_{b,cal} + R_{s,cal}}{\xi} = \frac{R_{c,cal}}{\xi} = Min\left\{\frac{\left(R_{c,cal}\right)_{mean}}{\xi_3}, \frac{\left(R_{c,cal}\right)_{min}}{\xi_4}\right\}$$

where $(R_{c,cal})_{mean}$ is the mean value of the pile’s resistance calculated from a number of ground tests; $(R_{c,cal})_{min}$ is the minimum value calculated from those tests; and δ_3 and δ_4 are correlation factors applied to these mean and minimum values, respectively.

Number of ground tests	1	2	3	4	5	7	10
ξ_3	1.55	1.47	1.42	1.38	1.36	1.33	1.30
ξ_4	1.55	1.39	1.33	1.29	1.26	1.20	1.15

Table 6.8. Correlation factors ξ_3 and ξ_4 given in the UK NA to BS EN 1997-1 [ii].

Note: Divide by 1.1 when piles can transfer loads from “weak” to “strong” piles.

Example:

A series of seven cone penetration tests gave the following pile capacities: 461, 478, 512, 483, 452, 493, and 488 kN.

The piles in the group are able to transfer loads from weak to strong piles, therefore ξ_3 = 1.33/1.1 = 1.21 and ξ_4 = 1.2/1.1 = 1.10.

Thus the characteristic resistance is calculated as the lesser of:

$$R_{k,3} = \frac{\overline{R_m}}{\xi_3} = \frac{\sum R_m}{n \times \xi_3}$$

$$= \frac{461 + 478 + 512 + 483 + 452 + 493 + 488}{7 \times 1.21}$$

$$= 398\,\text{kN}$$

and

$$R_{k,4} = \frac{R_{m,min}}{\xi_4} = \frac{452}{1.10} = 411\text{kN}$$

Therefore R_k = 398 kN.

6.13.3. Dynamic testing

This method particularly suits the testing of resistance and capacity of steel sheet piles and bearing piles and has the advantage of compatibility with driving equipment already mobilised to install the piles and yields quick efficient data.

The test pile is instrumented with strain transducers and accelerometers and is struck with the piling hammer. The force and velocity data are recorded and analysed. Using this data, methods are available that give an on-site estimate of the pile bearing capacity, although more rigorous and detailed analysis of the recorded data can be performed using a computer program such as the Case Pile Wave Analysis Program (CAPWAP). Using the program, an engineer can determine the pile bearing capacity, in terms of shaft resistance and toe resistance and the distribution of resistance over the pile shaft.

Eurocode 7 - Part 1 [i] allows the compressive resistance of a pile to be estimated using dynamic load tests, provided the tests are calibrated against static load tests on similar piles, with similar dimensions, installed in similar ground conditions. These requirements limit the applicability of dynamic load tests for design purposes – but they remain useful as an indicator of pile consistency and a detector of weak piles.

When designing piles on the basis of dynamic impact tests, the characteristic resistance of a pile ($R_{c,k}$) should be determined from the smaller of:

$$R_{c,k} = \frac{(R_{c,m})_{mean}}{\xi_5} \; or \; \frac{(R_{c,m})_{min}}{\xi_6}$$

where $(R_{c,m})_{mean}$ is the mean value of the pile's resistance measured in a number of tests; $(R_{c,m})_{min}$ is the minimum value measured in those tests; and ξ_5 and ξ_6 are correlation factors applied to these mean and minimum values, respectively.

Number of dynamic impact tests	≥ 2	≥ 5	≥ 10	≥ 15	≥ 20
ξ_5	1.94	1.85	1.83	1.82	1.81
ξ_6	1.90	1.76	1.70	1.67	1.66

Table 6.9. Correlation factors ξ_5 and ξ_6 given in the UK NA to BS EN 1997-1 [ii].

Note: Multiply by 0.85 when using signal matching.
Multiply by 1.1 when using pile driving formula with measurement of quasi-elastic pile head displacement during impact (or by 1.2 without).

6.13.4. Pile driving formulae

Direct use of traditional pile driving formulae is less suitable for modern pile driving equipment and is not recommended when superior methods of assessing pile capacity by dynamic testing is available. Pile driving formulae may not apply to sheet pile walls and combined system walls due to effects of interlock connections to neighboring piles and inconsistent behaviour of the compression wave and pile length.

6.14. Welding of steel piles

For all types of steel piles it is very important to recognise the quality and grades of the steel components for fabricated welding applications and assigning correct specifications and procedures especially for applications to carry vertical load. A suitable testing specification should also be assigned.

Steel bearing piles can be readily welded on-site by suitably qualified welders.

Splice welds to connect an extra length to a pile, either before or after an initial installation to increase the overall pile length. Welded splice joints to sheet piles should not be executed in positions of high bending moment. Guidance is provided in BS EN 12063 [ix].

All welding should be carried out to in accordance with specified requirements of BS EN 12063 and the latest relevant welding standards. ICE SPERW 2nd Edition 2010 gives further guidance.

6.15. Execution of bearing piles

The procedures for the installation of sheet piles and displacement piles are covered by execution standards EN 12063 for sheet pile walls [ix] and EN 12699 for displacement piles [x].

Execution of steel bearing piles necessitate similar plant and methods to sheet piles and is discussed in Chapter 11.

6.16. Driving shoes

Driving shoes may be specified on bearing piles where there is perceived risk of damaging the pile section where driving to achieve penetration into hard strata or rock may be anticipated. Driving shoes are usually specified when using comparatively weak materials such as timber or concrete but for steel bearing piles shoeing may not be necessary. The steel grade may also be sufficiently increased by the designer without needing to have a specially reinforced shoe at the toe of the pile. For standard sheet piles it is recommended to simply increase the steel grade up to S430GP and use appropriate technique as described in Chapter 11.

Box piles and primary elements of combined walls may be reinforced by welding thickening plates on the ends of the piles. H piles and box piles are usually reinforced on the inside perimeter face of the piles.

For pipe piles consideration has to be given to the design of the reinforcement and location. There are many types of shoes and fittings for pipe piles if required from reinforced plates for compression piles to thicker open ended pipes extended to the toe of the piles for tension and combi wall pipes.

Piles should never be overdriven into rock, it is essential not to damage the pile; driving shoe is for purposes of limiting potential damage and not for ensuring depth of penetration in hard strata.

Guidance for difficult driving is given in Chapter 11.

References:

[i] *EN 1997, Eurocode 7 – Geotechnical design, Part 1: General rules, European Committee for Standardization, Brussels.*

[ii] *NA to BS EN 1997-1: 2004, UK National Annex to Eurocode 7: Geotechnical design – Part 1: General rules, British Standards Institution, London.*

[iii] *Fleming, W. G. K., and Thorburn S. (1983) 'State of the art report on recent piling advances', Proc. Conf. on Piling and Ground Treatment, London: Thomas Telford.*

[iv] *Terzaghi, Karl, and Ralph B. Peck (1948) Soil mechanics in engineering practice, New York: John Wiley & Sons, Inc.*

[v] *Fleming, W.G.K., A.J. Weltman, M.F. Randolph, and W.K. Elson (1992) Piling engineering (2nd Edition), Glasgow: Blackie & Son Ltd.*

[vi] *Berezantzev, V.G., V.S. Khristoforov, and V.N. Golubkov (1961) "Load bearing capacity and deformation of piled foundations", 5th Int. Conf. Soil Mech. Found. Engng. Paris, pp 11-15.*

[vii] *Randolph, M. F., and Murphy, B. J. (1985) "Shaft capacity of driven piles in clay", Proceedings 17th Offshore Technology Conf. Houston, Texas, pp 371-378.*

[viii] *Jardine, R.J., F. Chow, R. Overy, and J. Standing (2005) ICP design methods for driven piles in sands and clay, London: Thomas Telford Publishing.*

[ix] *EN 12063, Execution of special geotechnical work – Sheet pile walls, European Committee for Standardization, Brussels.*

[x] *EN 12699, Execution of special geotechnical work – Displacement piles, European Committee for Standardization, Brussels.*

[xi] *BS EN 10219, Cold formed welded structural hollow sections of non-alloy and fine grain steels.*

[xii] *EAU 2012, Recommendations of the Committee for Waterfront Structures, Harbours and Waterways, Berlin, 2012. (Ernst & Sohn).*

[xii] *NF P94-262, Justification of geotechnical work - National application standards for the implementation of Eurocode 7 - Deep foundations. 2012.*

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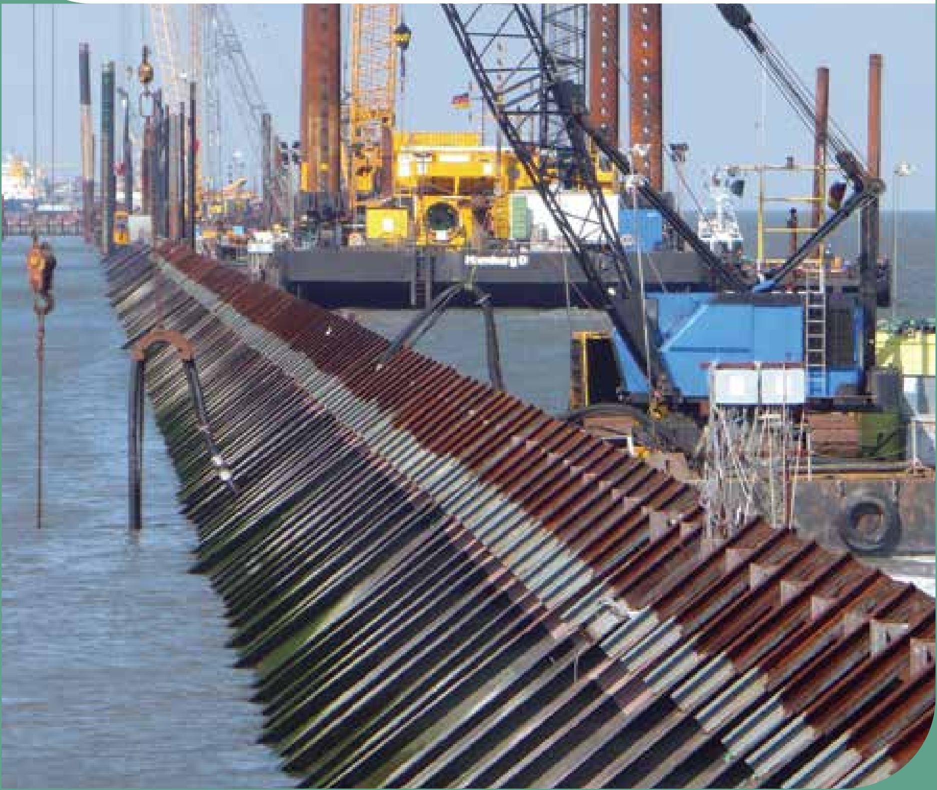
7 | Design of anchorages and tieback systems



Chapter 7 - Design of anchorages and tieback systems

Contents

7.1. Types of support system

Where it is not possible to use a simple cantilever retaining wall, support is required either at one or at several levels down the pile. This is usually achieved by using props or tied anchorage systems.

The support for a sheet pile wall may be provided by internal props or loading frames or by some form of anchorage into the ground behind the wall.

The anchorage system should be designed to provide sufficient resistance to movement under serviceability limit state conditions and sufficient resistance to satisfy ultimate limit state loads in the anchorage.

Typically an anchorage system is either an anchor or tie-rod secured to a dead-man (a system of shorter sheet piles, raking piles, concrete block, ...) or grouted anchorage system (prestressed).

Steel pre-stressed ground anchors should be designed according to EN 1537 [ii]. The method is briefly described in Chapter 7.5., but the detailed design is outside the scope of the Piling Handbook.

7.2. Deadman anchorages – balanced anchorages

Deadman anchorages can be formed as discrete units or as a continuous wall. The anchorage may comprise a secondary sheet pile wall, a laterally loaded pile or a concrete block. The anchorage in these systems is usually positioned at a minimum depth above the water table such, that the net passive pressure resistance is balanced at the tie bar level of the anchorage. Such anchorages are called balanced anchorages. The anchorages may be placed lower than the minimum level required and would in such cases have a greater resistance to adverse conditions affecting sliding or slip failure on stability of the whole system.

There is no specific guidance in Eurocode 7 - Part 1 [i] for the design of deadman anchorage systems. Therefore, it is assumed that components of a deadman system should be designed in accordance to the relevant Eurocodes using design actions derived as described in Chapters 4 and 5.

7.3. Location of anchorage support

For an anchorage to be effective it must be located outside the potential active failure zone developed behind a sheet pile wall - shown by the dashed lines in Fig. 7.1.

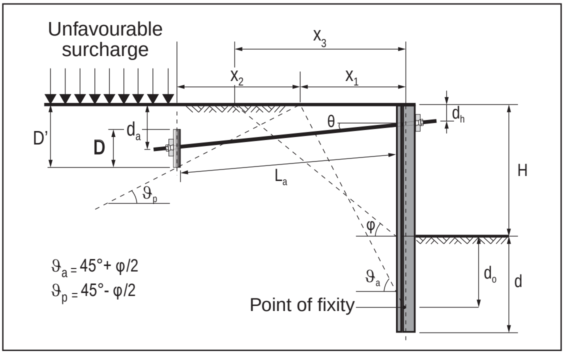


Fig. 7.1. Location of the anchorage support (balanced anchor wall).

For embedment depth d_O the anchor's free length L_a should satisfy:

$$L_a \geq \frac{x_1 + x_2}{\cos\theta} \text{ and } L_a \geq \frac{x_3}{\cos\theta}$$

in which the dimensions x_1, x_2, x_3 and θ are given by:

$$x_1 = (H + d_O) \times \tan\left(45° - \frac{\varphi}{2}\right)$$

$$x_2 = \frac{d_a + \frac{D}{2}}{\tan\left(45° - \frac{\varphi}{2}\right)}$$

$$x_3 = \frac{H}{\tan\varphi}$$

$$\theta = \text{asin}\left(\frac{d_a - d_h}{L_a}\right)$$

where φ is the soil's angle of shearing resistance and the dimensions H, d_O, d_h, d_a and D are defined in the sketch above.

And:

- $d_O = d$ for free earth support;
- $d_O = 3d/4$ for fixed earth support.

The anchorage's capacity is also impaired if it is located in unstable ground or if the active failure zone prevents the development of full passive resistance of the system.

If the anchorage is located within (x_1 + x_2), its resistance is reduced owing to intersection of the active and passive failure wedges shown. Although the theoretical reduction in anchor capacity may be determined analytically, it is better to lengthen the anchorage if at all possible.

The resistance of a deadman anchorage may be derived from the net passive resistance (passive minus active) in a similar manner to the main wall using the worst conceivable combination of circumstances. Wall friction should only be taken into account when deriving the earth pressure coefficients if the designer is confident that it can be realised under all loading conditions – the conservative approach is to ignore it. However, the effect of variations in the ground water level on soil strength properties and the application of surcharge loading to the active side of the anchorage only should be included to maximise the disturbing and minimise the restoring actions.

7.3.1. Balanced anchorages

The design of balanced anchorages assumes that the resistance afforded increases with depth below the ground surface giving a triangular pressure distribution. The top of the anchorage is assumed to be at a depth below the ground surface equal to one third of the overall depth to its toe, i.e.:

$$d_a - \frac{D}{2} = \frac{1}{3}\left(d_a + \frac{D}{2}\right) \Rightarrow d_a = D$$

The tie rod or tendon is placed such that it connects with the anchorage at two thirds of the overall depth to the toe (on the centre line of the anchorage element). This arrangement ensures that the tie rod force passes through the centre of passive resistance.

The entire passive wedge developed in front of the anchorage, including that above the top of the deadman unit, is effective in providing resistance.

When the design is based on the provision of discrete anchorage units, an additional force equal to that required to shear the wedge of soil in front of the anchorage from adjacent soil at each side can be added to the passive resistance to give the total anchorage resistance.

The additional resistance resulting from shearing of the soil can be calculated using the following equations:

$$\Delta R = \frac{1}{3}\gamma\left(d^a + D/2\right)^3 \times K_{ah} \times \tan\left(45^\circ + \frac{\varphi}{2}\right) \times \tan\varphi$$

for cohesionless soils and:

$$\Delta R = c_u\left(d_a + D/2\right)^2$$

for cohesive soils, where φ and c_u are the soil's angle of shearing resistance and undrained strength, respectively.

In the case of an anchorage in cohesive soil, the top metre of soil should be ignored if tension cracks are likely to develop parallel to the tie rods.

The maximum resistance that can be developed in the soil is that resulting from adoption of a continuous anchorage, so it is essential that a check is made to ensure that the resistance provided by a series of discrete anchorages does not exceed this figure.

7.3.2. Cantilever or fixed anchorages

Cantilever anchorages may be considered where good soil is overlain by a layer of poor material. This type of anchorage can be designed in the same manner as a cantilever wall where the piles must be driven to sufficient depth in a competent stratum to achieve fixity of the pile toes. The earth pressures can be assessed using conventional methods, but an additional load is introduced to represent the tie rod load and the whole system is then analysed to determine the pile length required to give rotational stability about the pile toe under the applied loads. An additional length of pile is then added to ensure that toe fixity is achieved. A check must be made to verify that the horizontal forces acting on the anchorage are in equilibrium.

The bending moments induced in this type of anchorage are generally large. Raking piles can sometimes be an economic alternative to this type of anchorage.

7.4. Stability - verification of stability at lower failure plane

For the design of cantilever walls, anchored sheet pile walls or double pile wall structures the stability of the system at the lower failure plane requires verification. Although the nature of the failure plane in theory may be complex, for the stability check, it is sufficient to take a uniform line from the deepest lowest point of zero shear on the main wall to an assumed high point of balanced support in the anchor wall. The verification of the stability is usually calculated using Kranz method (ref. Recommendations of the Committee for Waterfront Structures, Harbours and Waterways, see EAU 2012 [vi] (Chapter 8.4.9.1.). This method can also be adapted for soils with multiple soil layers.

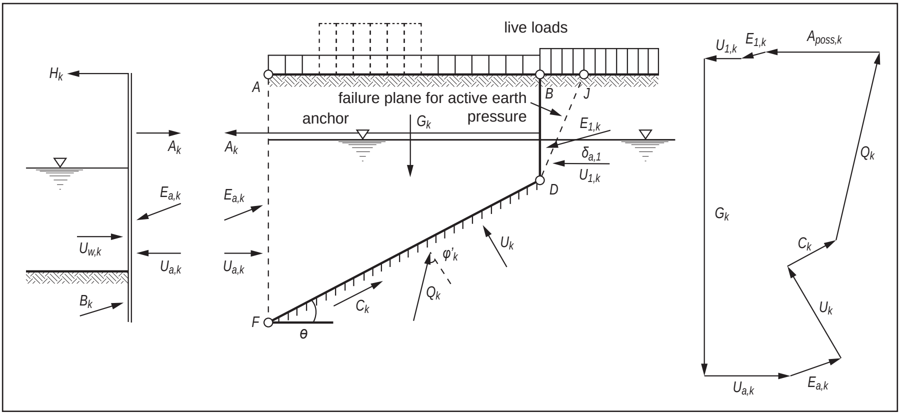


Fig. 7.2. Simplified diagram showing the Kranz failure plane.

7.5. Pre-stressed anchorages (Ground anchors)

Ground anchors are usually considered where suitable competent strata occur, possibly below the main sheet pile wall toe level, to provide anchorage for the system by drilling for the anchor at an angle from a position near the top of the retaining wall. Steel sheet piles are particularly suited for this method because a direct connection to the sheet pile wall is practical to construct enabling high tension forces to be accommodated by the sheet pile section in a location of low bending stress.

A pre-stressed grouted anchorage system consists of a tendon, either bar or strand, which is grouted over the anchor bond length, to transfer the tension load into the soil. The part of the tendon between the wall and the anchor bond length is left un-grouted to ensure the load transfer occurs beyond the potentially unstable soil mass adjacent to the wall. The installation of these anchors is usually carried out by specialist contractors from whom further information may be obtained.

Guidance on the design of grouted anchorages is currently split between Eurocode 7 - Part 1 [i] and EN 1537, the execution standard for ground anchors [ii]. Unfortunately some of this guidance is contradictory, as discussed in [iii]. Furthermore, EN 1537's testing requirements for ground anchors overlap the planned scope of EN ISO 22477-5, the testing standard for anchorages [iv]. CEN's Technical Committees are seeking to resolve the various issues to provide a self-consistent set of documents covering grouted anchorage design and construction.

The recommended method in Eurocode 7 - Part 1 for defining the characteristic pull-out resistance of a grouted anchorage is based on testing. Three types of testing are defined: investigation tests; suitability tests; acceptance tests.

Investigation tests are performed before working anchorages are installed, to establish the anchorage's ultimate pull-out resistance in the ground conditions at the site, to prove the contractor's competence and to prove novel types of ground anchorage. Investigation tests should be carried out when anchorages have not previously been tested in similar ground conditions or if higher working loads than previously tested are anticipated.

Suitability tests are normally carried out on a selected number of anchorages to confirm that a particular anchor design is adequate. Their intent is to examine creep characteristics, elastic extension behaviour and load loss with time. Anchorages subjected to suitability tests may be used as working anchorages.

Acceptance tests must be carried out on all working anchorages to demonstrate that a proof load P_p can be sustained, to determine the apparent tendon free length, to ensure the lock-off load is at its design level, and to determine creep or load loss characteristics under serviceability conditions.

The characteristic pull-out resistance of a grouted anchorage $R_{a,k}$ is the lowest of the following:

- bond resistance between the grout and ground (external resistance, $R_{e,k}$);
- bond resistance between the grout and tendon (internal resistance, $R_{i,k}$);

- tensile capacity of the tendon ($P_{t,k}$);
- capacity of the anchor head.

On the basis of acceptance tests the characteristic pull-out resistance $R_{a,k}$ may be taken as the proof load P_p and the design resistance $R_{a,d}$ is given by:

$$R_{a,d} = \frac{R_{a,k}}{\gamma_a}$$

with:

$R_{a,k}$ anchor's characteristic pull-out resistance;

γ_a partial safety factor.

Partial safety factors are given in the EN 1997-1 [i] (Table A.12.). Further information about safety values can be found in the National Annexes. If applicable, also other standards shall be respected, e.g. EN 1537 [ii].

Due to the current uncertainty, UK designers should ensure that the designs also comply with the guidance in BS 8081: 2015 [v].

7.6. Props and raking supports

For bottom up construction it is preferable to use an anchorage system as this reduces access constraints within the excavation. However this is often not possible due to:

- access restrictions;
- effects on adjacent structures;
- obstructions, services, infrastructure in the ground;
- way-leave requirements;
- when installing anchors beneath neighbouring properties.

For top down construction or where restrictions make anchorage systems unsuitable internal propping will be required. This may take the form of:

- use of concrete floor levels;
- strutting frames;
- raking props.

As for anchorage systems and ground anchors, internal props need to be designed using the relevant Eurocodes to accommodate design actions derived as described in Chapters 4 & 5. For conception and design, see Chapter 5 Sections 5.15. and 5.16.

7.7. Progressive collapse

Eurocode 7 and Eurocode 3 - Part 5 [viii] requires the designer to take into account the design situation for the event of loss of a support prop or anchor in order to prevent catastrophic progressive collapse. Further guidance for checking against progressive failure is discussed in section 5.15.4.2.

7.8. Detailing and components of anchorage systems

Combinations of various components can be used to support the main retaining wall and anchorage system. Walings are used to distribute the structural loading from props and anchorage systems to the sheet pile retaining wall. The wall may be tied back using rolled and threaded steel tie rods, ground anchors or raking piles. The waling is usually connected to steel walls using anchor bolts.

Design resistance of the sheet piles is required to take into account the configuration and details of connections and the transmission of forces from anchors and connections to the flanges and webs of the sheet pile section. Guidance is given in Chapter 8.13.

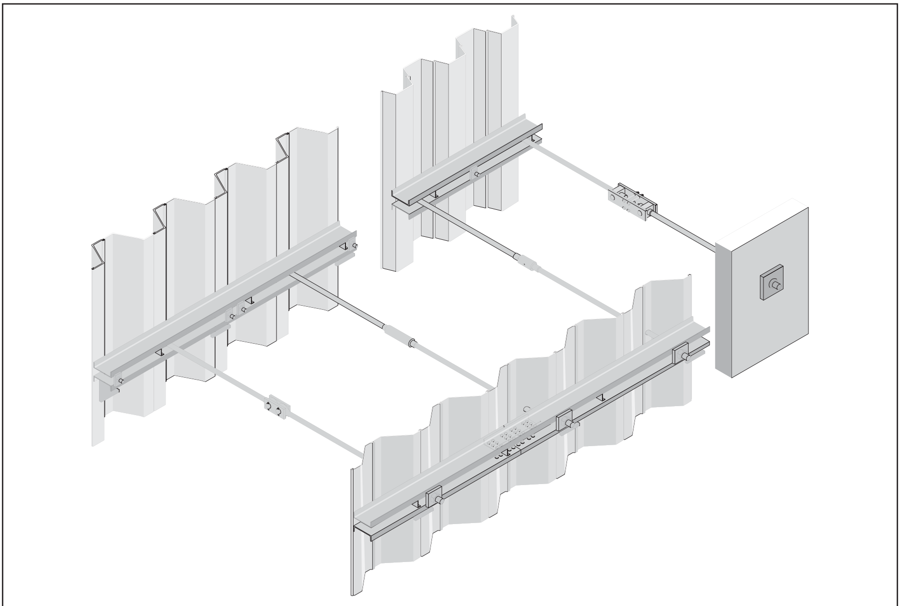


Fig. 7.3. Typical components of sheet pile wall anchorage system.

General arrangement of the sheet piles and tie bar connections as well as fittings may be different to take into account pile shape and section, position of the waling and the designer's preference for connection of the ties – direct to the waling or through the sheet pile.

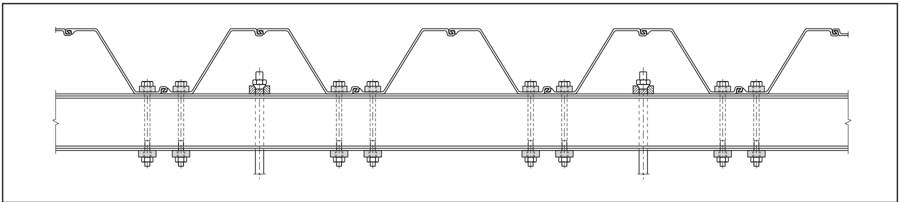


Fig. 7.4. Typical detail for the front wall using AZ piles with tie bar connected to the waling and waling bolted to sheets.

A benefit of this arrangement is to protect the tie bar from more severe exposure on the outside of the wall. Two anchor bolt connections at every pile waling connection are necessary and usually have a forged head to protect the thread. The integrity of the sheet pile wall is dependent on the design of the waling for durability for this method and attention to detail is required for splicing of the walings for continuous design.

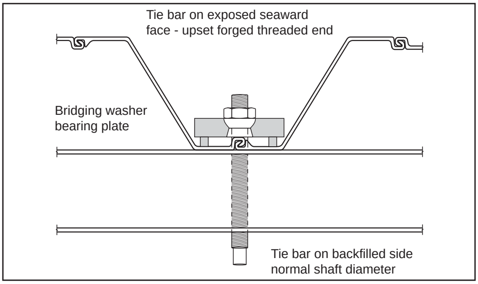


Fig. 7.5. Typical detail for the front wall connection using AZ piles and tie bar jointed through the sheet piles.

This traditional method for connecting of the tie bars to the sheet piles in the UK has the benefit that catastrophic failure of the wall is not dependent on the durability or damage to the waling. The ends of the tie bars, where exposed, can be designed with upset forged ends thicker than the shaft to suit durability requirements and also capped for protection if required.

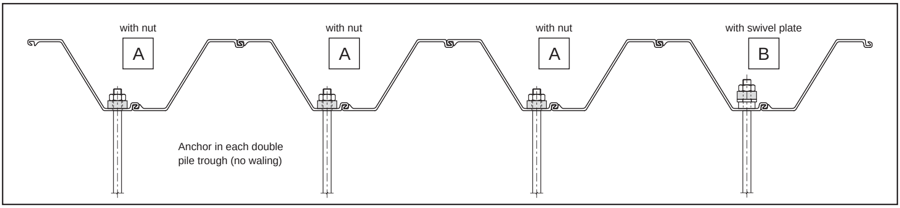


Fig. 7.6. Typical detail for the front wall connection using AZ piles and the eccentric anchorage method (without walings).

The principal benefit for the eccentric anchoring method is to utilize smaller ties without a waling bringing the tie bars at closer centres.

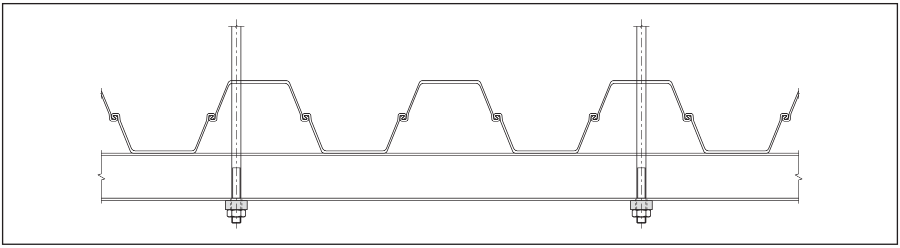


Fig. 7.7. Typical detail for continuous U-pile anchorage – waling behind the sheet piles.

The benefits for fixing the waling at the back of the anchor wall may make the need for detailing anchor bolts unnecessary. But good workmanship is required to ensure adequate bearing of the waling to the pans of the sheet piles. It is not necessary for the ties to coincide with the pans of the sheet piles provided that the waling has correct bearing.

Note: a safe method for fixing the tie bars through and behind the sheet piles is required when utilising this detail if excavation at depth is necessary.

7.9. Walings

Walings usually comprise two rolled steel channel sections placed back to back and spaced to allow the tie rods to pass between the channels. This spacing must allow for the diameter of the tie rod and the thickness of any protective material applied to the rod, and take into account any additional space required if the tie rods are inclined and will need to pass between the walings at an angle.

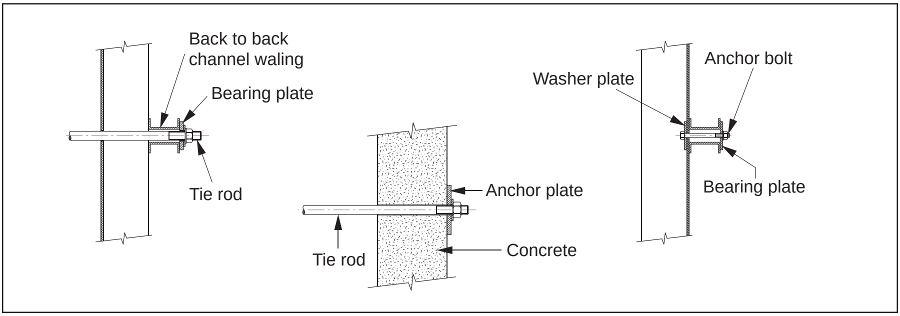


Fig. 7.8. Cross sections through main wall walings.

For high modulus walls and cofferdams the walings can comprise fabricated twinned beams and column sections in conjunction with tie bars, ground anchors and structural props for support.

The walings may be fixed either at the back or front of the retaining wall. For temporary works temporary walings are normally provided at the front or excavated side of the wall – this enables straightening using ties and is convenient to install as well as remove and minimize holing of the sheet piles by minimizing fixings.

For permanent works the waling is usually positioned behind the wall for protection and durability issues. It is necessary to use short anchor bolts and plates at every point of contact between the piles and the waling to connect them together. Placing the waling in front of the wall eliminates the need for connection bolts and this arrangement is therefore more economical.

Where the waling is connected behind the wall, if the tie bar is anchored directly to the waling, then the waling must be designed for durability and loading effects if one tie bar fails and the spacing doubles – bending moment increases proportionally to the spacing squared, but a plastic analysis may be envisaged.

Splices should be located at a distance of approximately one quarter of the tie rod spacing from a tie rod location as this will be close to the position of minimum bending moment in the waling, see Fig. 7.9. (zero point). The walings should be ordered longer than the theoretical dimensions to allow for any discrepancies or creep which may develop in the wall as the piles are installed, one end only of each length being pre-drilled for splicing (if the splice is to be achieved by bolting). The other end should be plain for cutting and drilling on site, after the actual length required has been determined by measurement of the driven piles.

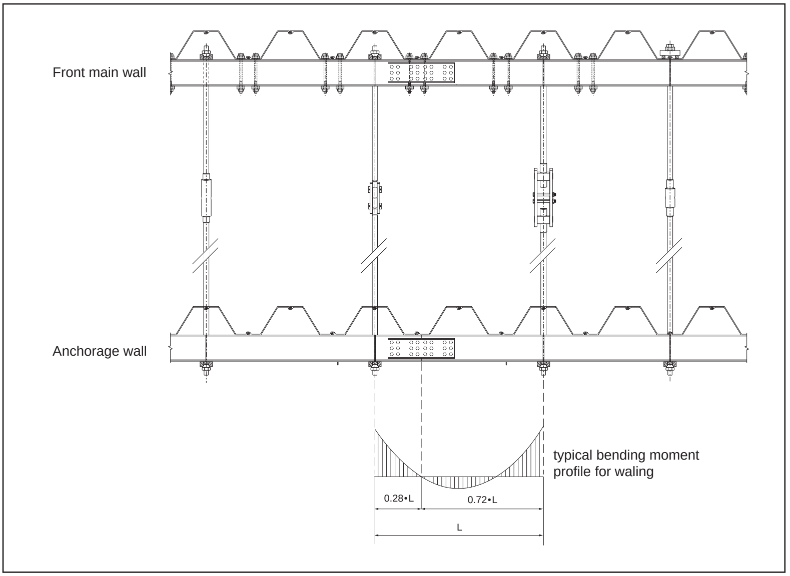


Fig. 7.9. Typical general arrangement of walings and fixings for front and back walls.

Where inclined ties are used, the vertical component of the anchor load must not be overlooked and provision must be made to support the waling, usually in the form of brackets or properly designed welded connections. It is not recommended to weld brackets and fixings to the sheet piles prior to driving the piles to level.

In order to prevent the build up of water on top of the waling after backfilling, holes should be provided at any low spots and generally at 3 m centres in the webs of the walings.

Where sheet pile anchorages are used, similar walings to those at the retaining wall are required. These are always placed behind the anchor piles and consequently no anchor bolts are required.

Where walings form part of the permanent structure they can be supplied with a protective coating. Sometimes the walings can be surrounded with concrete for protection but care must be taken not to impair the rotation of the ties if articulated couplings are used to restrict transfer of bending moment from the wall to the tie bar.

Damage incurred to coatings caused by repetitive handling and local connections should be repaired prior to covering up using an approved appropriate system.

7.9.1. Design of continuous walings

For design purposes, walings may be considered to be simply supported between the tie rods or anchors (which result in conservative bending moments) with point loads applied by the anchor bolts. The magnitude of the tie bolt load is a function of the bolt spacing and the design support load per metre run of wall.

Alternatively, walings can be considered as continuous with allowance being made for end spans. Although the waling is then statically indeterminate, it is usual to adopt a simplified approach where the bending moment M is assumed to be given by:

$$M \approx \frac{w \times L^2}{10}$$

where

w is the calculated load to be supplied by the anchorage system (acting as a uniformly distributed load);

L is the span between tie rods.

When checking an anchorage system for the loss of a tie rod or anchor, the load in the anchorage system is assessed on the basis of the requirements for a serviceability limit state analysis with no allowance being made for overdig at excavation level. The resulting bending moments and tie forces are considered to be ultimate values and are applied over a length of waling of $2L$.

In this extreme condition, it can be demonstrated that, with the exception of the ties at either end of the external spans, the bending moment in a continuous waling resulting from the loss of any tie rod will not exceed $0.3\,wL^2$

where

w is the support load calculated for this condition expressed as a uniformly distributed load and, for simplicity;

L is the original span between tie rods.

It is intended that this estimation is used for an initial assessment of the effect that loss of a tie rod will have on the structural requirements.

This simplification will enable a check to be made with minimum effort to ascertain whether the normal design conditions are the more critical design situations. If the anchorage design proves to be governed by this extreme case, it may be advantageous to carry out a more rigorous analysis of the waling arrangement with a view towards optimising the design.

Finally the section would need to be checked for steel stress verification after corrosion and considering possible combined loading effects – especially at connection positions with inclined ground anchors.

Combined bending, shear, and compression is outside the scope of the Piling Handbook. The reader should refer to Eurocode 3 - Part 1-1 [ix] for further details.

7.9.2. Initial sizing of parallel flange channel walings

Fig. 7.10. gives information on walings formed from ”back to back” channels in commonly used steel grades. It must not be overlooked that the calculated ultimate bending capacity of the waling will need to be reduced to take into account torsion, high shear loads and axial loading. The values are included as an aid to initial section sizing before corrosion is taken into account.

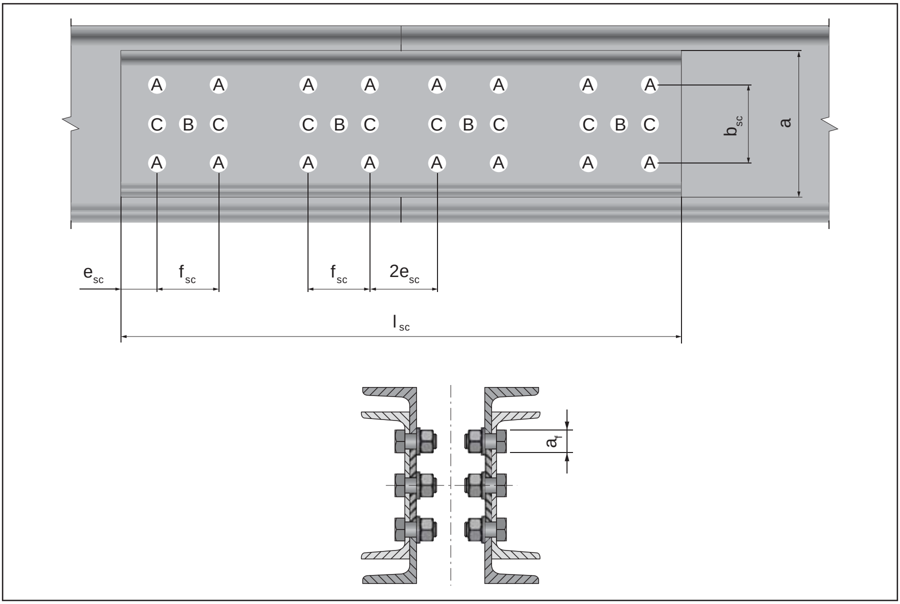


Fig. 7.10. Typical splicing details of walings.

	Waling	Splice		Hole pattern	Individual dimensions				Width across flat
UPN	$W_{el,y}$ cm^3	a mm	l_{sc} mm		b_{sc} mm	e_{sc} mm	f_{sc} mm		SW mm
180	300	140	560	A	**60**	40	60	32 X M20 X 45	30
200	382	140	640	A	**60**	40	60	32 X M20 X 45	30
220	490	160	680	A	**80**	40	60	32 X M20 X 45	30
240	600	180	740	A	**90**	50	75	32 X M24 X 50	36
260	742	200	800	A	**110**	50	75	32 X M24 X 50	36
280	896	220	840	AB	**120**	50	90	40 X M24 X 55	36
300	1070	220	920	AB	**120**	50	90	40 X M24 X 55	36
320	1358	240	1000	AB	**130**	60	110	40 X M30 X 65	46
350	1468	260	1000	AB	**140**	60	110	40 X M30 X 65	46
380	1658	300	1000	AC	**180**	60	90	48 X M30 X 65	46
400	2040	300	1000	AC	**180**	60	90	48 X M30 X 65	46

Table 7.1. Typical component details for waling splice plates.

7.10. Anchor bars or tie rods

7.10.1. Steel grades

Anchor bars or tie rods are usually specified in structural steel complying with EN 10025 [x]. Recommended grades for steel retaining wall applications are S 355, S 460 for yield stress 355 MPa and 460 MPa respectively but also bars manufactured from steel at 500 MPa and 700 MPa yield stress are readily available.

The choice of steel grade depends on a number of factors, whilst the higher strength steel will always produce the lightest weight anchor this may not be suitable for stiffness or durability requirements. Particular attention needs to be made for design of connections and steel grade chosen to accommodate design features such as forged end details with articulation to mitigate effects of settlement and induction of combined stresses. Consultation with specialist anchor providers is recommended. Eurocode 3 - Part 5, 7.2.2.4. [viii] recommends that steel with a specified yield strength of not greater than 800 MPa should be used.

Typical steel grades for design of tie bars are listed in Table 7.2. Please note that elongation properties may be different for other grades of steel.

Steel grade	Yield strength f_y	Tensile strength f_{ua}	Charpy value (min) at 0 °C [1)]
	MPa	MPa	J
S 355	355	510	27
S 460	460	610	27
S 500	500	660	27

Table 7.2. Common steel grades of tie rods.

1) For tie rods up to 100 mm diameter.

7.10.2. Anchor bars

Anchor bars may be manufactured from plain round bars with the threads formed in the parent metal such that the minimum tensile area will occur in the threaded portion of the bar. Alternatively, they may be manufactured with upset ends which involves forging the parent bar to create a larger diameter over the length to be threaded. Using this process, a smaller diameter bar can be used to create a given size of thread or the upset end is simply increased in diameter to accommodate more severe corrosion loss where exposed (particularly in marine environments) without increasing the diameter over the whole length of the bar. In this case a tie rod with upset ends may reduce the overall cost. Upset ended tie bars can be detailed to connect through and directly to the sheet pile outer face so that the wall design does not depend entirely on the durability of the waling to resist failure.

Anchors are generally manufactured from round steel bars with forged or threaded ends that allow a variety of connections to be made to the structure, dead man anchorage or to other bars. Connections can provide articulation or adjustment of the anchor in length.

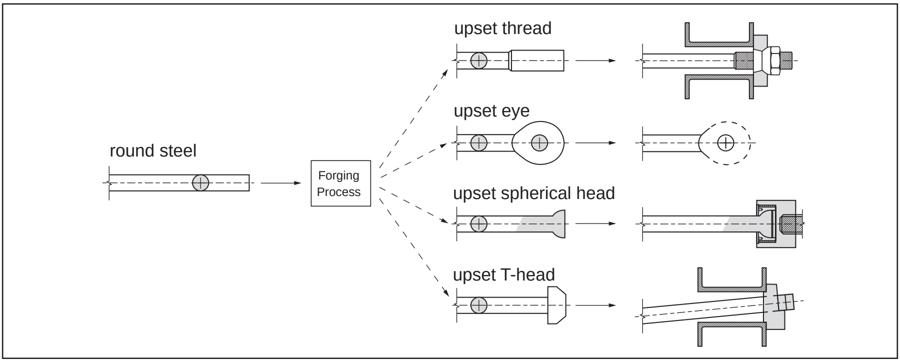


Fig. 7.11. Types of anchor bar connectors.

The advantage of forged end connectors is that it avoids welding and ensures steel quality is the same for the end fitting and the main shaft of the anchor bar.

On a plain steel round bar threads can be produced by cold rolling or machining such that the minimum tensile area will occur in the threaded portion of the bar. Alternatively threads may be manufactured on upset forged ends which involves forging the end of the parent bar to create a larger diameter over the threaded portion, the minimum tensile area is now in the shaft of the anchor.

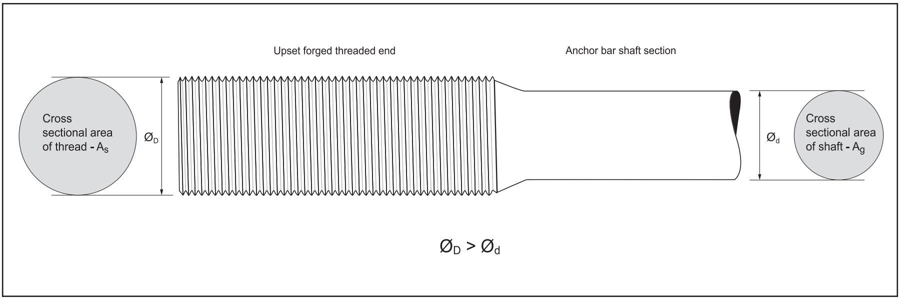


Fig. 7.12. Upset forged threaded end of anchor bar.

7.10.3. Design tensile resistance of anchor bar

Consideration should be taken for both the Serviceability and Ultimate Limit States. The sizing of the tie bar also takes into account the resistance of the section at both the threaded and shaft sections of the tie bar once corrosion has been considered.

7.10.3.1. Ultimate Limit State

In accordance with Eurocode 3 - Part 5, 7.2.3. [viii] the tensile resistance $F_{t,Rd}$ is the lesser of $F_{tt,Rd}$ and $F_{tg,Rd}$.

For the threaded section

$$F_{tt,Rd} = \frac{k_t \times f_{ua} \times A_s}{\gamma_{M2}}$$

For the unthreaded shaft section

$$F_{tg,Rd} = \frac{f_y \times A_g}{\gamma_{M0}}$$

With:

A_g gross section area of the tie bar;

A_s tensile stress area of thread;

f_y yield strength of anchor material;

f_{ua} tensile strength of anchor material;

k_t notch factor = reduction factor allowing for combined bending and tension in the thread.

Note: k_t is given in e.g. the UK National Annex. Recommended values of EC3-5 [viii] are:

- $k_t = 0.6$ where bending at the connection must be considered;
- $k_t = 0.9$ should only be used where the designer by structural detailing is satisfied that bending or combined stresses at the connections will not occur.

Partial factors γ_{M0} and γ_{M2} are given e.g. in the UK National Annex in accordance with Eurocode 3 Part - 5, 5.1.1. (4), in compliance with BS EN 1993 -1-1:

$\gamma_{M0} = 1.0$

$\gamma_{M2} = 1.25$

7.10.3.2. Serviceability Limit State

In accordance with Eurocode 3 - Part 5, 7.2.4., the anchor bar shall be designed to provide sufficient resistance to prevent deformations due to yielding. Characteristic load combinations are considered.

Characteristic axial force of the anchor

$$F_{t,ser} \leq \frac{f_y \times A_s}{\gamma_{Mt,ser}}$$

With

A_s min. gross cross-sectional area of the shaft, tensile stress area of threaded portion;

$\gamma_{Mt,ser}$ partial safety factor according to Eurocode 3 - Part 5, 7.1. (4), to be found in the UK National Annex is: $\gamma_{Mt,ser} = 1.25$

7.10.4. Initial sizing of the anchor bar

For sizing purposes for the threaded or upset end the following Table 7.3. may be used.

Thread size M	Stress area A_s	Grade	Tensile Resistance Thread $F_{tt,Rd}$ for sacrificial corrosion allowance (mm on radius)					
			no corrosion allowance	1,2	1,7	2,2	3,75	5,6
	mm²		kN	kN	kN	kN	kN	kN
64	2676	355	655	602	581	560	498	428
		500	848	779	752	725	644	554
		700	1156	1063	1025	988	878	755
68	3055	355	748	691	669	646	579	503
		500	968	895	865	836	749	652
		700	1320	1220	1180	1140	1022	888
72	3460	355	847	787	762	738	666	585
		500	1096	1018	987	956	862	757
		700	1495	1388	1345	1303	1176	1033
76	3889	355	952	888	862	837	760	673
		500	1232	1150	1116	1083	983	871
		700	1680	1568	1522	1477	1341	1188
80	4344	355	1063	996	968	941	860	767
		500	1376	1289	1253	1218	1113	993
		700	1877	1757	1709	1661	1517	1354
85	4948	355	1211	1139	1110	1081	993	894
		500	1567	1474	1436	1398	1285	1156
		700	2137	2010	1958	1907	1753	1577
90	5591	355	1369	1292	1261	1230	1136	1029
		500	1771	1672	1631	1591	1470	1332
		700	2415	2280	2224	2170	2005	1817
95	6273	355	1536	1454	1421	1388	1289	1175
		500	1987	1882	1839	1796	1668	1520
		700	2710	2566	2508	2450	2274	2073
100	6995	355	1712	1626	1591	1556	1451	1330
		500	2216	2105	2059	2014	1878	1721
		700	3022	2870	2808	2746	2560	2347
105	7755	355	1899	1808	1771	1734	1623	1495
		500	2457	2340	2292	2244	2100	1934
		700	3350	3190	3125	3060	2864	2638
110	8556	355	2094	1999	1960	1922	1804	1669
		500	2710	2587	2537	2487	2335	2160
		700	3696	3528	3459	3391	3184	2945
115	9395	355	2300	2200	2159	2119	1995	1853
		500	2976	2847	2794	2742	2582	2398
		700	4059	3882	3810	3739	3521	3270
120	10274	355	2515	2411	2368	2325	2196	2047
		500	3255	3119	3064	3009	2842	2648
		700	4438	4254	4178	4103	3875	3611
125	11191	355	2740	2631	2586	2541	2406	2250
		500	3545	3404	3346	3289	3114	2911
		700	4835	4642	4563	4485	4246	3970
130	12149	355	2974	2860	2814	2767	2626	2462
		500	3849	3702	3641	3581	3398	3187
		700	5248	5048	4965	4883	4634	4346
135	13145	355	3218	3100	3051	3003	2856	2685
		500	4164	4011	3948	3886	3696	3475
		700	5679	5470	5384	5299	5039	4738
140	14181	355	3471	3349	3298	3248	3095	2917
		500	4492	4333	4268	4203	4005	3775
		700	6126	5909	5820	5731	5461	5147
145	15256	355	3735	3607	3555	3503	3343	3158
		500	4833	4668	4600	4533	4327	4087
		700	6590	6365	6273	6181	5900	5574
150	16370	355	4007	3875	3821	3767	3602	3410
		500	5186	5015	4945	4875	4661	4413
		700	7072	6839	6743	6647	6356	6017
155	17524	355	4290	4153	4097	4041	3870	3671
		500	5551	5374	5302	5229	5008	4750
		700	7570	7329	7229	7131	6829	6477
160	18716	355	4582	4440	4382	4324	4147	3941
		500	5929	5746	5671	5596	5367	5100
		700	8085	7836	7733	7631	7319	6955

Nom. shaft Ø	Nom. Shaft Area	Bar W_t per m	Grade	Tensile Resistance Shaft $F_{tg,Rd}$ for sacrificial corrosion allowance (mm on radius)					
				no corrosion allowance	0,70	1,2	1,7	2,2	3
mm	mm²	kg		kN	kN	kN	kN	kN	kN
48	1810	14	355	643	606	580	555	531	492
			500	905	853	817	781	747	693
			700	1267	1194	1143	1094	1045	970
52	2124	17	355	754	714	686	659	632	590
			500	1062	1006	966	928	890	831
			700	1487	1408	1353	1299	1246	1164
56	2463	19	355	847	805	775	746	717	673
			500	1232	1171	1129	1087	1046	982
			700	1680	1596	1537	1480	1423	1335
60	2827	22	355	952	907	875	844	814	767
			500	1376	1312	1267	1222	1179	1111
			700	1877	1788	1726	1665	1605	1511
64	3217	25	355	1063	1015	982	949	917	866
			500	1567	1498	1450	1403	1356	1283
			700	2137	2042	1976	1910	1846	1745
68	3632	29	355	1211	1160	1124	1089	1055	1001
			500	1771	1698	1647	1596	1547	1469
			700	2415	2314	2243	2174	2105	1998
72	4072	32	355	1369	1315	1277	1239	1202	1145
			500	1987	1910	1855	1802	1749	1666
			700	2710	2603	2528	2454	2381	2267
80	5027	39	355	1712	1651	1609	1567	1525	1460
			500	2457	2371	2310	2250	2191	2098
			700	3350	3231	3147	3065	2983	2855
85	5675	45	355	1899	1835	1790	1746	1702	1633
			500	2710	2619	2556	2493	2431	2333
			700	3972	3842	3751	3661	3571	3431
90	6362	50	355	2258	2188	2139	2091	2043	1967
			500	3181	3083	3014	2945	2878	2771
			700	4438	4301	4204	4108	4014	3865
95	7088	56	355	2515	2441	2389	2338	2287	2207
			500	3544	3440	3367	3295	3223	3110
			700	4835	4692	4591	4491	4392	4236
100	7854	62	355	2740	2663	2609	2555	2502	2418
			500	3849	3741	3665	3589	3514	3397
			700	5248	5099	4993	4889	4786	4623
105	8659	68	355	2974	2894	2837	2781	2726	2638
			500	4164	4052	3972	3894	3816	3693
			700	6061	5900	5787	5675	5564	5388
110	9503	75	355	3218	3135	3076	3018	2960	2869
			500	4752	4632	4547	4463	4379	4248
			700	6590	6423	6304	6187	6071	5887
115	10387	82	355	3687	3598	3535	3472	3410	3312
			500	5186	5060	4972	4884	4796	4659
			700	7072	6898	6776	6654	6534	6344
120	11310	89	355	4007	3914	3848	3783	3718	3616
			500	5551	5421	5329	5238	5148	5005
			700	7570	7390	7264	7138	7013	6816
125	12272	96	355	4290	4194	4126	4058	3991	3885
			500	5929	5795	5700	5605	5512	5364
			700	8590	8399	8263	8129	7996	7785
130	13273	104	355	4712	4611	4540	4469	4398	4287
			500	6320	6181	6083	5986	5889	5736
			700	9167	8969	8829	8691	8553	8335
135	14314	112	355	5081	4976	4902	4828	4755	4639
			500	7138	6991	6886	6783	6680	6517
			700	9733	9529	9385	9242	9100	8875
140	15394	121	355	5465	5356	5279	5203	5127	5007
			500	7565	7413	7306	7199	7093	6925
			700	10316	10106	9958	9810	9664	9432
145	16513	130	355	5862	5749	5670	5590	5512	5387
			500	8005	7849	7738	7628	7519	7346
			700	11533	11311	11154	10998	10843	10597

Table 7.3. Tensile Resistance of anchor bars with upset forged threads.

Note: The values of Table 7.3. are based on nominal standardized diameters

Tensile Resistance $F_{tt,Rd}$ & $F_{tg,Rd}$ calculated with following assumptions

$\gamma_{M0} = 1$ $\gamma_{M2} = 1.25$ $k_t = 0.6$

F_{ua} - Grade 355 = 510, Grade 500 = 660, Grade 700 = 900 N/mm^2

To use Table 7.3. select nearest thread diameter and shaft diameter that exceeds design load (n.b. ensure the same grade of steel is selected from each table)

E_g: Design Ultimate load = 3500 kN;

Wall connection - threaded bar and nut in splash zone - assumed sacrifical corrosion 3.75 mm over lifetime of structure;

Remaining tie bar in compacted, non aggressive fill - assumed sacrifical corrosion 1.2 mm over lifetime of structure;

Anchor end & connections buried in same fill - assumed sacrifical corrosion 1.2 mm over lifetime of structure;

Steel Grade, f_y = 500 N/mm^2, F_{ua} = 660 N/mm^2.

From thread table - Select M135 Thread with $F_{tt,RD}$ of 3696 kN for the wall connection;

From shaft table - Select 100 mm dia with $F_{tg,Rd}$ of 3665 kN;

From thread table - Select M130 Thread with $F_{tt,RD}$ of 3702 kN for the anchor & buried connections.

Therefore suitable anchor is a 100 mm diameter grade 500 bar with upset M135 thread at the wall connection and M130 threads in buried connections.

$F_{t,Rd}$ = lesser of: $F_{tg,Rd}$ & $F_{tt,Rd}$ = 3665 kN

Note: Table 7.3. shows the tensile resistance of a variety of anchor diameters and steel grades. Care must be exercised when assessing the tensile resistance of anchors offered by different manufacturers because the assumptions made for the tensile area of threads, the k_t reduction factor and partial safety factors may differ. As can be seen, a smaller diameter parent bar can be used to create a tie rod with a given thread size when the ends are upset.

Taking manufacturing tolerances into account when defining the minimum thread diameter can reduce the tensile area (based on nominal values) by up to 3%, dependent upon the rod diameter.

Elongation of the anchors under the design load should be checked. This should be based on the shaft diameter of the anchor. The increased stiffness of the threaded area (if larger) can be neglected if the anchor is sufficiently long. Movement under imposed loads may be reduced in many cases by pre-loading the anchors at the time of installation to develop the passive resistance of the ground. Pre-loading of the anchor is most easily achieved at a threaded end of the anchor by means of a hydraulic jack, consideration to the practicality of this should be made at design stage.

The effect of sag of the tie rods and forced deflection due to settlement of fill should also be considered. Bending stresses induced at a fixed anchorage may significantly increase the tensile stress in the tie rod locally. Shear stresses may also be induced if a tie rod is displaced when the fill settles causing compound stresses which must be allowed for in the detailed design. This can often be overcome by provision of articulated joints.

It is highly recommended to adopt detailing of joints and connections to piles using purpose proprietary forged manufactured swivel and hinge coupler systems – examples are illustrated in Chapter 7.10.9. With the correct design and application the notch factor on tensile resistance of the thread k_t may be increased to 0.9 giving a 50% increase in design capacity.

In marine applications settlement ducts may be detailed but the designer should consider durability issues where both oxygen and saline water may be introduced to exposed or damaged areas of protective coating of the steel causing greater levels of corrosion. Using articulated joints or temporary support anchors can be set at levels to counteract effects of settlement. Care should also be taken to specify suitable fill to compact and surround the anchorage system.

7.10.5. Anchor bar fittings and detailing

Anchor assemblies will normally comprise of a series of lengths of round steel bar (depending on the maximum length capable of being produced by a manufacturer) coupled together with suitable threaded or forged connections and usually with a turnbuckle with right and left hand threads to permit length adjustment and to take out any sag. Connections to the front and anchorage walls can be made with simple bearing plates and nuts or articulated joints. All connecting components to an anchor should be designed to exceed the tensile resistance ($F_{t,Rd}$) of the bar, hence the bar shaft, or threaded part of bar determines the overall tensile resistance of the anchor. Where the axis of an anchor is not perpendicular to the structure, or construction methods make it difficult to achieve this with accuracy, a perpendicular connection with an articulated joint is preferable.

7.10.6. Anchor bolts

Anchor bolts are designed in the same way as the main anchor bars, however it is usual to provide two anchor bolt connections at each sheet pile connection to the waling at locations where the main anchor bar is not directly connected to the sheet piles.

If anchor bolts are used to straighten the piles to the waling or are pre-stressed this needs to be taken into account in the design. An additional loading of 25% is recommended to be taken into account for the required tensile resistance of an anchor bolt. The anchor bolts may be designed with a forged head to avoid exposure of the thread on the seaward face of the sheet pile in a marine application.

7.10.7. Anchor plates

Plates are required to transmit the load imposed on sheet piling to the anchors and from the anchors to the anchorages and vice-versa. Washer plates are used when the anchors are connected through the pans and bear directly on to the flat surface of the sheet piles and bearing plates when the load is transmitted through walings. When the load is taken to a concrete wall or block, anchorage plates are required to distribute the load to the concrete. The waling loads are transmitted to the anchorages by means of the anchor bolts which also require bearing plates and washers of sufficient size to provide adequate bearing on to the sheet piling, walings etc.

Detailing of the bearing plates to walings and piles need to take into account the pile type. Bridging washers may be required where the bearing to the pile is not flat. Taper or special washers are used when the axis of a tie rod is not perpendicular to its seating. In some instances it is desirable to allow for rotation of the axis to a tie rod relative to the bearing face, and spherical connector fittings are available for this purpose.

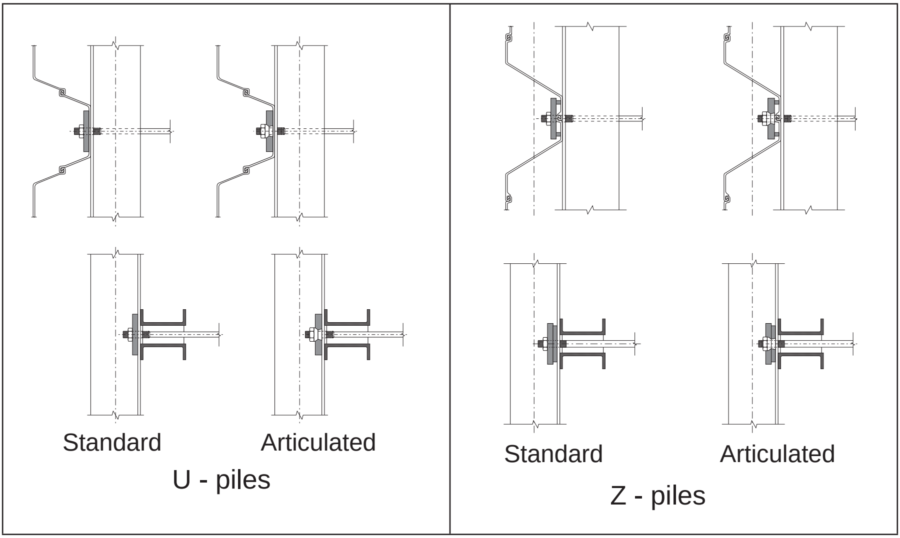


Fig. 7.13. Types of piles - Detailing of bearing to sheet piles.

7.10.8. Detailing anchor bar assembly

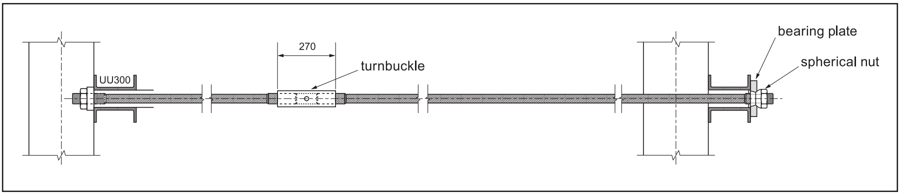


Fig. 7.14. Typical arrangement tie bar assembly, non-articulated.

Where inclination or angled position is required tapered or hemispherical washers should be used. The walings need to be adequately spaced apart for tie rods inclining in the vertical plane.

Purpose made fittings are available for detailing anchor connections inside reinforced concrete infilled steel pipes. Recommendations for specific details may be available from anchor bar manufacturers on request for the designer to consider. Practicalities of installing the fittings to the correct level and workmanship of the concrete surround require consideration.

7.10.9. Special fittings

7.10.9.1. High Modulus system fittings

Special fittings including T-bar and T-heads are used for heavily loaded anchors for high modulus wall systems with or without concrete surround.

It is good practice to design the connections to fit to the king piles in accordance with the manufacturer's recommendations.

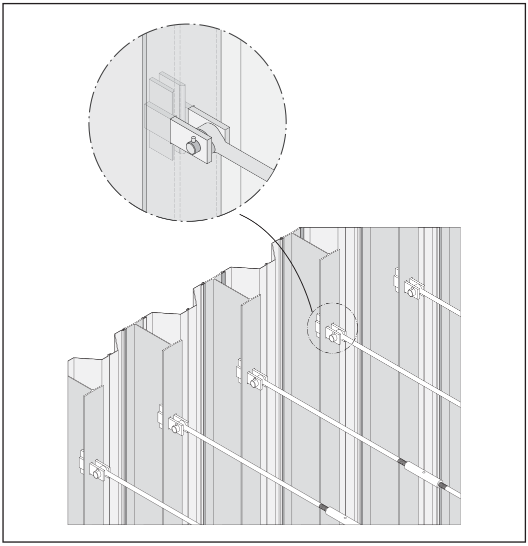


Fig. 7.15. Typical combi-wall anchor bar fittings to HZ®-M king piles. Typical arrangement tie bar assembly with articulated joints for HZ-M system.

The HZ-M system accommodates T-plate fittings which are inserted into slots cut in the flange after driving.

7.10.9.2. Couplings

The detailing of couplings will depend on the type of articulation and tolerances for installation required to fit the assembly accurately.

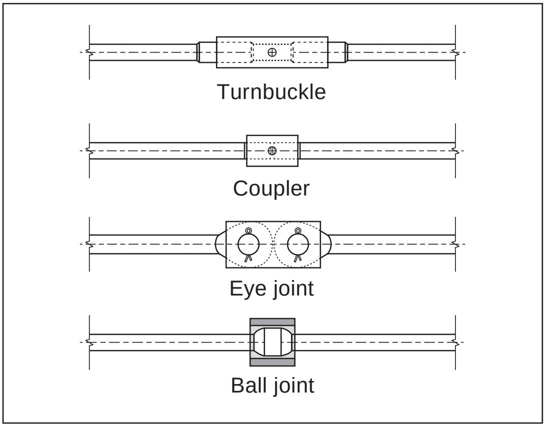


Fig. 7.16. Types of coupling fittings.

7.10.9.3. Fittings for angular rotation

Any bending in a tie rod, especially in the threaded length increases the stress locally with the possibility of yield or even failure if the bending is severe. In order to eliminate the risk of bending, several options are available which allow rotation of the axis of a tie rod while maintaining its tensile capacity. Options are available using nuts and washers with spherical seatings or pairs of taper washers which can be rotated to give any angle between zero and a predetermined maximum. The last two methods will cater for initial angularity but will not move to accommodate rotation in service.

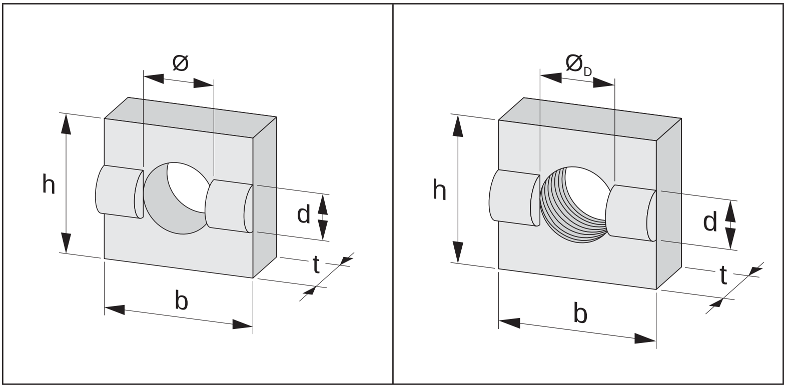


Fig. 7.17. Rocker plates – max 7 degrees.

7.10.10. Anchor corrosion protection

Sheet piles are used in many aggressive environments and consequently corrosion protection factors influencing effective life must be considered. It is especially important to consider the corrosion protection of the anchors at design stage and of particular importance is the connection to the front wall as the anchor is typically subjected to the most aggressive environment at this point.

Several options are available to the designer:

- often the most practical and robust method is to allow for steel loss in the anchor i.e. the anchor is increased in diameter to allow for anticipated corrosion. In this situation, consideration should be given to the probable corrosion rates and consequential loss of anchor section, in the thread, shaft and fittings depending on their position of exposure in the structure. Upset ends for tie bars and forged end heads for anchor bolts may be detailed or specified for advantage where the critical diameter threaded component would be otherwise exposed to an aggressive environment;
- surface protection: several options are available, such as painting, galvanising or wrapping. The most commonly used method is to wrap the anchor bar and fittings to give an appropriate level of corrosion protection. Often the anchor shaft is wrapped in factory conditions and shipped to site but connections cannot be wrapped until installed on site. It is important to ensure that protection to connections and the anchor head are correctly performed during installation, any damaged or unprotected areas must be repaired before backfilling. Any breaks in the wrapping system could lead to aggressive pitting corrosion and reduced durability of the anchor. Smaller diameter anchor bars in high strength steels are particularly at risk.

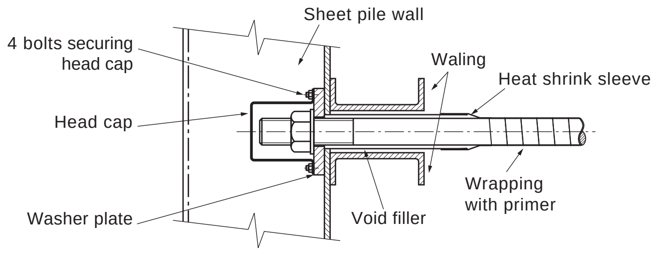


Fig. 7.18. Typical details for surface protection of anchor fittings.

7.11. Connections and plates

Guidance on the design of the transmission of forces into the sheet pile wall and vice versa to the anchorage system is given in Chapters 5 and 8.

It is recommended to contact the manufacturer of the anchor bars and walings for correct detailing and sizing of the bearing plates and fittings to walings as well as anchor bolts. Verification of resistance should be carried out according to EN 1993-1-8 [vii]. This is outside the scope of the Piling Handbook 9th Edition.

7.12. Installation

Anchor bars perform best in pure tension, so it is good practice to ensure that this is achieved with suitably designed fittings and accurate alignment. Good experienced workmanship, inspection supervision and safety are essential for successful execution of anchorage systems. Final inspection procedure should always take place to ensure any damaged protection is repaired, connections completed and bearing plate fittings aligned correctly before covering up. Sometimes anchor bolt bearing plates are tack welded to the walings to avoid slipping when the bolt is tightened up. Walings should be securely fixed to the sheet piles (straightening of sheet pile walls should be carried out using temporary walings if necessary to avoid overstressing the waling and anchor bolts).

The following is a recommended sequence of events to ensure that tie rods are installed and tensioned correctly:

1. backfill to approximately 150 mm below the finished level for the anchor bars;
2. place sand bags every 6 m or either side of a coupler/turnbuckle or articulated joint;
3. pre-camber in accordance with the design recommendation or fit settlement ducts over the ties;
4. assemble with turnbuckles set such that there is a 100 mm gap showing between the ends of the bars. Couplers should be fully engaged;
5. tension from the anchorage outside of the wall to take up the slack;
6. tension turnbuckles;
7. place sand fill over the ducts or carefully compact clean granular fill surround to the anchors;
8. backfill to required level taking care to complete the backfill and compaction in front of the anchorage first. This procedure applies to a simple situation and additional activities may be considered for example, applying pre-tensioning to pull the piles in before final backfilling, stressing after backfilling to prevent future movement due to subsequent loading;
9. it is essential not to overload the front wall with fill before the anchor bars and anchorage are completed. Cantilever walls tend to move in excess of predictions especially when subject to dynamic loading.

Further information on assembling tie bars and protection systems is usually available on request from tie rod manufacturers.

7.13. Worked example – anchorage location

7.13.1. Design situation

Consider a balanced dead-man anchorage which is required to support an embedded sheet pile wall retaining $H = 7$ m of ground. The layout including anchorage location is given in Fig. 7.1. The point of fixity of the pile toe has been calculated as $d_O = 3.45$ m. The anchorage is required to provide an ultimate resistance of at least $F_d = 125$ kN/m. The anchor head is connected to the wall at a depth $d_h = 1.15$m. The anchorage is $L_a = 14.5$ m long and is inclined at $\theta = 5°$ to the horizontal. It is connected to a restraint $D = 1.9$ m deep.

The anchorage is installed in a loose fine sand with characteristic weight density $\gamma_k = 18$ kN/m^3 and angle of shearing resistance $\varphi_k = 32°$. The ground is dry throughout the depth of the anchorage. A variable surcharge of $q_{Qk} = 10$ kPa may be applied at ground surface behind the anchor restraint. The lower part of the wall is Geometry.

Depth to bottom of anchor restraint is

$$D' = d_h + L_a \sin\theta + \frac{D}{2} = 3.36 \text{ m}$$

7.13.2. Actions

Vertical stress at base of anchor restraint is

$$\sigma_{v,k} = \gamma_k \times D' = 60.48 \text{ KN/m}^2$$

7.13.3. Material properties

Partial factor for Design Approach 1, Combination 2 from Set M2: $\gamma_{\varphi'} = 1.25$

Design angle of shearing resistance:

$$\varphi_d = \text{atan}\left(\frac{\tan\varphi_k}{\gamma_{\varphi'}}\right) = 26.6°$$

Design angle of shearing resistance of lower sand

Because the anchor restraint is close to ground surface, it will not be prevented from moving vertically under load. Hence it is safest to ignore wall friction by choosing $\delta_a = \delta_p = 0°$.

7.13.4. Effects of actions

Partial factors for Design Approach 1, Combination 2 from Set A2:
$\gamma_G = 1.0$, $\gamma_{G,fav} = 1.0$, and $\gamma_Q = 1.3$

Active earth pressure coefficient:

$K_{ah} = 0.381$

Active thrust from ground self-weight will be treated as a favourable action, according to the “single-source principle”, to match passive thrust.

Active thrust on anchor restraint:

$$P_{a,d} = K_{ah} \times \left[\left(\gamma_{G,fav} \times \sigma_{v,k} \times \frac{D'}{2} \right) + \left(\gamma_Q \times q_{Qk} \times D' \right) \right]$$

$$= 53.35 \, \frac{\text{kN}}{\text{m}}$$

Design force to be provided to wall: F_d = 125 kN/m.

Total horizontal thrust:

$H_{Ed} = F_d + P_{a,d}$ = 180.35 kN/m.

7.13.5. Resistance

Partial factor for Design Approach 1, Combination 2 from Set R1:

$\gamma_{R,e}$ = 1.00

Passive earth resistance coefficient:

K_{ph} = 2.622

Passive thrust will be treated as a favourable action, according to the “single-source principle”.

Passive thrust on anchor restraint:

$$P_{p,d} = K_{ph} \times \frac{\left(\gamma_{G,fav} \times \sigma_{v,k} \times \frac{d_{\max}}{2} \right)}{\gamma_{\mathrm{Re}}} = 266.41 \, \frac{\text{kN}}{\text{m}}$$

Total horizontal resistance:

$H_{Rd} = P_{p,d}$ = 266.41 kN/m

7.13.6. Verifications

Horizontal equilibrium H_{Ed} = 180.35 kN/m and H_{Rd} = 266.41 kN/m

Degree of utilization:

$$\Lambda_{GEO} = \frac{H_{Ed}}{H_{Rd}} = 68\%$$

Design is unacceptable if the degree of utilization is > 100%.

Check restraint is placed far enough behind the wall.

$$x_1 = \left(H + d_O \right) \times \tan\left(45° - \frac{\varphi_d}{2} \right) = 6.45 \text{ m}$$

$$x_2 = \frac{D'}{\tan\left(45° - \frac{\varphi_d}{2}\right)} = 5.44 \text{ m}$$

$x_1 + x_2 = 11.89$ m

$L_a \cos\theta = 14.44$ m

Design is acceptable if $L_a \cos\theta > x_1 + x_2$

$L_a \cos\theta = 14.44 \text{ m} > 11.89\ 0 = x_1 + x_2$

Design is unacceptable if the degree of utilization is > 100%.

$$x_3 = \frac{H}{\tan\varphi_d} = 13.98 \text{ m}$$

Check $L_a \cos\theta$ more than x_3:

$L_a \cos\theta = 14.44 \text{ m} > 13.98 \text{ m} = x_3$

7.13.7. Conclusion

Proposed solution is acceptable.

References:

The following documents are referenced by this chapter.

[i] EN 1997, Eurocode 7 - Geotechnical design, Part 1: General rules, European Committee for Standardization, Brussels.

[ii] EN 1537, Execution of special geotechnical work - Ground anchors. European Committee for Standardization, Brussels. 2013.

[iii] Bond A. J. and Harris A. J. (2008) Decoding Eurocode 7, London: Taylor and Francis, 616pp.

[iv] EN ISO 22477-5, Geotechnical investigation and testing - Testing of geotechnical structures, Part 5: Testing of anchorages, European Committee for Standardization, Brussels.

[v] BS 8081: Code of practice for ground anchorages, British Standards Institution. 2015.

[vi] EAU 2012. Recommendations of the Committee for Waterfront Structures, Harbours and Waterways, Berlin, 2012. (Ernst & Sohn).

[vii] EN 1993-8, Eurocode 3: Design of Steel Structures - Part 8: Design and joints. 2005.

[viii] EN 1993-5, Eurocode 3: Design of Steel Structures - Part 5: Piling 2007.

[ix] EN 1993-1, Eurocode 3: Design of Steel Structures - Part 1-1: General rules and rules for buildings. 2005.

[x] EN 10025: Hot rolled products of structure steel. 2004.

ArcelorMittal

8 | Structural design of sheet pile sections

Chapter 8 - Structural design of sheet pile sections

Contents

8.1. Introduction

According to Eurocode 3 - Part 5 [i], sheet pile sections should be verified against structural failure due to:

- bending and/or axial force;
- overall flexural bending, taking account of the restraint provided by the ground;
- local buckling due to overall bending;
- local failure where loads are applied (e.g. web crippling);
- fatigue.

The effects of actions resulting from changes in temperature over time should be taken into account, for example, in the design of struts if large temperature changes are expected. The design may prescribe measures to reduce the influence of temperature.

Simplified models of the loads acting on retaining walls from roads and/or railways may be used (for example, by replacing point and/or line loads with uniformly distributed loads).

> Eurocode 3 uses the terms “design moment resistance” (in Eurocode 3 - Part 5 [i]) and “design resistance for bending” (in Eurocode 3 - Part 1-1 [ii]) for the same quantity, which is termed “**design bending resistance**” in the Piling Handbook.

The design of straight web sheet piles is outside the scope of Chapter 8. The reader should refer to Chapter 10 and separate publications by ArcelorMittal [iii] for details of how to design these piles.

The design of combined walls is also outside the scope of the Piling Handbook. The reader should refer to separate publications by ArcelorMittal ([iv] and [xii]) for details of how to design these walls.

8.2. Section classification

The principles behind the classification of steel sections for plastic design are discussed in Chapters 4 and 5.

Sheet pile cross-sections are classified by Eurocode 3 - Part 5 [i], Table 5.1., according to their flange slenderness ratio (b/t_f), where b = flange width and t_f = flange thickness, as indicated in the following table.

Class	Rotation check	Ratio of b/t_f is less or equal than...	
		Z-profile	U-profile
1	Required	45ε	37ε
2	Not required		
3	Not required	66ε	49ε

Table 8.1. Classification ratios for sheet pile sections.

ε = coefficient that depends on steel grade (see Chapter 5).
Note: The b/t_f ratio is applied for the pile section properties after allowance for corrosion loss.

Class 4 sections are discussed in Chapter 8.11. These sections require checking for local buckling and combined stress effects, because these effects may determine the cross-section resistance.

Classification of a range of Z- and U-type sheet piles is given in Chapter 1 – but the values given there do not take into account corrosion.

Example 1:

A PU 18 section has flange width b = 288.5 mm and thickness t_f = 11.2 mm.
In grade S 355 GP steel, its yield strength is f_y = 355 N/mm^2 and ε = 0.81.
Hence the PU 18 is a Class 1 or 2 section, since:

$$\frac{b}{t_f} = \frac{288.5}{11.2} = 25.8 \leq 30.1 = 37 \times 0.81 = 37\ \varepsilon$$

Example 2:

An AZ 18-700 section has flange width b = 352.8 mm and thickness t_f = 9.0 mm.
In grade S 355 GP steel, the AZ 18-700 is a Class 3 section, since:

$$\frac{b}{t_f} = \frac{352.8}{9.0} = 39.2 > 36.6 = 45 \times 0.81 = 45\ \varepsilon \qquad \textit{(not class 2)}$$

$$\frac{b}{t_f} = \frac{352.8}{9.0} = 39.2 \leq 53.7 = 66 \times 0.81 = 66\ \varepsilon \qquad \textit{(class 3)}$$

8.3. Bending

Verification of bending resistance (in the absence of shear and axial compression) involves checking that the design bending moment M_{Ed} in each cross-section does not exceed its corresponding design bending resistance $M_{c,Rd}$. This is expressed in Eurocode 3 - Part 5 [i] (exp. 5.1.) by the inequality:

$$M_{Ed} \leq M_{c,Rd}$$

The design bending resistance of Class 1 and 2 cross-sections is (exp. 5.2):

$$M_{c,Rd} = \frac{\beta_B W_{pl} f_y}{\gamma_{M0}}$$

and of Class 3 cross-sections (exp. 5.3):

$$M_{c,Rd} = \frac{\beta_B W_{el} f_y}{\gamma_{M0}}$$

where

β_B is a factor that accounts for possible lack of shear force transmission in the interlocks between adjacent piles (for U-piles only);

W_{pl} is the cross-section's plastic section modulus;

W_{el} its elastic section modulus;

f_y is the yield strength of steel;

γ_{M0} is a partial factor whose value is given in Chapter 5.

Values of W_{pl} and W_{el} are given in Chapter 1 for a range of Z- and U-type sheet piles.

Values of f_y are given in Chapter 1 for hot-rolled (S 240 GP to S 430 GP and S 460 AP) and cold-formed (S 235 JRC to S 355 J0C) steel grades.

Values of β_B for single and double piles are given in the National Annex to Eurocode 3 - Part 5 [i]. Table 8.2. summarises the β_B-values given in the UK National Annex to Eurocode 3 - Part 5 [v].These values depend on the number of structural supports and the conditions under which the piles are installed.

For Z-type and for U-type triple piles, β_B = 1.0.

Structural support levels include any restraint, designed in accordance with relevant standards, that changes the sign of the shear force (i.e. from positive to negative, or vice-versa). The pile toe is not considered to be a restraint. The benefit of the restraint only applies in design situations that follow its installation.

Highly unfavourable conditions include:

- when the piles retain substantial depths of free water;
- significant presence of very low strength fine soil or very loose coarse soil (as defined in EN 14688-1 [vi]);
- artificial loosening of fine soil by pre-augering below final excavation level;
- artificial loosening of fine soil by water jetting at a rate greater than 240 l / min (see EN 12063 [vii]);
- artificial loosening of coarse soil by water jetting at a rate greater than 480 l / min (see EN 12063 [vii]).

Unfavourable conditions include:

- significant presence of low strength fine soil or loose coarse soil (as defined in EN 14688-1 [vi]);
- artificial loosening of coarse soil by pre-augering below final excavation level;
- artificial loosening of fine soil by water jetting at a rate between 60 and 240 l / min (see EN 12063 [vii]);
- artificial loosening of coarse soil by water jetting at a rate between 240 and 480 l / min (see EN 12063 [vii]).

Favourable conditions apply when:

- none of the highly unfavourable or unfavourable conditions apply.

Type of U-sheet pile unit	**No. of structural support levels**	**Conditions**		
		Highly unfavourable	**Unfavourable**	**Favourable**
Singles or uncrimped doubles	0	0.40	0.50	0.60
	1	0.55	0.60	0.70
	>1	0.65	0.70	0.80
Crimped or welded doubles	0	0.70	0.75	0.80
	1	0.80	0.85	0.95
	>1	0.90	0.95	1.00

Table 8.2. Values of β_B for U-type piles from the UK NA to EN 1993-5 [v].

Note : Values of β_B cannot exceed 1.0.
Other National Annexes may show different values.

The coefficients β_B may be increased by a certain value if:

	Conditions		
	Highly unfavourable	**Unfavourable**	**Favourable**
Interlock not treated with sealants or lubrication	+0.05	+0.05	+0.05
Structural weld to prevent slippage	+0.10	+0.15	+0.20

Table 8.3. Correction of β_B values according to UK NA to EN 1993-5 [v].

Example 1 (continued):

PU 18 steel sheet piles made of grade S 355 GP steel are to be driven as singles into favourable ground conditions (without pre-augering or lubrication of the interlock). The sheet piles will be supported by a single row of anchors. Hence, from the table above, $\beta_B = 0.70 + 0.05 = 0.75$.

The PU 18 is a Class 2 section, hence its plastic section modulus $W_{pl} = 2134$ cm^3/m may be used. With $\gamma_{M0} = 1.0$, the design bending resistance of the cross-section is calculated as:

$$M_{c,Rd} = \frac{\beta_B \, W_{pl} \, f_y}{\gamma_{M0}} = \frac{0.75 \times 2134 \times 355}{1.0 \times 10^3} = 568 \text{ kNm/m}$$

Example 2 (continued):

AZ 18-700 steel sheet piles installed in the same ground conditions has $\beta_B = 1.0$. In grade S 355 GP steel, the AZ 18-700 is a Class 3 section, hence its elastic section modulus $W_{el} = 1800$ cm^3/m must be used. With $\gamma_{M0} = 1.0$, the design bending resistance of the cross-section is:

$$M_{c,Rd} = \frac{\beta_B \, W_{el} \, f_y}{\gamma_{M0}} = \frac{1.0 \times 1800 \times 355}{1.0 \times 10^3} = 639 \text{ kNm/m}$$

8.4. Shear

Verification of shear resistance involves checking that the design shear force V_{Ed} in each cross-section does not exceed its corresponding design plastic shear resistance $V_{pl,Rd}$. This is expressed in Eurocode 3 - Part 5 [i] (exp. 5.4.) by the inequality:

$$V_{Ed} \leq V_{pl,Rd}$$

The design plastic shear resistance of each web is given by (exp. 5.5.):

$$V_{pl,Rd} = \frac{A_v \, f_y}{\sqrt{3} \, \gamma_{M0}}$$

where

A_v is the projected shear area of each web (exp. 5.6.):

$$A_V = t_w \left(h - t_f\right)$$

t_w is the web's thickness;

t_f the flange thickness;

h the overall height of the cross-section, as illustrated in Fig. 8.1.

(In this diagram, the dimension c is known as the section's "slant height" and its definition is given in Chapter 8.10.)

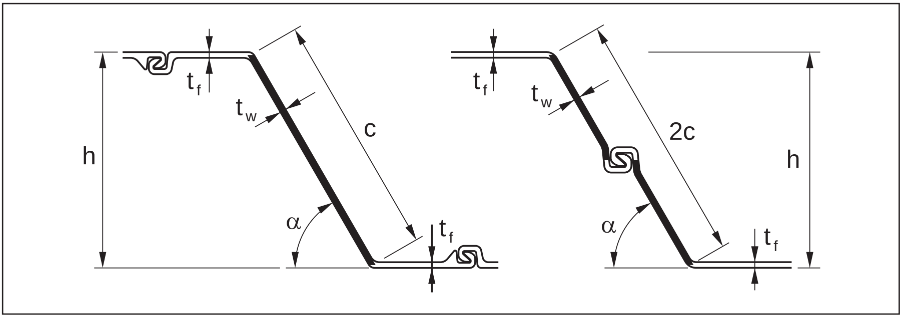


Fig. 8.1. Definition of cross-section variables and shear area.

Example 1 (continued):

The dimensions of a PU 18 section are t_w = 9.0 mm, t_f = 11.2 mm, and h = 430 mm. The clutch-to-clutch spacing is B = 600 mm.

The projected shear area *of each web* is therefore:

$$A_V = t_w(h - t_f) = 9.0 \times (430 - 11.2) = 3769 \text{ mm}^2$$

The design plastic shear resistance of the section in grade S 355 GP steel is then:

$$V_{pl,Rd} = \frac{A_v\ f_y}{\sqrt{3}\ \gamma_{M0}} = \frac{3769 \times 355}{\sqrt{3} \times 1.0 \times 10^3} = 772.5 \text{ kN}$$

which can be expressed *per metre run of wall* as:

$$V'_{pl,Rd} = \frac{V_{pl,Rd}}{B} = \frac{772.5}{0.6} = 1288 \text{ kN/m}$$

Example 2 (continued):

The dimensions of an AZ 18-700 section are $t_w = t_f$ = 9.0 mm and h = 420 mm. The clutch-to-clutch spacing is B = 700 mm.

The projected shear area of each web is therefore:

$$A_V = t_w(h - t_f) = 9.0 \times (420 - 9.0) = 3699 \text{ mm}^2$$

The design plastic shear resistance of the section in grade S 355 GP steel is then:

$$V_{pl,Rd} = \frac{A_v\ f_y}{\sqrt{3}\ \gamma_{M0}} = \frac{3699 \times 355}{\sqrt{3} \times 1.0 \times 10^3} = 758.1 \text{ kN}$$

which can be expressed per metre run of wall as:

$$V'_{pl,Rd} = \frac{V_{pl,Rd}}{B} = \frac{758.1}{0.7} = 1083 \text{ kN/m}$$

8.5. Combined bending and shear

Verification of resistance to combined bending and shear (in the absence of axial compression) involves checking that the design bending moment M_{Ed} in each cross-section does not exceed the *reduced* design plastic bending resistance of the section $M_{V,Rd}$. This is expressed by the inequality:

$$M_{Ed} \leq M_{V,Rd} \text{ but } M_{V,Rd} \leq M_{c,Rd}$$

The effects of shear on the plastic bending resistance of sheet piles may be neglected if:

$$V_{Ed} \leq \frac{V_{pl,Rd}}{2}$$

where

V_{Ed} is the design shear force;

$V_{pl,Rd}$ is the design plastic shear resistance defined in Chapter 8.4.

The reduced design plastic bending resistance for class 1 and 2 sections is given in Eurocode 3 - Part 5 [i] (exp. 5.9.) by:

$$M_{V,Rd} = \left(\beta_B W_{pl} - \frac{\rho A_V^2}{4 t_w \sin \alpha} \right) \frac{f_y}{\gamma_{M0}}$$

where ρ is defined in Eurocode 3 - Part 5, Section 5.2.3(12):

$$\rho = \left(\frac{2 V_{Ed}}{V_{pl,Rd}} - 1 \right)^2$$

and the other symbols are defined in Chapter 8.3.

For non-class 1 and 2 sections, the reduced design bending resistance should be taken as the design resistance of the cross section, calculated using a reduced yield strength $(1-\rho)\, f_y$ for the shear area.

The ratio of the reduced to the full plastic bending resistance of the cross-section is given by:

$$\frac{M_{V,Rd}}{M_{c,Rd}} = 1 - \left(\frac{A_V^2}{4 t_w \beta_B W_{pl} \sin \alpha} \right) \rho \leq 1$$

Example 1 (continued):

The PU 18 steel sheet pile is subjected to a design shear force V_{Ed} = 800 kN/m. The section's design bending resistance is $M_{c,Rd}$ = 568 kNm/m and its design plastic shear resistance is $V'_{pl,Rd}$ = 1288 kN/m (from example in Chapter 8.4).

The sections is a class 2 section (from example in Chapter 8.2.)

The section's projected shear area is A_v = 3769 mm², its web thickness t_w = 9.0 mm, slant angle α = 57.5°, and plastic section modulus W_{pl} = 1280 cm³ (i.e. 2134 cm³/m, section width 600 mm).

With β_B = 0.75, the reduced design plastic bending resistance is:

$$\frac{M_{V,Rd}}{M_{c,Rd}} = 1 - \left(\frac{3769^2}{4 \times 9.0 \times 0.75 \times 1280 \times 10^3 \times \sin 57.5°}\right)\left(\frac{2 \times 800}{1288} - 1\right)^2$$
$$= 1 - 0.487 \times 0.0598 = 0.971$$

$$M_{V,Rd} = 0.971 \times M_{c,Rd} = 0.971 \times 568 = 552 \text{ kNm/m} \leq M_{c,Rd}$$

Example 2 (continued):

The AZ 18-700 steel sheet pile is also subjected to a design shear force V_{Ed} = 800 kN/m. The section's design bending resistance is $M_{c,Rd}$ = 639 kNm/m and its design plastic shear resistance is $V'_{pl,Rd}$ = 1083 kN/m (from example in Chapter 8.4).

The sections is a class 3 section (from example in Chapter 8.2.)

The section's projected shear area is A_v = 3699 mm², its web thickness t_w = 9.0 mm, slant angle α = 51.2° and elastic section modulus W_{pl} = 1260 cm³ (i.e. 1800 cm³/m, section width 700 mm).

With β_B = 1.0., the reduced design plastic bending resistance is:

$$\frac{M_{V,Rd}}{M_{c,Rd}} = 1 - \left(\frac{A_V^2}{6 \times t_w \times \beta_B \times W_{el} \times \sin\alpha}\right)\rho$$
$$= 1 - \left(\frac{3699^2}{6 \times 9.0 \times 1.0 \times 1260 \times 10^3 \times \sin 51.2}\right)\left(\frac{2 \times 800}{1083} - 1\right)^2 = 0.94$$

$$M_{V,Rd} = 0.94 \times M_{c,Rd} = 0.94 \times 639 = 601 \text{ kNm/m} \leq M_{c,Rd}$$

Fig. 8.2. shows the reduction in available moment resistance $M_{V,Rd}$ with increasing design shear force V_{Ed} for the two sections, under the conditions specified in these worked examples.

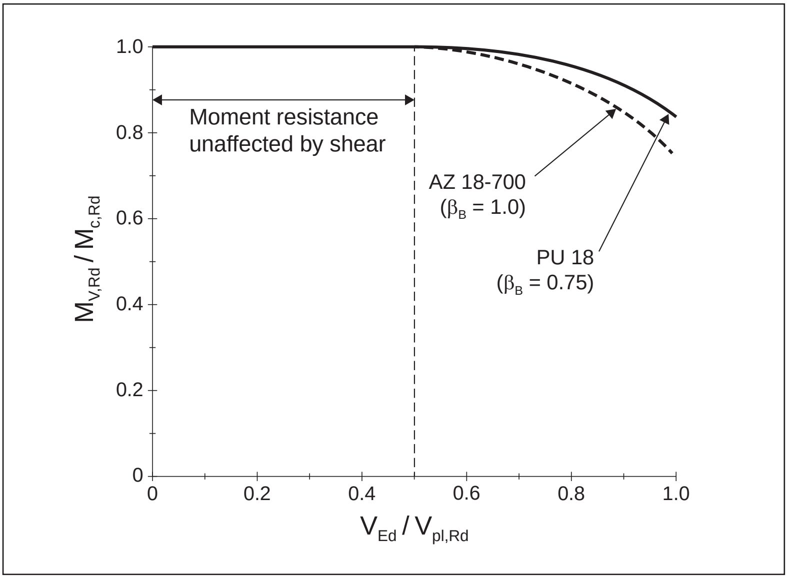


Fig. 8.2. Reduction of moment resistance with increasing shear force.

8.6. Compression

Verification of compressive resistance involves checking that the design axial compressive force N_{Ed} in each cross-section does not exceed its corresponding design plastic resistance $N_{pl,Rd}$. This is expressed in Eurocode 3 - Part 5 [i] (exp. 5.15.) by the inequality:

$$N_{Ed} \leq N_{pl,Rd}$$

The design plastic resistance is given by (exp. 5.16.):

$$N_{pl,Rd} = \frac{A f_y}{\gamma_{M0}}$$

where

A is the cross-sectional area;

f_y the yield strength of steel;

γ_{M0} the partial material factor given in Chapter 5.

Example 1 (continued):

The cross-sectional area of a PU 18 sheet pile is A = 163.3 cm²/m. Using grade S 355 GP steel with f_y = 355 N/mm², the pile's design plastic resistance is calculated as:

$$N_{pl,Rd} = \frac{A\,f_y}{\gamma_{M0}} = \left(\frac{163.3 \times 355}{1.0}\right) / 10 = 5797 \text{ kN/m}$$

Example 2 (continued):

The cross-sectional area of an AZ 18-700 sheet pile is A = 139.2 cm²/m. Using grade S 355 GP steel, the pile's design plastic resistance is calculated as:

$$N_{pl,Rd} = \frac{A\,f_y}{\gamma_{M0}} = \left(\frac{139.2 \times 355}{1.0}\right) / 10 = 4942 \text{ kN/m}$$

8.7. Combined bending and compression

Verification of resistance to combined bending and compression (in the absence of shear) involves checking that the design bending moment M_{Ed} in each cross-section does not exceed the *reduced* design plastic bending resistance of the section $M_{N,Rd}$. This can be expressed by the inequality:

$$M_{Ed} \le M_{N,Rd} \quad \text{but} \quad M_{N,Rd} \le M_{c,Rd}$$

The effects of axial compression on the plastic bending resistance of sheet piles may be neglected if:

$$\frac{N_{Ed}}{N_{pl,Rd}} \le \text{limiting value}$$

where

N_{Ed} = design axial compressive force;

$N_{pl,Rd}$ = design plastic resistance, as defined in Chapter 8.6.

The limiting value is:

- 0.10 for Z-profiles in Classes 1-3;
- 0.25 for U-profiles in Classes 1-2;
- 0.10 for U-profiles in Class 3.

See EN 1993-5 § 5.2.3. (10).

The reduced design plastic bending resistance is given in Eurocode 3 - Part 5 [i] (exp. 5.20.-5.22.) by:

$$M_{N,Rd} = k\, M_{c,Rd}\left(1-\frac{N_{Ed}}{N_{pl,Rd}}\right) \quad \text{but} \quad M_{N,Rd} \leq M_{c,Rd}$$

where *k* is a factor given in Table 8.4. (and the other symbols are as defined in Chapters 8.3 and 8.6).

Class	U-profiles	Z-profiles
1	1.33	1.11
2	1.33	1.11
3	1.00	1.00
4	See Annex A of EN 1993-5 [i]	

Table 8.4. Factor "k" to determine design plastic bending resistance.

The reduction in bending resistance with increasing axial compression is illustrated in the interaction diagram below.

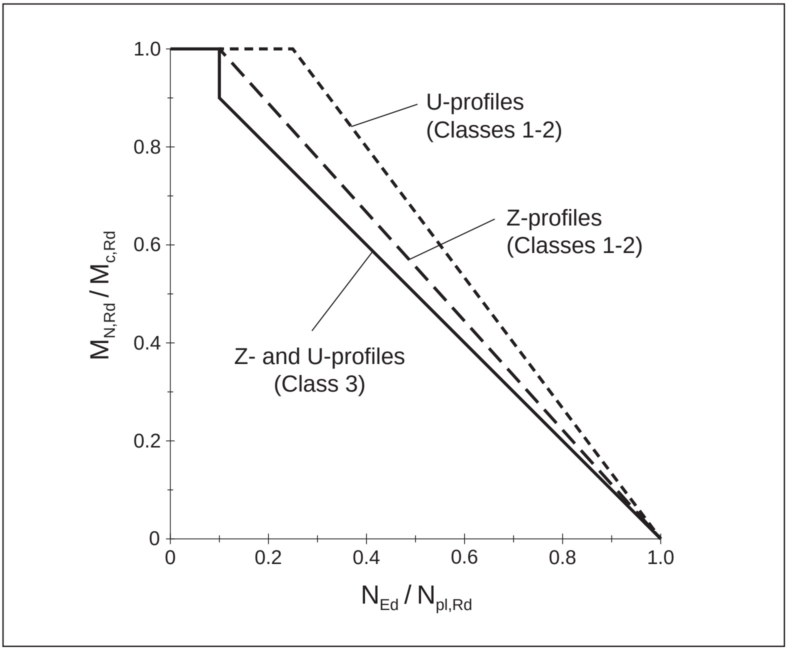


Fig. 8.3. Reduction of bending resistance with increasing axial compression.

Example 1 (continued):

The design bending resistance of the PU 18 sheet pile section from before is $M_{c,Rd}$ = 568 kNm/m and its design plastic resistance $N_{pl,Rd}$ = 5797 kN/m.
The PU 18 is a Class 2 section.

If a design axial compressive force N_{Ed} = 1500 kN/m is applied to the wall, then its effect on the wall's bending resistance must be calculated, since:

$$\frac{N_{Ed}}{N_{pl,Rd}} = \frac{1500}{5797} = 0.259 > 0.25$$

Therefore, the wall's design bending resistance is reduced to:

$$M_{N,Rd} = 1.33\, M_{c,Rd}\left(1 - \frac{N_{Ed}}{N_{pl,Rd}}\right) = 1.33 \times 568 \times \left(1 - \frac{1500}{5797}\right) = 560\ \text{kNm/m}$$

Example 2 (continued):

The design bending resistance of the AZ 18-700 sheet pile section from before is $M_{c,Rd}$ = 639 kNm/m and its design plastic resistance $N_{pl,Rd}$ = 4942 kN/m.
The AZ 18-700 is a Class 3 section.

If N_{Ed} = 1500 kN/m is applied to the wall, then its effect on the wall's bending resistance must be calculated, since:

$$\frac{N_{Ed}}{N_{pl,Rd}} = \frac{1500}{4942} = 0.304 > 0.10$$

Therefore, the wall's design bending resistance is reduced to:

$$M_{N,Rd} = 1.00\, M_{c,Rd}\left(1 - \frac{N_{Ed}}{N_{pl,Rd}}\right) = 1.00 \times 639 \times \left(1 - \frac{1500}{4942}\right) = 445\ \text{kNm/m}$$

8.8. Combined bending, shear, and compression

In the presence of shear, the design resistance of the cross-section to combined bending and axial compression is reduced if:

$$\frac{N_{Ed}}{N_{pl,Rd}} > \text{limiting value}$$

and

$$V_{Ed} > \frac{V_{pl,Rd}}{2}$$

where the symbols and the limiting value are as defined in Chapter 8.4. and 8.7.

In these circumstances, the design bending resistance of the section should be calculated with a reduced yield strength given by:

$$f_{y,\,red} = (1 - \rho)\, f_y$$

where f_y is the steel's specified yield strength and ρ is defined in Chapter 8.5.

Example 1 (continued):

The PU 18 steel sheet piles are subject to a combined design shear force V_{Ed} = 800 kN/m and design axial compressive force N_{Ed} = 1500 kN/m.

The design plastic shear resistance $V'_{pl,Rd}$ = 1288 kN/m for this Class 2 section, giving:

$$\frac{N_{Ed}}{N_{pl,\,Rd}} = \frac{1500}{5797} = 0.259 > 0.25$$

$$V_{Ed} = 800 > 644 = \frac{1288}{2} = \frac{V'_{pl,\,Rd}}{2}$$

and so the section's design moment resistance will be reduced by the presence of shear and compression.

The reduction factor is:

$$\rho = \left(\frac{2\,V_{Ed}}{V_{pl,\,Rd}} - 1\right)^2 = \left(\frac{2 \times 800}{1288} - 1\right)^2 = 0.059$$

Hence the steel's yield strength is reduced to:

$$f_{y,\,red} = (1 - \rho)\, f_y = (1 - 0.059) \times 355 = 334.2 \text{ MPa}$$

The PU 18 is a Class 2 section, hence its plastic section modulus W_{pl} = 2134 cm^3/m may be used. With the partial factor γ_{M0} = 1.0, the design bending resistance of the cross-section is calculated as:

$$M_{c,Rd} = \frac{\beta_B \, W_{pl} \, f_{y,red}}{\gamma_{M0}}$$

$$= \frac{0.75 \times 2134 \times 334.1}{1.0 \times 10^3} = 535 \text{ kNm/m}$$

The cross-sectional area of a PU 18 sheet pile is A = 163.3 cm^2/m. Hence the pile's design plastic resistance is:

$$N_{pl,Rd} = \frac{A \, f_{y,red}}{\gamma_{M0}} = \left(\frac{163.3 \times 334.2}{1.0}\right) / 10 = 5456 \text{ kN/m}$$

With a design axial compressive force N_{Ed} = 1500 kN/m applied to the wall, the design bending resistance is reduced to:

$$M_{N,Rd} = 1.33 \, M_{c,Rd}\left(1 - \frac{N_{Ed}}{N_{pl,Rd}}\right)$$

$$= 1.33 \times 535 \times \left(1 - \frac{1500}{5456}\right) = 516 \text{ kNm/m}$$

Example 2 (continued):

The AZ 18-700 steel sheet piles are subject to a combined design shear force V_{Ed} = 800 kN/m and design axial compressive force N_{Ed} = 1500 kN/m.

The design plastic shear resistance $V'_{pl,Rd}$ = 1083 kN/m for this Class 3 section, giving:

$$\frac{N_{Ed}}{N_{pl,Rd}} = \frac{1500}{4942} = 0.304 > 0.10$$

$$V_{Ed} = 800 > 541 = \frac{1083}{2} = \frac{V'_{pl,Rd}}{2}$$

and so the section's design moment resistance will be reduced by the presence of shear and compression.

The reduction factor is:

$$\rho = \left(\frac{2V_{Ed}}{V_{pl,Rd}} - 1 \right)^2 = \left(\frac{2 \times 800}{1083} - 1 \right)^2 = 0.228$$

Hence the steel's yield strength is reduced to:

$$f_{y,red} = (1 - \rho)\, f_y = (1 - 0.228) \times 355 = 274.1 \text{ MPa}$$

The AZ 18-700 is a Class 3 section, hence its elastic section modulus W_{el} = 1800 cm³/m must be used. With the partial factor γ_{M0} = 1.0, the design bending resistance of the cross-section is calculated as:

$$M_{c,Rd} = \frac{\beta_B\, W_{el}\, f_{y,red}}{\gamma_{M0}} = \frac{1.0 \times 1800 \times 274.1}{1.0 \times 10^3} = 493 \text{ kNm/m}$$

The cross-sectional area of an AZ 18-700 sheet pile is A = 139.2 cm²/m. Hence the pile's design plastic resistance is:

$$N_{pl,Rd} = \frac{A\, f_{y,red}}{\gamma_{M0}} = \left(\frac{139.2 \times 274.1}{1.0} \right) / 10 = 3815 \text{ kN/m}$$

With a design axial compressive force N_{Ed} = 1500 kN/m applied to the wall, the design bending resistance is reduced to:

$$M_{N,Rd} = 1.00\, M_{c,Rd} \left(1 - \frac{N_{Ed}}{N_{pl,Rd}} \right) = 1.00 \times 493 \times \left(1 - \frac{1500}{3815} \right) = 299 \text{ kNm/m}$$

8.9. Member buckling

Verification of resistance to member buckling under combined bending and compression involves checking that the design bending moment M_{Ed} and design compression force N_{Ed} in each cross-section satisfy the following inequality (adapted from Eurocode 3 - Part 5 [i] exp. 5.13.):

$$\frac{N_{Ed}}{\chi\, N_{pl,Rd}} + 1.15\,\frac{M_{Ed}}{M_{c,Rd}} \le \frac{\gamma_{M0}}{\gamma_{M1}}$$

where

$N_{pl,Rd}$ is the plastic design resistance of the cross-section (see Chapter 8.6.);

$M_{c,Rd}$ is the design bending resistance of the cross-section (see Chapter 8.3.);

χ is a buckling coefficient, and $\chi \le 1$;

γ_{M0} and γ_{M1} are the partial material factors given in Chapter 5.

The member buckling check may be omitted if the design compression force N_{Ed} does not exceed 4% of the sheet pile wall's elastic critical load N_{cr}, see also Eurocode 3 - Part 5 (exp. 5.11. or 15.12.), i.e.:

$$N_{Ed} \le 0.04\, N_{cr} = 0.04\left(\frac{E\, I\, \beta_D\, \pi^2}{\ell^2}\right)$$

where the symbols are as defined below.

The value of the buckling coefficient χ may be obtained from Fig. 8.4.

Fig. 8.4. Buckling coefficient.

Fig. 8.4. relates the buckling coefficient χ to the wall's non-dimensional slenderness ratio $\bar{\lambda}$, given by:

$$\bar{\lambda} = \sqrt{\frac{A f_y}{N_{cr}}} = \frac{\ell}{\pi} \sqrt{\frac{A f_y}{E I \beta_D}}$$

where

N_{cr} the wall's elastic critical load;

ℓ the wall's buckling length (see Fig. 8.5.);

E modulus of elasticity of steel;

I the section's second moment of area;

β_D a reduction factor accounting for insufficient shear force transmission in the interlocks (values of which are given below and only affect U profiles).

The value of χ can also be calculated with following equations:

$$\chi = \frac{1}{\Phi + \sqrt{\Phi^2 - \bar{\lambda}^2}}$$ but $\chi \leq 1$ (EN 1993-1-1 [ii]; eq. (exp. 6.49.)

where

$$\Phi = 0.5 \left(1 + \alpha \left(\bar{\lambda} - 0.2 \right) + \bar{\lambda}^2 \right)$$

with $\alpha = 0.76$ (EN 1993-1-1 [ii]; Table 6.1., curve *d*).

The buckling length ℓ of an embedded wall is defined in Eurocode 3 - Part 5 [i] as shown in Fig. 8.5.

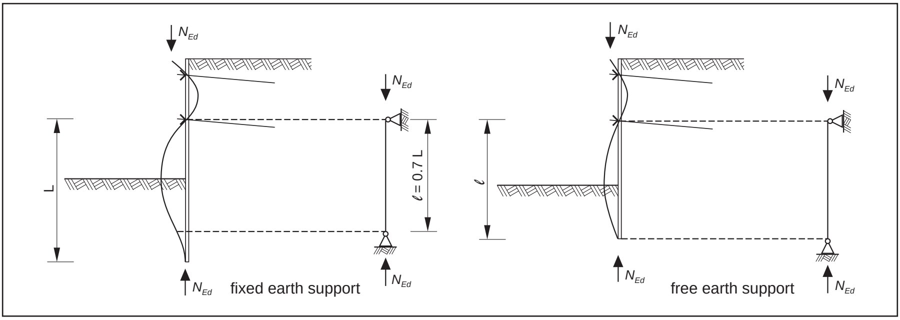


Fig. 8.5. Definition of buckling length.

A more rigorous approach is needed for cantilever walls, based on structural analysis without the presence of soil. This is outside the scope of the Piling Handbook.

Values of β_D for single and double piles are given in the National Annexes to Eurocode 3 - Part 5 [v]. Summarized in the Table 8.5. are the β_B-values from the UK National Annex [v]. These values depend on the number of structural supports and the conditions under which the piles are installed (see Chapter 8.3.).

For Z-type and for U-type triple piles, $\beta_D = 1.0$.

Type of U-sheet pile unit	No. of structural support levels	Conditions		
		Highly unfavourable	Unfavourable	Favourable
Singles or uncrimped doubles	0	0.30	0.35	0.40
	1	0.35	0.40	0.45
	>1	0.45	0.50	0.55
Crimped or welded doubles	0	0.60	0.65	0.70
	1	0.70	0.75	0.80
	>1	0.80	0.85	0.90

Table 8.5. Values of β_B for U-piles from the UK NA to BS EN 1993-5 [v].

Note: Values of β_B cannot exceed 1.0.

The coefficients β_D may be increased by a certain value if:

	Conditions		
	Highly unfavourable	Unfavourable	Favourable
Interlock not treated with sealants or lubrication	+0.05	+0.05	+0.05
Structural weld to prevent slippage	+0.15	+0.20	+0.25

Table 8.6. Correction of β_B values according to UK NA to BS EN 1993-S [v].

Example 1 (continued):

PU 18 steel sheet piles made of grade S 355 GP steel are to be driven as uncrimped doubles into favourable ground conditions (without pre-augering or lubrication of the interlock). The sheet piles will be supported by a single row of anchors, installed 1m below ground surface. Hence, from the table above, $\beta_D = 0.45 + 0.05 = 0.50$. The sheet piles will act under free earth conditions, with a buckling length $\ell = 3.5$ m.

The sectional properties of the PU 18 sheet pile are $A = 163.3$ cm^2/m and $I = 38650$ cm^4/m. The modulus of elasticity and yield strength of the steel are $E = 210$ GPa and $f_y = 355$ N/mm^2, respectively.

$$N_{cr} = \frac{E\,I\,\beta_D\,\pi^2}{\ell^2}$$
$$= \frac{210 \times 10^{-2} \times 38650 \times 0.5 \times \pi^2}{3.5^2}$$
$$= 32697 \text{ kN/m}$$

And:

$$N_{Ed} = 1500 > 1308 \text{ kN/m} \quad = 0.04\,N_{cr}$$

The slenderness ratio is then calculated as:

$$\bar{\lambda} = \frac{\ell}{\pi}\sqrt{\frac{A\,f_y}{E\,I\,\beta_D}} = \frac{3.5}{\pi}\sqrt{\frac{163.3\times 355\times 10^2}{2.10\times 10^3\times 38650\times 0.5}}$$
$$= 0.421$$

and hence (from the chart) $\chi = 0.84$.

The design bending resistance of the PU 18 sheet pile section is $M_{c,Rd} = 568$ kNm/m and its design plastic resistance is $N_{pl,Rd} = 5797$ kN/m.
The PU 18 is a Class 2 section. If a design axial compressive force $N_{Ed} = 1500$ kN/m is applied to the wall, then the applied design moment M_{Ed} must satisfy:

$$M_{Ed} \le \frac{M_{c,Rd}}{1.15}\left(\frac{\gamma_{M0}}{\gamma_{M1}} - \frac{N_{Ed}}{\chi\,N_{pl,Rd}}\right)$$
$$\le \frac{568}{1.15}\times\left(\frac{1.0}{1.1} - \frac{1500}{0.84\times 5797}\right) = 297\ \text{kNm/m}$$

8.10. Shear buckling

Eurocode 3 - Part 5 [i] requires the shear buckling resistance of the cross-section to be verified if:

$$\frac{c}{t_w} > 72\,\varepsilon$$

where c is the slant height of the web (see Fig. 8.1.).
For Z-profiles, the value of c is given by:

$$c = \frac{h - t_f}{\sin\alpha}$$

and for U-profiles by:

$$c = \frac{h - t_f}{2\sin\alpha}$$

where α is the inclination of the web (see Fig. 8.1.).

Section	c	t_w	S 240 GP $\varepsilon = 0.99$	S 270 GP $\varepsilon = 0.93$	S 320 GP $\varepsilon = 0.86$	S 355 GP $\varepsilon = 0.81$	S 390 GP $\varepsilon = 0.78$	S 430 GP $\varepsilon = 0.74$	S 460 AP $\varepsilon = 0.71$
	mm	mm				$(c/t_w)/\varepsilon$			
AZ-700 and AZ-770									
AZ 12-770	527.5	8.5			72.4	76.3	79.9	83.9	86.8
AZ 13-770	526.7	9.0					75.4	79.2	81.9
AZ 14-770	527.5	9.5						75.1	77.7
AZ 14-770-10/10	526.7	10.0							73.7
AZ 12-700	449.6	8.5							74.0
AZ 13-700	449.6	9.5							
AZ 13-700-10/10	449.6	10.0							
AZ 14-700	449.6	10.5							
AZ 17-700	528.0	8.5			72.5	76.3	80.0	84.0	86.9
AZ 18-700	527.4	9.0					75.5	79.3	82.0
AZ 19-700	528.0	9.5						75.2	77.8
AZ 20-700	527.4	10.0							73.8
AZ 24-700	545.3	11.2							
AZ 26-700	545.3	12.2							
AZ 28-700	545.3	13.2							
AZ 24-700N	543.8	9.0				74.3	77.8	81.7	84.5
AZ 26-700N	543.8	10.0						73.6	76.1
AZ 28-700N	543.8	11.0							
AZ 36-700N	542.2	11.2							
AZ 38-700N	542.2	12.2							
AZ 40-700N	542.2	13.2							
AZ 42-700N	538.9	14.0							
AZ 44-700N	538.9	15.0							
AZ 46-700N	538.9	16.0							
AZ									
AZ 17	450.1	8.5							74.1
AZ 18	450.1	9.5							
AZ 18-10/10	450.7	10.0							
AZ 19	450.1	10.5							
AZ 25	485.6	11.2							
AZ 26	485.6	12.2							
AZ 28	485.6	13.2							
AZ 46	488.2	14.0							
AZ 48	488.2	15.0							
AZ 50	488.2	16.0							

Table 8.7. Table of Z-sections with $(c/tw)/\varepsilon > 72$.

Note: No U-type sheet pile from ArcelorMittal's existing range exceeds this ratio.

Design for shear buckling involves verifying that the design shear force V_{Ed} in each cross-section does not exceed its corresponding design shear buckling resistance $V_{b,Rd}$. This is expressed in Eurocode 3 - Part 5 [i] by the inequality:

$$V_{Ed} \leq V_{b,Rd}$$

The design shear buckling resistance *of each web* is given by:

$$V_{b,Rd} = \frac{A_V\ f_{bv}}{\gamma_{M0}}$$

where f_{bv} is the steel's shear buckling strength.

Values of f_{bv} are given in Eurocode 3 - Part 1-3 [viii] and summarized in the table below.

Relative web slenderness $\bar{\lambda}$	**Value of** f_{bv}
≤ 0.83	$0.58\ f_{yb}$
0.83 - 1.40	$0.48\ f_{yb} / \bar{\lambda}$
≥ 1.40	$0.67\ f_{yb} / \bar{\lambda}^2$

Table 8.8. Values of f_{bv} from Eurocode 3 - Part 1-3 [viii].

where

f_{yb} basic yield strength of steel (i.e. before cold-forming);

$\bar{\lambda}$ is the web slenderness, according to Eurocode 3 - Part 5 [i] (exp. 5.8.).

$$\bar{\lambda} = 0.346 \frac{c}{t_w} \sqrt{\frac{f_y}{E}}$$

8.11. Thin walled steel sheet piling

The resistance and stiffness of steel sheet piling with Class 4 cross-sections should be designed according to Annex A of Eurocode 3 - Part 5 [i].

As explained in Chapter 8.2., Class 4 sections are those whose flange slenderness ratio is given by:

$$\frac{b}{t_f} > 66\,\varepsilon \qquad \text{for Z-profiles}$$

$$\frac{b}{t_f} > 49\,\varepsilon \qquad \text{for U-profiles}$$

where

b = flange width;

t_f = flange thickness;

ε is a coefficient that depends on steel grade (see Chapter 5).

The design resistance of a Class 4 cross-section should be determined either by calculation (in accordance with Annex A.6. of Eurocode 3 - Part 5) or by testing (in accordance with Annex A.7.). In design by calculation, the resistance of the cross-section should be verified for:

- bending (together with local transverse bending);
- local transverse forces;
- combined bending and shearing;
- combined bending and axial compression;
- combined bending and local transverse forces.

The effect of local buckling should be taken into account by using effective cross-sectional properties as specified in Annex A.4 of Eurocode 3 - Part 5.

Detailed procedures are given in Eurocode 3 - Part 1-3 [viii] to allow the effects of local and distortional buckling on the resistance and stiffness of cold-formed members and sheeting to be determined. These procedures are outside the scope of the Piling Handbook. Fig. 8.6. illustrates the influence of member length on the buckling resistance of steel members. Not to lose efficiency, the avoidance of class 4 sections by e.g. the use of a lower steel grade is recommended, if possible.

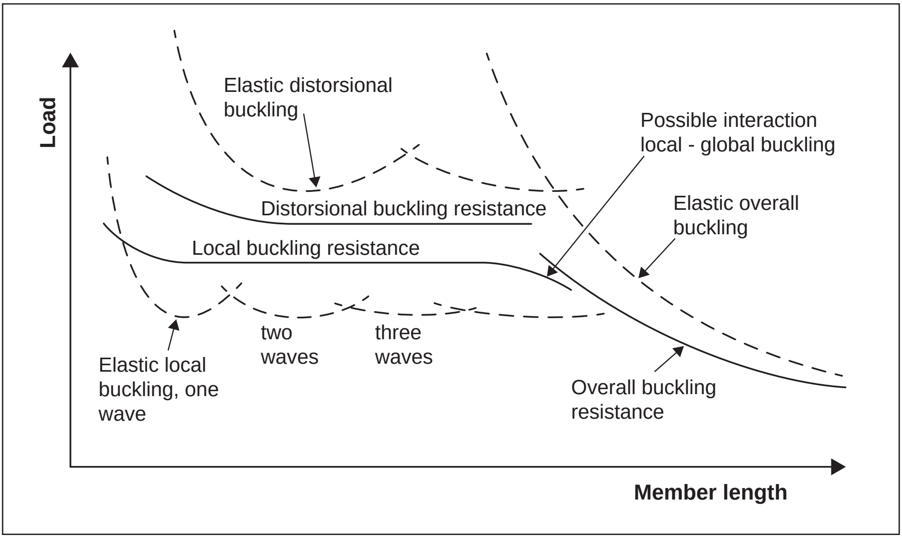


Fig. 8.6. Correlation member length with buckling resistance.

8.12. Local effects of water pressure

The overall bending resistance of a sheet pile wall may be reduced by transverse local plate bending, which can occur when the wall retains water at different levels on its opposing sides (resulting in differential water pressures across the wall).

In the presence of differential water pressure, the design bending resistance of the cross-section (see Chapter 8.3.) should be calculated with a reduced yield strength $f_{y,red}$ given by:

$$f_{y,red} = \rho_p \, f_y$$

where

f_y is the steel's full yield strength;

ρ_P is a reduction factor that depends on the slenderness ratio $(b/t_{min})\ \varepsilon$;

where

b is the pile's flange width;

t_{min} is the smaller of its flange thickness t_f and its web thickness t_w;

the coefficient ε is defined in Chapter 5.

Values of ρ_P taken from Eurocode 3 - Part 5 [i] (Table 5.2.) are given in Table 8.9.

$(b/t_{min})\varepsilon$	Differential head, w (m)			
	5	**10**	**15**	**20**
20	1.00	0.99	0.98	0.98
30	1.00	0.97	0.96	0.94
40	1.00	0.95	0.92	0.88
50	1.00	0.87	0.76	0.60

Table 8.9. Values of the reduction factor ρ_P [i].

Note:
ρ_P = 1 for Z-piles with welded interlocks
b should not be less than $c/\sqrt{2}$

The local effects of water pressure on overall bending resistance may be neglected if the difference in water level Δh_w across the wall is:

$$\Delta h_w \leq 5 \text{ m}$$

for Z-profiles with uncrimped or unwelded interlocks or

$$\Delta h_w \leq 20 \text{ m}$$

for U-profiles.

Example (modified from Section 8.3.):

AZ 18-700 steel sheet piles made of grade S 355 GP steel are to retain water with w = 10 m of differential head. The section's dimensions are b = 352.8 mm, t_f = 9.0 mm, and t_w = 9.0 mm (hence t_{min} = 9.0 mm).
From Chapter 8.3, β_B = 1.0 for a Z-pile.

The modified slenderness ratio of the web is:

$$\left(\frac{b}{t_{min}}\right)\varepsilon = \left(\frac{352.8}{9.0}\right) \times \sqrt{\frac{235}{355}} = 31.9$$

Interpolating from the table above, the reduction factor ρ_P = 0.967 and hence the steel's yield strength is reduced to:

$$f_{y,red} = \rho_p\, f_y = 0.967 \times 355 = 343.3 \text{ N/mm}^2$$

The AZ 18-700 is a Class 3 section, hence its elastic section modulus W_{el} = 1800 cm^3/m should be used. With the partial factor γ_{M0} = 1.0, the design bending resistance of the cross-section is calculated as:

$$M_{c,Rd} = \frac{\beta_B\, W_{el}\, f_{y,red}}{\gamma_{M0}} = \frac{1.0 \times 1800 \times 343.3}{1.0 \times 10^3} = 617.9 \text{ kNm/m}$$

This compares with $M_{c,Rd}$ = 639 kNm/m, as calculated in Chapter 8.3., in the absence of a differential head of water.

8.13. Connections

Structural analysis should take into account any connections that have a major influence on the distribution of internal forces and moments in the structure. Anchors may be modelled as simple supports or springs.

Eurocode 3 - Part 5 [i] requires the resistance of sheet piles to the introduction, via walings or washer plates, of anchor or strut forces into their webs, as illustrated in the figure below for the following situations:

1. connection on pile's in-pan, with waling behind wall
   a. using an anchor;
   b. using a tie-rod;
2. connection on pile's out-pan, without waling;
3. connection on pile's in-pan, without waling;
4. connection on waling in front of wall.

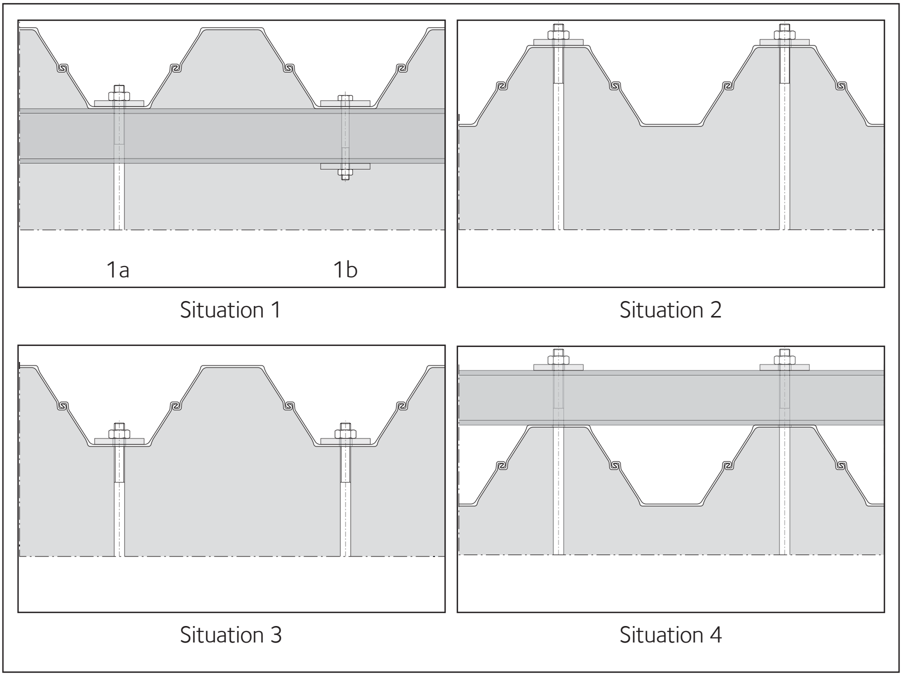


Fig. 8.7. Possible tie-back connections in U-pile walls.

8.13.1. Dimensions of washer plate

The dimensions of the washer plate in situations 1a, 1b, and 3 above should satisfy the following inequalities:

$$b_a \geq 0.8\, b$$

where:

b_a is the width of the washer plate;

b is the flange width of the sheet pile (see Figure in Table 5.1. of EC 3-5);

and also:

$$t_a \geq 2\, t_f$$

where:

t_a is the thickness of the washer plate;

t_f is the sheet pile's flange thickness.

A smaller value of b may be assumed, provided flange bending is checked.
The washer plate should also be checked for bending.

Example:

A washer plate is needed to support a connection between an anchor and a PU 18 steel sheet pile. The width of the section's web is b = 288.5 mm and its flange thickness t_f = 11.2 mm. The plate's breadth and thickness should therefore satisfy:

$$b_a \geq 0.8\, b = 0.8 \times 288.5 = 230.8\,\text{mm}$$

and

$$t_a \geq 2\, t_f = 2 \times 11.2 = 22.4\text{ mm}$$

The washer plate should also be checked for bending.

8.13.2. Shear resistance of flange

The shear resistance of the pile's flange in situations 1a, 1b, and 3 above may be verified by checking that the applied design local transverse force F_{Ed} does not exceed the flange's shear resistance $R_{Vf,Rd}$. This is expressed in Eurocode 3 - Part 5 [i] by the inequality:

$$F_{Ed} \leq R_{Vf,Rd}$$

The flange's shear resistance is given by:

$$R_{Vf,Rd} = 2\left(b_a + h_a\right) t_f \, \frac{f_y}{\sqrt{3}\,\gamma_{M0}}$$

with

b_a and h_a the width and length, respectively, of the washer plate;

t_f is the pile's flange thickness;

f_y is the yield strength of steel;

γ_{M0} a partial factor whose value is given in Chapter 5.

The value of h_a in this equation must be limited to 1.5 b_a.

Example:

A design local transverse force F_{Ed} is applied to the flange of a PU 18 steel sheet pile made of grade S 355 GP steel. A washer plate with dimensions b_a = 250 mm and h_a = 175 mm is used to transfer the force to the sheet pile. The section's flange thickness is t_f = 11.2 mm and the steel's yield strength f_y = 355 N/mm^2. With the partial factor γ_{M0} = 1.0, the flange's design shear resistance is calculated as:

$$R_{Vf,Rd} = 2 \times \left(250 + 175\right) \times 11.2 \times \frac{355}{\sqrt{3} \times 1.0 \times 10^3} = 2410\,\text{kN}$$

The design local transverse force F_{Ed} must then satisfy:

$$F_{Ed} \leq R_{Vf,Rd} = 2410\,\text{kN}$$

8.13.3. Tensile resistance of webs

The tensile resistance of the pile webs in situations 1a, 1b, and 3 above may be verified by checking that the applied design local transverse force through the flange F_{Ed} does not exceed the web's tensile resistance $R_{tw,Rd}$. This is expressed in Eurocode 3 - Part 5 [i] by the inequality:

$$F_{Ed} \leq R_{tw,Rd}$$

The tensile resistance of two adjacent webs is given by:

$$R_{tw,Rd} = \frac{2\, h_a\, t_w\, f_y}{\gamma_{M0}}$$

where

h_a is the length of the washer plate;

t_w is the pile's web thickness;

f_y is the yield strength of steel;

γ_{M0} a partial factor whose value is given in Chapter 5.

Example:

The PU 18 sheet pile from the example in Chapter 8.13.2. has a web thickness t_w = 9.0 mm. The web's design tensile resistance is calculated as:

$$R_{tw,Rd} = \frac{2\, h_a\, t_w\, f_y}{\gamma_{M0}} = \frac{2 \times 175 \times 9.0 \times 355}{1.0 \times 10^3} = 1118 \text{ kN}$$

The design local transverse force F_{Ed} must then satisfy:

$$F_{Ed} \leq R_{tw,Rd} = 1118 \text{ kN}$$

8.13.4. Compressive resistance of webs

The compressive resistance of the pile webs in situations 2 and 4 above may be verified by checking that the design local transverse force applied to each web F_{Ed} satisfies the following inequality

$$\frac{F_{Ed}}{R_{c,Rd}} + 0.5\,\frac{M_{Ed}}{M_{c,Rd}} \leq 1.0$$

where

$R_{c,Rd}$ is the design compressive resistance of the web;

$M_{c,Rd}$ is the sheet pile's design bending resistance (see Chapter 8.3.);

M_{Ed} is the design bending moment applied to the pile at the location of the anchor or strut force.

Verification of the compressive resistance of the pile webs in situations 2 and 4 may be omitted if:

$$F_{Ed} \leq \frac{R_{c,Rd}}{2}$$

where the symbols are defined above.

The web's compressive resistance is given by the smaller of its elastic and plastic compressive resistances, $R_{e,Rd}$ and $R_{p,Rd}$ respectively.

The web's elastic compressive resistance $R_{e,Rd}$ is given by:

$$R_{e,Rd} = \frac{\varepsilon}{4\,e}\left(s_s + 4\left[\frac{2\,\pi\,r_0\,\alpha}{180^\circ}\right]\right)\sin\alpha\,\left(t_w^2 + t_f^2\right)\frac{f_y}{\gamma_{M0}}$$

where

ε is a coefficient that depends on steel grade (see Chapter 5);

e is the eccentricity of the force introduced into the web (defined below);

s_S is the length of stiff bearing determined in accordance with Eurocode 3 - Part 1-5 [ix];

r_0 is the outside radius of the corner between the flange and the web;

α is the web's inclination (entered in degrees);

t_w and t_f are the section's web and flange thicknesses, respectively;

f_y is the yield strength of steel;

γ_{M0} is a partial factor.

The web's plastic compressive resistance $R_{p,Rd}$ is given by:

$$R_{p,Rd} = \frac{\chi\, R_{p0}}{\gamma_{M0}} = \left(0.06 + \frac{0.47}{\sqrt{R_{p0}/R_{cr}}}\right)\frac{R_{p0}}{\gamma_{M0}}$$

where

the bracketted term χ (not the same as the buckling coefficient in (Chapter 8.9.) must be ≤ 1 ($\chi \le 1.0$)

γ_{M0} is defined above;

and the intermediate terms R_{p0} and R_{cr} are given by:

$$R_{p0} = \sqrt{2}\ \varepsilon\, f_y\ t_w\ \sin\alpha\left(s_s + t_f\sqrt{\frac{2\,b\sin\alpha}{t_w}}\right)$$

and:

$$R_{cr} = 5.42\,E\,\frac{t_w^3}{c}\sin\alpha$$

where

E is the modulus of elasticity of steel;

c is the section's slant height (as defined in Chapter 8.4.);

and the other terms are defined above.

The eccentricity of the force introduced into the web e is given by:

$$e = r_0\ tan\left(\frac{\alpha}{2}\right) - \frac{t_w}{2\sin\alpha} \quad \text{but} \quad e \ge 5\text{ mm}$$

In situation 2, the dimensions of the washer plate should be greater than the sheet pile's flange width, to avoid increasing the eccentricity above the value calculated by this equation.

Example:

A PU 18 sheet pile has dimensions t_w = 9.0 mm, t_f = 11.2 mm, h = 430 mm, α = 57.5 °, and r_0 = 15 mm. For Grade S 355 GP steel, ε = 0.81 and E = 210 GPa.

The eccentricity of loading is:

$$e = r_0\ \tan\left(\frac{\alpha}{2}\right) - \frac{t_w}{2\sin\alpha}$$

$$= 15\times\tan\left(\frac{57.5^\circ}{2}\right) - \frac{9.0}{2\times\sin 57.5^\circ}$$

$$e = 2.89\text{ mm} \quad \text{but} \quad e \ge 5\text{ mm}$$

A minimum value e = 5 mm must be assumed. Hence, the web's design elastic compressive resistance, assuming the length of stiff bearing s_S = 43 mm, is:

$$R_{e,Rd} = \frac{0.81}{4 \times 5} \times \left(43 + 4 \times \left[\frac{2 \times \pi \times 15 \times 57.5°}{180°} \right] \right)$$
$$\times \sin 57.5° \times \left(9.0^2 + 11.2^2\right) \times \frac{355}{1.0}$$
$$= 409 \text{ kN}$$

The web's design plastic compressive resistance is calculated from:

$$R_{p0} = \sqrt{2} \times 0.81 \times 355 \times 9.0 \times \sin 57.5°$$
$$\times \left(43 + 11.2 \times \sqrt{\frac{2 \times 288.5 \times \sin 57.5°}{9.0}} \right)$$
$$R_{p0} = 380 \text{ kN}$$

The slant height of a U-profile is calculated from (see Chapter 8.4.):

$$c = \frac{h - t_f}{2 \sin \alpha} = \frac{430 - 11.2}{2 \times \sin 57.5°} = 248 \text{ mm}$$

Hence:

$$R_{cr} = 5.42 \times 210 \times 10^3 \times \frac{9.0^3}{248} \times \sin 57.5° = 2822 \text{ kN}$$

$$\chi = 0.06 + \frac{0.47}{\sqrt{380/2822}} = 1.34$$

but $\chi \le 1.0$

$$R_{p,Rd} = \frac{1.0 \times 380}{1.0} = 380 \text{ kN}$$

The design compressive resistance $R_{c,Rd}$ is then the lesser of $R_{e,Rd}$ = 409 kN and $R_{p,Rd}$ = 380 kN. To avoid the more complicated verification involving interaction with bending moments, the design local transverse force F_{Ed} must satisfy:

$$F_{Ed} \le \frac{380}{2} = 190 \text{ kN} = \frac{R_{c,Rd}}{2} \quad \text{(per web)}$$

8.13.5. Crimps and intermittent welds

Threaded AZ double piles are recommended for facilitating the installation process. AZ double piles are not crimped for statical reasons. However, due to customer demand, most of our AZ piles are crimped according to our standard specification, for the following reasons:

- single piles easily bend around the weak axis under driving;
- faster installation progress with double piles.

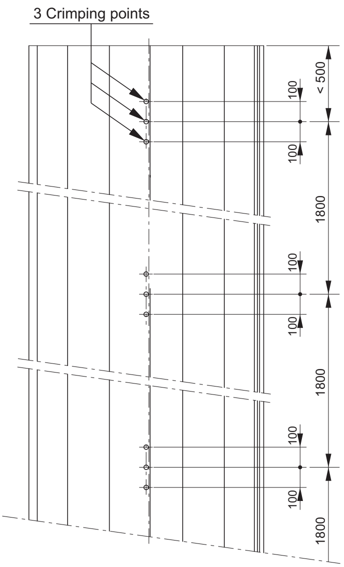
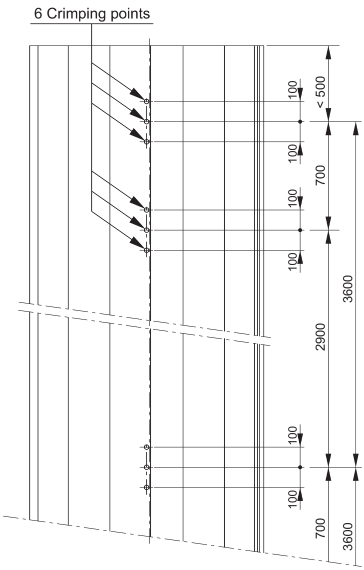
<table>
<tr><th colspan="2">AZ standard crimping pattern</th></tr>
<tr><td>Pile length < 6 m:
3 crimping points per 1.8 m = 1.7 crimping points per m[1)]</td><td>Pile length ≥ 6 m:
6 crimping points per 3.6 m = 1.7 crimping points per m[1)]</td></tr>
<tr><td>
</td><td>
</td></tr>
</table>

Fig. 8.8. AZ Standard crimping pattern.

Notes:

1) Amount and layout of crimping points may differ at both ends. Special crimping on request.

2) Based on EN1993-5. See Table 8.2. and 8.4. for reduction factors β_B and β_D .

For more detailed information, see EN 1993 - Part 5 (§5.2.2 14(P) and (15), §6.4.).

Contrary to Z-piles, the interlocks of U-piles have to transmit shear forces on the neutral axis. To guarantee proper shear force transmission, the interlocks of U-sections can be delivered as crimped double piles or with intermittently welded common interlocks. See sketch for ArcelorMittal's standard crimping pattern.

Please note that the theoretical section properties of a continuous wall may have to be reduced even for double piles crimped[2)].

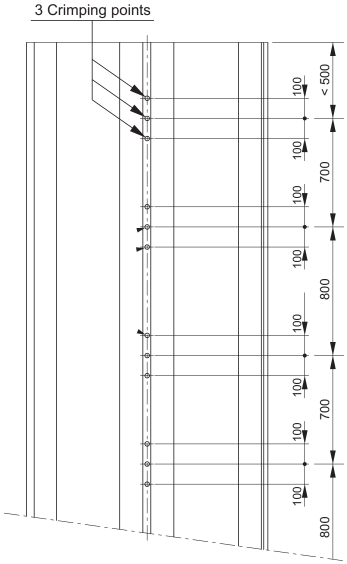
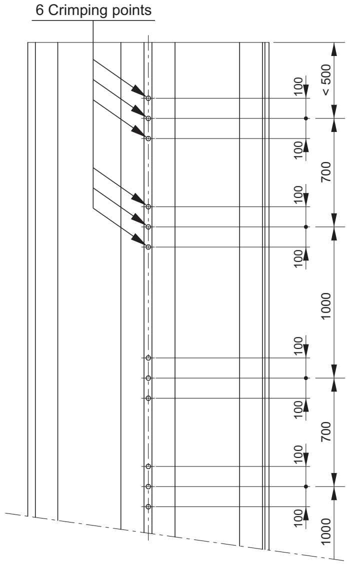
AU standard crimping	PU/GU standard crimping
3 crimping points per 0.75 m = 4 crimping points per m[1)]	6 crimping points per 1.7 m = 3.5 crimping points per m[1)]
	

Fig. 8.9. U-pile standard crimping pattern.

Notes:
1) Amount and layout of crimping points may differ at both ends. Special crimping on request.
For more detailed information, see EN 1993 - Part 5 (§5.2.2 14(P) and (15), §6.4.).

At **ULS**, it shall be verified that the crimped points are able to transmit the shear stresses t_{Ed} :

$$t_{Ed} = V_{Ed} \frac{S}{I}$$

where:

V_{Ed} is the design value of the shear force at ultimate limit state;

S is the static moment of the cross-section portion to be connected, referred to the centroidal axis of the connected sheet pile wall;

I is the moment of inertia of the connected sheet piling.

If the spacing of triple crimp points does not exceed 1.0 m, each crimp point may be assumed to transmit an equal shear force of

$$T_{Ed} \leq \frac{R_k}{\gamma_{M0}}$$

where:

R_k is the characteristic resistance of the crimped point determined by testing according to EN 10248;

γ_{M0} is a partial safety factor.

R_k varies by sheet pile section and steel grade. Please contact the technical department to obtain these values.

At **SLS**, it must be checked that the crimped points are able to transmit the required interlock shear stress. This criteria must be guaranteed by the manufacturer / supplier.

The shear stresses t_{ser} in the interlocks are determined by the equation:

$$t_{ser} = V_{ser} \frac{S}{I}$$

where:

V_{ser} is the design value of the shear force at serviceability limit state,

S and I are defined above.

It must be checked that:

$$T_{ser} \leq R_{ser}$$

where:

T_{ser} is the shear force per crimp at serviceability limit state;

R_{ser} is the resistance of the crimped point at serviceability limit state, and R_{ser} = 75 kN/crimp as per EN 1993-5.

Provided the spacing criterion given above is fulfilled, the verification of the crimps is done per segments assuming a mean value of the shear forces over this segment length. The segment is defined as the length between a zero shear force point and the adjacent maximum shear force point.

The shear force per crimp T_{ser} in the segment is obtained as

$$T_{ser} = \frac{T_{ser,\,Res}}{n}$$

where:

n is the number of crimps in the segment;

$T_{ser,Res}$ is the resultant shear force calculated by integrating the interlock shear stresses t_{ser} over the segment.

At ULS the same procedure applies.

Note: The manufacturer has verified by testing according to EN 10248 [x] that the stiffness of the crimped point is higher than 15 kN/mm (which corresponds to a shear force of 75 kN at a displacement of 5 mm)

For interrupted welds, the shear stress to section 4.9. of EN 1993-1-8 [xi] should be set correspondingly higher.

The verification of the welds is to be carried out according to section 4.5.4. of EN 1993-1-8, in which the plastic analysis - assuming a uniform shear stress - according to (1) of section 4.9. of EN 1993-1-8 is permitted. For steel grades with yield stresses not covered by table 4.1. of EN 1993, the β_w value may be obtained through linear interpolation.

In practice, the verification of the crimping can be done as described below.

Subdivide the length of the pile in several segments (shear force diagram).

At ULS, from the bending moment diagram, or as a simplification from the shear force diagram (assume a linear distribution of the shear force), calculate for each relevant segment the shear force that needs to be transmitted over the "segment".

If ℓ_V is the length of the segment, and M_1 and M_2 the bending moments at both extremities, then the shear force over the segment V'_{Ed} is calculated as:

$$V'_{Ed} = |M_1| + |M_2|$$

Hence, per interlock, the shear force to be transmitted over this segment is:

$$T'_{Ed} = (V'_{Ed}\ 2\ B)\frac{S}{I}$$

The number of crimping points n' over the length ℓ_V required to resist this shear force is

$$T'_{Ed} \leq n\ \frac{R_k}{\gamma_{M0}}$$

$$n' \geq \frac{T'_{Ed}}{R_k/\gamma_{M0}} = \frac{(V'_{Ed}\ 2\ B)\frac{S}{I}}{R_k/\gamma_{M0}}$$

Finally, n is the minimum number of crimping point per running metre of interlock, calculated as:

$$n \geq \frac{\left(V'_{Ed}\ 2\,B\right)\dfrac{S}{I}}{\ell_V\,R_k/\gamma_{M0}}$$

Note: from a practical point of view, whenever possible, consider the maximum crimping pattern required (n) for all the segments and apply it to the whole pile.

Repeat the same procedure for the serviceability limit state (SLS) with V_{ser} and R_{ser} :

$$n \geq \frac{\left(V'_{ser}\ 2\,B\right)\dfrac{S}{I}}{\ell_V\ R_{ser}}$$

Example:

A PU 18 sheet pile has static moment $S = 1055$ cm^3/m, a moment of inertia $I = 38650$ cm^4/m, and a characteristic resistance of each crimped point $R_k = 98.5$ kN (at a displacement of 10 mm based on EN 1993-5). The width of a single pile is $B = 600$ mm. Sheet piles are delivered as double piles with each common interlock crimped.

From the bending moment diagramm, the sheet pile can be subdivided in 4 segments of shear transmission, see Fig. 8.10.

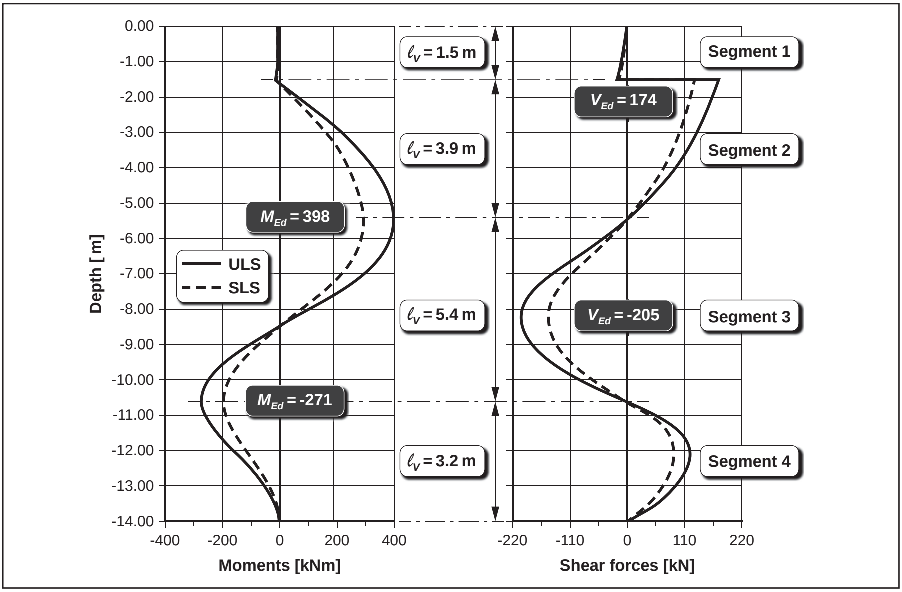


Fig. 8.10. Moment distribution example along a pile.

For segment 2, $\ell_V = 3.9$ m.

From the bending moment diagram (see Fig. 8.10.):

$$V'_{Ed} = |-15| + |398| = 413 \text{ kN}$$

At ultimate limit state, the minimum number of crimping point per running metre *n* is :

$$n \geq \frac{\left(V'_{Ed}\ 2\,B\right)\dfrac{S}{I}}{\ell_V\ \dfrac{R_k}{\gamma_{M0}}}$$

$$n \geq \frac{(413 \times 2 \times 0.6) \times \dfrac{1055}{38650}}{3.9 \times \dfrac{98.5}{1.0}} = 3.52$$

At SLS, the stiffness guaranteed by the manufacturer, based on laboratory testing, is 25.3 kN/mm > 15 kN/mm.

Additionally:

$$V'_{ser} = |-11| + |295| = 306 \text{ kN}$$

$$n \geq \frac{\left(V'_{ser}\ 2\,B\right)\dfrac{S}{I}}{\ell_V\ R_{ser}}$$

$$n \geq \frac{(306 \times 2 \times 0.6) \times \dfrac{1055}{38650}}{3.9 \times 75} = 3.43$$

Conclusion: the standard crimping pattern for U-type piles of 3.5 crimps per metre of interlock is sufficient.

8.14. Fatigue

In general, fatigue is a phenomenon that does not affect sheet pile walls. One exception might be structures that are submitted to cyclic loads, like waves.

For instance, it is advised to avoid building breakwaters with cantilever walls, unless the quite complex influence of this particular load case is taken into account. An anchor or strut will limit the deflections and the stresses.

Quay walls are backfilled after the installation of the sheet piles. However, if this backfill will only be executed years after the installation, temporary measures might be required to prevent damage of the structure during that temporary phase. For more information, please contact our technical service.

References:

[i] EN 1993, Eurocode 3 – Design of steel structures, Part 5: Piling, European Committee for Standardization, Brussels.

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[iv] Arbed Group (undated). AZ sheet piles in combined walls.

[v] NA to EN 1993-5: 2007, UK National Annex to Eurocode 3: Design of steel structures – Part 5: Piling, British Standards Institution, London.

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[viii] EN 1993, Eurocode 3 – Design of steel structures, Part 1-3: Supplementary rules for cold-formed members and sheeting, European Committee for Standardization, Brussels.

[ix] EN 1993, Eurocode 3 – Design of steel structures, Part 1-5: Plated structural elements, European Committee for Standardization, Brussels.

[x] EN 10248: Hot rolled steel sheet piling. 1995.

[xi] EN 1993-1-8, Eurocode 3: Design of steel structures, Part 1-8: Design of joints. 2005.

[xii] The HZ®-M Steel Wall System. ArcelorMittal 2014.

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9 | Cofferdams

Chapter 9 - Cofferdams

Contents

9.1. Introduction

The purpose of a cofferdam is to exclude soil and/or water from an area in which it is required to carry out construction work to a depth below the surface. Total exclusion of water is often unnecessary, and in some instances may not be possible, but the effects of water ingress must always be taken into account in any calculations.

For basement construction the designer should always consider incorporating the cofferdam into the permanent works. Considerable savings in both, time and money, can be achieved by using the steel sheet piles as the primary permanent structural wall. The wall can be designed to carry vertical loading, see Chapter 6 and, by the use of a suitable sealant system, be made watertight. Details of suitable sealant systems can be found in Chapter 2.

Where control of ground movement is a specific concern the use of top down construction should be considered. This will ensure that movement at the top of the wall is restricted with the introduction of support at ground level prior to excavation starting. Further it will also remove the possibility of secondary movement occurring when the lateral soil loading is transferred from the temporary supports, as they are removed, to the permanent structure.

There are two principal approaches to cofferdam design. Single skin structures are most commonly used but for very large or deep excavations and marine works, double wall or cellular gravity structures may be preferred.

9.2. Requirements of cofferdams

The design of a cofferdam must satisfy the following criteria:

- the structure must be able to withstand all the various loads applied to it;
- the quantity of water entering the cofferdam must be controllable by pumping;
- at every stage of construction the formation level must be stable and not subject to uncontrolled heave, boiling or piping;
- deflection of the cofferdam walls and bracing must not affect the permanent structure or any existing structure adjacent to the cofferdam;
- overall stability must be shown to exist against out of balance earth pressures due to sloping ground or potential slip failure planes;
- the cofferdam must be of an appropriate size to suit the construction work to be carried out inside it;
- temporary cofferdams must be built in such a way that the maximum amount of construction materials can be recovered for reuse.

9.3. Planning a cofferdam

The designer of a cofferdam must have an established set of objectives before commencing the design. The sequence of construction activities must be defined in order that the design can take into account all the load cases associated with the construction and dismantling of the cofferdam. From this sequence, the designer can identify the critical design cases and hence calculate the minimum penetrations, bending moments and shear forces to determine the pile section and length required.

As part of the analysis of the construction activities, the designer should undertake a risk assessment of the effect of any deviation from the planned sequence. Such deviations may be in the form of over excavation at any stage, inability to achieve the required pile penetration, installation of the support at the wrong level or the imposition of a large surcharge loading from construction plant or materials. If any stage in the cofferdam construction is particularly vulnerable, then contingency plans should be developed to minimise any risk and the site management should be informed to limit the possibility of critical conditions being realised.

The majority of cofferdams are constructed as temporary works and it may be uneconomic to design for all possible loading cases. Decisions will have to be taken, normally involving the site management, to determine the level of risk that is acceptable when assessing the design cases; such a situation may occur when assessing hydraulic loading on a cofferdam. Flood conditions tend to be seasonal and provision of a cofferdam which will exclude water at all times may involve a substantial increase in pile size and strength as well as increased framing. In an extreme flood condition, the design philosophy may involve evacuation of the cofferdam and allowing it to overtop and flood. Under these conditions the designer must allow for the overtopping, considering the effect of the sudden ingress of water on the base of the cofferdam and the effect that any trapped water may have on the stability when the flood subsides.

Prior to the commencement of construction the site area should be cleared to permit plant and guide frames to be set up. Excavation should not begin until all the plant and materials for supporting the piles are readily available including pumping equipment where necessary.

Once excavation is complete the cofferdam and support frames should be monitored to ensure that they are performing as expected and to provide as early as possible a warning of any safety critical problems. It is good practice to maintain a written record of such monitoring - in the UK this is a legal requirement. Some possible causes of failure are given in Chapter 9.4. and it will be seen that a number of them relate to problems that may well occur after the cofferdam is finished.

9.4. Causes of failure

There are many possible causes of cofferdam failure but in practice it can generally be attributed to one or more of the following:

- lack of attention to detail in the design and installation of the structure;
- failure to take the possible range of water levels and conditions into account;
- failure to check design calculations with information discovered during excavation;
- over excavation at any stage in the construction process;
- inadequate framing (both quantity and strength) provided to support the loads;
- loading on frame members not taken into account in the design such as walings and struts being used to support walkways, materials, pumps etc.;
- accidental damage to structural elements not being repaired;
- insufficient penetration to prevent piping or heave;
- failure to allow for the effect on soil pressures of piping or heave;
- lack of communication between temporary works and permanent works designers, designers and site management or site management and operatives.

In many cases failure may result from the simultaneous occurrence of a number of the above factors, any one of which might not have been sufficient, on its own, to cause the failure.

9.5. Support arrangements

The arrangement of supports to a cofferdam structure is the most critical part of a cofferdam design. The level at which the support is provided governs the bending moments in the sheet piles and the plan layout governs the ease of working within the structure. Whilst structural integrity is paramount, the support layout must be related to the proposed permanent works construction activities causing the minimum obstruction to plant and materials access. As a general rule simplicity should always be favoured.

Support frames should be located such that concrete lifts can be completed and the support load transferred to the permanent works before the frame is removed. Clearance to starter bars for the next lift should be considered when positioning frames.

The clear space between frame members should be optimised to provide the largest possible uninterrupted area without the need for excessively large structural elements. Positioning of support members is often a matter of experience.

9.6. Single skin cofferdams

Single skin cofferdams are typically formed of sheet piles supported either by means of internal props or external anchors. The mechanics of single skin cofferdam design are based on embedded retaining wall design already outlined in Chapters 4 & 5. The piles are considered to be simply supported between frames and below the lowest frame and will need to be driven to such a level, depending on the type of soil, as to generate sufficient passive resistance. However, where there are at least two frames, if the cut-off of the piles below the excavation is insufficient to provide the necessary passive support the wall might still be stable and the pile below the lowest frame can be considered as a cantilever. This will, however, give rise to large loads in the lowest frame and should be avoided whenever possible.

In all cases the penetration below formation level will need to be sufficient to control the infiltration of water into the excavation.

Records should be kept during driving for any indication of declutching of the piles. In such a case it may be necessary to grout behind the piles in order to control seepage. Cantilever pile cofferdams can be formed, but have the same limitations as cantilever retaining walls, particularly in terms of the achievable retained height.

When the cofferdam has very large plan dimensions, but relatively shallow depth, it is often more economical to incorporate inclined struts or external anchorages similar to those described in Chapter 7. It should not however be forgotten that the installation of external anchorages requires space which is outside the cofferdam area and way-leaves may be required to install the anchors under adjacent properties.

For a typical cofferdam with a depth exceeding 3 m, a system of internal frames in the form of steel sections or proprietary bracing equipment is normally employed.

The design should be undertaken in stages to reflect accurately the construction process. Typically the sequence of operations would be to excavate and dewater to just below top frame level then install the first frame; this procedure being repeated for each successive frame. In the case of cofferdams in water, it should be noted, that the stresses occurring during dewatering and frame installation may be considerably in excess of those in the completed cofferdam. For cofferdams in water it is advisable to use a proprietary interlock sealant as described in Chapter 2.

When a cofferdam is to be used solely for the purpose of excluding water, and the depth of soil to be excavated is only nominal, it is often more efficient, to install all the framing under water before commencing dewatering. The Fig. 9.1. shows the optimum spacing of frames for this method of construction. The spacing results in approximately equal loading on the second and successive frames.

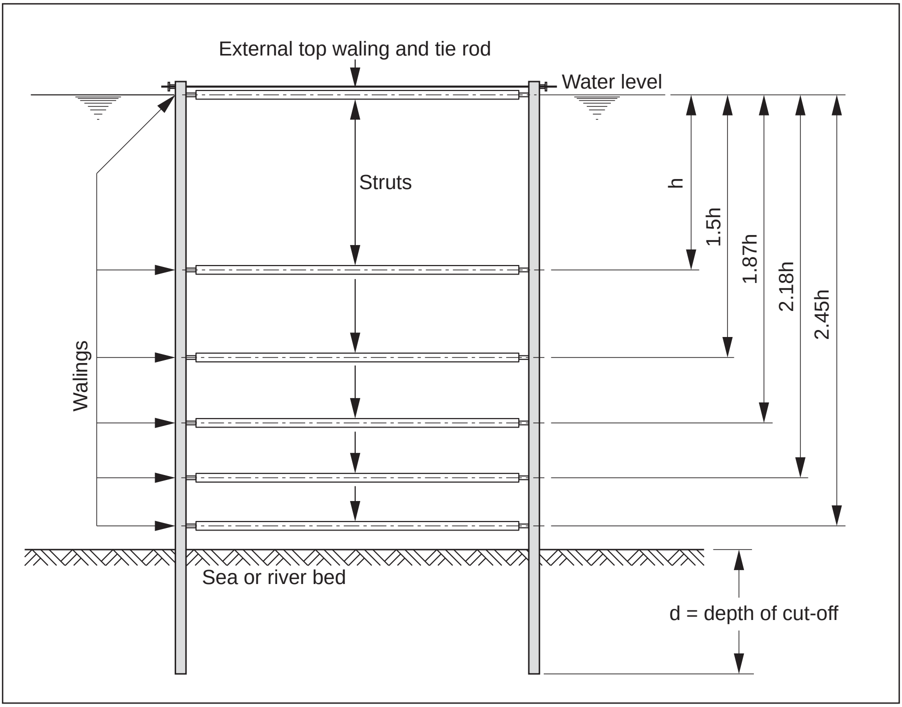

Fig. 9.1. Recommended spacing of frames for cofferdams with framing prior to dewatering.

9.7. Design of temporary framing and struts

Major loads need to be supported in cofferdam design, so temporary cofferdam framing is usually constructed in steel. Fabricated framing and supports can be assembled on site, combining steel components such as structural beams, columns and steel tubes.

A typical internal framing support arrangement for a rectangular cofferdam is illustrated Fig. 9.2. These systems are explained in more detail in other publications such as CIRIA SP 95 [vii].

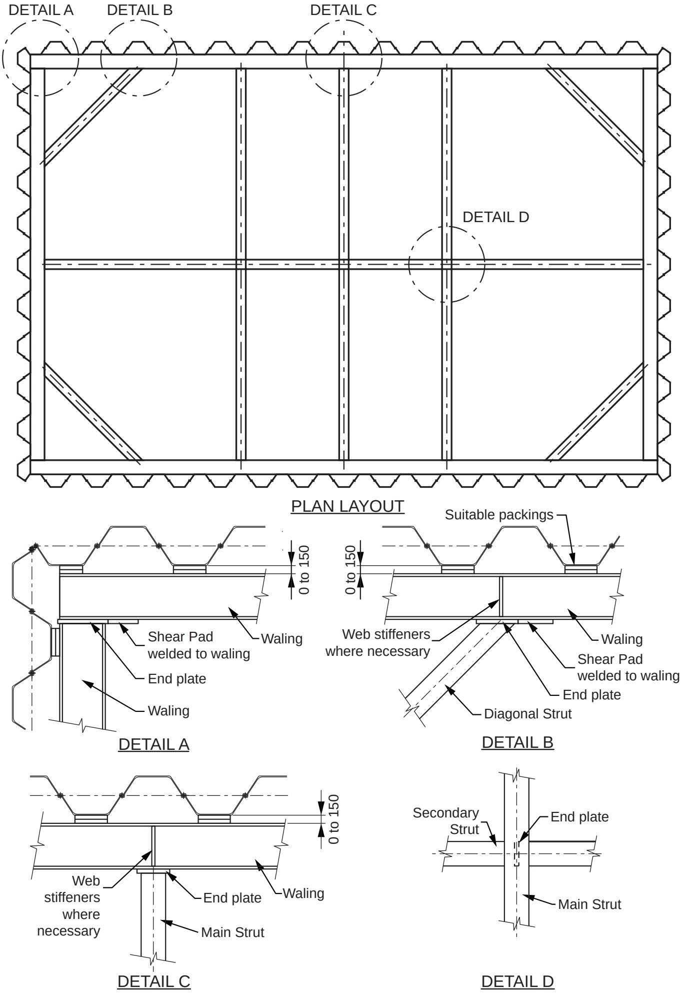

Fig. 9.2. Typical framing arrangement for rectangular cofferdams.

For small sized cofferdams, which will only be open for a relatively short period, it is probably more economic to use a proprietary frame or frames on hire. These frames use hydraulic rams to apply a pre-load and have been developed from the support systems used for trenches for more than thirty years.

However, if large span walings are proposed, deflections should be checked, since these may well be large and will permit significant movement of the wall and the ground behind.

As an alternative, and for larger cofferdams beyond the scope of the proprietary equipment, purpose made frames utilising universal beams and column sections, respectively tubes, will be necessary. These members may require suitable stiffeners to prevent local buckling.

The way the framing is detailed can make a significant difference to the ease with which it is erected and dismantled. Waling beams should be supported at regular intervals either with brackets welded to the piles or with hangers, possibly chains, from the top of the piles. Struts should be fitted with a hanger to support their weight on the waling while being aligned and fixed in position. Prop design must include for accidental impact by materials or machinery especially during excavation or filling operations and the designer should ensure, by discussion with the contractor's site management, that the allowance is adequate for the size of machines being used. Various sources give guidance on this in the range 10-50 kN. Temporary columns or tubular piles should be used to support the props or frames from adverse vertical loads if necessary. Where the walings do not bear directly on the piles, suitable packers will be required which may be of timber, either softwood or hardwood, concrete filled bags, or steel plates depending on the loads to be transferred.

If a strut fails, there is unlikely to be any warning (such as gradual movement) or any time to take remedial measures. Since the consequences of strut failure can be very serious, a conservative approach to the design of struts and their connection may be appropriate [i]. Walings are designed so that catastrophic failure of the cofferdam will not occur if a strut accidentally fails or is removed. Verification of the section capacity is important, but overdesign may be commercially advantageous if steel is re-used or recovered.

Where necessary, the effects of actions (see Chapter 5.8.), together with effects arising from variations in temperature with time, should be taken into account [i].

9.8. Cofferdams with unbalanced loading

This type of cofferdam is usually subjected to greater loading on the landward side, due to soil pressure plus construction loads, hence special precautions may be needed to overcome the resulting unbalanced loading. The method used will, of course, depend upon the specific site conditions, but the following methods are suggested as general practice subject to approval by the relevant supervising authority:

- Method A – the removal of soil from the landward side;
- Method B – the use of "fill" on the water side of the cofferdam;
- Method C – the use of external anchorages to the landward side;
- Method D – the use of raking struts inside the cofferdam.

These methods are illustrated in Fig. 9.3.

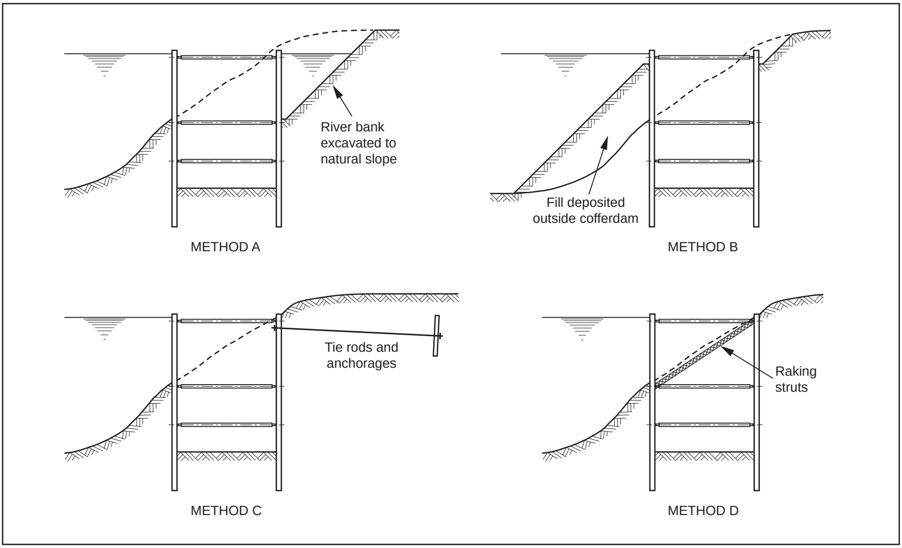

Fig. 9.3. Cofferdams with unbalanced loading.

9.9. Circular cofferdams

Chapter 1.4.5. gives the approximate minimum diameters of cofferdams constructed in AZ, AU, PU and GU sheet piling. ArcelorMittal hot rolled sheet piles are able to be constructed in circular form by rotating at the interlock at no more than 5 degrees per interlock. The tables are intended as a guide only, since the minimum diameter will depend upon several other factors, such as type of ground, length of piles and penetration required.

Smaller diameters can be achieved by introducing individual bent piles.

On site it is usually advantageous to pitch the whole circle before driving, to ensure the circle can be closed. The piles are subsequently being driven in stages as the hammer works its way several times around the circumference. However for larger circles, or when using a leader rig, this may be impractical, but great care will be needed to ensure that the final piles close the ring without departing too far from the required line.

Earth pressures are calculated as for straight-sided cofferdams. Circular ring beams, instead of walings and struts, may support the piles, leaving the central area clear of obstructions. The ring beams will work in hoop compression and are thus normally subjected to axial loads only, which are calculated from:

$$N_{Ed} = W_d \times r$$

N_{Ed} is the design axial load;

W_d is the design waling load;

r is the radius of the cofferdam.

Ring beams may be designed in steel or reinforced concrete and may be deep in section for larger diameters. It is very important, that the self weight of heavy walings are well supported by brackets and tension stays to prevent torsion loading.

The structural design of permanent and temporary structural circular walings is outside the scope of the Piling Handbook and requires specialist expertise.

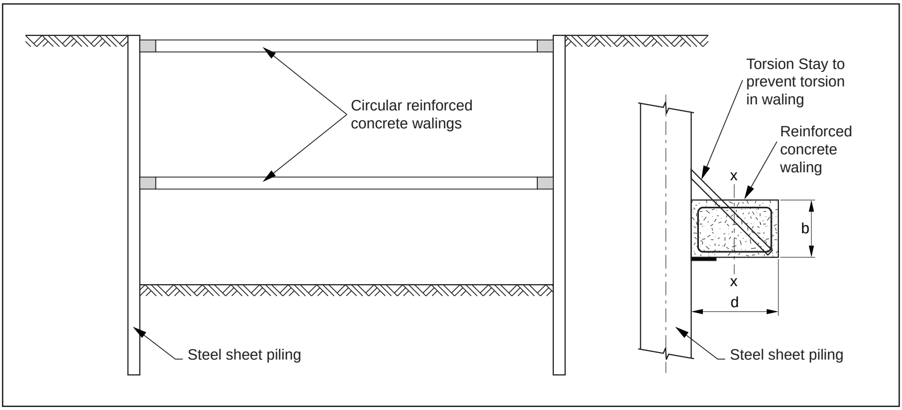

Fig. 9.4. Example for circular waling.

9.10. Double-walled filled cofferdams

Filled cofferdams are self supporting “gravity type” structures (N.B. not strictly a gravity wall under Eurocode definition), either parallel-sided double-wall cofferdams or cellular cofferdams. The stability of both types is dependent on the properties of the fill and the soil at foundation level, as well as on the arrangement and type of the steel sheet piling. The fill must be a suitable granular free draining and practical compactable material. Typical uses are as dams to temporarily seal off dock entrances, so that work below water level can be carried out in the dry and in the construction of permanent walls for land reclamation, quays, wharves and dolphins. It is strongly recommended that the structure is compartmentised into cells, using permanent or temporary sheet pile diaphragm walls for buildability and protection from systematic collapse, should any wall be damaged. Guidance for Cellular Construction is given in Chapter 10.

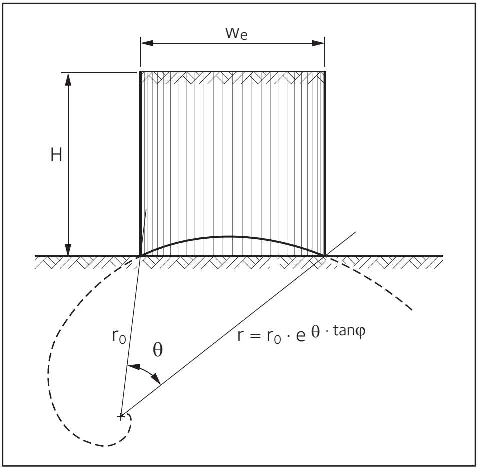

Fig. 9.5. Jelinek logarithmic spiral method.

9.11. Double skin wall cofferdams

Double wall cofferdams comprise two parallel lines of sheet piles connected together by a system of steel walings and tie rods at one or more levels. The space between the walls is generally filled with granular material, such as sand, gravel, or broken rock.

The exposed or inner wall is designed as an anchored retaining wall while the outer line of piles acts as the anchorage. U or Z profile sheet piles as well as the HZ®/AZ® system are appropriate to this form of construction.

The wall as a whole should be analysed as a gravity structure and, in order to achieve adequate factors of safety against overturning and sliding, the width will generally be found to be not less than 0.8 of the retained height of water or soil.

It is recommended that the overall stability of the structure is checked, using the logarithmic spiral method devised by Jelinek.

Transverse bulkheads should be provided to form strong points at the ends and at intermediate positions to assist construction and confine any damage that might occur. The strong points may comprise a square or rectangular cell tied in both directions.

The water regime both inside and outside the structure is critical. It is recommended that weep holes are provided on the inner side of the structure near the bottom of the exposed portion of the piles, to permit free drainage of the fill material reducing the pressures on the inner wall and preventing a decrease in the shear strength of the fill with time. Weep holes are only effective for small structures and complete drainage of the fill may not always be practical. Well points and pumping offer an alternative option and will provide fast drainage if required. However the designer should always make allowance for any water pressure acting on the piles. It is essential that clay or silt is not used as fill material and any material of this type, occurring above the main foundation stratum, within the cofferdam should be removed prior to fill being placed.

The piles must be driven into the soil below excavation or dredge level to a sufficient depth, to generate the required passive resistance. In this condition the structure will deflect towards the excavated side and the lateral earth pressures on the retained side may be taken as active. When cohesionless soils occur at or below excavation level, the penetration of the piling must also be sufficient, to control the effects of seepage. The bearing capacity of the founding stratum should be checked against the weight of the structure and any superimposed loading.

The presence of rock at excavation level makes this type of cofferdam unsuitable unless:

- the rock is of a type that will allow sheet piles to be driven into it to an adequate penetration;
- tie rods can be installed at low level (probably underwater);
- a trench can be preformed in the rock into which the piles can be placed and grouted;
- the pile toes can be pinned with dowels installed in sockets in the rock.

If the piles are driven onto hard rock, or to a nominal depth below dredged level, the resistance to overturning and sliding should be developed by base friction and gravitational forces alone. In this condition the lateral earth pressure on the retained side will be in a condition between at rest and active, depending on the amount of deflection.

The internal soil pressures acting on the outer walls are likely to be greater than active, due to the non uniform distribution of vertical stresses within the cofferdam (due to the moment effects) and hence the design should be based on increased active pressure values, for instance 1.25 times the active values.

9.12. Cellular Circular Cofferdams

The design and construction of cellular cofferdams is discussed in Chapter 10.

9.13. Limit state HYD

Eurocode 7 - Part 1 [viii] defines the limit state HYD as "hydraulic heave, internal erosion or piping in the ground caused by hydraulic gradients".

Verification of limit state HYD involves checking that destabilizing effects of actions do not exceed the corresponding stabilizing effects. This is expressed in Eurocode 7 - Part 1 by the inequalities:

$$u_{d,\,dst} \leq \sigma_{d,\,stb}$$

and

$$S_{d,\,dst} \leq G'_{d,\,stb}$$

with

$u_{d,dst}$ design total water pressure that is destabilizing the soil column;

$\sigma_{d,stb}$ stabilizing design total stress that resists the design pore pressure;

$S_{d,dst}$ design seepage force destabilizing a soil column;

$G'_{d,stb}$ design submerged weight of the soil column.

To verify limit state HYD, it is only necessary to ensure that the inequality above is satisfied. Factoring of the values used to calculate $u_{d,dst}$ and $\sigma_{d,stb}$ (or $S_{d,dst}$ and $G'_{d,stb}$) provides the necessary reliability (safety).

The first inequality adopts a total stress approach to verifying hydraulic failure; the second inequality uses seepage forces instead.

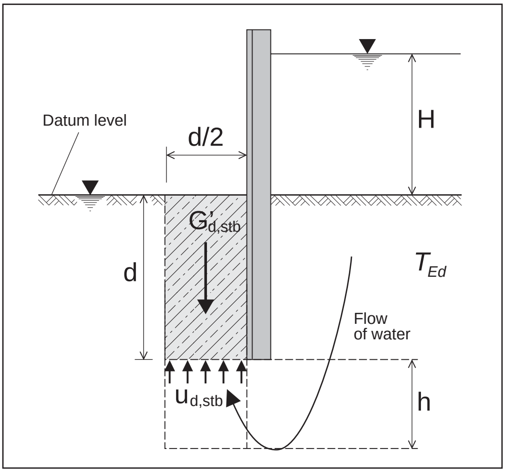

Fig. 9.6. Illustration of hydraulic heave.

Fig. 9.6. indicates the key dimensions of limit state HYD for an embedded retaining wall. A block of soil (shaded) of width *d/2* is susceptible to piping failure, if the hydraulic gradient over the depth of embedment *d* exceeds a critical value i_{crit}. The critical hydraulic may be approximately taken as 1.0 for sand and gravels.

Taking excavation level as the datum, the total head acting over the base of the shaded soil column may be approximated by:

$$h \approx -d + \frac{(H+d)+d}{2} = \frac{H}{2}$$

where *H* is the height of the retained water above formation level. This assumes that the head loss caused by seepage into the excavation is equal on both sides of the wall. Alternatively, the total head (and consequent pore pressure) may be derived from a flow net, as shown in Fig. 9.7., and the hydraulic gradient at any point determined.

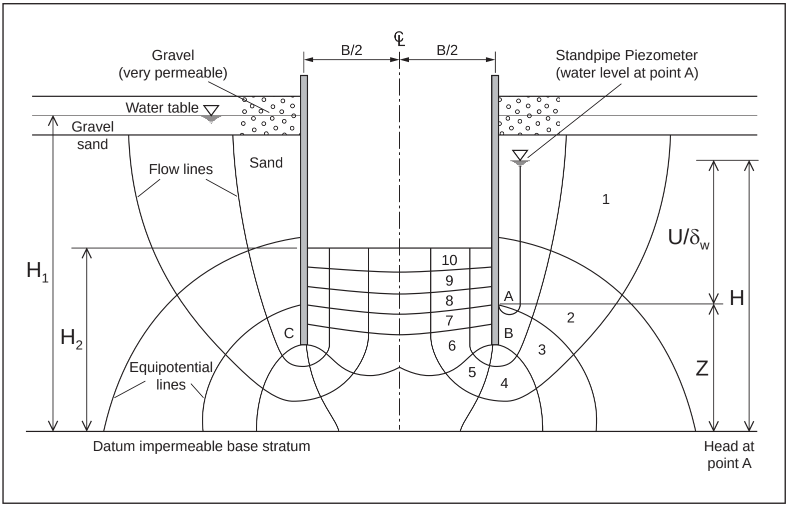

Fig. 9.7. Illustration of the flow net.

For comparison with traditional practice an equivalent global factor of safety may be calculated. This is the ratio of i_{crit} to i_d, where i_d is the design hydraulic gradient. Using the two inequalities given in Eurocode 7 - Part 1 [viii] for limit state HYD values of i_d may be derived. For the analysis based on water pressures and total stress an equivalent global factor of safety of 3.0 is derived and for that based on seepage force and submerged weight 1.5.

The traditional factor of safety on i_{crit} quoted in the literature is between 1.5 and 4.0. Thus the inequality based on seepage forces provides a level of safety at the bottom of the traditional range and should be treated with caution. It is recommended that gross forces and pressures be used, wherever possible, as in the total stress approach.

Piping is a particular form of limit state HYD and occurs when the pressure on the soil grains due to the upward flow of water is so large that the effective stress in the soil approaches zero. This is represented by the total stress inequality above. In this situation the soil has no shear strength and assumes a condition that can be considered as a quicksand, which will not support any vertical load. This is obviously a very dangerous situation for personnel operating in the excavation and will also lead to a significant reduction in passive resistance afforded to the embedded wall by the soil. In extreme cases this can lead to a complete loss of stability and failure of the embedded wall. The likelihood of piping for a given cross section should be assessed as shown above.

Care should be taken when designing circular cofferdams or at the corners of rectangular structures where the three dimensional nature of the situation may lead to higher hydraulic gradients than for a long wall. For square or circular cofferdams, this has the effect of further concentrating the head loss within the soil plug between the sheet pile walls. The following correction factors may be applied to the head loss on the inside face of a cofferdam.

Structure	Use parallel wall values time
Circular cofferdams	1.3
In corners of square cofferdam	1.7

Table 9.1. Correction factors to head loss on inside of cofferdam.

Should limit state HYD not be satisfied, the characteristic hydraulic gradient must be reduced. This may be achieved by:

- installing the piles to greater depth (i.e. lengthening the flow path);
- placing filter material on top of the excavation (i.e. lengthening the flow path and increasing the total stress);
- pumping from well points located inside the cofferdam at or below the pile toe level (i.e. reducing the head loss).

When the stability or ease of operation of a cofferdam involves pumping, it should be remembered that reliability of the pumps is of paramount importance and back up capacity must be available to cope with any emergencies.

9.14. Empirical methods for wall displacement and basal heave

Case histories of walls embedded in stiff soil, with a traditional lumped factor of safety of at least three against basal heave, indicate that wall deflections and associated ground movements are insensitive to wall thickness and stiffness. It follows that flexible sheet pile walls will be more economic in stiff soils than equivalent concrete alternatives, without increasing ground movements.

The maximum lateral (i.e. horizontal) wall movement $\delta_{h,max}$ may be estimated from Fig. 9.8. [ii], which relates $\delta_{h,max}/H$ (where H is the wall's retained height) to system stiffness ρ_s and the overall factor of safety against basal heave F_{bh} .

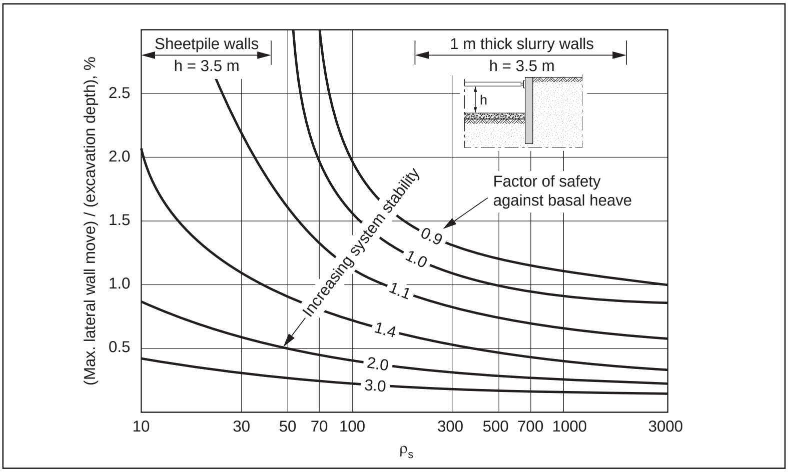

Fig. 9.8. Lateral wall movement curves.

"System stiffness" is defined as [ii]:

$$\rho_s = \frac{EI}{\gamma_w h_{avg}^4}$$

where E and I are the retaining wall's modulus of elasticity and second moment of area, respectively; γ_w is the weight density of water; and h_{avg} is the average vertical prop spacing of a multi-propped system.

Table 9.2. summarizes a method of calculating h_{avg} for cantilever and single-propped walls, which was established for UK soils [iii].

No of props	**Approximate value of h_{avg} in ... soil**		
	Soft	Medium	Stiff
None	2.4 *H*	1.8 *H*	1.4 *H*
Single	1.6 *H*	1.4 *H*	1.2 *H*
Multiple	Use maximum vertical spacing		

Table 9.2. Equivalent "average vertical prop spacing" for cantilever, single-propped, and multi-propped walls.

The factor of safety against basal heave used in the diagram above is that defined by Terzaghi [iv]:

$$F_{bh} = \frac{N_c c_u}{\gamma H - (H/d_b)c_u}$$

where N_c is a stability number (Annex D of [viii]); γ and c_u are the soil's weight density and undrained strength, respectively; H is the wall's retained height; and the depth d_b is given by:

$$d_b = \frac{B}{\sqrt{2}}$$

in the absence of a rigid layer; and by:

$$d_b = D$$

when one is present.

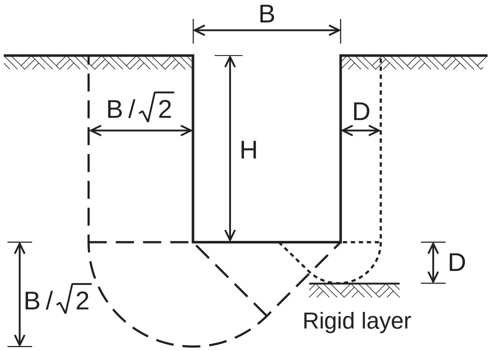

Fig. 9.9. Example geometry.

Example:

A PU 18 sheet pile wall is to retain H = 3.5 m of stiff clay with characteristic weight density γ_k = 20 kN/m^3 and undrained strength c_{uk} = 80 kPa. The wall's Young's modulus E = 210 GPa and its second moment of area I = 38650 cm^4/m. The breadth of the excavation is B = 25 m.

Without propping, the equivalent average prop spacing is calculated as:

$$h_{avg} = 1.4\,H = 4.9 \text{ m}$$

and the wall's system stiffness is:

$$\rho_s = \frac{EI}{\gamma_w h_{avg}^4} = \frac{210 \times 10^6 \times 38650 \times 10^{-8}}{9.81 \times 4.9^4} = 14.4$$

In the absence of a rigid stratum beneath the excavation, the factor of safety against basal heave is:

$$F_{bh} = \frac{N_c c_u}{\gamma H - \left(\sqrt{2}\,H/B\right) c_u} = \frac{(\pi + 2) \times 80}{20 \times 3.5 - \left(\sqrt{2} \times 3.5/25\right) \times 80}$$

$$= 7.6$$

Result following Fig. 9.8.:

$$\frac{\delta_{h.\max}}{H} \approx 0.37\% \Rightarrow \delta_{h.\max} \approx \frac{0.37}{100} \times 3500 = 13 \text{ mm}$$

Ground settlement becomes negligible at horizontal distances more than $2H$ behind embedded retaining walls in sands and soft to medium clays and at distances more than $3H$ behind walls in stiff clay [vi].

Modification factors α_i may be applied to the maximum horizontal wall movement $\delta_{h,max}$ to take account of various influences listed in the table 9.3.:

$$\delta^*_{h,\max} = \alpha_M \alpha_W \alpha_S \alpha_P \alpha_D \alpha_B \delta_{h,\max}$$

where $\delta^*_{h,max}$ is the modified maximum horizontal wall movement.

Influence of ...	Factor	Typical value
Wall stiffness and strut spacing	α_W	0.5 - 1.1
Strut stiffness and spacing	α_S	0.4 - 1.2
Depth to underlying firm layer	α_D	0.7 - 1.0
Excavation width	α_B	1.0 - 2.0
Strut preload	α_p	0.5 - 1.0
Soil's stiffness to strength ratio	α_M	0.4 - 1.7

Table 9.3. Modification factors to maximum horizontal wall movement [vi].

For full details of the modifications, the reader is referred to [vi].

9.15. Limit state EQU

Eurocode 7 - Part 1 [viii] defines the limit state EQU as “loss of equilibrium of the structure or the ground as a rigid body where the strength of the ground or the materials is insignificant”.

Verification of limit state EQU involves checking that destabilizing effects of actions do not exceed the corresponding stabilizing effects, plus any resistance that enhances those stabilizing effects. This is expressed in Eurocode 7 by the inequality:

$$E_{d,dst} \leq E_{d,stb} + R_d$$

with

$E_{d,dst}$ design effect of destabilizing actions;

$E_{d,stb}$ design effect of stabilizing actions;

R_d any design resistance that helps to stabilize the structure (this value should be small).

To verify limit state EQU, it is only necessary to ensure that the inequality above is satisfied. Factoring of the values used to calculate $E_{d,dst}$, $E_{d,stb}$, and R_d provides the necessary reliability (safety).

For embedded retaining walls and bearing piles it is unlikely that limit state EQU will need to be considered as stability is not governed by the structure failing as a rigid body, i.e. toppling is highly unlikely to occur.

9.16. Pump sumps

Although a sheet pile wall can prevent the ingress of water into an excavation, it is not possible to give any guarantee that a cofferdam will be watertight. In order to deal with any water that enters the excavation it is often desirable to install a drainage system that can channel water to a sump from which water can be pumped away.

As the hydraulic gradient adjacent to the corner of a cofferdam is at its largest, it is advisable to place any sumps at excavation level as far as possible from any corner and wall.

It should not be forgotten that pumps are able to remove soil as well as water and a suction hose laid in the bottom of a cofferdam can disturb the base of the excavation with subsequent movement of the wall, if the hose is badly located. Consideration should be given to forming a sump using a perforated drum into which the hose can be fixed to limit damage.

9.17. Sealing sheet pile interlocks

Sealing sheet pile interlocks is dealt with in detail in Chapter 2.

While cofferdams on land will generally have sufficient soil within the interlocks to restrict the flow of water, the use of sealants should not be discounted. In open granular soils particularly, a suitable sealant in the interlock may restrict the volume of water entering the cofferdam such, that the reduction in pumping costs will be significantly greater than the initial cost of the sealant. For cofferdams in water the problem of sealing a cofferdam that is leaking badly is such, that it is advisable to use a sealant from the start as a matter of course, it will be far more difficult to prevent ingress of water through the interlocks after driving, if the piles have not been pre-treated with sealants.

References:

[i] EN 1993, Eurocode 3: Design of steel structures – Part 5: Piling, European Committee for Standardization, Brussels.

[ii] Clough, W. G., Smith, E. M., and Sweeney, B. P. (1989) "Movement control of excavation support systems by interactive design", Proc. Am. Soc. Civ. Engrs Found. Engng, Current Principles and Practices, pp 869-884.

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10 | Circular cell construction design & installation

Chapter 10 - Circular cell construction design & installation

Contents

10.1. Introduction

Cellular cofferdams are self-supporting gravity structures, constructed using straight web sheet piles to form various shapes. The piles are interlocked and driven to form closed cells, which are then filled with cohesionless material. To achieve continuity of the wall, the circular cells are connected together using fabricated junction piles and short arcs.

Provided that the material on which they are to be founded is solid, they require only nominal penetration to be stable. Pile penetration will assist in the resistance of any lateral loads occurring during the construction phase, in the vulnerable period before the fill has been placed and the cell has become inherently stable.

Cellular cofferdam structures are used to retain considerable depths of water or subsequently placed fill. They are commonly used as dock closure cofferdams, or to form quay walls and breakwaters. The straight web pile section and particularly the interlocks have been designed to resist the circumferential tension which is developed in the cells due to the radial pressure of the contained fill. At the same time they permit sufficient angular deflection to enable cells of a practical diameter to be formed. In cellular construction no bending moments are developed in the sheet piles, which enables the steel to be disposed in such a manner, that the maximum tensile resistance is developed across the profile. The sections have therefore very little resistance to bending and are not suitable for normal straight sheet pile wall construction. Walings and tie rods are not required.

Technical information and available products for circular cell constructions are given in Chapter 1.6. and are again summarised in Chapter 10.2. - 10.6.

The design and construction of cellular cofferdams is very complex and further information can be obtained from the brochure "Design & Execution Manual AS500 Straight Web Steel Sheet Piles" [i] and from the technical department of ArcelorMittal Sheet Piling in Luxembourg.

10.2. Straight web piling

Tolerances	AS 500®
Mass	± 5 %
Length	± 200 mm
Height	–
Thickness	t > 8.5 mm: ± 6%
Width single pile	± 2%
Width double pile	± 3%
Straightness	0.2 % of the length
Ends out of square	2 % of pile width

Table 10.1. Tolerances for straight web piles to EN 10248 – Part 2 [ii].

10.2.1. Dimensions and properties for AS 500® straight web piles

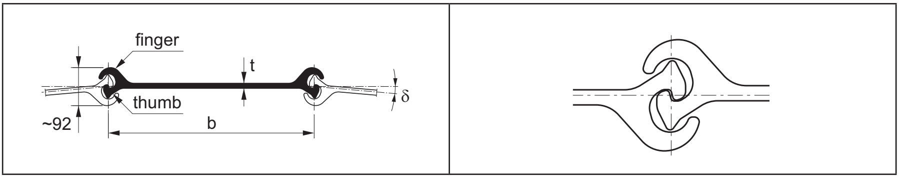


Fig. 10.1. Dimensions and properties for AS 500® straight web piles.

Section	Nominal width[1)]	Web thickness	Deviation angle[2)]	Perimeter	Steel section	Mass	Mass per m² of wall	Moment of inertia	Section modulus	Coating area[3)]
				(single pile)				(single pile)		
	b mm	t mm	δ °	cm	cm²	kg/m	kg/m²	cm⁴	cm³	m²/m
AS 500 - 9.5	500	9.5	4.5	138	81.3	63.8	128	168	46	0.58
AS 500 - 11.0	500	11.0	4.5	139	89.4	70.2	140	186	49	0.58
AS 500 - 12.0	500	12.0	4.5	139	94.6	74.3	149	196	51	0.58
AS 500 - 12.5	500	12.5	4.5	139	97.2	76.3	153	201	51	0.58
AS 500 - 12.7	500	12.7	4.5	139	98.2	77.1	154	204	51	0.58
AS 500 - 13.0[4)]	500	13.0	4.5	140	100.6	79.0	158	213	54	0.58

Table 10.2. AS 500 sheet piles.

Note: All straight web sections interlock with each other.

[1)] The effective width to be taken into account for design purposes (layout) is 503 mm for all AS 500 sheet piles.

[2)] Max. deviation angle 4.0° for pile length > 20 m.

[3)] One side, excluding inside of interlocks.

[4)] Please contact ArcelorMittal Sheet Piling for further information.

10.3. Interlock strength

The interlock complies with EN 10248 [ii]. In Table 10.3., the maximum interlock strength F_{max} for a steel grade S 355 GP is listed. However, higher steel grades are available.

Section	$R_{k,s}$ [kN/m]
AS 500 - 9.5	3000
AS 500 - 11.0	3500
AS 500 - 12.0	5000
AS 500 - 12.5	5500
AS 500 - 12.7	5500
AS 500 - 13.0	6000

Table 10.3. Interlock strength.
For the related steel grade, please contact ArcelorMittal Sheet Piling.

For verification of the strength of piles, both yielding of the web and failure of the interlock should be considered. The tensile resistance $F_{ts,Rd}$ in the pile can be obtained from Eurocode EN1993-5:2007 [iii], Chapter 5.2.5:

$F_{ts,Rd} = \beta_R R_{k,s} / \gamma_{M0} \leq t_w f_y / \gamma_{M0}$

where

f_y is the yield strength;

$R_{k,s}$ is the characteristic interlock resistance given in Table 10.3;

t_w is the web thickness;

β_R is the reduction factor for interlock resistance

$$\beta_R = 0.8 \text{ }^{1)}$$

γ_{M0} is the partial factor given in the Eurocode EN 1993-5: 2007, Chapter 5.1.1 (4)

$$\gamma_{M0} = 1.0 \text{ }^{1)}$$

1) Recommended values from Eurocode EN1993-5:2007. In the National Annexes different values may be provided.

When two different sections are used in the same section of wall, the lowest allowable tensile resistance is to be taken into account.

The resistance to structural failure of the plain sheet pile shall be verified in accordance with Eurocode EN1993-5: 2007, Chapter 5.2.5:

$F_{t,Ed} \leq F_{ts,Rd}$

where

$F_{ts,Rd}$ is the design tensile resistance as shown above;

$F_{t,Ed}$ is the design value of the circumferential tensile force determined with:

$$F_{t,Ed} = p_{m,Ed} \; r_m$$

where:

r_m is the radius of the main cell;

$p_{m,Ed}$ is the design value of maximum internal pressure acting in the main cell due to water pressure and at-rest pressure of the fill and surcharges.

10.4. Junction piles

In general junction piles are assembled by welding in accordance with EN 12063 [iv]. The connecting angle θ can be up to 90° (recommended $\theta = 30°$ to 45°).

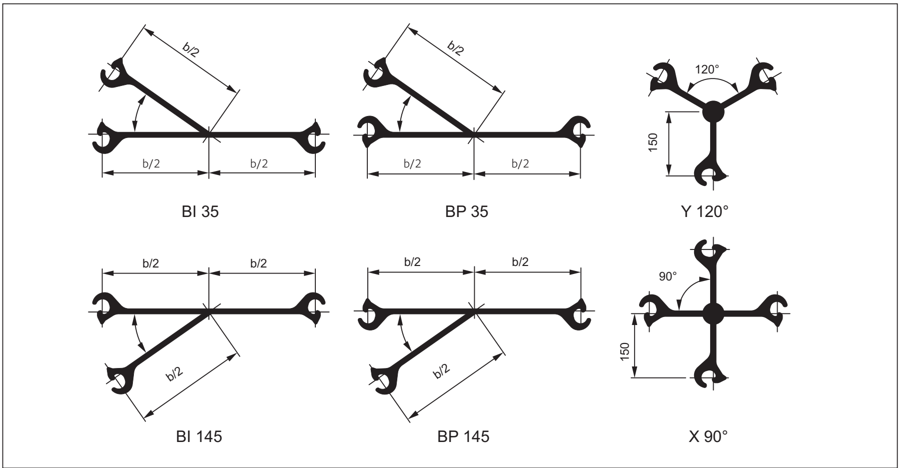


Fig. 10.2. Junction piles.

where:

$F_{ts,Rd}$ is the design tensile resistance as shown at chapter 10.3;

$F_{tm,Ed}$ is the design tensile force in the main cell given by

$$F_{tm,Ed} = p_{m,Ed}\, r_m$$

where:

$p_{m,Ed}$ and r_m as per chapter 10.3.;

b_T is the reduction factor taking into account the behaviour of the welded junction piles as shown in Fig. 10.2 at Ultimate Limit States and which should be taken as follows:

$$\beta_T = 0.9 \times (1.3 - 0.8 \times r_a/r_m) \times (1 - 0.3 \times \tan \varphi_k)$$

where:

r_a is the radius of the connecting arc;

r_m is the radius of the main cell;

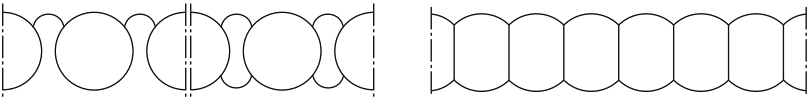
φ_k is the characteristic value of the internal friction angle of the fill material.

10.5. Bent piles

If deviation angles exceeding the values given in table 10.2. have to be attained, piles pre-bent in the mill may be used. Generally, β should be limited to 12°.

Fig. 10.3. AS 500 bent piles.

10.6. Types of cell



Circular cells with 35° junction piles and one or two connecting arcs.

Diaphragm cells with 120° junction piles.

Fig. 10.4. Types of cells.

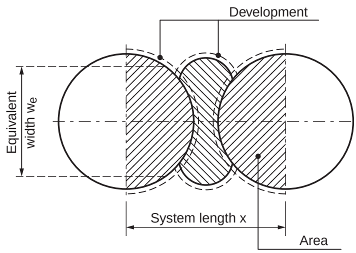
10.7. Equivalent width and ratio

The equivalent width w_e which is required for stability verification, determines the geometry of the chosen cellular construction.

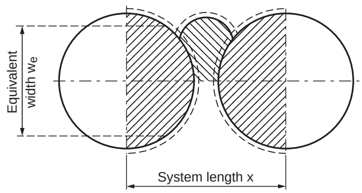
- for circular cells

$$w_e = \frac{\text{Area within 1 cell + Area within 1 (or 2) arc(s)}}{\text{System length } x}$$

circular cell with 1 arc



circular cell with 2 arcs

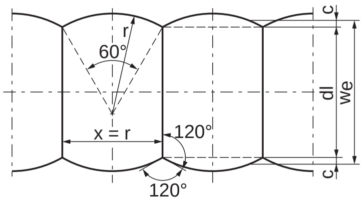


- for diaphragm cells

w_e = diaphragm wall length (dl) + 2 x c

with

$$c = \frac{\text{Area of arc segment}}{\text{System length } x}$$



The ratio R_a indicates how economical the chosen circular cell will be:

$$R_a = \frac{\text{Development 1 cell + Development 1 (or 2) arc(s)}}{\text{System length } x}$$

10.8. Geometry

Once the equivalent width has been determined, the geometry of the cells is to be defined. This can be done with the help of tables or with computer programs. Several solutions are possible for both, circular and diaphragm cells, with a given equivalent width.

10.8.1. Circular cells

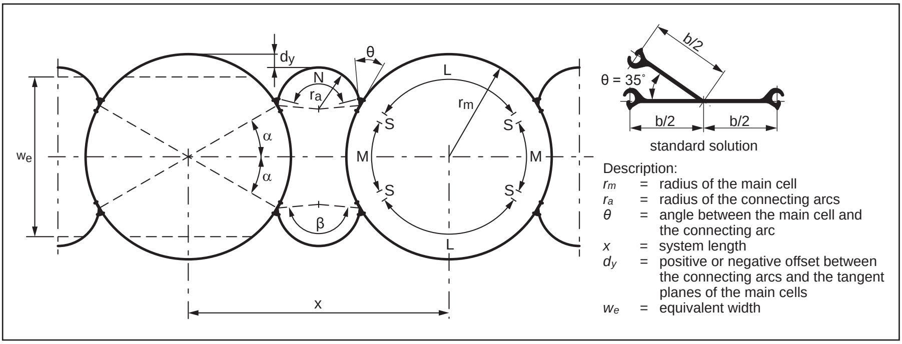


Fig. 10.5. Geometry of circular cells.

Junction piles with angles θ between 30° and 45°, as well as $\theta = 90°$, are possible on request. Table 10.4. shows a short selection of solutions for circular cells with 2 arcs and standard junction piles with $\theta = 35°$.

Nb. of piles per						**Geometrical values**						**Interlock deviation**		**Design values**	
Cell				**Arc**	**System**							**Cell**	**Arc**	**2 Arcs**	
Total pcs.	L pcs.	M pcs.	S pcs.	N pcs.	pcs.	$d = 2 \times r_m$ m	r_a m	x m	d_y m	α °	β °	δ_m °	δ_α °	w_e m	R_a
100	33	15	1	25	150	16.01	4.47	22.92	0.16	28.80	167.60	3.60	6.45	13.69	3.34
104	35	15	1	27	158	16.65	4.88	24.42	0.20	27.69	165.38	3.46	5.91	14.14	3.30
108	37	15	1	27	162	17.29	4.94	25.23	0.54	26.67	163.33	3.33	5.83	14.41	3.27
112	37	17	1	27	166	17.93	4.81	25.25	0.33	28.93	167.86	3.21	6.00	15.25	3.35
116	37	19	1	27	170	18.57	4.69	25.27	0.13	31.03	172.07	3.10	6.15	16.08	3.42
120	39	19	1	29	178	19.21	5.08	26.77	0.16	30.00	170.00	3.00	5.67	16.54	3.38
124	41	19	1	29	182	19.85	5.14	27.59	0.50	29.03	168.06	2.90	5.60	16.82	3.35
128	43	19	1	31	190	20.49	5.55	29.09	0.53	28.13	166.25	2.81	5.20	17.27	3.32
132	43	21	1	31	194	21.13	5.42	29.11	0.33	30.00	170.00	2.73	5.31	18.10	3.39
136	45	21	1	33	202	21.77	5.82	30.61	0.36	29.12	168.24	2.65	4.95	18.56	3.35
140	45	23	1	33	206	22.42	5.71	30.62	0.17	30.86	171.71	2.57	5.05	19.39	3.42
144	47	23	1	33	210	23.06	5.76	31.45	0.50	30.00	170.00	2.50	5.00	19.67	3.39
148	47	25	1	35	218	23.70	5.99	32.13	0.00	31.62	173.24	2.43	4.81	20.67	3.44
152	49	25	1	35	222	24.31	6.05	32.97	0.34	30.79	171.58	2.37	4.77	20.95	3.42

Table 10.4. Standard solutions for circular cells with 2 arcs.

10.8.2. Diaphragm cells

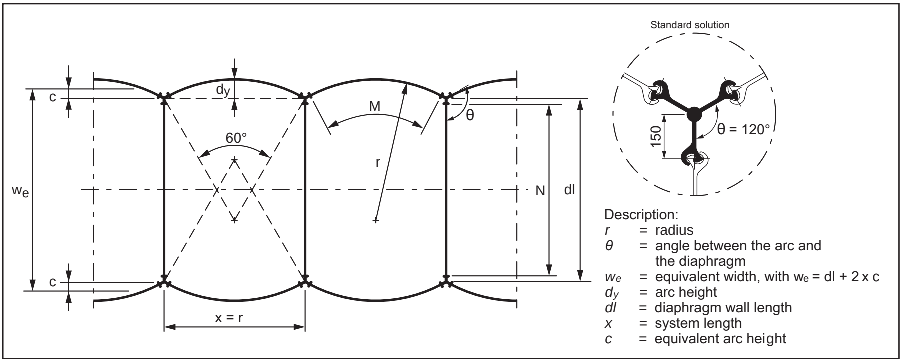


Fig. 10.6. Geometry for diaphragm cells.

The two parts of the Table 10.5. should be used separately, depending on the required number of piles for the diaphragm wall and the arcs.

Geometry diaphragm wall		Geometry arc (Standard solution)				
Number of piles	Wall length	Number of piles	Radius System length	Arc height	Equivalent arc height	Interlock deviation
N	dl	M	$x = r$	d_y	c	δ_a
pcs.	m	pcs.	m	m	m	°
11	5.83	11	5.57	0.75	0.51	5.17
13	6.84	13	6.53	0.87	0.59	4.41
15	7.85	15	7.49	1.00	0.68	3.85
17	8.85	17	8.45	1.13	0.77	3.41
19	9.86	19	9.41	1.26	0.86	3.06
21	10.86	21	10.37	1.39	0.94	2.78
23	11.87	23	11.33	1.52	1.03	2.54
25	12.88	25	12.29	1.65	1.12	2.34
27	13.88	27	13.26	1.78	1.20	2.17
29	14.89	29	14.22	1.90	1.29	2.03
31	15.89	31	15.18	2.03	1.38	1.90
33	16.90	33	16.14	2.16	1.46	1.79
35	17.91	35	17.10	2.29	1.55	1.69
37	18.91	37	18.06	2.42	1.64	1.60
39	19.92	39	19.02	2.55	1.73	1.52
41	20.92	41	19.98	2.68	1.81	1.44
43	21.93	43	20.94	2.81	1.90	1.38
45	22.94					
47	23.94					
49	24.95					
51	25.95					
53	26.96					
55	27.97					
57	28.97					
59	29.98					

Table 10.5. Standard solutions for circular cells.

10.9. Handling straight web piles

Unlike piles designed to resist bending moments, straight web sheet piles have low flexural stiffness, which means that care must be taken over their handling.

Incorrect storage could cause permanent deformation, making interlock threading difficult, if not impossible. It is therefore vital to have a sufficient number of wooden packing pieces between each bundle of stacked sheet piles, and to position these pieces above each other to limit the risk of deformation.

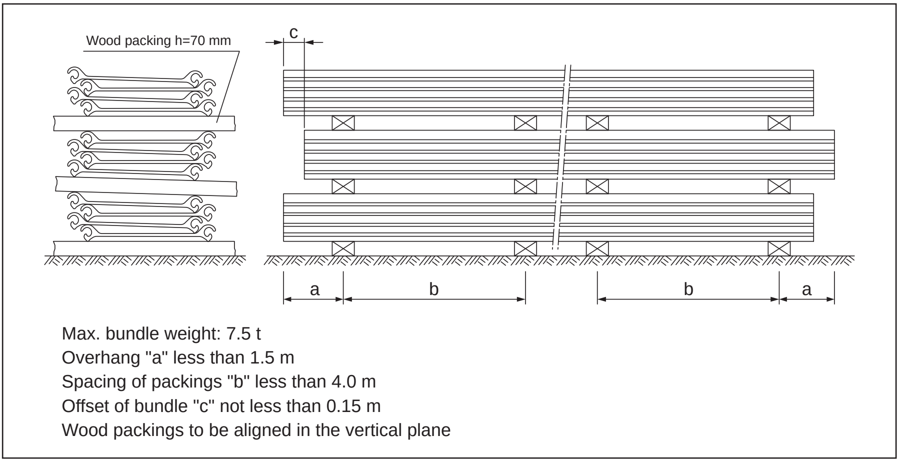


Fig. 10.7. Storage of straight web sheet pile.

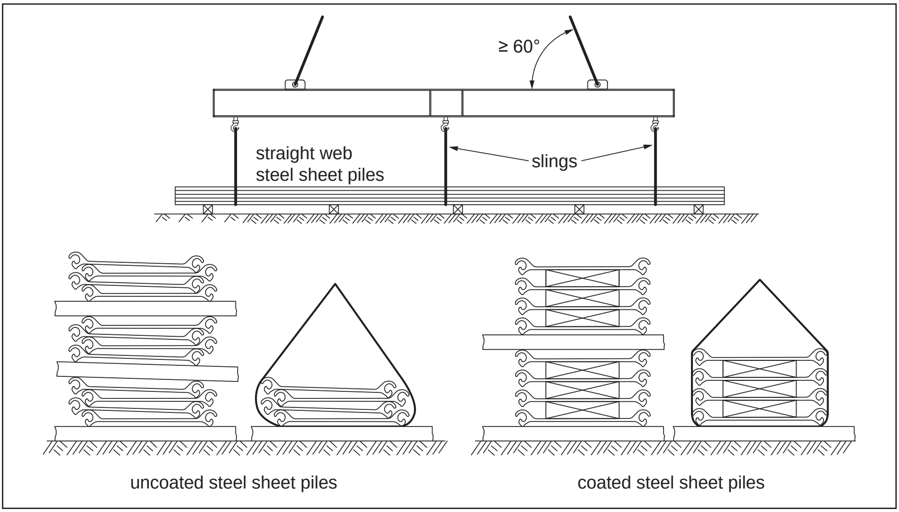


Fig. 10.8. Storage and handling of straight web sheet piles.

When sheet piles have to be moved from the horizontal storage position to another storage location, lifting beams or brackets, made from pile sections threaded into the interlocks prior to lifting, should be used.

When pitching piles up to 15 m long into the vertical position, only one point of support near the top (the handling hole) is necessary.

Straight-web sheet piles more than 15 m long should be lifted at two or even three points, in order to avoid plastic distortion as illustrated in Fig. 10.9.

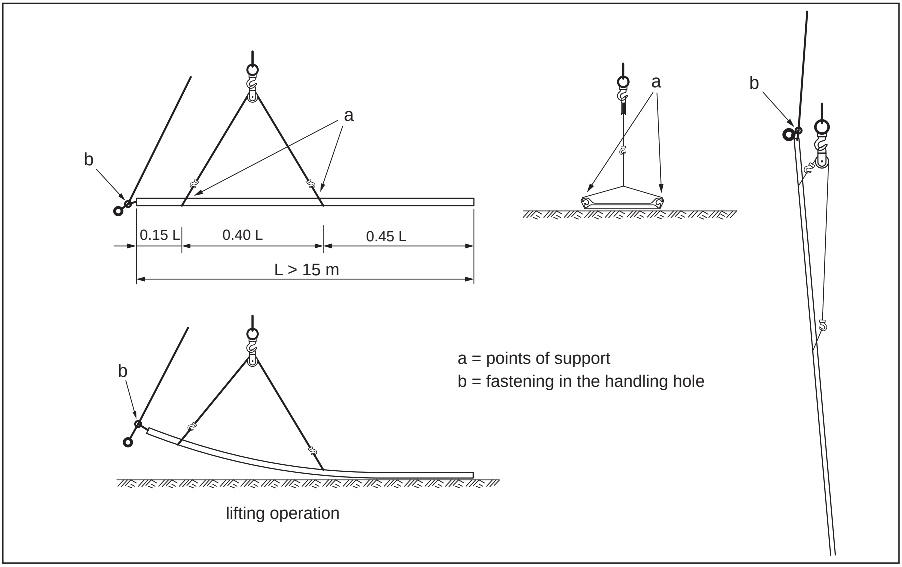


Fig. 10.9. Lifting of long straight web sheet piles.

References:

[i] ArcelorMittal Sheet Piling: Design & Execution manual AS 500® Straight Web Steel Sheet Piles. 2009.

[ii] EN 10248-2, Hot rolled steel sheet piling - Part 2: Tolerances on shape and dimensions. 2006.

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11 | Installation

Chapter 11 - Installation

Contents

11.1. Introduction

This chapter provides an introduction to the modern methods of installing sheet piles, taking into account the equipment available for safe working practice with due consideration of site environmental requirements.

Knowledge of the characteristics of the steel and the section are not enough to guarantee good results prior to installation and this chapter briefly describes the practical information to be considered to ensure proper product installation. It also indicates how pile drivability can be predicted following a thorough evaluation of the ground conditions.

This chapter also contains information on pile driving equipment which is current at the time of writing and includes impact hammers, vibratory pile drivers, hydraulic press-in piling and special systems. Descriptions of driving methods, ancillary equipment and guideline procedures to assist in the adoption of good practice when installing sheet piles are also included.

Finally some guidance is given on optimization of driving techniques to minimise noise and vibration during installation and extraction.

Note: the terms “pre-augering” and “augering” used in this chapter refer to a procedure consisting in loosening the soil with an auger without removing the soil from the hole (as far as practically possible). It is sometimes also called “pre-drilling” or “drilling”.

11.2. Driving methods

11.2.1. General

Whilst it is recognised that, in common with most civil engineering projects, a measure of flexibility is desirable to meet site conditions, every precaution must be taken to maintain the necessary standards of safety whilst giving the required alignment and verticality of the installed piles.

Therefore principal consideration must be given to access of plant and labour as well as working positions for handling the piles and threading the sheets together. The length of the piles and height, from which they can be pitched and driven safely and accurately, is also important.

Whenever possible, sheet piles should be driven in pairs. The first sheet piles in a wall must be installed with great care and attention to ensure verticality in both planes of the wall. Control of the sheet pile installation must be maintained during both, the pitching and driving phases of the installation process. Twisting of sheet piles at the interlocks should be minimized and prevented to be less than 5 degrees rotation.

The principal pile driving methods available to installers are pitch and drive and panel driving, or a method which entails a combination of both by final driving the individual piles in panel form. The features, advantages and disadvantages of each method are described in this chapter.

11.2.2. Pitch and drive method

This method requires equipment to control the verticality of the pile during installation so that piles can be pitched and driven one by one. The pitching operation can be carried out close to ground level, meaning that operatives are potentially at less risk. Downtime in windy conditions can be reduced. Piles can be installed to final level by this method. For example, it is necessary when using the Japanese press-in piling machines, or left at a higher level before final driving using panel driving techniques, as a second stage operation. Usually a different hammer is then to be used. This is called back-driving and is carried out in a particular sequence. Generally heavier more powerful hammers are required in a second stage driving operation for driving into more difficult strata. The piles should always be paired up if possible for the second stage back-driving.

The pitch and drive method for completing pile driving is the simplest way of driving piles but is only really suited to loose soils and short piles. For dense sands and stiff cohesive soils or in the case of possible obstructions, pitch and drive is not recommended. However, in suitable conditions, productivity is maximised.

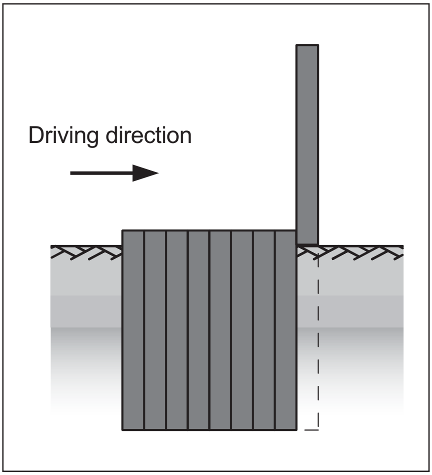


Fig. 11.1. Pitch and drive illustration.

It is more difficult to control forward lean using the pitch and drive method because the leading lock has less resistance than the trailing or connected lock as a result of soil and interlock friction. Although the piling may commence from a true vertical position, the top of the piles will have a natural tendency to lean in the direction of driving. Very careful site supervision will be needed, otherwise this situation will get progressively worse if not countered. When anticipating driving long straight sections of wall with a planned pitch & drive method, it may be advisable for Engineers to incorporate design features giving changes of direction to the pile line at approximately 50 m intervals. This is important to consider when using some silent press-in piling machines, for example in basement projects. It is to be noted, that it may not be possible to revert back to a panel driving system to avoid or correct the forward lean problem without the use of specially fabricated tapered piles.

With pitch and drive, the free leading interlock is constantly in danger of rotation in plan which increases, the deeper the free end penetrates the ground, as it is unsupported during the driving operation. When a pile rotates during installation, friction develops in the connected locks, making driving progressively more difficult.

When using leader rigs, a ground guide waling is recommended to prevent the piles being excessively rotated at the interlock.

Fig. 11.2. Pitch and drive operation using guide walings at ground level to prevent excessive rotation of piles.

11.2.3. Panel driving

Piles may be threaded together above the ground in a support frame to form a panel prior to driving. In this situation, both interlocks are engaged before any driving takes place. This balancing of the friction forces ensures maximum control and accuracy. The piles are then driven in stages and in sequence into the ground. Sequential driving enables verticality to be maintained.

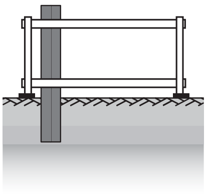
Sheet piles should be installed or back driven using the panel-driving technique to ensure that good verticality and alignment is achieved and to minimise the risk of driving difficulties or declutching problems.

This technique is important for maintaining accuracy when driving long piles or driving into difficult ground.

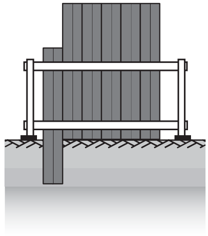
When a whole panel of piles has been pitched, there, is no need to drive all piles completely to maintain progress of the piling operations. During driving, the toe of alternate pairs of piles can be kept close to the same level. This is ensuring that the maximum stiffness of the piles is maintained and allows for the pile toe to be driven through soil of greater resistance without undue deviation.

The interlocks of adjacent piles are effectively guiding the pile being driven through more difficult ground. The lead on the driven pile should be kept to a minimum (refer to Table 11.10.). If obstructions are encountered, individual piles can be left high without fear of disruption to the overall efficiency of the installation process. Engineering decisions can then be taken to attempt to remove the obstruction or drive piles carefully at either side of the obstruction before trying once more to drive or punch through it if further penetration is necessary.

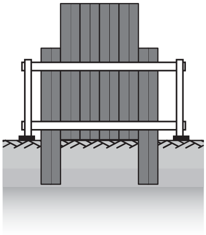
Panel driving is the best method for driving sheet piles in difficult ground or for penetrating rock - which is unlikely to be possible with the pitch and drive method, except press-in piling equipment with a crus auger attachment. Piles are usually paired up or neighbouring sheets are leveled up at the head before commencing the hard driving operation with a heavier hammer. Care should be taken when piles are firstly pitched and installed in singles and driven in the first stage with a vibro-hammer. It is easier to execute two stage driving in pairs if the piles are pre-ordered and installed in crimped pairs. Difficulty of pairing up in the panel is avoided in this way, piles are straighter in plan, and safer more efficient operation of impact hammers can be ensured.



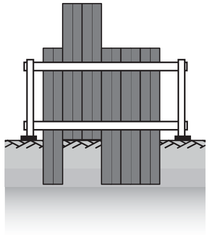
1. Pitch, align and plumb 1st pair.



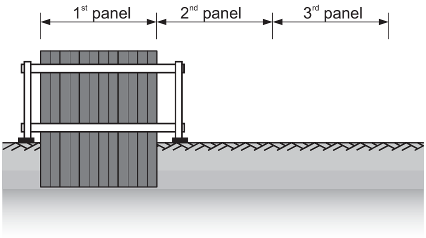
2. Drive 1st pair carefully & accurately pitch rest of panel.



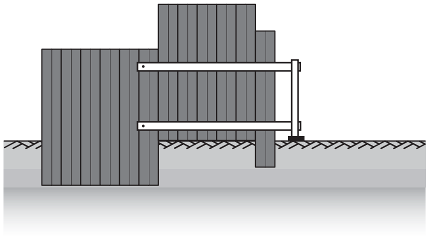
3. Ensure last pair are accurately positioned & plumbed, drive last pair.



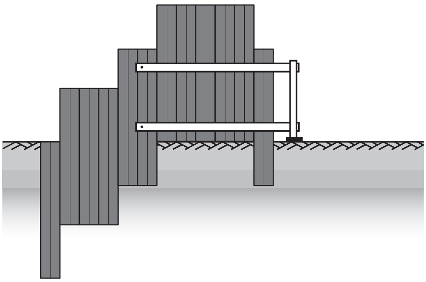
4. Drive reminder of panel – working back towards 1st pair.



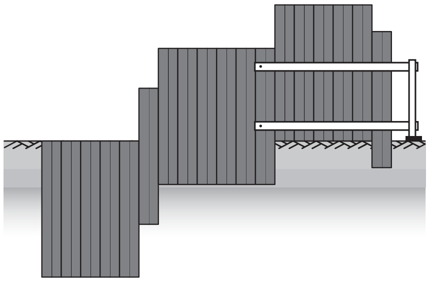
5. 1st panel part driven.



6. 2nd panel pitched. Last pair of 1st panel become 1st pair of 2nd panel. Wales supported by through bolting to last driven pair.



7. 1st panel driven to final level in stages. Last pair of 2nd panel plumbed & driven accurately The lower frame is usually left in position after removal of the upper frame until driving is sufficiently progressed for it to be removed.



8. 1st panel completed. 2nd panel part driven. 3rd panel pitched. Last pair of 2nd panel becomes 1st pair of 3rd panel.

Fig. 11.3. Panel driving.

11.2.4. Staggered driving

It is essential that the heads of adjacent piles or pairs are kept close together to maximise the pile performance when driving in hard conditions. This means that the installer should keep moving the hammer from one pile pair to another in sequence to advance the toe of the piling with less risk of damage or refusal. This technique is known as staggered driving. It is not recommended that piles are advanced more than 2 metres beyond neighbouring piles unless driving conditions are relatively easy for the pile section and equipment used (refer to Table 11.10.).

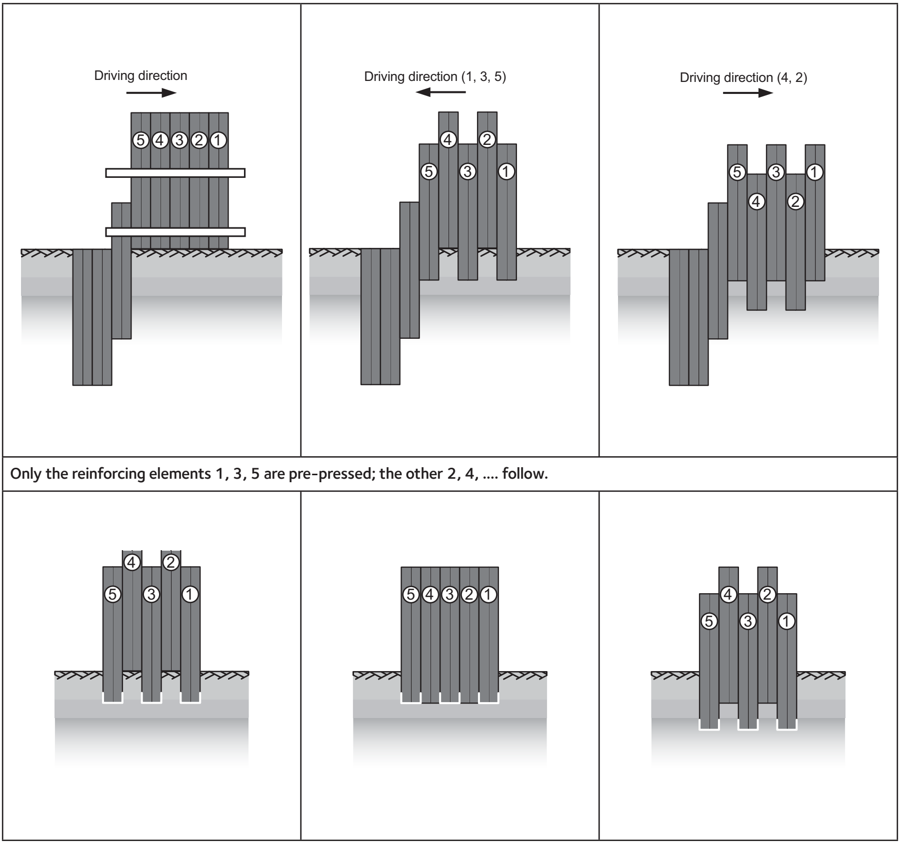


Fig. 11.4. Staggered driving.

11.2.5. Cofferdam and closure installation techniques

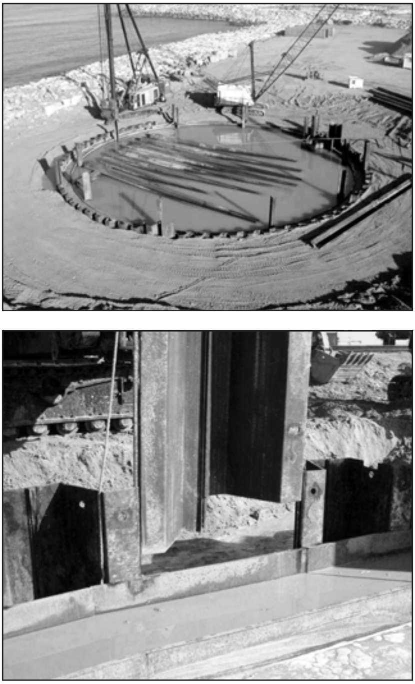


Fig. 11.5. Circular cofferdam and closure.

When installing cofferdams or combined walls, accuracy is essential - particularly where it is necessary to pitch a pile of significant length into both adjacent pile interlocks to close a gap. If the gap tapers, it will be very difficult to interlock and drive the closure pile successfully. Therefore the panel driving method is the favoured method for installing structures of this type. It is recommended to pitch all the piles in a cofferdam structure and close all the free interlocks fully before driving. In general Z-piles are to be preferred over U-piles in this case, because of higher flexibility due to the existing middle interlock. U-piles have the interlock in the wall axis, thus having less rotation capacity. The sheet piles need to be fully supported by temporary framing. Stability needs to be assured by using temporary piles if necessary. With any sheet pile project, the risk of declutching should be minimised especially when it is required to work in dewatered cofferdams. When joining walls or closing to fixed positions, panel installation methods are obligatory to maintain accuracy. It is necessary to avoid the risks and potential disaster caused by declutched or damaged piles when planning, designing and executing the works. The panel driving technique is also best for the control of wall length and creep by using appropriate guide walings to facilitate setting out and adjustments. This may be important when dimensions are critical. Curved walls can also be set out using this method with curved walings to suit.

11.3. Driving systems

The choice of a suitable driving system is of fundamental importance to ensure successful pile installation with due regard to the safety of operatives and environmental disturbance.

The basic driving methods are:

Impact driving

This is the best method for driving piles into difficult ground or final driving of piles to level in panel form. With a correctly selected and sized hammer it is in most conditions the most effective way of completing deep penetration into hard soils. The downside is that it can be quite noisy and not suitable for sensitive or restricted sites as well as driving time is usually longer than with the use of a vibratory hammer.

Vibro-driving

This is usually the fastest and most economical method of pile installation but usually needs loose or cohesionless soil conditions for best results. Vibration and noise occurs, but this can be kept to a minimum provided the right equipment is used and when conditions are suitable as well as the site does not expressly require a vibrationless method of installation.

Press-in piling

Otherwise known as silent vibrationless hydraulic jacking. Machines of various types are now widely used. This method is very effective in fine cohesive soils but less so in dense coarse soils unless pre-drilling or jetting techniques are used. This is the most effective method to use when installing sheet piles in sensitive locations where piling would have not been considered in the past. Length limitations are given by soil conditions.

11.4. Types of hammers

11.4.1. Impact hammers

There are several types of impact hammers available to suit the particular requirements of the job site. Most impact hammers will involve a piston or ram and an anvil block with a driving cap which spreads the blow to the pile head. The machines are usually supported by a heavy frame or chassis and normally need leg guides set up to fit snugly to the pile section being driven to maintain a vertical position during operation. Alternatively, the hammers can be set up to be supported and aligned by a leader rig. It is very important that, because of the height and slenderness of these types of hammer, the hammer is prevented from rocking or swaying when delivering powerful blows to the piles.

The main differences between hammers are size and mechanism for delivering the blow from the ram. Some hammers deliver the blow freely under gravity; others are able to accelerate the fall of the ram and are described as double acting. In all cases the effectiveness of driving will depend on the power and efficiency of the blow. Modern hammers are in widespread supply and, provided they are adequately maintained, can be expected to totally outperform the older types of pile hammers. Therefore the impact hammer types described in this chapter are those that are most commonly in use. Descriptions and detail of small older types such as air hammers can be found in previously published installation guides. Hydraulic hammers usually outperform diesel hammers in terms of efficiency, are more environmentally acceptable and are less likely to damage the head of the pile when transmitting the driving force.

11.4.1.1. Single acting hammers

Fig. 11.6. Diesel hammer on fixed lead.

These hammers act on the principle of free fall for the ram weight to deliver high blow energy to the top of the steel pile. This type of hammer consists of a segmental ram guided by two external supports; the ram is lifted by hydraulic pressure to a preset height and allowed to free-fall onto the anvil or driving cap. The modern hammers are usually either hydraulically powered by a powerpack, which supplies hydraulic oil flow to control the hammer, or by fuel pre-filled in a tank in the chassis in the case of diesel hammers. Both types are suited for crane suspended operation. Hammers can be set upon the piles that need to be driven to the final stage. These hammers are usually too wide to fit on a single pile and are particularly suitable for driving in pairs. Alternatively the hammers may be mounted on a fixed leader type piling rig.

Hydraulic hammers can be adapted with heavy block ram weights and they are particularly suitable for prolonged driving into thick clay strata. The weight and the height of drop of the ram can be varied to suit the pile section and the site conditions. Ram weights are usually set up in 3, 5, 7 or 9 tonnes modes for standard sheet piles up to 6000 cm^3/m section modulus, or to drive primary elements of combined walls. The drop height is variable up to approximately 1.2 metres, or even up to 1.5 metres with the larger hammers. Hydraulic hammers can easily adjust the impact energy with the help of an electronic steered control box: the drop height can be changed from approximately 10 cm up to 120 cm, depending on the size of the hammer.

Diesel hammers may be designed to deliver the blow from the ram weight or piston from varying height and have a rope controlled throttle to deliver fuel for different settings to vary the impact energy. The efficiency of the blow is improved by mounting on a leader rig as it is more difficult to maintain stability of the hammer driving a sheet pile when rope suspended on full power.

Fig. 11.7. Diesel hammer mounted on leader rig driving sheets.

Both types of hammers are usually noisy and with diesel hammers precautions are sometimes necessary to avoid fuel or oil spills when working over water. For driving in stiff clays, it is always preferable to use a heavy ram, with short stroke to minimise pile head damage and noise emission levels. Besides, the bow rate is also higher with a short stroke than with the full stroke. The hammer controls are precise, and used correctly hydraulic hammers can achieve 75-90% of rated output energy.

11.4.1.2. Hydraulic double-acting hammers

Fig. 11.8. BSP CX85 driving AZ piles.

These hammers can be used on single or pairs of piles. They are particularly suited to drive U-piles or Z-piles with reinforced shoulders in hard driving situations and with rapid blow action can be used effectively to penetrate very dense sands, gravels as well as rock.

This type of hammer consists of an enclosed ram which is lifted by hydraulic pressure. On the downward stroke, additional energy is delivered to the ram, producing acceleration above that from gravity alone and powerful blows to strike the anvil or driving cap which is purpose built to fit the pile section.

When set up for use with standard sheet piles, these hammers may deliver a maximum energy/blow of 10 kNm to 100 kNm with a blow rate from approximately 150 to 40 blows per minute. The electronic control system ensures optimum control of the piling process. The ram weight of the machines suitable for standard sheet pile sections range from 1.0 t to 9.0 t. Bigger machines are available for driving large non-standard pile sections, such as box piles, tubular piles and HZ-M piles for combined wall systems and offshore projects. The total weight of the hammer ranges from approximately 2.5 t to 20 t (and possibly up to 50 t for offshore tubular piles – note the driving cap and bell insert may be very heavy for large diameter tubular piles).

The machines are usually suspended from a crane and, because even the lighter machines are very powerful, effective driving systems are available at significant reach using large crawler cranes.

Under normal site conditions it is usual to select a ram weight that is in the range 0.75 to 2 times the weight of the pile plus the driving cap.

11.4.1.3. Transmitting the blow to the pile

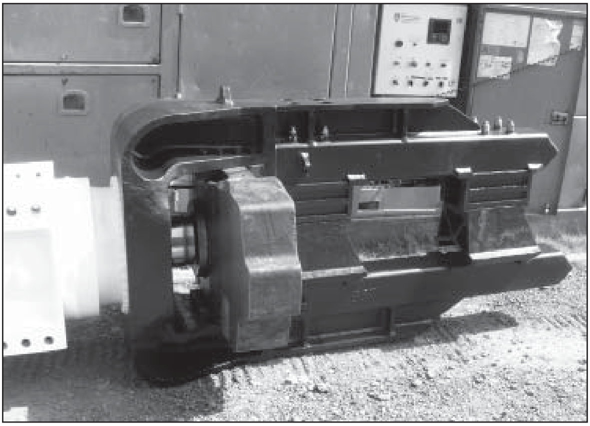


Fig. 11.9. Hydraulic hammer and anvil plate.

Any pile section can be set up to be driven with a suitable impact hammer. However it is not only important to size the hammer correctly but it is imperative that the driving cap and / or anvil plate fits well and is correctly sized to suit the pile section being driven - especially on wide piles or pairs of piles. The hammers should not be used to drive piles of different widths without changing the fittings. The central axis of the ram should always align with the centre of the driven pile section in plan and the blow spread evenly over the full cross sectional area of the pile.

11.4.1.4. Control and settings

These hammers can usually be operated on different settings to suit the pile and ground conditions. For instance a heavy ram weight ratio hammer on wide piles can be used with a low setting to suit driving in clay and double acting hammers on a rapid blow setting can be used to drive single piles in dense sandy soils. Equipment to provide digital readout of energy and blow count, for driving records and control, is available to be fitted to most machines.

11.4.1.5. Impact hammers and driving stresses

The driving stresses in the pile, when using impact hammers, are likely to be greatest at the head of the pile. This is known as the peak head stress value. It is important to assess the peak driving stress when checking the section capacity of the pile for driveability (refer to Chapters 11.7.5. and 11.7.6.).

Where the impact hammer has a low efficiency (for instance, diesel hammers may rate at 30% - 40% efficiency), the yield stress of the steel section may be exceeded by the peak driving stress causing buckling at the pile head.

Also note that for highly efficient hydraulic hammers, which usually operate at 85% - 95% efficiency, the hammer energy may be transmitted effectively to the toe of the pile. It is therefore important that the pile continues to penetrate the ground when driving for a sustained period because toe damage can occur when the penetration rate is low or refusal sets are exceeded.

11.4.1.6. Refusal criteria - hard driving

It is crucial to set refusal criteria for hard driving with impact hammers. A penetration of 20 mm per 10 blows should be considered as the limit for the use of all impact hammers in accordance with the hammer manufacturer's recommendations. Under certain circumstances a penetration of 1 mm per blow could be allowed for a few minutes. Longer periods of time at this blow rate will cause damage to the hammer as well as ancillary equipment and may also result in damage to the pile head.

11.4.2. Vibratory hammers

11.4.2.1. Types of vibratory hammer

Fig. 11.10. Rope suspended HFVM vibratory hammer.

Vibratory hammers are available in a wide range of sizes and also operate in different frequency modes. The standard machines usually operate at a frequency from 0 to 1800 rpm (rotations per minute), whereas the high frequency or variable moment ones can go up to 2500 rpm. The power available is described by the centrifugal force of the hammer which ranges from 400 to 2000 kN for typical leader rig mounted units, and up to 10000 kN for crane suspended models.

Higher frequency drivers are also available extending the range up to 3000 rpm. The high vibrations developed attenuate very rapidly limiting any problems to adjacent properties. The variable (resonance free) high frequency vibratory hammers allow the excentrics to be adjusted at start up and shut down to eliminate resonance and the generation of unwanted vibrations through the upper strata on sensitive sites and close to buildings. Together with pre-drilling or waterjetting techniques it is possible to vibro-drive piles successfully in locations which are considered to be environmentally sensitive.

11.4.2.2. Ground conditions and use of vibratory hammers

The soils best suited to vibration work are non-cohesive soils, gravel or sand, especially when they are water-saturated and provided the soil is not too dense. If SPT's over 50 prevail then driving will be difficult. Waterjetting or pre-drilling can be used to loosen dense cohesionless soil. Vibratory hammers operating at higher amplitudes are normally more effective in difficult soils. With mixed or cohesive soils, vibro-drivers can also be very effective where there is a high water content and the ground is loose or soft. Clay soils have a damping effect and reduce the energy available for driving the pile. Vibratory driving is difficult where firm or stiff clay soils are encountered but once again an high amplitude is likely to yield the best results.

11.4.2.3. Gripping the pile

Fig. 11.11. Vibro-hammer with double clamps.

All pile sections can be driven with vibratory hammers but attention should be given to the area where the machine jaws grip the top of the pile. For example, the thick part of the pan on U type piles is most suited for this when driving or extracting piles singly. If the jaws need to be attached to the web of a pile section – for instance on Z-piles - care should be taken to avoid ripping the steel especially during extraction. Tearing can be a particular problem with wide piles if the vibro-driver is equipped with small size grips and attaches to the pile near the handling hole level. Multiple clamps are available and it is recommended that they are used on paired sections especially when driving wide piles. A correctly fitting clamp should have jaws in good condition – particularly when it is being used for extraction - and recesses to accommodate the pile interlock if used in the centre of paired units.

11.4.2.4. Sizing vibro-drivers

There is less risk of damage to the pile section where conditions allow the use of vibratory hammers. However, when driving becomes more difficult, the selection process for these hammers is different. When using a vibratory hammer it is imperative that the pile continues to penetrate the soil at an appropriate speed. If pitch and drive techniques are used, the recommendations in Fig. 11.1. and 11.2. should be followed. If control of alignment and good rates of penetration cannot be achieved, panel driving techniques or using other types of hammers can be considered. Generally vibratory hammers with greater power and self-weight as well as higher amplitude will perform better in harder and deeper strata. If it is necessary to complete driving with a vibratory hammer, Fig. 11.12. may assist to identify the size of machine required. Leader rigs may add an additional pull down force of aproximately up to 300 kN but this may not be sufficient to penetrate thick clay or dense soil strata. For harder driving conditions associated with the installation of long piles with up to 20 m penetration and vibratory driving, the apparent resistance significantly increases and hammers with much greater power are necessary unless impact driving techniques are used. It is important to ensure that the vibratory hammer is capable of supplying the necessary centrifugal force to the head of the piles to drive them, and that the power pack or carrier machine is capable of supplying sufficient power for the vibratory hammer to operate at its maximum output. Apart from the machine consideration, the pile itself must be able to transfer the energy from the head to the toe of the pile. Soil, pile and vibratory always have to be seen as an interacting system.

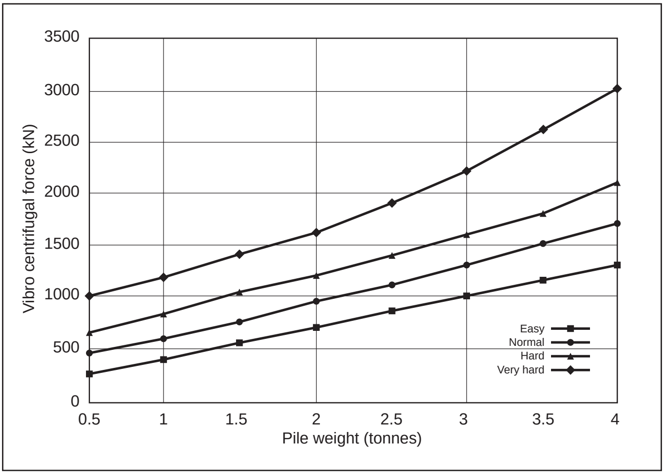


Fig. 11.12. Guidance for size of vibratory hammer (rated by centrifugal force) in terms of pile weight and driving conditions.

The necessary centrifugal force can be calculated with the following empiric formula (see EAU 2012, chapter 8.1.23.4.) [v]:

$$F = 15 \times \left(t + \frac{2 \text{ x } m_R}{100}\right) \times E$$

With	F	centrifugal force	[kN];
	t	driving depth	[m];
	m_R	pile weight	[kg];
	E	adaption factor for driving conditions (value 1.0 - 1.2).	

The amplitude is also an important factor when sizing vibro-drivers:

$$\text{amplitude} = \frac{2000 \text{ x eccentric moment (kgm)}}{\text{dynamic weight (kg)}}$$

where the dynamic weight includes the clamp and sheet pile.

	Easy driving	Normal driving	Hard driving
Typical amplitude	< 4 mm	6 mm	8 mm

Table 11.1. Minimum typical working amplitude requirement for vibro-driving.

11.4.2.5. Refusal criteria, limitations for vibro-driving

Formulae to determine the size of vibratory driver needed for a given set of conditions vary and readers should obtain guidance from their machine manufacturing company on this topic if in doubt. Vibrators are also used for installing vertical or battered bearing piles and high modulus or HZ-M king piles.

Note that performance on sheet piles and isolated piles is different and care should be taken not to undersize the hammer or overstress the pile.

It is essential that movement is maintained when driving or extracting piles with vibratory hammers and, it is generally recognised that a penetration rate of approximately 25-50 cm per minute shall be used as a limit. This not only acts as a control on possible vibration nuisance but also as a precaution against the detrimental effects of overdriving.

When refusal occurs and the pile installation rate is below the refusal rate quoted above, the energy being input by the vibro-driver will be converted into heat through friction in the interlocks of the pile being driven. The steel can sometimes melt and damage the interlocks themselves or the sealants being used as well as also the hammer may be damaged if prolonged driving in refusal conditions takes place. Clearly when refusal is reached with vibratory systems an alternative installation technique or more powerful hammer must be employed to gain the final designed penetration depth.

Piles shall not be left short of the designed penetration depth without specific permission from the engineer / designer.

11.4.2.6. Setting up the hammer and driving methods

Fig. 11.13. Telescopic rig with vibratory hammer.

The driving method to be adopted needs to be taken into account when choosing the type and model of hammer. Crane suspended machines are best if heavy extraction or pitching in panels are used. Small free riding vibratory hammers are sometimes used as starter hammers for long piles or when the hammer needs to operate at distance from the crane.

Vibratory hammers can also be mounted on tall masted leader rigs. Double clamps can be used to centralise the driving action on long paired piles and the equipment is specially suited for use with pitch and drive methods. For wide AZ or AU double piles the use of double champs is highly recommended to avoid energy loss and minimize noise during driving.

Telescopic leader rigs generally use high frequency, resonance free or variable moment vibratory hammers and can apply a crowding force from the telescopic pistons which adjust the height of the mast to deliver additional driving or withdrawal force. These machines can therefore press and vibrate the piles simultaneously. The length of the mast, size of the rig and hammer will determine the capability of the installation when using pitch and drive methods.

The operator of the rig may be trained to recognize ground conditions during the installation to enable control of amplitude and frequency settings to optimize the driving through variable strata.

11.4.2.7. Excavator mounted vibro-drivers

Fig. 11.14. Excavator mounted with vibratory hammer.

Small excavator mounted, high frequency hammers can be used for installing short piles. Care should be taken when handling piles because excavators are not built for this process and it is not as safe as using purpose built lifting equipment when threading the piles together. However, it should only be used when installing short and light piles (usually below 7 m in length) in loose soils when accuracy is not of paramount importance.

Side grip type vibro-drivers controlled from excavators require extra care to thread piles safely and prevent damage to the pile/pile interlock during installation.

If the piles are too long to pitch safely and also preventing accurate driving to level then this method should not be attempted.

For both methods (top driven and side grip) sturdy guide walings at ground level are critical to control alignment of the piles to prevent twisting or rotation of the pile during the driving operation.

When guide walings are not used untidy pile lines usually result and premature refusal can occur when piles rotate and deviate off the theoretical line.

11.4.2.8. Use as an extractor

The vibratory pile driver is also a very efficient pile extractor. The pull force applied to the sheet pile will depend on the size of the vibrator and the crane pulling force that can be applied to the pile from a safe stable position. This force will be a function of the capacity of the crane or rig and the distance it is located from the pile line. When sheet piles have a tendency to deviate off line, it is necessary to withdraw the pile and re-drive in panel form to ensure verticality.

11.4.3. Vibration free sheet pile press-in piling

Today, there are different machine setups available, but the principle of operation remains the same. They represent a means by which sheet piles can be installed with less noise and no vibration, often called press-in piling or silent vibration-free hydraulic jacking. All press-in piling systems operate by using reaction from installed piles to develop the necessary pressing force to drive adjacent sheet piles in the line to progress.

Press-in piling is an excellent method to avoid noise and vibration problems when driving sheet piles on sensitive sites.

As the sheet piles can be installed permanently close to buildings and boundaries, more space is available for basements and property development. This yields a major commercial benefit, which can be of more value than the cost of the wall itself.

Generally speaking, pressing takes more time to install the piles. For standard double sheet pile sections the technical limit may be just above 22 m length, depending on soil conditions and hydraulic force of the press-in piling machine. Waterjetting and pre-augering can be used to facilitate the works.

11.4.3.1. Press-in equipment, self supporting type

Press-in equipment are now well established and are widely available for the installation of hot rolled steel sheet piles, including wide AZ® sections in singles or pairs. Many different models have been developed providing improved operational features which are suited to particular installation situations.
The “Japanese” press-in piling machines have been developed to enable the plant to “walk” over the tops of the driven piles. The system, including the press-in pile driver, a clamp crane and a pile transporter. It is able to work independently and remotely from access roads as well as also over water.

These machines, which are especially suited for use in cohesive and granular soils, are hydraulically operated and derive most of their reaction force from the friction between the soil and previously driven piles.

The most readily available machines are used to drive single and double Z-piles and single U-piles. Note that press-in piling machines have been built that can drive pairs of piles but it is important to note that machines may be set up to drive different pile sections and combinations of similar configuration. Presses for telescopic and fixed lead machines with 2, 3 or 4 pressing cylinders are available from different manufacturers.

Fig. 11.15. Walking press.

11.4.3.1.1. Procedure and control

The most widely used machine is the Japanese silent press-in piling which jacks one pile after another to full depth, using a pitch and drive procedure, while walking on the previously set piles. These machines work independently from a crane which is used only to handle the piles.

The sheet piles are fed by a crane into the enclosed chuck or pressing jaws of the machine which acts as a guide to align the piles without the need for guide wailings. Setting out control is executed by using a laser light beam focused on the leading interlock of the pile being driven. The operator adjusts the verticality and position of the leading lock by remote control and a push pull action on the pile during driving. The press-in piling machine is able to move itself forwards (“walk”) using remote control. The machine raises its body and travels forward to the next position without crane support. Curves and corner can be built by rotating the machine head in the desired direction.

Fig. 11.16. Leader guided pressing system.

11.4.3.1.2. Starting off

A reaction stand weighted down with kentledge or delivered sheet piles is used to commence the pile line using a few temporary piles to precede the first working pile to be driven. A crane is used to initially lift the machine onto the reaction stand but there is usually no need to lift it off again until completion of the pile line. Ancillary equipment is also available that has been designed to "walk" along the top of the installed piles, including a special crane, to enable the whole piling operation to be carried out on the top of the sheet piles without any other means of access.

11.4.3.1.3. Operational issues and suitability

The press-in piling can also be used for withdrawing temporary sheet piles using this silent, vibrationless method. The machines work best in clayey, slightly cohesive or fine grained soils and can be equipped with jetting devices for low or high pressure water jetting. This is necessary to loosen fines in cohesion-less strata or to lubricate dry cohesive soils to make driving easier. For difficult dense or high strength cohesive soils, cohesive or with the presence of potential obstacles, pre-drilling can be required to loosen or mix the soil. Superficial obstructions are dealt with by digging a lead trench and either backfilling with suitable material or using the trench to control surplus water and arisings when jetting.

Previously driven piles cannot be progressed with walking type press-in piling machines without using pile extensions. Where a pitch and drive technique is necessary, the pile section choice is influenced by the stiffness and length of the pile being driven in addition to the ground conditions. Guidance is given in table 11.5. Great care shall be taken when applying water jetting techniques near the toe of the piling or close to existing foundations. Jetting should be terminated before the end of the drive for cut-off walls.

11.4.3.2. Crush Pilers

Fig. 11.17. Crush piler.

Where vibration free techniques are essential and for difficult soil conditions where water jetting techniques would also be precluded, pile driving is now made possible by use of the Super Crush Piling System. A development of the Japanese silent press-in piling machine, this system uses an integral rock auger inside a casing to penetrate hard ground. The press-in action is carried out while simultaneously extracting the auger. As for all situations where use of an auger is involved, care has to be taken not to remove the soil.

This technique enables silent piling into rock and allows sheet piles to be designed to take significant vertical loads in end bearing. Piles may also be extended by butt welding on site to build deep sheet pile walls that otherwise would not be considered feasible using traditional installation method (48 m long piles have been installed in Tokyo using this type of machine).

Consideration should be given to the integrity of the seating of the pile and the effectiveness of the water cut-off provided when using this technique. Injection grouting or re-seating of the pile, using vibratory or impact driving, may be necessary to repair holes or voids in the soil strata caused by the augering process.

The advantage of using these machines in city centres for deep basement construction can be very important for sustainable solutions. Machines can be set up for double AZ crimped piles, tubes and continuous HZ®-M beams as well as standard single piles.

11.4.3.3. Pressing equipment for leader guided machines

Leader guided presses with up to 100 t pressing force have the advantage that sheet pile panels can be pressed with a vibration free system, allowing quicker installation sequences. The machine can be adapted easily to other piling techniques.

Lighter sections may be used compared to the pitch and drive method used by the walking presses, because the piles may be finally installed by a panel driving technique.

Fixed or telescopic leader guided presses have a minimum of 2 rams, sometimes 3 and usually 4 rams, alternately operating using reaction by gripping onto a driven pile while the other rams press. The rams (hydraulic cylinders) are connected to the piles in such a manner that both tensile and compressive forces can be applied. Pressurising the rams in sequence while the others are locked enables the piles to be pushed into the ground, one or two at a time, to the full extent of the rams. The cycle is then repeated to completion. It should be noted that panel type pressing machines are also suitable for dense coarse soil strata with pre-drilling and they are compatible with water jetting equipment. This is an important machine to consider for installation near party walls and buried sensitive structures or services.

The machines can be set up to drive standard sheets AZ, AU, PU and GU piles in pairs, triples or a set of four, but they must be supplied in uncrimped form. Note the AZ section is preferred because the force is applied down the axis of the pile, which coincides with the centre of wall and machine. Threading of piles on-site can be done in case of need.

The smaller type of multi-ram pressing machine can be mounted on a large tracked excavator for installation of short sheet piles.

To reduce interlock with Beltan, grase or loam. A simple bolt to close the end of the pile interlocks also proved to be effective, see Chapter 2.

11.4.3.3.1. Types of pressing machines – 200 t Power push

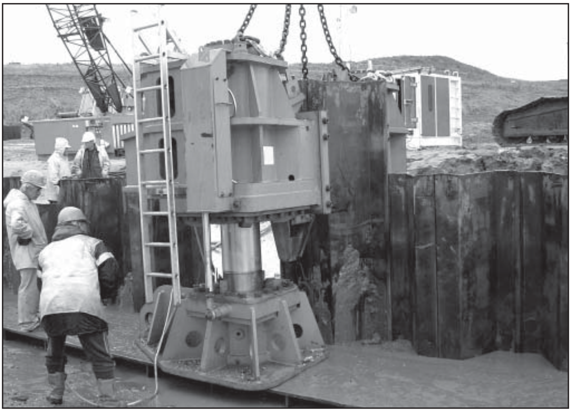


Fig. 11.18. Pile extractor.

The machine is normally used for installation using panel driving techniques. Rams or cylinders can be arranged in multiples of paired units to deliver push-pull forces to the piles for either driving or withdrawal. The machine can be mounted on a leader rig or suspended by a crane.

Each double acting cylinder can generate 200 tonnes pressing force. Reaction is derived from the weight of the press, from the piling rig and by gripping adjacent piles to mobilise static skin friction. The cylinder and hydraulic jaws can be reconfigured to suit different pile types and layouts including box piles formed from sheet piles (see Fig. 11.19). Up to 4 cylinders can be used on a leader rig and 8 cylinders in line can be used when crane mounted.

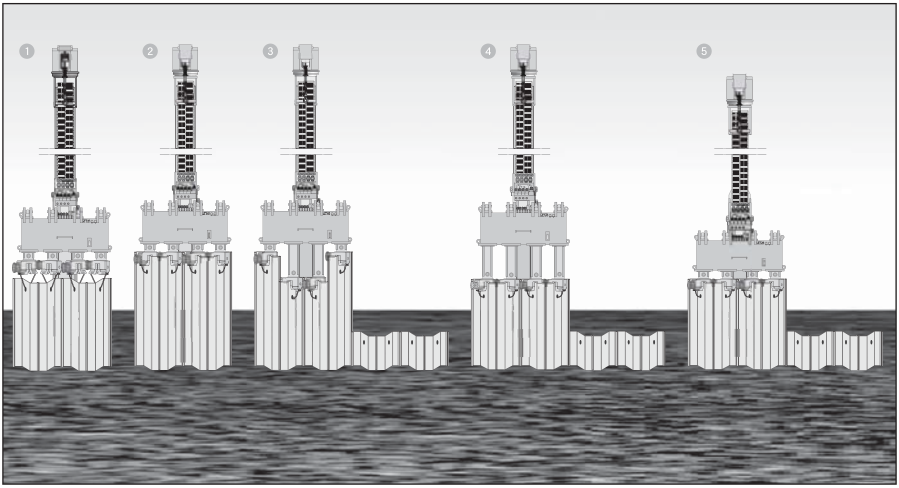


Fig. 11.19. Quad pile pressing diagram.

The pressing of combined walls is technically difficult and not recommended.

11.4.3.3.2. Types of Multi-ram pressers – 80 t press

The multi-ram press 80 t is smaller than the DCP Power push and can deliver up to 80 tonnes pressing force on four rams which clamp directly to the piles. The connection is on the web for AZ piles and on the pan for PU/AU/GU piles.

This machine is usually mounted on a leader rig but for short piles can be mounted on a heavy excavator.

11.5. Influence of soil conditions on installation

11.5.1. Site conditions

For the successful driving of sheet piles, it is essential that a good knowledge of the site conditions is available to enable an accurate assessment to be made of environmental and geological conditions.

The local environment of the site will influence working restrictions such as noise and vibration. Each site will be subject to its own unique set of restrictions which varies according to the proximity and nature of neighbouring buildings, road category, underground services, power supplies, material storage areas etc.

Geological conditions refer to the vertical characteristics of the soil strata. In order to achieve the required penetration of the sheet piles, site investigation of the soils together with field and laboratory tests provided in a Ground Investigation Report (GIR) is necessary for installation assessment.

This provides necessary information which affects pile section choice and installation method such as:

a) historical data;

b) depth and stratification of the subsoil;

c) soil type particle size, soil properties, shape distribution & uniformity;

d) level of the groundwater table;

e) permeability and moisture content of the soil;

f) geotechnical test results.

11.5.2. Identification of soil characteristics

Identification and classification of soil and rock is detailed in section 4.5. Geotechnical parameters are described in section 4.7.

11.5.3. Driving system characteristics of various soils

Different types of soil demand different installation techniques. The most efficient driving method for a job site can be determined by in-situ testing. Local experience is also very important. Brief notes on each system are given in this chapter.

11.5.3.1. Vibratory driving

Non cohesive soil, such as round-grain sand and gravel as well as soft soils are especially suited to vibratory driving. Easy driving should be expected when soils are described as loose. Dense angular grain material or cohesive soils with firm consistency are much less suited. Difficult driving may be experienced when dominant SPT values are greater than 50 or significant thicknesses of cohesive strata are encountered.

It is also found that dry soils give greater penetration resistance than those which are moist, submerged or fully saturated.

If the granular subsoil is compacted by prolonged vibrations, then penetration resistance will increase, quickly leading to refusal. For difficult non-cohesive soil water jetting can be considered. Pre-drilling is an alternative.

11.5.3.2. Impact driving

The use of impact hammers is principally possible in all kind of soils. The machines are most effective in cohesive and hard soils. If the vibro hammer has reached its refusal criteria, finishing the pile with impact driving is common practice. Keep in mind that the other way round is not possible.

Penetration into weak rock will be possible with a powerful hammer and adequate sheet pile sections and high steel grades. Toe strengthening or special HZ-M toe cutting can be considered. Otherwise, rock-bolting may be an alternative solution to provide additional resistance at the toe.

11.5.3.3. Press-in piling

This method is especially suited to soils comprising soft cohesive and fine material. Easy driving is usually experienced in soft clays and loose non-cohesive soils. This technique can employ jetting assistance to loosen silt and sand particles to be able to advance the piles by press-in piling. Successful installation will also depend on the soil providing sufficient resistance to the reaction piles.

Difficult soil conditions are found when dense sands and gravels or soil containing cobbles or any large particles are encountered. In the presence of boulders or rock reaction failure or refusal may occur. Lead trenches can be of assistance for the removal of obstructions encountered near the surface.

Pre-drilling may facilitate the press-in piling technique in difficult soil conditions; otherwise piles will have to be driven to final level by percussive means or a press-in piling machine with a crush auger attachment can complete the piles to final level (see Chapter 11.4.3.2.).

Wet soil conditions are also favourable for pressing. In dry, stiff clay strata, it is normal practice to use low pressure jetting to lubricate the soil to pile interface and make driving easier.

11.6. Choice of sheet pile section for driving

11.6.1. Influence of pile section properties

Effective construction with sheet piles will depend on the selection of an adequate pile section for the chosen method of installation, taking into account environmental restrictions and ground conditions over the full driven length of the pile. The pile section selected must fit the structural requirements, but shall also be suitable for driving through the various strata to the required penetration depth. The stresses developed in the pile may be many times higher than required for the structural requirements.

The driveability of a pile section is a function of its cross-section properties, stiffness, length, steel grade, quality, preparation and the method of installation. The piles also need to be driven within the predefined tolerances to retain their driveability characteristics. Piles allowed to deviate off line or twist will cause following piles not to drive well and are likely to refuse prematurely.

The driving force required to achieve the necessary penetration is affected by the soil properties and the resistance to driving that develops on the pile profile. This pile related resistance will develop through three main factors:

- skin friction along the pile surface that is in contact with the soil;
- toe resistance or plugging-effect;
- interlock friction.

11.6.2. Influence of driving resistance

Whatever force is required to drive the pile, it is necessary to overcome the total resistance and move the pile without damaging it. While the pile is moving, the hammer energy is consumed to overcome the driving resistance. When the pile ceases to move, the energy from the piling hammer will have to be absorbed by the pile section and the soil. This situation will increase the stress level in the steel as well as the probability of pile damage. Deformation of the pile usually occurs at the head or toe of the pile, sometimes on both locations. The use of bigger equipment or driving assistance shall be considered in this case.

11.6.3. Influence of steel grade and shape

The stress that the piles can withstand increases with the yield strength. The higher yield strength steel piles are more resistant to head or toe deformation than the same section in a lower steel grade. High steel grades are recommended when hard driving conditions prevail, or multiple re-use of the piles is considered.

In a similar manner, it can be seen that the larger the area of steel in a profile, the higher the load it can carry. Hence, the heavier pile sections will have increased driveability when compared to light, thin sections. However, it must not be forgotten that under certain driving conditions, a large cross section area may result in an end bearing resistance that exceeds the increase in driveability. Careful consideration of the soil layers and appropriate parameters will enable to assess the expected driving resistance and to select a suitable sheet pile section.

11.6.4. Influence of method of installation

It is very important to consider the installation technique to be used. Pitch and drive (P&D) methods will reduce the driveability of the section as discussed in section 11.7.8. When silent press-in piling machines are used, the stiffness of the pile is of paramount importance to maximise driveability as the machine operates on pure P&D methods.

By experience, rules of thumb have been developed to assess the driveability of particular profiles. One such relationship uses the section modulus of the pile profile as key factor. However, it is not possible to derive the most suitable choice of pile section by consideration of section modulus alone. In fact for pitch and drive the length of the pile and moment of inertia of the section are key parameters to assess driveability.

Because of limited rotation capacity at the interlocks, double piles drive better for longer lengths than single pile sections. The section required to be commercially most effective depends on consideration of a number of factors and the following selection procedure provides guidance:

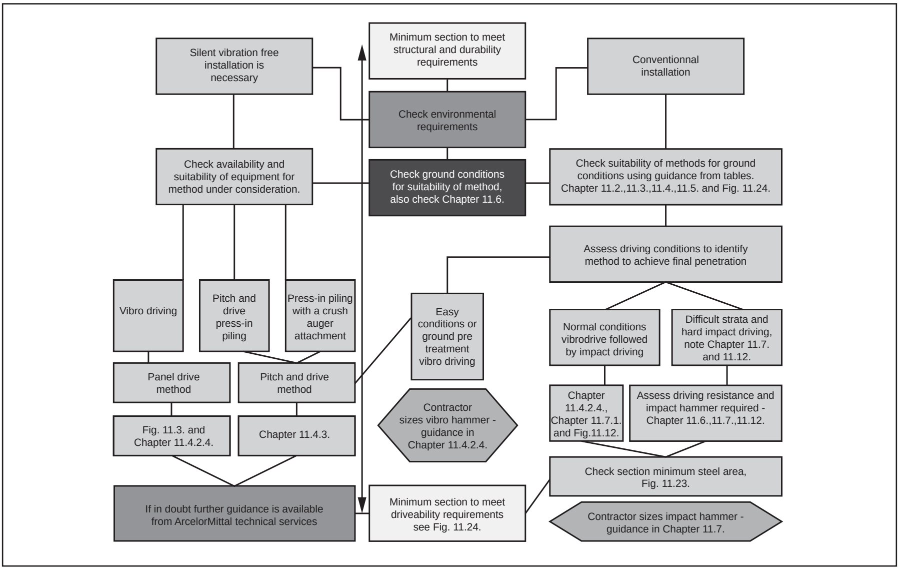


Fig. 11.20. Summary diagram showing influences on choice of section.

11.6.5. Influence of soil type

To assess the prevailing soil characteristics and corresponding driveability, the following tables may assist in the identification of a suitable range of sections. The two distinct methods of installation, panel driving or pitch & drive and three methods of driving are taken into account. The choice of section and suitability of the driving method will also depend on whether the piles are driven in singles or pairs.

Tables 11.2. and 11.3. give guidance on methods to consider for soil types, Table 11.4. takes into account resistance effects from strata thickness. Table 11.5. gives guidance for press-in piling by pitch and drive only.

The selection of a suitable pile section for driving into cohesive soil is a complex process and the section choice is often based on previous experience. However it is possible to assess the driving resistance using the surface area of the piling profile and the characteristics of the cohesive strata. The following table may be used for preliminary assessment.

	Driving method		
SPT value	**Vibro drive**	**Impact drive**	**Press-in singles (with jetting)**
0 -10	Very easy	Runaway problem ⇒ use vibro method to grip pile	Stability problem & insufficient reaction
10 - 20	Easy	Easy	Suitable
21 - 30	Suitable	Suitable	Suitable
31 - 40	Suitable	Suitable	Difficult ⇒ consider pre-auger
41 - 50	Very difficult ⇒ consider pre-auger	Suitable ⇒ consider HYS	Difficult ⇒ consider pre-auger or crush piler
50+	Very difficult Pre-auger essential	Suitable ⇒ require HYS	Very difficult ⇒ crush piler required

HYS: High Yield Strength Steel

Table 11.2. Driving in coarse soils or predominantly cohesionless ground.

	Driving method		
C_u value	**Vibro drive**	**Impact drive**	**Press-in**
0 - 15	Easy	Runaway problem ⇒ use vibro method to grip pile	Possible stability problem & insufficient reaction
16 - 25	Suitable	Easy	Easy
26 - 50	becoming less effective with depth	Normal	Normal
51 - 75	Very difficult	Normal	Normal
76 - 100	Not recommended	Suitable ⇒ drive in pairs	Difficult
100+	Not recommended	Suitable ⇒ essential to drive in pairs	Difficult ⇒ consider ground pretreatment or jetting

Table 11.3. Driving in cohesive strata.

Impact driving - Driveability of pairs

C_u value	0 - 2m penetration	2 - 5m penetration	> 5 m penetration
0 - 15	Runaway problem ⇒ use vibro method to grip pile	Easy	Easy
16 - 25	Easy	Normal	Normal
26 - 50	Easy	Normal	Suitable
51 - 75	Normal	Suitable	Suitable ⇒ consider HYS
76 - 100	Normal	Suitable ⇒ consider HYS	Suitable ⇒ consider HYS
100+	Suitable consider HYS	Hard ⇒ HYS recommended	Hard ⇒ consider pretreatment or HYS

HYS: High Yield Strength Steel

Table 11.4. Consideration of driveability characteristics relative to cohesive strata thickness.

Pile Length (not exceeding) (m)	Press-in force < 80 t Small multi-ram press (single pile press-in)	Press-in force 80t - 150t Japanese Walking press (possible jetting) (single pile press-in)	Press-in force 100t - 200t Large Multi-ram ; Japanese Crush Piler (without jetting) (double pile press-in)
8	PU 8, GU 8N	AZ 14-700, PU 12, GU 13N	-
10	AZ 12-700, PU 12, GU 13N	AZ 20-700, PU 18-1, GU 16N	-
12	AZ 17-700, PU 18-1, GU 16N	AZ 24-700, PU 22+1, GU 23N	AZ 17-700, PU 18-1, GU 16N
14	Consult machine supplier	AZ 36-700N, PU 28-1, GU 27N	AZ 20-700, PU 22+1, GU 23N
16	-	AZ 44-700N, PU 28+1, GU 30N	AZ 26-700, PU 28-1, GU 27N
18	-	AZ 46-700N, PU 32, GU 32N	AZ 36-700N, PU 28+1, GU 30N
20	-	Consult machine supplier	AZ 40-700N, PU 32, GU 32N
22	-	-	AZ 46-700N
> 22	-	-	Consult machine supplier

Table 11.5. Press-in piling – Recommended section sizes for maximum length of pile in suitable ground conditions. This table is based on minimum section sizes for suitable conditions noting that loosening soil by pre-drilling or waterjetting may be used in non cohesive soil.

Note: For the new AZ-800 range, please consult your machine suppliers.

11.6.6. Driving dynamics and driving characteristics for impact driving sheet piles

Whichever method is adopted it is important that an acceptable rate of penetration is maintained to prevent machine or pile damage. The size and the type of hammer must be suitable for the length and weight of the pile being driven and it is assumed that the ground is penetrable.

For impact driving the rate of penetration, or blow count, is the most recognisable indicator. The mean driving stress, σ_m, in the pile section is a function of the resistance and section properties of the pile:

$$\sigma_m = R_{app} / A_{act}$$

where R_{app} is the total apparent resistance and A_{act} is the actual cross section area of the pile in contact with the anvil plate or driving cap.

The driving stress in the pile section can be used as an indication of the expected driving difficulty; an approximate guide is given in table 11.6.

Driving condition for impact driving	**Easy**	**Normal**	**Hard**
Driving stress	25% f_y	25-50% f_y	50-75% f_y
Rate of penetration (blows per 25 mm)	< 2	2 – 8	> 8

Table 11.6. Driving stress based on expected driving difficulty.

Note: The driving stress should not exceed 75% of the yield stress in order to prevent damage to the pile section. Prolonged driving should not exceed 10 blows per 20 mm penetration (see Chapter 11.4.1.6). A more powerful hammer should be used provided the pile section and steel grade are adequate.

11.7. Resistance to driving

In penetrable ground, sheet piles are regarded as minimal displacement piles. The driving resistance of a sheet pile (single or pair) is simplified by the following relationship:

R_{app} = soil resistance (skin friction & toe resistance) + interlock resistance

The apparent resistance depends on both, the soil conditions and the length of embedment. Unlike tubes, H piles and other relatively closed sections, skin friction will dominate over plugging effects. The wider the sheet pile is, the less plugging effect is noticeable.

The interlock resistance will depend on the type of interlock, method of driving and whether the interlocks are treated with lubricants or sealants, to prevent small soil particles entering the interlock. Very important is the straightness and verticality of the already installed piles. Damage to piles after transport and handling or poor general condition of used sheet piles will also increase resistance significantly.

For effective pile driving, the total resistance has to be overcome by such a margin, that the pile progress into the soil is at a high enough rate, so that damage to pile or hammer is unlikely to occur. In order to achieve this with impact driving, the momentum of the hammer, namely the product of the ram mass and the velocity at impact, must be sufficient. The delivered kinetic or potential energy are often used as criteria for selecting an appropriate size of impact hammer (see Table 11.8.). It is essential, that the impact hammer is able to deliver the blow to the full cross section area of the pile in a balanced way, with a correctly fitted driving cap or leg guides to centralize the blow.

11.7.1. Impact hammer efficiency

The delivered energy = hammer operating rated energy x efficiency of the blow.

Please note that some hammers have adjustable output.

The impact hammer efficiency (ζ) takes into account losses of energy at impact, in the pile driving cap and the effect of absorption into the pile. Hammers of different types with different caps, plates and guides have various efficiency ratings. The poorer the fit to the pile, the lower the efficiency of the hammer and hence the amount of energy delivered. Table 11.7. indicates the potential difference in efficiency of hammers when used on sheet piling. Efficiency of rope suspended hammers on sheet piling can also be affected by a tendency to rock and move position during the driving process, resulting in inaccurate alignment of the central axis of hammer and sheet pile section. This can occur if poorly fitting leg guides are used, especially with slim powerful hammers with a large drop, such as diesel hammers.

Impact hammer type	Efficiency rating Leg guides and fittings in ...		
	Poor condition	**Good condition**	**Excellent condition**
Hydraulic – rope suspended	< 80%	80 - 85%	85 - 90%
Diesel – rope suspended	< 35%	35 - 50%	50 - 65%
Diesel – leader rig mounted	< 50%	50 - 65%	65 - 75%

Table 11.7. Impact hammer efficiency (fitting of the correct driving cap is essential).

The efficiency of the blow is also affected by the absorption of energy into the pile and the ratio of the impact hammer's ram weight (W) to the weight of the pile and driving cap (P).

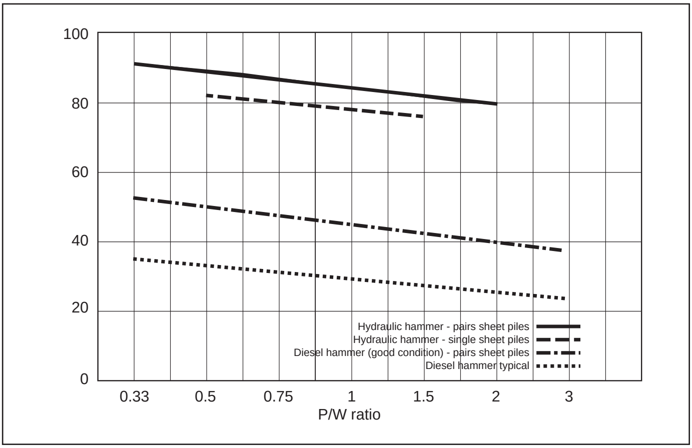


Fig. 11.21. Demonstrates the effect on efficiency by comparing pile to ram weight ratios of different types of hammer.

Note that driving in pairs doubles the mass of the piles to be driven. Larger hammers with heavier rams can be used on pairs, but it may not be possible to fit such a hammer on single piles.
If the hammer does not fit correctly, the efficiency reduces considerably.

11.7.2. Delivered energy

For impact hammers the delivered energy can be calculated as follows

a) for free fall hammers, using gravity force

$E = W \times h \times \xi$

with

W weight of the ram;

h drop height;

ξ overall efficiency of the blow.

b) for double acting or accelerated hammers

$E = E_{op} \times \xi$

with

E_{op} = hammer operational delivered energy set by the operator.

The maximum deliverable energy E_{max} is

$E_{max} = E_R \times \xi$

with

E_{max} = hammer manufacturers maximum energy rating;

$E_R = 0.5 \times m \times v^2$, where m is the ram weight and v its velocity at impact.

11.7.3. Measuring the delivered energy

The delivered energy can be measured

a) by means of the hammer operation and control equipment

For a drop hammer, the free fall distance is measured or set by the operator - if the control equipment provides the facility, a digital readout may be obtained for hydraulic drop hammers. Diesel hammers are usually difficult to assess. Although different settings are available on the controls, the usual method of assessing the energy is by recording the blow rate and referring to graphs provided by the hammer manufacturer.

For double acting hammers, timing the blow rate may also be necessary, but by far the best way is to have the controls calibrated and fitted with digital readout equipment. Some hammers can be controlled by pre-setting the required delivered energy in this way. The readouts can also be connected to a portable laptop to store and monitor the readout for driving record purposes.

b) by means of dynamic monitoring

Pile Driving Analyser (PDA) equipment is obtainable from specialist companies and transducers can be fitted to the sheet piles, so that measurements can be taken for:

- hammer efficiency;
- internal driving stresses;
- pile capacity.

The blow count and hammer stroke can be measured by using a Saximeter or similar equipment. A software program for analysing measured force such as the Case Pile Wave Analysis Program Model (CAPWAP) can be used to determine site specific soil parameters.

11.7.4. Sizing the impact hammer

Studies have shown that modern hydraulic hammers, operating at impact velocities in the order of 5m/sec, are able to overcome approximately 100 tonnes of apparent resistance per tonne of ram mass at maximum performance. On the basis that these hammers operate at 80% to 95% efficiency, it is possible to relate the actual delivered energy to the apparent driving resistance at a rate of penetration approaching refusal as illustrated in Fig 11.22.

The line on this graph represents the boundary between acceptable performance and effective refusal, defined here as 10 blows per 20 mm penetration; the chosen hammer should operate on or below the line.

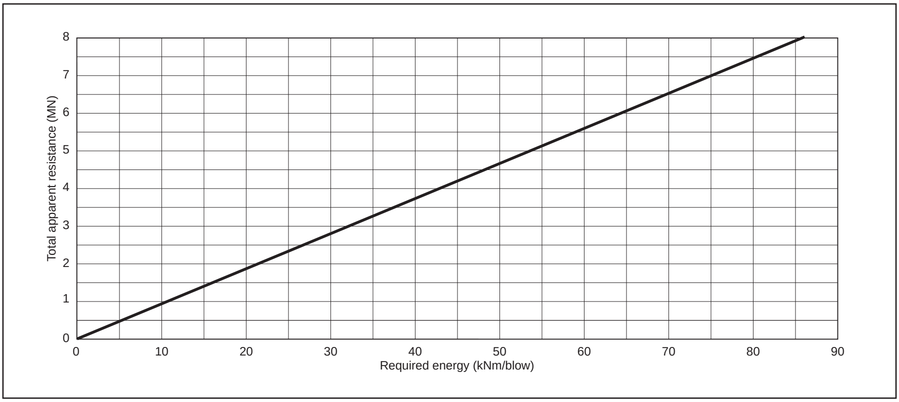


Fig. 11.22. Relationship between required delivered energy and apparent driving resistance near refusal.

Specialist advice from hammer manufacturers is recommended for offshore projects, combi-wall king piles or conditions with apparent resistance above 8000 kN.

The total apparent resistance may be estimated on the following basis:

$R_{app} = R_s \times F_d$

where R_s is the sum of the skin friction resistance over the embedded length of the driven pile.

The end bearing resistance is ignored for the purposes of sizing the hammer, as the pile is assumed to be driven at a rate less than 10 blows per inch (≈ 2,5 cm) and end bearing usually only becomes significant as refusal is approached.

Skin friction = area of pile in contact with the soil × unit frictional resistance.

See Chapter 6 for guidance for calculation of static skin friction soil resistance.

F_d is a dynamic resistance factor for sheet pile driving which depends on the velocity at impact, damping effects and interlock friction. Damping effects and interlock friction will also depend on soil characteristics and length of embedment of the pile.

For a conservative approach, if the hammer is unknown, a value of F_d = 2.0 may be appropriate for piles less than 20 m long, and F_d = 2.5 for piles longer than 20 m.

Depth of pile embedment	Hammer ram velocity at impact	
m	< 4 m/sec	≥ 4 m/sec
< 5	1.2	1.2
5 to 15	1.2 to 1.5	1.2 to 2.0
> 15	Not recommended	> 2.0

Table 11.8. Empirical guide to estimate F_d if the hammer is known and piles are driven accurately.

A suitable pile hammer can be selected using the procedure described above.

Please note that the selection of appropriate driving equipment is an iterative process as the apparent driving resistance is a function of the pile size and depth of embedment.

11.7.5. Driving dynamics and selection of suitable pile section and steel grade for impact driving

Taking into account above tables, criteria for selection of the pile section can now be established for impact driving in penetrable ground.

When identifying a suitable pile section, it is recommended that the peak driving stress should generally not exceed 75% of the yield stress. After selecting a section, the mean driving stress can be estimated by dividing the apparent resistance by the section area.

Please note that for R_{app} > 8000 kN, high yield strength steel grades may be appropriate.

We recommend contacting our Technical Assistance department for further guidance on selecting appropriate products and installation methods.

Fig. 11.23. may be used to estimate the minimum area of steel pile to be driven before selecting a pile section and steel grade.

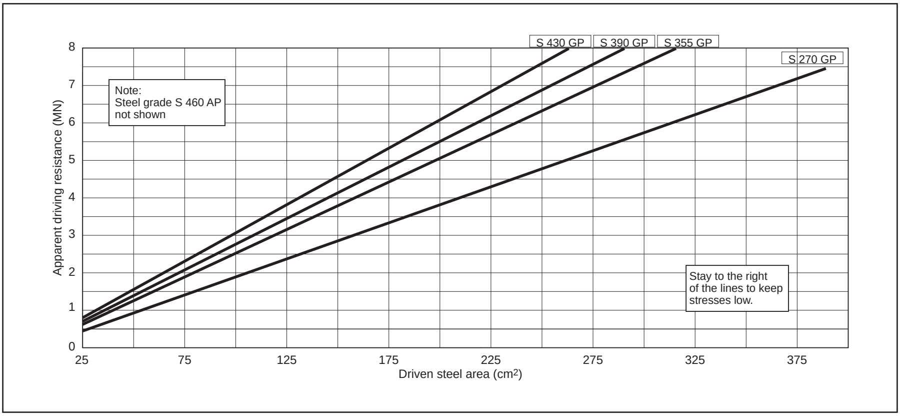


Fig. 11.23. Minimum steel area to be driven for a given apparent driving resistance.

To further reduce the risk of head damage, the area of steel provided should be assessed on the basis of the area actually covered by the hammer anvil, not the cross section area of the pile.

11.7.6. Relationship between peak stress and hammer efficiency

For hammers with low efficiency it is possible that peak stresses will be significantly higher than mean stresses.

Table 11.9. below is based on the equation

$$\sigma_p = \sigma_m \left(\left(2 / \sqrt{\xi} \right) - 1 \right)$$

ξ	0.9	0.8	0.7	0.6	0.5	0.4	0.3
Factor σ_p/σ_m	1.108	1.236	1.390	1.582	1.828	2.16	2.65

Table 11.9. Factor for peak stresses in the pile section.

The effect of reduced hammer efficiency can thus be taken into account by multiplying the calculated apparent mean driving stress by the factor from table 11.9. to obtain the estimated peak driving stress. If this is greater than 0.75 f_y then a larger section or higher steel grade should be tried and the mean and peak stresses re-calculated.

11.7.7. General comments on driveability and use of tables

The method indicated above for the selection of pile section and hammer does not take into account a specific driving method. Generally, a heavier section will drive better than a lighter section and panel driving will yield better results than pitch & drive techniques. In this respect there is a limit to the suitability of any particular pile section in respect of a pure pitch & drive technique.

11.7.8. Influence of stiffness of pile and driving method

Excessive penetration of any pile beyond its adjacent pile can result in the driven pile deviating from the theoretical line, since the stiffness of the projecting pile will diminish as a function of the non-interlocked length. The stiffness is required to resist deviation of the pile especially in non homogeneous or hard ground. Hot rolled interlocks on sheet piles are purposely manufactured to resist high forces induced at the toe of the pile when driving into hard ground. The interlocks, when connected to neighbouring elements, act as a restraint to guide the pile through the ground and enhance the stiffness of the section being driven.

The distance any pile is driven below its neighbour should be limited for panel driving. Recommendations are as follows:

Driving method	Soil characteristic			
	Easy	**Normal**	**Hard**	**Extremely hard e.g. rock < 30 MPa**
Panel driving – impact driving in pairs	8 m	4 m	2 m	0.5 m
Pitch & drive – vibro-driving – refer to 11.2.2. or press-in	singles up to 16 m pairs up to 25 m	depends on moment of inertia on weakest axis of section driven	not recommended ⇒ consider pre-drilling or water jetting	unsuitable

Table 11.10. Maximum length a pile is driven beyond neighboring pile.

11.7.9. Other factors affecting choice of section

After identifying a suitable pile section for driveability, the following factors should also be taken into consideration to adjust the final choice of section and steel grade:

Aspect	Influence on pile section choice
Environmental conditions	If vibration free methods are required, then appropriate sections for machine availability must be considered
Difficult strata – cobbles and rock, SPT > 40	Choose high steel grades, consider increase in section size or reinforce pile toe
Pre-drilling or water jetting	Reduction in section size if vibro-driving possible. Reconsider structural and design implications and risk of greater deflections than calculated. Toe stability to be guaranteed
Watertightness	Wider sections with fewer interlocks improves performance for watertightness
Secondary piles of combi walls	AZ piles best to accommodate tolerances of driven king piles

Table 11.11. Other factors influencing the choice of a section (sheet piles conforming to EN 10248).

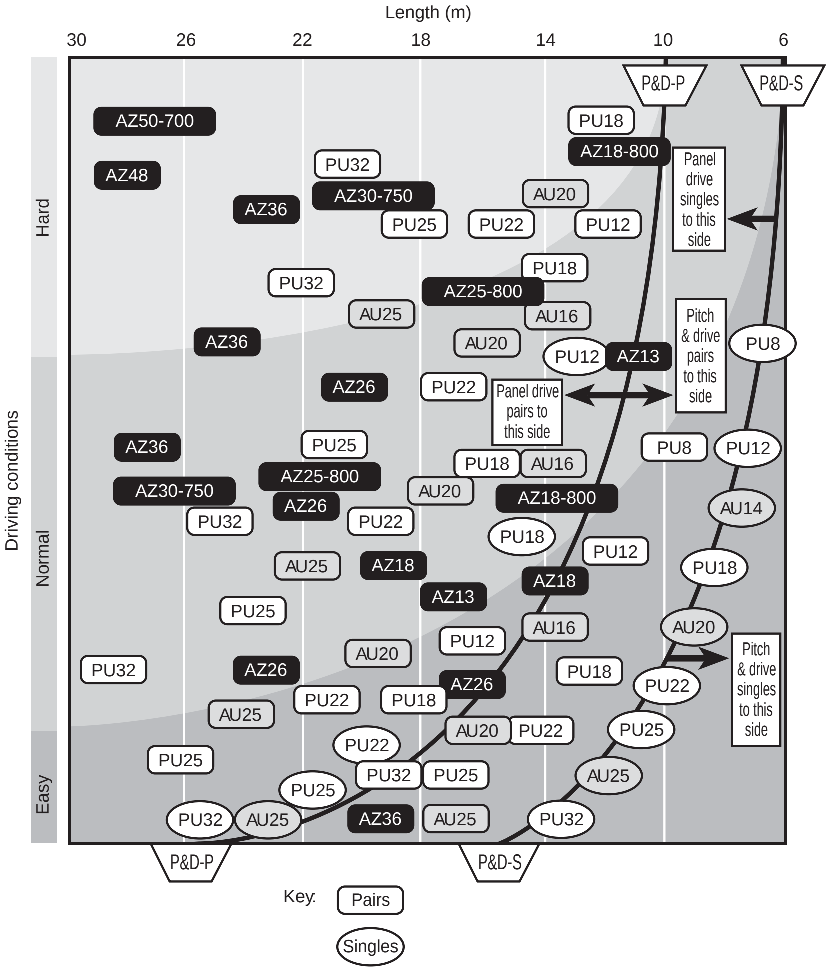


Fig. 11.24. Recommended section modulus in regard to pile length and driving conditions.

11.8. Guiding the piles and controlling alignment

11.8.1. General

A rigid guiding system must be employed when driving steel sheet piles. The guide can be manufactured on site or prefabricated. If a free suspended vibro or hammer is used, a two-level guiding frame is recommended. The two levels should have a minimum distance of 3 m. If a leader guided machine is used, a simple frame on the ground is usually sufficient, as the main verticality control is done by the operator and machine.

Steel systems are used for rigidity and facilitate temporary connections by tack welding the driven sheets to the steel guide walings to control alignment.

Walkways of the correct width, handrails and proper access ladders must be provided to comply with Health & Safety regulations. Supporting trestles are quick to erect, strip and move, and can be dismantled and neatly stacked for transportation. Safety features are incorporated to provide safe access and working space when assembly is either partially or fully complete. Walkway walings provide safe access to the work area and a secure working space. They are stiff box-girder beams and therefore will also serve as a rigid guide and straight edge for accurate pile alignment. Regular cleaning, a non-slip surface and the provision of drain holes is recommended.

11.8.2. Guide walings

The functions of the guide walings are to

1. support piles in the vertical plane during pitching operations;
2. restrain the sheet piles during driving and prevent lateral flexing;
3. control parallelism of the pans or flanges of the piles;
4. minimise rotation of the interlocks and thereby minimize friction in the lock;
5. act as setting out restraints and a physical check on the correct alignment of the pile line;
6. provide access for personnel to pitch the piles, carry out welding and access the piles effectively, provided they are wide enough to function as a walkway;
7. facilitate fixing of permanent walings to structurally support the sheet pile wall;
8. act as a template when constructing walls with complex and irregular shapes, setting out corner and junctions accurately and construction of circular cofferdams;
9. give adequate control of wall length.

It is particularly important that sheet piles are maintained in the correct horizontal and vertical alignment during installation. This is achieved by the use of effective temporary works and guide frames which should provide support to the piles at two levels. To be effective, the top and bottom guides must be rigid. The temporary works may be pinned to the ground using temporary H piles to prevent movement of the whole frame.

The effectiveness of the guides and accuracy of driving will be improved by maximising the distance between the two support levels. Very long sheet piles may need intermediate guides to prevent flexing and other problems associated with the axial loading of long, slender structural members. Pile installation may exert large horizontal forces on the guides and it is essential that the temporary works used to support the guide walings are adequately designed and rigidly connected so that movement or collapse does not occur during driving operations.

To prevent pile twist within the guide frame, the free flange of a Z type sheet pile or free leg of a U type pile should be secured by a guide block or strap connected across the waling beam during driving.

When driving piles in water the lower frame can be attached (above or below water) to temporary bearing piles.

When installing in marine conditions it is possible to use tube pile sections as the horizontal walings to facilitate pitching if the lower guide is expected to become submerged by the incoming tide. The curved upper surface of the tube will ensure that the pile being pitched is guided into the correct location between the walings.

Ladder access must comply with H&S regulations and because of the inherent danger, it is essential that sheet pile pitching is not carried out from ladders. Access platforms must be positioned to enable safe access throughout all operations. Purpose built trestles and walkways are designed so that the top guide waling can be removed at the appropriate time to allow the pile to be driven with the bottom guide in place maintaining control in the intermediate stages of driving.

It is recommended that two levels of guide walings are used for panel driving and pitching piles using a crane. Crane suspended hammers are normally used for the initial stages of panel driving because they can be used at greatest reach with the crane for withdrawing or lifting the pile if adjustments are necessary. Already in the preparation of the job site, boom length and crane capacity should be checked, to avoid under-dimensionning of equipment.

Temporary works provide support to the upper level guide waling for the piles. To be effective it should be at least a third of the pile length above the lower guide and preferably located as close to the top of the pitched piles as possible.

11.8.3. Guiding the piles when installing with fixed or telescopic leaders

Leader rigs are often used for pitching and driving, if access is available close to the pile line. With this method it is usual for both the hammer and the pile to be guided by the leader. As a result there is less need for upper guide walings but it is nevertheless recommended that a rigid, ground level guide waling shall be used to prevent excessive twisting of the piles during the driving and correction process.

It is important that the leader is always vertical and that the hammer delivers its energy through the centroid of the pile profile.

Robust guide walings are critical to achieving a straight pile wall. The spacing of the beams must be maintained by spacers to suit the theoretical depth of the paired pile section + approximately 10 mm. Therefore if AZ 26-700 piles are to be used the spacing of the pile guide walings should be 460 + 10 = 470 mm.

When pitching and driving a guide element consisting of spreader and bracket should be located adjacent to the sheet piles being driven to prevent frame bulging. The wider the guide walings are set apart the more freedom for rotation occurs making the wall untidy and more difficult to drive. Correct guide waling spacing will also ensure good control of wall length.

11.9. Handling, sorting and lifting the piles on site

11.9.1. Stacking

It is essential that the piles are stacked safely on dunnage with spacers on firm level ground before handling for installation. This is not only important to prevent accidents caused by stacks toppling and trapping personnel, but also to minimise damage whilst piles are stockpiled on site. If piles are painted, special care has to be taken.

11.9.2. Bundles of piles

Bundles of piles should be lifted from the delivery truck using a crane of adequate size. Lifting chains shall be managed and fitted by experienced and trained piling crews.

Bundles of piles can be heavy, so it is essential that adequate stability of ground and equipment is provided for unloading operations.

Unloading piles using excavators without appropriate lifting tools is not recommended.

11.9.3. Splitting bundles and lifting individual piles

These operations need special equipment which is designed for this purpose. Makeshift equipment and use of inappropriate plant, such as excavators, should be avoided.

Simple cast-steel shoes have been designed to slide between each pile in a stack, enabling them to be easily separated and moved horizontally. The shoes are usually attached to long steel rope slings which allow them to be attached at both ends of the pile. U-piles are easy to handle in this way, because single bars balance well in the horizontal position. When handling pairs of piles the shoes have to be attached to the same individual bar to prevent the two piles from sliding apart.

Spreader beams are sometimes needed for handling very long piles and straight web sections.

It is inevitable that piles cut by hot or cold sawing may still have sharp saw rag when arriving on site. It is recommended that an inspection is carried out and any saw rag should be removed to prevent accidental cuts and injuries. Rigger gloves or dedicated steel handling gloves should always form part of the Personal Protective Equipment.

11.9.4. Lifting shackles

A variety of special "quick" ground release shackle (QRS) are available and should be a standard tool in the sheet pile installer's equipment.

The shackles enable releasing the crane connection to the pile from ground level or walkway waling level. This is a fast, efficient and safe way to work and eliminates the risk of personnel climbing ladders to release the lifting device from a pitched pile. The shackle uses a lifting hole in the head of the pile through which a shear pin passes. The slinging holes in the piles can be ordered or cut on site to suit the QRS or other lifting equipment to be used. It is necessary for personnel to be trained to attach and check the QRS to ensure correct insertion of the lifting pins in the slinging hole before the signal to lift the pile is given.

11.9.5. Lifting chains

When telescopic leader rigs are used for pile installation, the process of lifting the pile off the ground is usually achieved by attaching chains fastened at the driving equipment. Holes of adequate size to accommodate the lifting chains are usually cut in the webs of the sheet piles about 300 mm from the top of the piles before pitching. This enables the pile to be lifted up to the hammer jaws near the top of the mast. The pile is then driven and the chains are released near to ground level before the hammer or mast needs to be moved away from the pile.

In some cases, the watertightness of the sheet pile wall is essential to the project and has to be guaranteed also at the head of pile. If not covered by a concrete capping beam, the handling holes can be closed either by welding a plate over the opening, or by using special sheet pile plugs made of plastic.

11.10. Pitching - connecting the interlocks when pitching the piles

Interlocking the piles together in the vertical position is called pitching. The greatest risk of injury to piling personnel occurs during pile pitching so it is important to develop a safety plan and an approved method of working before work commences.

The following actions or conditions must be ensured to comply with H&S regulations:

1. the lifting equipment must be securely connected to the top of the pile until the pile is fully threaded and supported by the ground. Piles should not be allowed to free fall;
2. personnel threading the piles or handling the free end of the pile being lifted must operate from a safe working platform or ground level. Operatives must not stand on ladders or balance on the tops of piles when piles are being pitched;
3. sufficient personnel need to be available for handling the size of pile being pitched especially in windy conditions. One or two operatives should restrain the pile from swaying – using ropes if necessary. The crane operatives should

avoid slewing or moving the jib when operatives are attempting to pitch the piles by hand. The piles should only be lowered when the correct signal is given by a qualified banksman.

11.11. Threading devices

The sheet pile threader is designed to interlock any steel sheet pile accommodating the different profiles and interlock types without the need for a man to be employed at the pile top. Use of a pile threader allows pile pitching to continue in windy conditions which would stop manual interlocking, making the work both safer and more efficient.

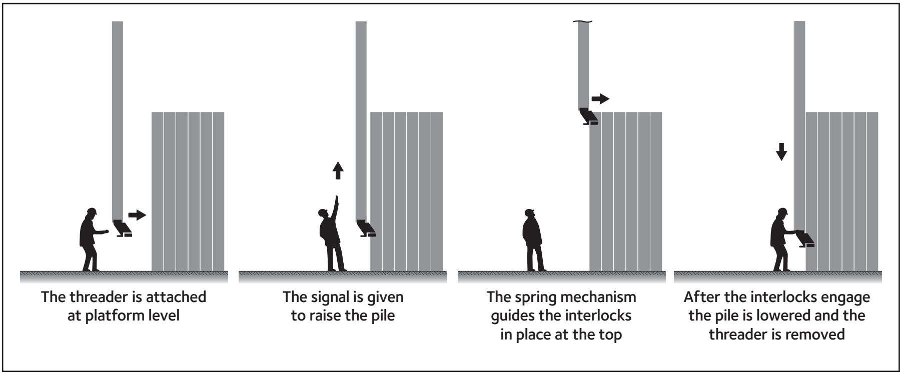


Fig. 11.25. Using threading device to pitch piles safely at height.

11.12. Driving assistance

11.12.1. General

In case sheet piles have to be installed in hard to very hard soil conditions, several methods of facilitating the piling works can be used. Impact driving, vibrating and pressing of piles can be made easier with the help of water jetting. The process delivers water through pipes to the pile toe, where it loosens the soil, reducing toe resistance and skin- and interlock friction. The water is delivered at controlled pressure by standard pumps connected to the pipes. Sufficient water supply has to be guaranteed.

The effectiveness of jetting is influenced by the density of the soil and the content of fines present, the available water pressure and the number of jetting pipes. Piles installed in a leading trench will help to control the disposal of jetting water and to keep operations and the site as tidy as possible.

Care must be taken to ensure that this form of ground treatment does not endanger adjacent structures. A loss of fines can occur in the soil near the pile. Small settlements and a temporary reduction of the friction angle of the soil along the wall cannot be ruled out. The effect of this on the wall should be assessed and taken into account by the design engineer.

Control of the jetting equipment during operation is highly recommended. The least amount of water should be used to advance the piling. The piles should be driven to final level without jetting wherever possible, to ensure cut-off function and bearing capacity.

Jetting techniques and details should be agreed between contractor and consulting engineer in a method statement before commencement.

Airlifting can be an option in specific cases such as for removal of soil within enclosed pile sections such as box piles.

In case of shallow rock, toe pinning or rock-dowelling can be considered for developing additional resistance at the toe of the piles to compensate for insufficient passive resistance where soil penetration is limited.

11.12.2. Press-in piling and jetting

Press-in piling machines work best in loose and soft cohesive soils. The efficiency of the press in non-cohesive soils can be optimized with water jetting. A water supply and powerful pumps are required. Systems to recover the jet pipes from the ground are available from some manufacturers.

11.12.3. Low pressure jetting

Low pressure jetting is mainly used in loose to dense soils. It helps by lubricating in dry higher strength cohesive soils.

In general the soil characteristics are only slightly modified, although special care must be taken when piles have to carry vertical loads. See ArcelorMittal's publication "Jetting-assisted sheet pile driving" for further details.

11.12.4. High pressure jetting

This type of jetting may be used for driving in extremely dense soil layers. High pressure jetting should only be carried out with the consulting engineer's consent and an agreed method statement.

11.12.5. Pre-drilling

If dense soils are encountered, pre-drilling in the pile axis loosens the soil and allows easier installation with vibro-driving or pressing. The auger diameter should be approximately 1/3 of the pile width, which is about 20 – 50 cm. The best position and number of drillings is usually tested on site. Larger diameters or overlapping drills are occasionally utilized. Depths of up to 12 m can be treated usually with the same base machine, just by changing the tools on the telescopic or fixed lead.

When loosening the soil by drilling, care must be taken not to remove too much of the soil when withdrawing the auger. Holes can be left in the ground when attempting to drill into high strength cohesive soils. Any holes that do occur should be filled with granular soil before driving piles. Pre-drilling should be avoided in the passive zone near the toe of the piles and where artesian water can be encountered.

It must not be forgotten, that drilling effectively changes the nature of the soil and possibly the water table regime in which the pile is located. This fact may invalidate the design assumptions, the effective angle of wall friction and soil stiffness may be reduced. It is also likely that wall deflections will increase especially in any temporary construction stage or cantilever condition.

Any pre-drilling must be agreed with the consulting engineer before commencement.

Fig. 11.26. Predrilling for a combined wall installation.

11.12.6. Blasting

This process is applicable if the ground consists of rock with a compressive strength above 20 MPa. If a trench of blasted rock is created pile installation may be possible by vibrodriving or impact driving using panel techniques. However blasting is not always 100% successful first time and further treatment may be required.

With the development of new piling techniques and pre-treatment, blasting methods are becoming increasingly rare. Please contact the local ArcelorMittal technical department service to discuss suitability for your specific site.

11.13. Pile driving corrections

11.13.1. Correction of leaning forward or backward

Care should be taken to pitch the first piles vertically and maintain them in a vertical position within permitted tolerances.

Leaning forward is a phenomenon sometimes seen in loose soils, while leaning backward can happen in high strength soils. In order to avoid the tendency of sheet piling to lean, the hammer should be positioned over the centre of gravity of the piles being driven and should be held vertically and firmly on the piles by means of efficient grips. When driving in pairs the adjacent piles should be square and straight at the top and the hammer blow shall spread evenly across the maximum area of steel by means of a correctly sized and fitting anvil or driving cap. Transverse leaning of sheet piles is eliminated by the use of efficient guide walings. If the piles develop a transverse lean which needs to be corrected, the piles should be extracted and re-driven in shorter steps to maintain control, see 11.2.3. "panel driving".

Longitudinal leaning in the direction of driving can be caused by additional friction between the previously driven pile and the pile being driven, or by incorrect use of the hammer. Immediate counter acting is required when leaning occurs. If left unchecked, the lean can become uncontrollable, requiring piles to be withdrawn until an acceptably vertical pile is found. Pile installation can then continue using panel methods to reduce the risk of further lean.

Prevention is better than cure and when using pitch and drive methods, driving should cease before the lean approaches the maximum permitted verticality tolerance limits. In conjunction with the above method, longitudinal lean may be corrected by pulling the misaligned piles back while displacing the hammer from the centre of the pair towards the last driven piles. When a lean cannot be eliminated and piles cannot be withdrawn or replaced, the error may be corrected by introducing taper piles, but only with the consent of the consulting engineer.

As an alternative, impact driving can be used instead of vibro-driving. Where the problem is encountered locally the simplest means of prevention is to tack weld the pile being drawn down to the temporary guide walings - however these must be adequately supported, so that they do not move or collapse when driving the piles. The problem is less likely to occur if the piles are installed with good alignment and verticality and the problem may be alleviated by introducing a sealant to the interlocks to prevent the ingress of soil to the interlock area during driving. Grease and lubricants shall not be placed in the interlocks without the prior consent of the consulting engineer.

11.13.2. Control of wall length

In general, the tendency of creep is rather limited on U-piles with Larssen interlocks, as rotation capacity is limited to 5 degrees. When using uncrimped piles, an effective wall length control can be achieved by adjusting the distance between the guide walings. If accurate theoretical wall dimensions have to be achieved, it may be necessary to introduce a fabricated pile at the end.

Note: rolling tolerances on the width of the sheet piles (according to EN 10248 [vi]) and installation tolerances (according to EN 12063 [ii]) should be taken into account, above all for long straight walls for which it is strongly recommended to order a few spare piles in case quick delivery of additional piles might be an concern. Trying to stretch out or squeeze a Z-piles to extend the theoretical width may affect the section properties of the wall detrimentally. See also section 11.14.

11.13.3. Drawing down

When piles are driven in soft or loose soils, the pile being driven may draw down the adjacent pile below its intended final level. The problem sometimes occurs when the pitch & drive method is used and is caused when more friction develops in the interlock connected to the pile being driven than is available in the interlock connected to the previously driven piles.

This may happen when either or all of the following occurs

- the piles are leaning forward;
- the piles have been allowed to rotate causing additional interlock friction;
- vibro-driving action has compacted sand into the interlocks during installation;
- interlocks have not been cleaned before driving;
- interlocks have been damaged or bind together on one side of the pile.

A solution is to re-level the heads by withdrawing the piles and tack welding them together in pairs or triples and proceeding with a panel back driving method. Pairs of piles will usually be driven easily in soft ground where this problem is usually found.

11.14. Driving tolerances

Theoretical position and orientation of the sheet piles are usually indicated in the driving plan and on working drawings. Deviations from this theoretical layout may occur due to rolling tolerances, soil conditions and driving procedure.

General tolerances for a straight and plumb sheet pile wall should be in accordance with EN 12063 [ii]:

- deviation in plan normal to the wall line at the top of the pile ± 50 mm (± 75 mm for silent press-in piling);
- deviation of verticality for panel driving - all directions 1 in 100;
- deviation of verticality along line of piles for pitch & drive 1 in 75;
- finished level deviation from nominal level at top of pile ± 20 mm (higher values may be accepted for certain structures).

More stringent tolerances for verticality may be necessary for very long sheet pile elements, or for combined walls with piles over 26 m length. This is necessary to minimize the risks for pile damage or declutching of secondary elements in a combi-wall. Piles that deviate greater than the allowed tolerance should be withdrawn and corrective methods submitted to the approval of the consulting engineer.

Note: Tolerances for plan and verticality are accumulative. Designers and architects should allow for this, especially when considering design of internal fittings or within a cofferdam.

11.15. Special aspects of installation

11.15.1. Test-driving

For large projects or in case driveability of the soil is difficult to assess, test-driving is recommended.

Test-driving is undertaken to determine the most suitable pile section for the given situation. Besides that, the influence of pre-drilling or water jetting can be checked. Test installations should preferably be carried out on the line of the final wall. The number of test piles depends on the size of the project and on the expected variations in the underlying strata. Hammer and pile performance have to be recorded for later evaluation. Subsequent extraction of the piles may give supplementary information. In case piles cannot be removed after the test, they shall not obstruct the final wall.

In the event that the sheet piles are designed to carry axial loads, the test piles can be used to perform a load test. Modern pile driving machines are equipped with data recording systems which allow the consulting engineer to evaluate the ultimate capacity of a pile for carrying vertical loads. Equipment for static or dynamic load testing of the piles is available on the market.

11.15.2. Pile driving in restricted headroom areas

Under bridges or other existing structures, the free working height between soil and structure is often insufficient to allow normal pile threading and driving. One possibility is to drive the piles in short lengths, connection done by butt-weld or with fish-plates as driving proceeds. Another way of overcoming the height problem is to pre-assemble a panel of piles horizontally on the ground, the length of the piles being less than the headroom. The panels should be bolted to temporary walings and moved into position. In any case, the headroom may be increased by the excavation of a trench along the proposed line of the piling. Driving is commenced using a double-acting hammer mounted in a cradle, suspended at the side of the pile. As soon as sufficient headroom is available, the hammer can be moved to the normal driving position.

Over the last years a growing number of methods and plant types to suit restricted headroom challenges have been introduced into the market. A low headroom press-in pile driver may provide a solution for extremely tight conditions.

Please consult the local ArcelorMittal technical department for advice on specific circumstances.

11.16. Extracting

11.16.1. General

Sheet piling can be designed to serve only as temporary protection for other construction works. It can be extracted for multiple re-use up to 10 times, depending on local circumstances. Suitable extractors are usually reverse working hammers, vibrators or jacking machines.

To evaluate the required pulling force, the previous establishment of driving records for each pile is highly recommended. This helps identifying the piles with the lowest resistance, thus defining the most advantageous starting-point for the extraction work. If driving records for the piles are not available, then the first pile to be extracted should be selected with care. Piles near the centre of a wall should be tried first, until one pile begins to move. If difficulty is experienced, a few hammer blows may be used to loosen a pile. It may also be necessary to reinforce the head of the piles to aid the successful extraction of the initial pile. Accurate driving of the piles will make extraction easier, greasing the interlocks will reduce friction even after some months in the ground. Drilling close to the pile can be considered to loosen the soil and break the bond between earth and pile.

When designing temporary works, it might be worth increasing the section modulus to ensure good driveability and minimise damage to the piles. The commercial success of the operation can depend on the quantity of piles recovered with minimal damage.

11.16.2. Extraction by vibrator or reverse acting hammer

Vibrators and hammer extractors of various sizes are available on the market. The horizontal and vertical movement breaks the bond between soil and pile, loosens the pile from its initial position, so that it can move with the help of the pulling force of the crane. The technical limits of extractors and cranes given by the manufacturer must be respected for safe working conditions. The connection between pile and extractor must allow for the maximum pulling force of crane and extractor.

In general, the machine used for extraction shall be at least the same size as the one used for installation.

11.16.3. Vibration-free extraction by press-in piling machines

Press-in piling equipment are excellent for extracting piles in sensitive locations as no vibrations are generated. Piles that have been pre-treated with sealants are easier to extract. Both self-supporting and leader guided machines can be used to extract piles. The maximum pulling force depends on the model of the equipment used.

11.16.4. Extraction using the Universal Sheet Pile Extractor

This powerful tool can be used to extract long and heavy sheet piles. A maximum pulling force of approximately 400 - 1000 tonnes can be developed, depending on the machine specification chosen. It requires at least 1.5 m working space on either side of the pile line and a firm hard-standing to work from. The machine needs to be positioned at the open end of a pile line and work backwards. A heavy crane is usually needed to move the extractor to different positions and withdraw and separate the piles.

11.17. Installing combined HZ-M/AZ or high section modulus walls

High section modulus walls consist of HZ-M king piles or fabricated box piles, combined with standard sheet piles. The king piles (also called primary piles) are stiff structural elements, which are connected by intermediate secondary sheet piles (also called infill sheets) with much lesser stiffness. The HZ-M system is a very efficient system that has been used for many decades in projects all over the world. It provides a “straight” face, suitable for marine projects and deep berths. The HZ-M beams are rolled up to lengths of 33 m. They can be extended to longer shippable lengths by welding. Combined sheet pile walls should be installed by experienced contractors equipped with heavy equipment, which should be suitable for offshore conditions where appropriate.

Successful piling demands installation of the primary elements first, using the pilgrim step method, before putting the secondary elements in place. It is essential, that the primary elements are vertical and in correct position before placing the usually shorter secondary piles. The use of a rigid and stable two-level guiding frame is highly recommended, if no appropriate leader guided rig is available. The standard procedure to drive the piles is to start with a vibrator and finish the pile with a sufficient dimensioned impact hammer whenever required (for instance, in case refusal criteria with vibratory hammer reached). The vibrator can be used for the primary and secondary piles.

The HZ-M/AZ system combines all advantages of hot rolled products and mechanical joints for best wall integrity. It is the best system available for high section modulus walls when installed correctly. Other systems may combine fabricated primary elements such as box piles welded together, or tubes with interlocks welded on either side. In such cases, all welding of interlocks and splicing together needs to be executed in accordance with the relevant codes to high standards of workmanship and testing.

To allow arching effects to develop in various soils (predominantly granular soils) the free space between the king piles shall be relatively small. As a rule of thumb, for harbour structures, systems with secondary elements that are narrower than 1.80 m for U-type piles, and 1.60 m for Z-type piles, can be designed taking into account arching effects on the active earth pressure side and a three dimensional effect on the passive earth resistance side (design as a continuous wall even if the infill sheets have been curtailed, provided the embedment depth is sufficient). Larger spans between king pile require an additional verification of the arching effect and of the allowable stresses in the infill sheet piles.

The effects of water pressure difference on infill sheet piles have to be analyzed in detail. Incorrect installation of infill sheets that will be submitted to high water pressure difference may have a significant influence on their stability.

In general, the intermediary piles consist of at least two, maximum three, suitable single sheet piles. The interlocks must be capable of withstanding additional stresses from the action of forcing the sheets between the stiff primary elements and driving them with large hammers. Pairs of AZ piles with a Larssen type interlock are considered the most appropriate elements as secondary piles in combined walls, as they allow for some rotation in the common interlock.

It is recommended to prevent sand particles to clog the connectors of the HZ-M king piles in dense fine sands. This can be achieved by filling partially the interior of the interlocks with a bituminous filler material (such as the Beltan® Plus), or by other means.

11.18. Environmental considerations

11.18.1. Noise and vibration

Modern piling techniques can enable noise and vibration to be eliminated from the installation process for steel piles. When ground conditions are appropriate, hydraulic pile press-in technology enables piles to be driven almost silently and without causing any noticeable vibrations. This technology gives engineers the opportunity to use steel piling in areas where this type of construction would previously have been unthinkable. Sheet piling can now be considered as a first choice material for sites where environmental disturbance will not be tolerated, such as adjacent to hospitals, urban areas, alongside sensitive services and structures. Modern vibro-drivers offer the fastest rate of installation of any piling system, especially in granular soils, and they are less disruptive than impact hammers. Engineering advances have given operators the ability to vary the frequency and amplitude of the machine, so that the system can be tuned to suit best the existing ground conditions. Modern piling technology has also eliminated the severe vibrations generated when the vibro-driver passes through the resonance frequency of the surrounding ground and buildings during run up and run down. Impact hammers cause high levels of noise, but can drive piles into any type of soil and may be the only method available for driving into stiff, cohesive soils or soft rock in addition to a press-in piling machine crush auger attachment. Operation of this hammer type has changed over the past half century: steam has given way to diesel power, which in turn has been replaced by hydraulic forces. As a result, modern hydraulic drop hammers are much less environmentally damaging than their predecessors.

When planning the job site, a useful method to minimize noise and vibration is to consider the pre-drilling technique to loosen the soil and use high frequency vibratory hammers. It is then possible to drive piles in sensitive areas, but it is advisable to monitor vibration, using suitable instrumentation such as accelerometers in these situations. Today it is possible to design and plan the piling project using various vibration free press-in piling systems. The circumstances at each site and the acceptable levels of noise and vibration should be analyzed before choosing the pile driving method. Not every site demands silent and vibration-free pile installation, and cost and time savings may be achieved, if it is acceptable to adopt a less environmentally-sensitive method of installation. The opportunity to adopt combinations of driving methods should also be taken into account.

11.18.2. The effects of vibration

Pile driving using an impact hammer or vibro-driver generates ground vibrations, which are greatest close to the pile. Humans are very sensitive to ground vibrations, and it should be noted that even minor vibrations may attract complaints from people living or working in the area. However, sheet piles are driven by minimal displacement of soil. Damage to buildings caused by careful sheet piling installation is extremely rare.

Heavy ground vibrations may also disturb soils. Piling vibrations may destabilise slopes or lead to compaction settlements of very loose saturated granular soils. Vibrationless installation should be considered where this risk is foreseeable.

11.18.3. Regulatory guidance

Local authorities may stipulate and impose their restrictions prior to and during piling operations. To avoid this situation, a preferable approach is to arrange prior consent. Discussion with the local authority prior to commencement of the work is recommended to agree methods, timing, durations and generally embodying the “best practicable means” for the work. Although present British Standards do not give rigid limits on levels of vibration or noise, helpful guidance on these issues is given in BS 5228 parts 1 & 2, (2009): “Noise control on construction and open sites” ([iii] and [iv]). Alternatively, maximum wave speed values near existing buildings can be found for example in DIN 4150, Part 3 [i].

Clearly any piling project must comply with the applicable regulatory standards in regard to noise, vibration and general environmental requirements.

11.18.4. Good practice

Excessive ground vibrations can also be avoided by following good piling practice. Particular care should be taken to ensure that the pile is maintained in a vertical position by using well designed guide frames or a fixed-lead machine. An appropriate size of impact hammer or vibratory hammer should be selected, and the hammer should strike the centroid of the pile along its axis. Equipment should be in good condition and piling should be stopped if any head deformation occurs, until the problem is identified. Hammering against any obstructions or continued driving when refusal is reached may lead to excessive and unnecessary vibrations.

Specialist measures to overcome this problem include for example the excavation of a lead trench up to a depth of 2 m to avoid old concrete, brick or timber foundations.

Water jetting can be used to penetrate dense fine and coarse soils much quicker, with a lighter pile, and less vibrations.

Pre-drilling is used to break up hard soils, to reduce driving resistance and minimise requirements for impact driving. In all cases with careful planning and by adopting the appropriate piling technique to each specific project and its site conditions, it is possible to minimise noise and vibrations, making this type of problem a thing of the past.

Good liaison with local authorities, local residents and the consulting engineer, careful choice of plant and technique will allow successful and trouble free completion of the piling. For situations where the piling is within 15 m of a sensitive structure, it is recommended that appropriate techniques are chosen. Main contractors for a sheet piling project shall select only specialized sub-contractors with good equipment, to ensure that no unnecessary risks are taken for the pile installation.

11.18.5. Piling in the marine environment

11.18.5.1. Regulation and planning

Legislation has provided for control and mitigation of potential underwater noise and vibration effects which are detrimental to the marine environment. Particularly at risk are marine species and mammals which are sensitive to high levels of underwater transmission of sound waves from piling operations. Also seasonal factors such as disturbance on fish spawning and migratory sea birds has to be taken into account.

Features of conservation are important in aspects of planning for projects to safeguard harmful impacts on species which are protected by European and domestic law. It is essential for this aspect to be included, which forms an intrinsic part of an Environmental Impact Assessment (EIA) required in the process for planning and consents for marine and off-shore projects.

11.18.5.2. Methods to mitigate noise and disturbance in the marine environment

In the UK the Joint Nature Conservation Committee (JNCC) have produced guidance and documentation for protocol of best practice in the marine environment for piling operations to mitigate the risks and protect marine species.

It is necessary to consider the type of structure involving piles foundations and the practicalities involved reducing noise and vibration. Techniques such as pre-treatment of hard soil strata using pre-drilling (loosening) or jetting methods (in fine and coarse cohesionless soils) are important to minimise or eliminate prolonged impact driving. Use of variable frequency vibratory systems or press-in to drive steel piles enables environmentally suitable methods to be employed for all or a major part of the pile driving programme.

Local conditions for a project are taken into account and usually a trained Marine Mammal Observer (MMO) may be assigned with specific duties in respect of construction operations to keep watch and communicate for prescribed procedures to allow commencement and breaks in noisy operations.

Other methods such as shrouding the impact piling hammer can be planned and executed effectively but are more appropriate for driving the primary elements of combined wall systems, where the main piles are driven first, and bearing piles for dolphin structures and open deck jetties.

More expensive methods such as dissipating the noise using bubble curtain barriers are sometimes specified for offshore mono piles and difficult to implement for ports and harbour quay walls. Tidal flows dissipate bubbles and the effectiveness of this type of system.

Lastly the timing and programming of the piling operation for the project can solve the problem so the lead time and procurement of the piles should be taken into account in the planning for commencement of the project.

References:

[i] *DIN 4150. Vibrations in buildings - Part 1: Prediction of vibration parameters. DIN. Berlin. Germany*

[ii] *EN 12063:1999. Execution of special geotechnical work - Sheet-pile walls. CEN. Brussels. Belgium*

[iii] *BS 5228-1: 2009+A1: 2014. Code of practice for noise and vibration control on construction and open sites. Noise. BSI. London. UK.*

[iv] *BS 5228-2: 2009+A1: 2014. Code of practice for noise and vibration control on construction and open sites. Vibration. BSI. London. UK.*

[v] *EAU (2004 + 2012) Recommendations of the Committee for Waterfront Structures, Harbours and Waterways. (2004 & 2012). Ernst & Sohn. Berlin. Germany.*

[vi] *EN 10248: Hot rolled steel sheet piling - Part 2: Tolerances on shape and dimensions. 2006.*

ArcelorMittal

12 | Worked example: anchored retaining wall - tidal river

Chapter 12 - Worked example: anchored retaining wall - tidal river

Contents

Part 1. Design of main wall

12.1. Subject

Worked example for Category 2 anchored retaining wall: anchored retaining wall in a tidal river. *EC 7, 2.1. (10)*

12.1.1. Scope

Stability, section and pile length verification of a sheet pile wall for 6.7 m retained height.

Global stability i.e. slip circles and global sliding is outside scope of this worked example.

12.1.2. Design assumptions

Top of (sheet pile) wall: 0.0 m.

MHWS: -1.0 m.

MLWS: -5.0 m.

Extreme low water excavation side: -6.1 m.

Design formation level -6.7 m plus allowance for uncertainty of level, Δ_a max. = 0.5 m *EC 7, 9.3.2.2. (2)*

Anchor level: -1.0 m.

Flood design water level behind wall at 0.0 m (no reliable drainage in wall). *EC 7, 9.6.(3)P*

This is the highest possible level of groundwater behind the wall in a flood event return period not less than 50 years

Normal highest water level behind the wall -3.0 m. *BS 6349 Fig. 8 (see 1.7.)*

Durability: 50 year design life.

Surcharge loading – characteristic UDL 10 kN/m^2 all cases.

Accidental impact loading on parapet – horizontal point load 200 kN on section of parapet 2.4 m long.

No permanent softening to formation.

Sheet pile has adequate penetration into medium strength clay to prevent significant seepage around toe.

12.1.3. Installation

Conventional driving – pre-augering may be permitted in active zone depth.

12.1.4. Design standards and analysis

Eurocode 7 - Part 1, and its UK National Annex.

Eurocode 3 - Part 1 and Eurocode 3 - Part 5 and its UK National Annex.

BS 6349 Part 1-3 for water level interpolation in tidal design situation.

Design Approach 1 ULS case for stability and sheet pile section verification Combination 1 and Combination 2 – UK National Annex GEO and STR limit states.

EC 7, NA 2.4.7.3.4.1.(P)

12.1.5. Ground surface excavation rule *EC 7-1, 9.3.2.2. (2)*

Eurocode 7 requires an allowance to be made, in verifications of ultimate limit states, for uncertainties in the level of resisting soil surfaces in front of the wall.

In the case of a river, the most likely cause would be scour of the river bed. Maximum long term scour depth is 0.5 m in this case where it is unlikely to be higher.

The design retained height H_d is calculated as: $H_d = H_{nom} + \Delta_a$ *PHB 9, 5.18.2.1.*

Height between formation and support level

$H_b = H_{nom} - 1.0 = 6.7 - 1.0 = 5.7$ m

$\Delta_a =$ 10% x (6.7 – 1.0) = 0.57 > 0.5 m

Therefore the design “excavation” depth is $H_d = 6.7 + 0.5 = 7.2$ m.

12.1.6. Design groundwater levels *EC 7-1, 9.6.(3)P; BS 6349 Fig. 8 (b)*

For verification of ultimate limit states, Eurocode 7 requires the ground water levels to be considered as “the most unfavourable that could occur during the design working life of the structure”. Hence, it has been assumed for the ULS Combination 2 calculation that the water level on the back face of the wall may possibly rise to ground level (flood defence level, $d_{w,a} = 0$ m) and on the front face may drop to low tide level ($d_{w,p} = 6.1$ m) – this is the most unfavourable combination of high ground water level and the maximum possible fall in 24 hours in a flood event. There is no further safety margin applicable on water pressure for Combination 2.

For the Combination 1 calculation the highest normal groundwater would be taken on the backface of the piles (not in a flood event) at -3.0 m in accordance with BS 6349 1-3 2012.

From Fig 8 (c) of BS 6349 1-3:

- MHWS = -1.0 m;
- MLWS = -5.0 m;
- assumed ground water level:

$$-3.0\text{ m} = \left(\frac{MHWS + MLWS}{2} = \frac{-1.0 + (-5.0)}{2} \right)$$

For Combination 1 only the hydrostatic pressure difference is a permanent action and the effect of the action is factored (1.35). This method of application of the partial factor on effects is called DA1-1 and no further allowance or margin on the water pressure differential is required.

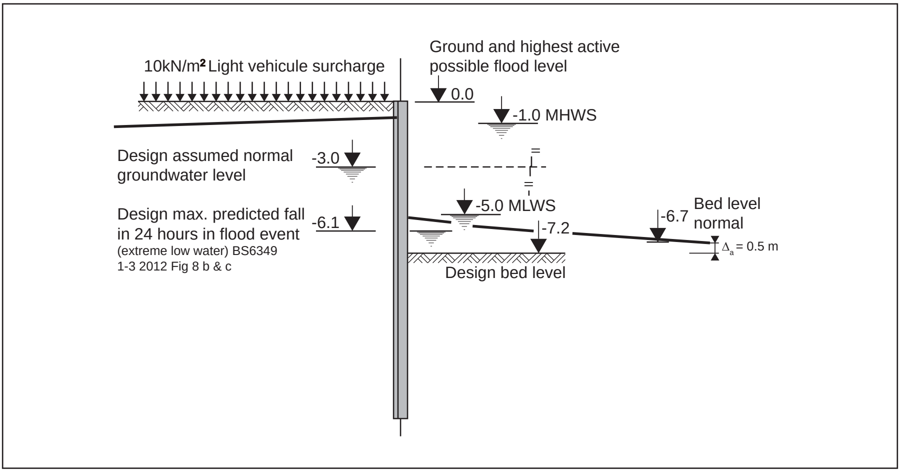


Fig. 12.1. Typical cross section showing ground surfaces and design water levels.

BS 6349 1-3:2012 Fig. 8 b) and c)

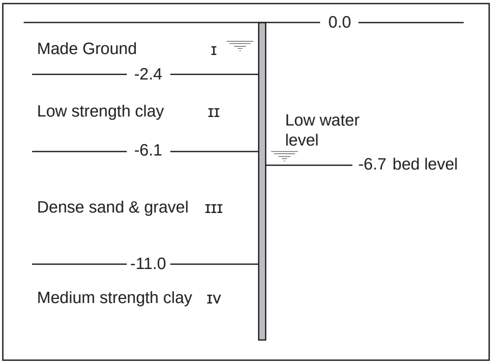


Fig. 12.2. Ground model showing strata levels from G.I for geotechnical design. Typical section.

12.1.7. Abbreviations

In the next chapters, following abbreviations will be used quite often:

LEM Limit Equilibrium Method;

SGRM Subgrade reaction Method;

SSI Soil Structure Interaction;

ULS Ultimate Limit State;

SLS Serviceability Limit State.

12.2. Geotechnical design – ULS case

In the UK, for Design Approach 1, the ULS case is normally checked for the following cases:

- Combination 1;
- Combination 2;
- Accidental loading design situations.

Where loading actions and surcharges are not significant Combination 2 is usually the critical combination for design. For this example Limit Equilibrium Method calculations applying to Combination 2 are demonstrated.

Section 12.3.3 of this example checks the tie load and also Combination 1 and the Accidental loading ULS checks using computer software to take into account soil structure interaction.

12.2.1. Establish characteristic values of soil parameters

Earth and water pressures: persistent ULS design situation
Density and strength of soils – characteristic values

Layer		Depth	Characteristic values				
			γ_{sat}	φ'_{peak} [1)]	φ'_{cv} [2)]	c'	c_u
		m	kN/m³	deg	deg	kPa	kPa
I	Made ground	0.0 - 2.4	19.1	30	30	0	-
II	Low strength clay	2.4 - 6.1	17.2	20	20	0	25
III	Sand and gravel	6.1 - 11.0	20.6	40	35	0	-
IV	Medium strength clay	11.0 - 16.5	18.6	25	20	2	65

Table 12.1. Characteristic values of soil data.

1) Peak value.
2) Constant-volume (i.e. critical state) value.

12.2.2. Design values of ground parameters

EC 7, A.3.

In Eurocode 7, design values of geotechnical parameters (with subscript *d*) are calculated from characteristic values (with subscript *k*) by dividing by the appropriate partial material factors (γ_M):

$$\gamma_d = \frac{\gamma_k}{\gamma_\gamma},\ \tan\varphi_d = \frac{\tan\varphi_k}{\gamma_\varphi},\ c_d = \frac{c_k}{\gamma_c},\ c_{u,d} = \frac{c_{u,k}}{\gamma_{cu}}$$

In Design Approach 1, the required embedment of a retaining wall is normally governed by Combination 2's set of partial factors, for which the numerical values for geotechnical parameters are:

$\gamma_\gamma = 1.0$

$\gamma_\varphi = \gamma_c = 1.25$

$\gamma_{cu} = 1.4$

Earth and water pressures: persistent ULS design situation
Density and strength of soils – design values

Layer		Depth	Design values				
			γ_{sat}	$\varphi'_{peak,d}$ 1)	$\varphi'_{cv,d}$ 2)	c'_d	$c_{u,d}$
		m	kN/m³	deg	deg	kPa	kPa
I	Made ground	0.0 - 2.4	19.1	24.8	24.8	0	-
II	Low strength clay	2.4 - 6.1	17.2	16.2	16.2	0	17.9
III	Sand and gravel	6.1 - 11.0	20.6	33.9	29.3 3)	0	-
IV	Medium strength clay	11.0 - 16.5	18.6	20.5	16.2 3)	1.6	46.4

Table 12.2. Design values of soil data.

1) Peak value.

2) Constant-volume (i.e. critical state) value.

3) Could take higher value of $\varphi'_{cv,d}$ if design value is selected directly (provided $\varphi'_{cv,d} \leq \varphi'_{peak,d}$).

The design weight density of water is $\gamma_{w,d} = \gamma_{w,k} / \gamma_\gamma = 9.81$ kN/m³.

12.2.3. Vertical stresses and earth pressure calculations

PHB 9, 4.8.

The design vertical total stress (aka "overburden pressure", $\sigma_{v,d}$) at any depth z is calculated by summing the weight of the overlying layers and surcharge, after suitable factoring:

$$\sigma_{v,d} = \int_0^z \gamma_d \, dz + q_d = \sum_j \gamma_{d,j} \; t_j + \gamma_Q \; q_k$$

where

γ_d = design weight density of the ground at depth z;

$\gamma_{d,j}$ = design weight density (assumed constant) in layer j;

t_j = thickness of layer j;

q_d = design surcharge;

q_k = characteristic surcharge at the ground surface;

γ_Q = appropriate partial factor on variable actions.

In Design Approach 1, Combination 2 (which normally governs calculation of wall embedment), the values of the partial factors on actions are

$\gamma_G = 1.0$

$\gamma_Q = 1.3$

Hence the design surcharge $q_d = \gamma_Q \times q_k = 13$ kPa. Also, since $\gamma_\gamma = 1.0$, it can be ignored.

The design pore water pressure u_d at any depth z is calculated by multiplying the weight of water by the distance to the water table:

$$u_d = \gamma_{w,d} \times \left(z - d_w\right)$$

where

$\gamma_{w,d}$ = design weight density of groundwater;

d_w = depth of groundwater table.

The design vertical effective stress σ' at any depth z is calculated using Terzaghi's principle of effective stress as:

$$\sigma'_{v,d} = \sigma_{v,d} - u_d$$

The next step in the worked example is to calculate the vertical effective stress at each soil layer interface down to the tip of the pile which in this case has been selected at -15.0 m.

Earth and water pressures: persistent ULS design situation (DA1-2)
Effective stress analysis - vertical stresses - design values

Layer	Depth	γ_{sat}	t_j	$\gamma_{sat} \times t_j = \Delta\sigma_v$	Vertical total stress σ_v	Pore water pressure u	Vertical effective stress σ'_v
	m	kN/m³	m	kPa	kPa	kPa	kPa
Active side							
	0.0	-	-	-	13.0	0.0	13.0
I	-2.4	19.1	2.4	45.8	58.8	23.5	35.3
II	-6.1	17.2	3.7	63.6	122.5	59.8	62.6
III	-11.0	20.6	4.9	100.9	223.4	107.9	115.5
IV	-15.0	18.6	4.0	74.4	297.8	147.2	150.7
Passive side							
	-6.1	-	-	-	0.0	0.0	0.0
Water	-7.2	9.8	1.1	10.8	10.8	10.8	0.0
III	-11.0	20.6	3.8	78.3	89.1	48.1	41.0
IV	-15.0	18.6	4.0	74.4	163.5	87.3	76.2

Table 12.3. Vertical stresses – Design values - Combination 2.

12.2.4. Earth pressure coefficients

PHB 9, 4.8.2/3

In the limiting equilibrium method of analysis, design horizontal effective stresses are obtained from design vertical effective stresses using active and passive earth pressure coefficients K_a and K_p. The active earth pressure coefficient K_a is a function of the soil's design angle of shearing resistance of the soil φ_d, the design angle of wall friction $\delta_{d,a}$, and the angle of inclination of the ground surface on the active side β_a:

$$K_{a,d} = f\{\varphi_d, \delta_{d,a}, \beta_a\}$$

Likewise, the passive earth pressure coefficient K_p is a function of φ_d, $\delta_{d,p}$, and the inclination of the ground on the passive side β_p:

$$K_{p,d} = f\{\varphi_d, \delta_{d,p}, \beta_p\}$$

Values of the earth pressure coefficients K_a and K_p are given in Annex C of Eurocode 7.

For this worked example inclination on the passive side is zero because beneficial effect of contribution to passive resistance by any soil above -7.2 m is ignored.

Also for sheet pile walls supporting granular soils, Eurocode 7 requires:

$$\delta_d \leq \pm {}^2/_3\ \varphi_{cv,d}$$

where $\varphi_{cv,d}$ = the soil's design angle of shearing resistance under constant-volume conditions (i.e. the "critical state" angle of shearing resistance).

If $\varphi_{cv,d}$ is not known then $\delta_d \approx \pm 1/2\ \varphi_d$.

And also if wall friction is ignored due to effects of pre-augering then $\delta_d = 0$.

Two further earth pressure coefficients applied to the design effective cohesion c_d can be calculated from:

$$K_{ac,d} = 2\sqrt{K_{a,d}\left(1 + \frac{a_d}{c_d}\right)}$$

$$K_{pc,d} = 2\sqrt{K_{p,d}\left(1 + \frac{a_d}{c_d}\right)}$$

where a_d = design adhesion between the ground and the wall.

The UK National Annex to Eurocode 7 - Part 1 limits the values of K_{ac} and K_{pc} to:

$$K_{a,c} \leq 2.56\sqrt{K_a} \text{ and } K_{a,c} \leq 2.56\sqrt{K_p}$$

Earth pressure coefficients may vary slightly according to the method of calculation and therefore may affect the results of an analysis differently. It is recommended to follow the sample procedure to evaluate the coefficients in accordance with EC 7 Annex C after determining the limit of the wall friction for the pile. Values of K_a and K_p may also be obtained from Annex C of EC 7.

The following table gives numerical values for these quantities for the current example.

Earth pressure coefficients for effective stress analysis – design values

Layer	Active conditions				Passive conditions			
	δ/φ_{peak}	K_a	a/c	K_{ac}	δ/φ_{peak}	K_p	a/c	K_{pc}
I	0[1]	0.41	-	-		Above formation		
II	0[1]	0.56	-	-		17.9		
III	0[1]	0.28	-	-	0[1]	3.52	-	-
IV	0.5[2]	0.44	0.5	1.62[3]	-0.5[2]	2.54	0.5	3.90[3]

Table 12.4. Earth pressure coefficients – design values.

- *Ignored because c' = 0.*

1) *Set to zero because of pre-augering to -11.0 m.*

2) *With δ/φ_{cv} = 2/3 and φ_{cv} = 16.2° → δ = 10.8°, hence with φ_{peak} = 20.5° $\delta/\varphi_{peak} \approx 0.5$.*

3) *Calculated from $K_{ac}=2\sqrt{K_a\left(1+\frac{a}{c}\right)}$ and $K_{pc}=2\sqrt{K_p\left(1+\frac{a}{c}\right)}$*

12.2.5. Horizontal stresses on each side of the wall

In the limiting equilibrium method of analysis, the design horizontal effective stress ($\sigma'_{h,d}$) at any depth z is calculated by multiplying the design vertical effective stress by the appropriate earth pressure coefficients.

On the retained side of the wall, where active conditions are assumed, the design active horizontal effective stress ($\sigma'_{a,d}$) is given by:

$$\sigma'_{a,d} = K_{a,d}\ \sigma'_{va,d} - K_{ac,d}\ c_d$$

$$= K_{a,d}\ \sigma'_{va,d} - 2\,c_d\sqrt{K_{a,d}\left(1+\frac{a_d}{c_d}\right)}$$

where

$\sigma'_{va,d}$ and c_d are as defined earlier (the subscript a denotes on the active side);

$K_{a,d}$ = design active earth pressure coefficient for self-weight, based on the design angle of shearing resistance of the soil φ_d and the design angle of wall friction β_d;

$K_{ac,d}$ = design active earth pressure coefficient for effective cohesion;

a_d = design adhesion between the ground and the wall.

On the restraining side of the wall, where passive conditions are assumed, the design passive horizontal effective stress $\sigma'_{p,d}$ is given by:

$$\sigma'_{p,d} = K_{p,d}\ \sigma'_{vp,d} + K_{pc,d}\ c_d$$

$$= K_{p,d}\ \sigma'_{vp,d} + 2\,c_d\sqrt{K_{p,d}\left(1+\frac{a_d}{c_d}\right)}$$

Earth and water pressures: persistent design situation (DA1-2)
Effective stress analysis - horizontal stresses - design values

Layer	Depth	Earth pressure coefficient	Vertical effective stress	Earth pressure coefficient	Cohesion	Horizontal effective stress	Pore water pressure	Horizontal total stress
		K_a ; K_p	σ'_v	K_{ac} ; K_{pc}	c'	σ'_h	u	σ_h
	m		kPa		kPa	kPa	kPa	kPa
Active side								
I	0.0	0.41	13.0	0.00	0.0	5.3	0.0	5.3
	-2.4		35.3			14.5	23.5	38.0
II	-2.4	0.56	35.3	0.00	0.0	19.8	23.5	43.3
	-6.1		62.6			35.1	59.8	94.9
III	-6.1	0.28	62.6	0.00	0.0	17.5	59.8	77.4
	-11.0		115.5			32.3	107.9	140.3
IV	-11.0	0.44	115.5	1.62	1.6	48.2	107.9	156.1
	-15.0		150.7			63.7	147.2	210.9
Passive side								
Water	-6.1	0.00	0.0	0.00	0.0	0.0	0.0	0.0
	-7.2		0.0			0.0	10.8	10.8
III	-7.2	3.52	0.0	0.00	0.0	0.0	10.8	10.8
	-11.0		41.0			144.3	48.1	192.4
IV	-11.0	2.54	41.0	3.90	1.6	110.4	48.1	158.5
	-15.0		76.2			199.7	87.3	287.0

Table 12.5. Horizontal stresses – design values – Combination 2.

12.2.6. Earth pressure diagram

To understand the effects of earth pressure actions it is necessary to understand and analyse the earth pressure diagramme from Table 12.5. where earth pressure force components and lever arms are subsequently used in the calculations to check stability and ultimate load effects.

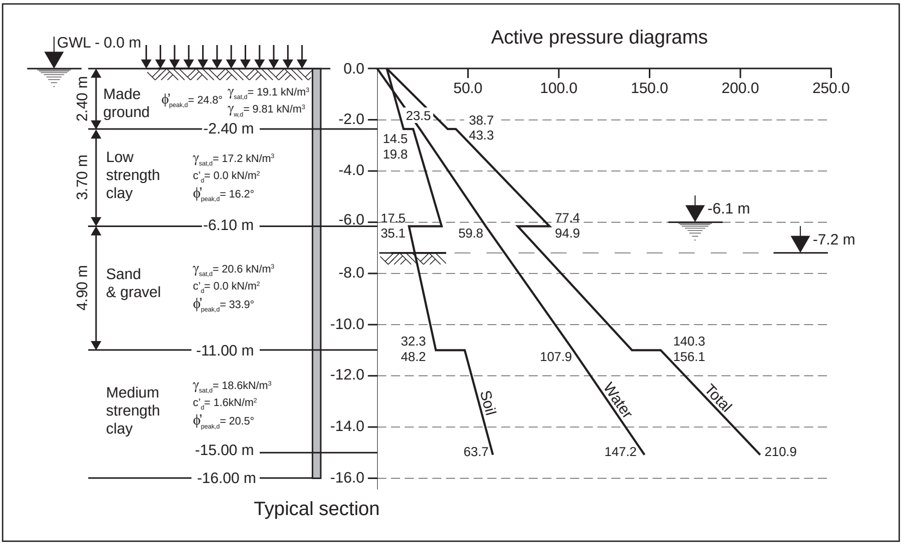


Fig. 12.3. Example of 2 dimensional sketch of active pressure components.

The passive pressures are calculated in the same way to build up a two dimensional diagram as shown in section 12.3.1. The resultant force vectors and lever arms are then calculated to build up the calculation for the stability check described in sections 12.3. and 12.3.1.

12.3. Stability

Check for failure by rotation at the anchor level on the main wall – toe failure: this is the ULS stability calculation for determination of minimum pile length.

12.3.1. Check moment equilibrium by Limit Equilibrium Method (LEM) *PHB 9 – 5.14.2.*

The resistance of the ground against overturning of the wall is calculated by taking moments about the level of the support. The free earth support method of analysis is recommended.

The most convenient way to do this in hand calculations is to divide the total horizontal pressure diagram into triangles or trapeziums and to calculate the overturning moment M_O from:

$$M_O = \sum_j F_{a,j} \; L_{a,j}$$

and the restoring moment M_R from:

$$M_R = \sum_j F_{p,j} \; L_{p,j}$$

Where L_j is the distance of the force F_j to the support level. The subscript a refers to the active side. the subscript p refers to the passive side.

The resultant of the horizontal stresses F_j and the lever arms L_j for each layer are calculated as follows:

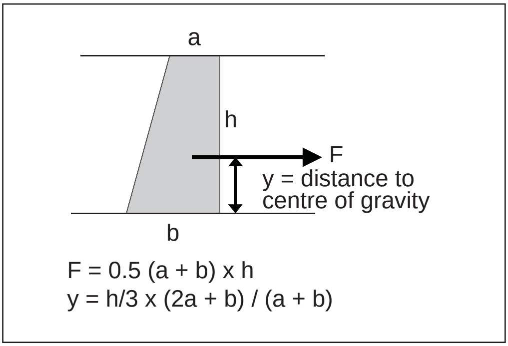


Fig. 12.4. Calculation of horizontal earth forces and lever arm components of a trapezium.

a_j and b_j are the total pressures in layer j at interface levels;

h_j is the thickness of layer j;

F_j is the total horizontal force acting at distance y_j above the lower interface level of layer j.

$$F_j = \frac{a_j + b_j}{2}\, h_j$$

$$y_j = \frac{2a_j + b_j}{a_j + b_j}\, \frac{h_j}{3}$$

This computation is carried out for all the earth pressure resultant forces for each soil layer.

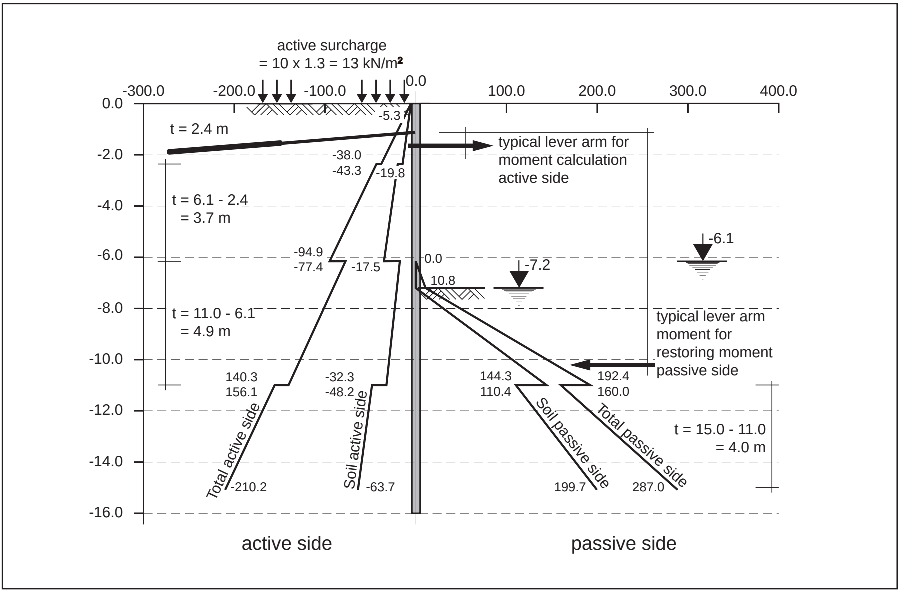


Fig. 12.5. ULS horizontal pressure diagram showing selected values.

Note: All the force vectors and lever arms necessary for calculation of stability are not shown in the diagram for clarity.

To verify the wall is stable. Eurocode 7 requires:

$$\sum M_O \leq \sum M_R$$

but to determine bending moments and shear forces in the wall requires the wall to be in equilibrium. i.e.:

$$\sum M_O = \sum M_R$$

The following table gives numerical values for these quantities for the current example.

Earth and water pressures: persistent design situation (DA1-2)
Check overturning by taking moments about the support level

Layer	Depth	*a*	*b*	*h*	*F*	*y*	*L*	*M*
	m	kN/m²	kN/m²	m	kN/m	m	m	kNm/m
Active side								
I	0.0	5.3	-	2.40	52.0	0.90	0.50	26
	-2.4	-	38.0					
II	-2.4	43.3	-	3.70	255.7	1.62	3.48	890
	-6.1	-	94.9					
III	-6.1	77.4	-	4.90	533.3	2.21	7.79	4154
	-11.0	-	140.3					
IV	-11.0	156.1	-	4.00	732.6	1.90	12.10	8864
	-15.0	-	210.2					
Total					1572			**13934**
Passive side								
Water	-6.1	0.0	-	1.10	5.9	0.37	5.83	35
	-7.2	-	10.8					
III	-7.2	10.8	-	3.80	386.1	1.33	8.67	3346
	-11.0	-	192.4					
IV	-11.0	158.5	-	4.00	890.0	1.81	12.19	10849
	-15.0	-	287.0					
Total					1282			**14230**

Table 12.6. Calculation of moments about the support.

12.3.2. Equilibrium of wall - Length of pile

Comparing the overturning and restoring moments from the table:

$$\sum M_O = 13934 \text{ kN}m/m < 14230 \text{ kN}m/m = \sum M_R$$

Length of pile driven to the chosen toe depth of 15.0 m is acceptable.

Bending moment and shear forces effects in the sheet pile wall can be calculated from a set of earth pressures provided the wall is in moment and horizontal equilibrium under those set of pressures.

The ratio of the overturning and restoring moments from the previous section is:

$$\frac{\sum M_O}{\sum M_R} = \frac{13934}{14230} = 98\%$$

which indicates that the wall is (almost) in equilibrium under the earth pressures presented earlier.

Bending moments and shear forces can be calculated more easily from net earth pressures (i.e. active less passive):

$$\sigma_{h,net} = \sigma_{ha} - \sigma_{hp}$$

The maximum bending moment in the wall occurs where the shear force changes sign.

Earth and water pressures: persistent design situation (DA1-2)
Net earth presures and effects of actions

Layer	Depth	Horizontal total stress			Shear force	Bending moment
		active	passive	net	V_{ed}	M_{ed}
	m	kPa	kPa	kPa	kN/m	kNm/m
I	0.0	5.3	-	5.3	0.0	0.0
	-1.0[1)]	18.9	-	18.9	12.1	-4.8
	-1.0	18.9	-	18.9	-301.0	-4.8
	-2.4	38.0	-	38.0	-261.1	392.2
II	-2.4	43.3	-	43.3	-261.1	392.2
	-6.1	94.9	-	94.9	-5.5	946.8
III	-6.1	77.4	0.0	77.4	-5.5	946.8
	-6.18	78.4	0.8	77.6	0.7	947.1
	-7.2	91.5	10.8	80.7	81.5	906.2
	-11.0	140.3	192.4	52.1	135.9	334.4
IV	-11.0	156.1	158.5	-2.4	135.9	334.4
	-14.7	206.1	277.0	-70.9	0.295	0.0
	-15.0	210.2	287.0	-	-	-

Table 12.7. Shear force and bending moment at indicated depth.

1) Depth of anchor.
Note: Intermediate values have been interpolated from values above and below.

The maximum bending moment effect M_{Ed} that the sheet pile must be designed for is:
M_{Ed} = 947 kN*m/m* at a depth of 6.18 m.

The maximum shear force effect that the sheet pile must be designed for is:

V_{Ed} = 301 kN/*m* at a depth of 1.0 m (i.e. at the level of the anchor).

The anchor load at 1.0 m depth from shear force calculation on Table 12.7. is 301 + 12.1 = 313.1 kN/m.

12.3.3. Design at ULS with an SSI or SGRM software

Check for maximum bending moment. prop / anchor and shear force with SSI or SGRM software to calculate anchor loads in ULS limit state Combinations 1 & 2 – using AMRetain software.

Note this is necessary to check the ultimate anchor load in the normal operation condition followed by checks with the collision on parapet accidental loading.

Note: For the SGRM check it is sometimes necessary to trial a longer pile length than the minimum length calculated in the LEM calculation. In this case checks are carried out for a toe depth of 16.0 m.

If the minimum pile length for equilibrium in "free earth support" is input into a SSI or SGRM calculation the analysis may indicate movement at the toe which may not be desirable. Therefore choosing a slightly longer pile would in some instance develop partial fixity at the toe and minimise movement – this is considered to be a recommended cautious approach.

12.3.3.1. Input for soil parameters. Soil and wall stiffness for SGRM software AM Retain - 16 m long pile

SOIL PROPERTIES	**COMB 1**														
Layer	z	*sat Dens.*	*sub. Dens.*	φ	c	k_0	$k_{a\gamma}$	$k_{p\gamma}$	k_d	k_r	k_{ac}	k_{pc}	k_h	δ_a/φ	δ_p/φ
	m	kN/m³	kN/m³	°	kN/m²								kN/m³		
SLS_MG	0.0	19.1	10.3	30	0	0.500	0.333	3.000	0.500	0.500	0.000	0.000	25000	0.0	0.0
SLS_LOW_STR_CLAY	–2.4	17.2	7.3	20	0	0.658	0.490	2.050	0.658	0.658	0.000	0.000	10000	0.0	0.0
SLS_GRAVEL	–6.1	20.6	10.3	40	0	0.357	0.217	4.600	0.357	0.357	0.000	0.000	50000	0.0	0.0
SLS_MED_STR_CLAY	–11.0	18.6	8.6	25	2	0.577	0.365	3.319	0.577	0.577	1.367	4.764	30000	0.5	–0.5
WALL PROPERTIES															
Section	*EI*														
	kNm²/m														
1 : AZ 26-700N	125559														

Table 12.8. SGRM check Combination 1. Soil properties and coefficients. SLS values.

SOIL PROPERTIES	**COMB 2**														
Layer	z	*sat Dens.*	*sub. Dens.*	φ	c	k_0	$k_{a\gamma}$	$k_{p\gamma}$	k_d	k_r	k_{ac}	k_{pc}	k_h	δ_a/φ	δ_p/φ
	m	kN/m³	kN/m³	°	kN/m²								kN/m³		
ULS_MG	0.0	19.1	10.3	24.8	0.0	0.581	0.409	2.434	0.581	0.581	0.000	0.000	25000	0.0	0.0
ULS_LOW_STR_CLAY	–2.4	17.2	7.3	16.2	0.0	0.721	0.565	1.784	0.721	0.721	0.000	0.000	10000	0.0	0.0
ULS_GRAVEL	–6.1	20.6	10.3	33.9	0.0	0.442	0.285	3.546	0.442	0.442	0.000	0.000	50000	0.0	0.0
ULS_MED_STR_CLAY	–11.0	18.6	8.6	20.5	1.6	0.643	0.432	2.658	0.643	0.643	1.496	4.172	30000	0.5	–0.5
WALL PROPERTIES															
Section	*EI*														
	kNm²/m														
1 : AZ 26-700N	125559														

Table 12.9. SGRM check Combination 2. Soil properties and coefficients. ULS design values.

12.3.3.2. Staged construction sequence for SGRM check using AMRetain

Stage 1 – Drive piles behind existing wall.

Stage 2 – Install anchorage system and anchor ties.

Stage 3 – Demolish existing wall and excavate to bed level at -6.7 m.

Stage 4 – Apply surcharge and design water table loading.

12.3.3.3. Ultimate loading effect. AM Retain results of Combination 1 and Combination 2 - 16 m long pile

For Combination 1 the ultimate load effects are calculated by applying the partial factor 1.35 to the "characteristic" loading effects of the SGRM analysis.

PHB 9. 5.13.1.1.

COMB 1	Characteristic loading effects				Ultimate loading effects			
Phase	Max displacement	Max moment	Max shear	Anchor load	Partial factor	Max moment	Max shear	Anchor load
	mm	kNm/m	kN/m	kN/m		kNm/m	kN/m	kN/m
4	61	406	150	159	1.35	548	202	215

Table 12.10. SGRM analysis - Summary ULS action effects Combination 1.

COMB 2	Ultimate loading effects		
Phase	Max moment	Max shear	Anchor load
	kNm/m	kN/m	kN/m
4	985	311	327

Table 12.11. SGRM analysis - Summary ULS action effects Combination 2.

12.3.4. Calculation of effects of actions for ULS accidental loading limit state

12.3.4.1. Combination 1 Loading example in 12.3.3. is considered but without unplanned excavation

The accidental loading event in this case is considered in normal operational conditions at highest normal ground water level and without unplanned excavation allowance. Therefore design excavation level for this check is -6.7 m.

A parapet accidental collision action characteristic loading of 200 kN / 2.4 m = 83.3 kN/m with a variable nominal surcharge loading of 10 kN/m^2 (equivalent to a 200 kN vehicle acting on an area of 20 m^2) is applied and the partial factor for accidental load is 1.0.

Computing this action effect – for the input – the variable actions are increased by 1.5 / 1.35 = 1.11 and the result effects are increased by 1.35.

Therefore for input:

- surcharge loading = 10 x 1.11 = 11.1 kN/m^2;
- parapet load = 83.3 x 1.00 = 83.3 kN/m.

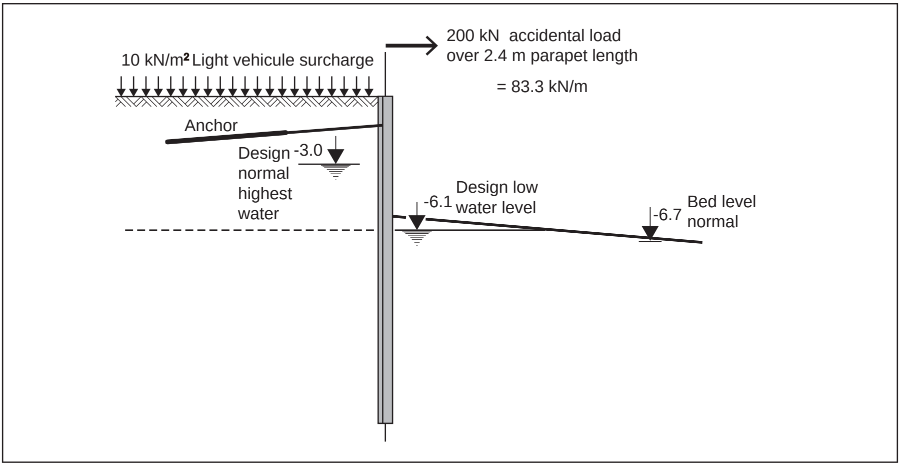


Fig. 12.6. Accidental loading case diagram – Combination 1.

12.3.4.2. Calculation of ultimate accidental loading effects for Combination 1

The characteristic loading effects from the SGRM analysis are factored by 1.35 for the ultimate loading effect values.

Results from SGRM analysis using AMRetain for AZ 26-700N pile 16.0 m toe depth are summarized in the table below.

COMB 1	Characteristic loading effects				Ultimate loading effects			
Phase	Max displacement	Max moment	Max shear	Anchor load	Partial factor	Max moment	Max shear	Anchor load
	mm	kNm/m	kN/m	kN/m		kNm/m	kN/m	kN/m
4	36	303	135	225	1.35	409	182	304

Table 12.12. SGRM analysis - Summary accidental loading action effects Combination 1.

12.3.5. Summary of analysis of ultimate limit state load effects

Three ULS cases have been checked for Design Approach 1: a Combination 2 check with highest possible flood water levels; a Combination 1 check with factored permanent and variable load action effects as well as a variation of Combination 1 with accidental loading from impact loading on a parapet connected to the top of the wall.

Sheet pile toe depth is 15.0 m for the LEM - Combination 2 and 16.0 m for the SGRM analysis.

	ULS Case	Analysis method	Calculation software check	Retained height including unplanned allowance	Toe depth	Hydrostatic head difference	Max moment	Max shear	Max anchor load
				m	m	m	kNm/m	kN/m	kN/m
1	Comb. 1	SGRM	AMRetain	7.2	16.0	3.1	548	202	215
1A	Comb. 1 + Accidental	SGRM	AMRetain	6.7[1)]	16.0	3.1	409	182	304
2	Comb. 2	SGRM	AMRetain	7.2	16.0	6.1	985	311	327
2	Comb. 2	LEM	ReWard 2.7	7.2	15.0	6.1	947	301	313

Table 12.13. Summary ULS effects of actions.

1) Unplanned excavation allowance not included in this accidental collision loading situation.

12.4. Structural verification check for ultimate limit state

The structural design involves the following steps, described in detail in the following paragraphs:

- estimate corrosion rates to be applied;
- assess the effects of corrosion on section properties;
- calculate revised section properties;
- revise the section classification;
- determine the effect of loss of shear transfer in interlocks (for U-type sheet piles);
- determine the impact of water pressure difference (for Z-type sheet piles);
- verify bending resistance;
- verify shear resistance;
- verify resistance to combined bending and shear;
- verify compression resistance.

12.4.1. Section choice and data

Driveability is not an issue in this example because pre-augering is permitted in the dense soils for the scope. A verification procedure follows for suitable Z pile and U pile sections.

The following paragraphs detail the calculations needed in each step of this procedure. The calculations are presented for two different sections: an AZ 26-700N and a PU 32, the properties of which are given by the manufacturer.

Property	Symbol	Units	Sheet pile section	
			AZ 26-700N	PU 32
Overall width	B	mm	700	600
Overall height	H	mm	460	452
Flange thickness	t_f	mm	13.5	19.5
Web thickness	t_w	mm	10.0	11.0
Flange breadth	b_f	mm	371	349
Slant angle	α	°	55.2	68.1
Sectional area	A	cm^2/m	176.4	242.3
Elastic section modulus	W_{el}	cm^3/m	2600	3200
Plastic section modulus	W_{pl}	cm^3/m	3015	3687
Moment of inertia	I	cm^4/m	59790	72320
Mass		kg/m^2	138.5	190.3
Class[1)]	-	-	2	2

Table 12.14. Initial section properties before corrosion. *PHB 9, Ch.1*

[1)] *For steel grades S 240 GP to S 460 AP.*

12.4.2. Section properties after corrosion *EC 3-5, 4.1.*

The design working life of the structure is t_{DWL} = 50 years. The river on the front face of the wall is assumed to be common fresh water.

The position along the sheet pile where the largest design bending moment effect occurs falls within the permanent immersion zone, defined in EC 3 Part 5. A slightly smaller moment occurs in the low water zone immediately above that position. The low water zone is a zone of high attack.

From Table NA1 EC 3 Part 5 National Annex, we obtain the loss of thickness Δ_{tfront} for the front face of the wall as:

Δ_{tfront} = 0.9 mm *EC 3-5, UK NA.1*

assuming the wall is in contact with common fresh water (with high attack) for t_{DWL} = 50 years.

Likewise, we obtain Δ_{tback} for the back face of the wall as:

Δ_{tback} = 0.6 mm

assuming the wall is in contact with undisturbed natural soils.

Hence the total loss of thickness of the sheet pile section is:

$\Delta_t = \Delta_{tfront} + \Delta_{tback} = 1.5$ mm

12.4.3. Revised section properties after corrosion

With Δ_t = 1.5 mm loss of thickness, the properties of the PU 32 and AZ 26-700N sheet piles are reduced as follows:

$H = H_0 - \Delta_t$

$t_f = t_{f,0} - \Delta_t$

$t_w = t_{w,0} - \Delta_t$

where the subscripts *0* denote orginal (ex-factory) properties. The sectional area, elastic section modulus, and plastic section modulus are also reduced.

Property	Symbol	Units	Sheet pile section	
			AZ 26-700N	**PU 32**
Overall width[1)]	B	mm	700[1)]	600[1)]
Overall height	H	mm	458.5	450.5
Flange thickness	t_f	mm	12.0	18.0
Web thickness	t_w	mm	8.5	9.5
Flange breadth[1)]	b_f	mm	371[1)]	349[1)]
Sectional area	A	cm^2/m	155.9	219.6
Elastic section modulus	W_{el}	cm^3/m	2340	2930
Plastic section modulus	W_{pl}	cm^3/m	2690	3355
Moment of inertia	I	cm^4/m	53580	66080
Class[2)]	-	-	2[2)]	2[2)]
Mass		kg/m^2	138.5	190.3

Table 12.15. Reduced section properties.

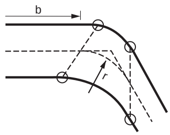
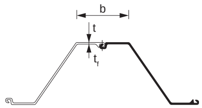
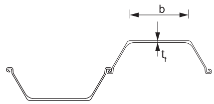
1) Assume property unchanged by loss of thickness.
2) See calculation in next section.

Note: Sheet pile properties and data should be directly provided by the manufacturer. Caution must be exercised when using web site data and information not hosted or provided by the sheet pile manufacturer.

12.4.4. Check section classification

EC 3-5, 5.2.1.

This is required to EC 3 Part 5 section 5 and is shown on Table 5-1 of the standard. The classification of a sheet pile section is potentially modified by loss of thickness. Therefore the section classification must be checked although the section class may be listed in product literature (before corrosion) when new.

Classification	Z profile	U profile
		
Class 1	the same boundaries as for class 2 apply a rotation check has to be carried out	
Class 2	$\frac{b/t_f}{\varepsilon} \leq 45$	$\frac{b/t_f}{\varepsilon} \leq 37$
Class 3	$\frac{b/t_f}{\varepsilon} \leq 66$	$\frac{b/t_f}{\varepsilon} \leq 49$

$\varepsilon = \sqrt{\frac{235}{f_y}}$	f_y (N/mm^2)	240	270	320	355	390	430
	ε	0.99	0.93	0.86	0.81	0.78	0.74

Fig. 12.7. Sheet piling class of a section to EC 3 Part 5.

$$\frac{b_f}{t_f} \leq 37\,\varepsilon$$

where

b_f = the section's flange breadth;

t_f = its flange thickness;

ε = a normalizing parameter for the steel grade, given by

$$\varepsilon = \sqrt{\frac{235}{f_y}}$$

where f_y = the steel's yield strength given in MPa.

For steel grade S 355 GP:

With Δ_t=1.5 mm loss of thickness, the PU 32's flange slenderness ratio is reduced to:

$$\varepsilon = \sqrt{\frac{235}{355}} = 0.814 \text{ and } \frac{b_f}{t_f} \leq 37\varepsilon = 30$$

Thus the PU 32 section remains in class 2.

Z-profiles are Class 2 if:

$$\frac{b_f}{t_f} \leq 45\,\varepsilon$$

For an AZ 26-700N with Δ_t=1.5 mm loss of thickness and for S 430 GP steel:

$$\varepsilon = \sqrt{\frac{235}{430}} = 0.739$$

$$\frac{b_f}{t_f} = \frac{371}{12} = 30.9 \le 45\,\varepsilon = 45 \times 0.739 = 33$$

Hence, AZ 26-700N in S 430 GP remains in class 2 at end of design life.

12.4.5. Effect of loss of shear transfer on interlocks *EC 3-5, 5.2.2. (2)*

The design bending resistance of a sheet pile section may be reduced by loss of shear transfer in the interlock. The factor β_B is required to be introduced into the calculation of bending resistance to take this phenomenon into account. Values for β_B are given in the National Annex Table NA-2 of EC 3-5.

The number of structural support levels for this design example is one (anchor at -1.0 m).

Because the sheet piles may be installed with pre-augering to -11 m (passing through coarse soil – sand and gravel – below formation level at -7.2 m), the ground conditions should be classified as unfavourable.

The piles will be installed (backdriven) as uncrimped doubles and the interlocks will not be treated with sealants.

For the PU 32 sheet pile section, the value of β_B for uncrimped piles in unfavourable conditions is:

β_B = 0.6 + 0.05 (for untreated interlocks) = 0.65

For the AZ 26-700N section, as for all Z-piles:

β_B = 1.0 (i.e. no loss of shear transfer occurs).

12.4.6. Verification of bending resistance at point of maximum moment *EC 3-5, 5.2.2.*

The bending resistance is verified if:

$$M_{Ed} \le M_{c,Rd}$$

where

M_{Ed} = design bending moment effect calculated in the geotechnical design check;

$M_{c,Rd}$ = design moment resistance of the cross-section.

From Table 12.3.5., the highest maximum bending moment for the verification check

M_{Ed} = 985 kNm/m

The design bending resistance $M_{c,Rd}$ of a sheet pile section is: *EC 3-5, 5.2.2. (2)*

$$M_{c,Rd} = \frac{\beta_B\, W f_y}{\gamma_{M0}}$$

where

W is the appropriate section modulus (plastic or elastic, depending on the section's classification);

f_y is the yield strength of steel;

γ_{M0} is a partial factor whose value is 1.0 (in the UK National Annex).

For the Class 2 PU 32 sheet pile section in steel grade S 355 GP:

$$M_{c,Rd} = \frac{0.65 \times 3355 \times 355}{1.0} \times 10^{-3} = 774 \text{ kNm/m}$$

and so the bending resistance is inadequate, since:

$$\frac{M_{Ed}}{M_{c,Rd}} = \frac{985}{774} = 127\%$$

Two changes can make this section work. First, the steel grade is increased to S 430 GP (thereby improving the moment resistance by 21%) and the sections will be welded at the top after installation (before excavation) to improve the shear transfer through the interlocks. In this case, β_B increases by +0.15, improving the moment resistance by 25%.

The section classification must be rechecked, based on the new steel grade:

$$\frac{b_f}{t_f} = \frac{349}{18} = 19.4 \leq 27.3 = 37 \times \sqrt{\frac{235}{430}} = 37\varepsilon$$

so it is remains Class 2.

The combined result is:

$$M_{c,Rd} = \frac{0.8 \times 3355 \times 430}{1.0} \times 10^{-3} = 1154 \text{ kNm/m}$$

and so the bending resistance is now adequate, since:

$$\frac{M_{Ed}}{M_{c,Rd}} = \frac{985}{1154} = 85\%$$

For the AZ 26-700N sheet pile section in Grade S 430 GP steel, using W_{pl}:

$$M_{c,Rd} = \frac{1.0 \times 2690 \times 430}{1.0} \times 10^{-3} = 1157 > 985 \text{ kNm/m} = M_{Ed}$$

and so the bending resistance is adequate (Note: steel grade S 390 GP would be sufficient).

12.4.7. Local effects of water pressure *EC 3-5, 5.2.4.*

Transverse local plate bending can occur when a sheet pile wall retains water at different levels on its opposing sides (resulting in differential water pressures across the wall).

The differential head in this worked example is:

$\Delta h_w = d_{w,p} - d_{w,a} = 6.1 - 0.0 = 6.1$ m

As discussed in EC 3 - Part 5 5.2.4 (1) this effect may be neglected if the difference in water level Δh_w across the wall is:

$\Delta h_w \leq 5$ m

for Z-profiles with uncrimped or unwelded interlocks or

$\Delta h_w \leq 20$ m

for U-profiles.

Hence transverse local plate bending is not relevant to the design of the PU 32 section, but is for the AZ 26-700N, for which a reduced yield strength of steel must be used, given by:

$f_{y,red} = \rho_P \, f_y$

where the reduction factor ρ_P depends on the value of Δh_w and the parameter:

$$\frac{b_f}{t_{min}} \varepsilon = \frac{b_f}{t_{min}} \sqrt{\frac{235}{f_y}}$$

where

b_f = the section's flange breadth;

t_{min} = the smaller of its web and flange thicknesses.

For the AZ 26-700N in S 430 GP

t_{min} = min (t_f, t_w) = min (12.0, 8.5) = 8.5 mm

and so the value of the parameter is:

$$\frac{b_f}{t_{min}} \varepsilon = \frac{371}{8.5} \times 0.739 = 32.3$$

The value of the reduction factor can be obtained from the EC 3 - Part 5 Table 5.2 and is $\rho_P = 0.99$ in this instance (i.e. a reduction of less than 1%, which may be neglected for practical purposes). *EC 3-5 Table 5.2.*

However ρ_P may be assumed = 1.0 if the Z-pile interlocks are welded.

12.4.8. Check on shear resistance

The shear resistance is verified if: *EC 3-5, 5.2.2. (4)*

$V_{Ed} \leq V_{pl,Rd}$

where

V_{Ed} = the design shear force effect calculated in the geotechnical design check (Comb 2):

V_{Ed} = 311 kN/m

The design plastic shear resistance of each web is given by:

$$V_{pl,Rd} = \frac{A_V f_y}{\sqrt{3}\,\gamma_{M0}} = \frac{t_w (h-t_f) f_y}{\sqrt{3}\,\gamma_{M0}}$$

and hence, per metre run of wall is:

$$V_{pl,Rd} = \frac{V_{pl,Rd}}{B} = \frac{t_w\ (h-t_f) f_y}{B\sqrt{3}\,\gamma_{M0}}$$

where B is the width of a single pile.

For the PU 32 sheet pile section in steel grade S 430 GP:

$$V_{pl,Rd} = \frac{V_{pl,Rd}}{B} = \frac{9.5 \times (450.5 - 18.0) \times 430}{600 \times \sqrt{3} \times 1.0} = 1700 \text{ kN/m}$$

and hence the shear resistance is adequate, since:

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{311}{1700} = 18\%$$

For the AZ 26-700N sheet pile section in steel grade S 430 GP:

$$V_{pl,Rd} = \frac{8.5 \times (458.5 - 12.0) \times 430}{700 \times \sqrt{3} \times 1.0} = 1346 \text{ kN/m}$$

and hence the shear resistance is adequate:

$$\frac{V_{Ed}}{V_{pl,Rd}} = \frac{311}{1346} = 23\%$$

12.4.9. Combined bending and shear at point of maximum shear

The maximum shear force in the wall (other than at the level of the anchor) occurs at a depth of 9.5 m, where, from the LEM calculation and Table 12.7. it can be demonstrated that

V_{Ed} = 175 kN/m, M_{Ed} = 576 kNm/m

However the effects of shear on bending resistance may be neglected if:

EC 3-5, 5.2.2. (8)

$$V_{Ed} \leq \frac{V_{pl,Rd}}{2}$$

For the PU 32 sheet pile section in steel grade S 430 GP:

$$V_{Ed} = 175 \text{ kN/m} \leq 850 \text{ kN/m} = \frac{1700}{2} = \frac{V_{pl,Rd}}{2}$$

and so no explicit check is needed for combined bending and shear.

For the AZ 26-700N sheet pile section in steel grade S 430 GP:

$$V_{Ed} = 175 \text{ kN/m} \leq 673 \text{ kN/m} = \frac{1346}{2} = \frac{V_{pl,Rd}}{2}$$

and so no explicit check is needed for combined bending and shear.

12.4.9.1. Other verifications

Although not shown here, the verifications of shear buckling resistance; combined bending and axial compression; combined bending, shear, and axial compression; web buckling; and flange tension – for both the PU 32 and AZ 26-700N sections – all pass comfortably although normally all these checks should also be verified in accordance with EC 3 - Part 5 after taking into account loss of thickness of the section at the critical level of relevant exposure.

12.4.9.2. Section recommendation

Ground may be pre-augered so vibrodriving is anticipated to -11.0 m level.

For 16 m long doubles Table 11.4. of Piling Handbook 9th edition recommends for 2-5 m penetration into soil of c_u = 51-75 kPa that normal driving is anticipated so no difficulty expected by driving sections in this range.

However the AZ 26-700N would offer a weight saving of around 37% and fewer piles to drive, therefore this section would be recommended.

The pile length recommended is 16.0 m. The AMRetain programme calculates an ultimate earth resistance ratio at the toe of the pile of 1.104 which is 10% greater than at free earth support equilibrium - see comment in section 12.3.3.

Main wall	AZ 26-700N	driven as crimped or uncripmed pairs
Steel grade	S 430 GP	
Pile length	16.0	m
Head level	0.0	m
Toe level	-16.0	m
Anchor tie level	-1.0	m

Part 2. Design of the anchorage system

12.5. Design model for tie bars connected to a cantilever anchor wall

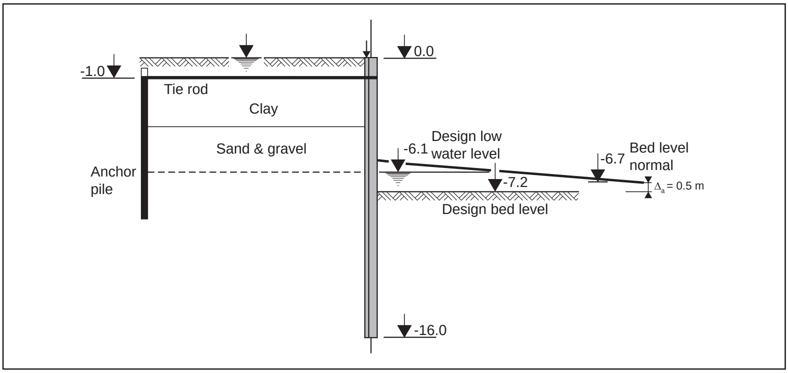


Fig. 12.8. Cross section with anchor wall.

12.5.1. Anchor pile length

The anchor pile needs to be embedded in the sand and gravel layer and the tie level at the anchor pile fixed preferably above the normal water table level, which is -3.0 m.

The anchor pile needs sufficient embedment to resist downward settlement and provide sufficient passive resistance to prevent forward movement by rotation and sliding.

12.5.1.1. Anchor pile design assumptions

Following assumptions are made:

- ultimate anchor load from tie bar derived from case Combination 2 (submerged ground either side of anchor pile);
- retained side in flood, so no surcharge acting;
- passive resistance benefit above tie bar level ignored.

12.5.1.2. Earth pressure coefficients

Earth pressure coefficients are different for the anchor wall because pre-augering is not anticipated or required and the piles are to have sufficient penetration into Layer III to resist downward settlement.

Earth and water pressures: persistent ULS design situation (DA1-2)
Earth pressure coefficients for effective stress analysis – design values for anchor wall

Layer	Active conditions				Passive conditions			
	δ/φ	K_a	a/c	K_{ac}	δ/φ	K_p	a/c	K_{pc}
I	0.5	0.369	[1]	–	-0.5	3.287	[1]	–
II	0.5	0.521	[1]	–	-0.5	2.089	[1]	–
III	0.5	0.254	[1]	–	-0.5	5.790	[1]	–
IV	0.5	0.440	0.5	1.62[3]	-0.5	2.540	0.5	3.90[2]

Table 12.16. Earth pressure coefficients – Design values.

[1] *Ignored because c' = 0.*

[2] *Calculated from* $K_{ac} = 2\sqrt{K_a\left(1+\frac{a}{c}\right)}$ and $K_{pc} = 2\sqrt{K_p\left(1+\frac{a}{c}\right)}$.

12.5.1.3. Calculation of vertical stresses for anchor pile

Earth and water pressures: persistent design situation (DA1-2)
Effective stress analysis - vertical stresses - design values

Layer	Depth	Saturated density	Layer thickness	γ_{sat} x t_j =	Vertical total stress	Pore water pressure	Vertical effective stress
		γ_{sat}	t_j	$\Delta\sigma_v$	σ_v	u	σ'_v
	m	kN/m³	m	kPa	kPa	kPa	kPa
Active & passive side: same values for this anchor pile situation							
-	0.0	–	–	–	0.0[1]	0.0	0.0
I	-2.4	19.1	2.4	45.8	45.8	23.5	22.3
II	-6.1	17.2	3.7	63.6	109.5	59.8	49.6
III	-11.0	20.6	4.9	100.9	210.4	107.9	102.5
IV	-15.0	18.6	4.0	74.4	284.8	147.2	137.7

Table 12.17. Vertical stresses – Design values - Combination 2

[1] *No surcharge load assumed.*

12.5.1.4. Calculation of horizontal stresses on anchor pile

Earth and water pressures: persistent ULS design situation (DA1-2)
Effective stress analysis - horizontal stresses - design values

Layer	Depth	Earth pressure coefficient	Vertical effective stress	Earth pressure coefficient	Cohesion	Horizontal effective stress	Pore water pressure	Horizontal total stress
		K_a ; K_p	σ'_v	K_{ac} ; K_{pc}	c'	σ'_h	u	σ_h
	m		kPa		kPa	kPa	kPa	kPa
Active side								
I	0.0	0.369	0.0	0.00	0.0	0.0	0.0	0.0
	-2.4		22.3			8.2	23.5	31.7
II	-2.4	0.521	22.3	0.00	0.0	11.6	23.5	35.2
	-6.1		49.6			25.9	59.8	85.7
III	-6.1	0.254	49.6	0.00	0.0	12.6	59.8	72.4
	-11.0		102.5			26.0	107.9	133.9
IV	-11.0	0.440	102.5	1.62	1.6	42.5	107.9	150.4
	-15.0		137.7			58.0	147.2	205.1
Passive side								
I	0.0	3.287	0.0	0.00	0.0	0.0	0.0	0.0
	-2.4		22.3			73.3	23.5	96.8
II	-2.4	2.089	22.3	0.00	0.0	46.6	23.5	70.1
	-6.1		49.6			103.7	59.8	163.5
III	-6.1	5.790	49.6	0.00	0.0	287.4	59.8	347.3
	-11.0		102.5			593.5	107.9	701.4
IV	-11.0	2.540	102.5	3.90	1.6	266.6	107.9	374.5
	-15.0		137.7			355.9	147.2	503.1

Table 12.18. Horizontal stresses – Design values – Combination 2.

12.5.1.5. Net passive horizontal total pressure on anchor pile

Earth and water pressures: persistent design situation (DA1-2)
Effective stress analysis - net horizontal pressures

Layer	Depth	Horizontal total pressure active side	Horizontal total pressure passive side	Horizontal total net pressure
		σ_{ha}	σ_{hp}	σ_{hnp}
	m	kPa	kPa	kPa
I	0.0	0.0	0.0	0.0
	-2.4	31.7	96.8	65.1
II	-2.4	35.2	70.1	35.0
	-6.1	85.7	163.5	77.8
III	-6.1	72.4	347.3	274.8
	-11.0	133.9	701.4	567.5
IV	-11.0	150.4	374.5	224.1
	-15.0	205.1	503.1	297.9

Table 12.19. Net passive horizontal total pressure at indicated depth.

12.5.1.6. Ultimate actions on anchor pile at limit equilibrium

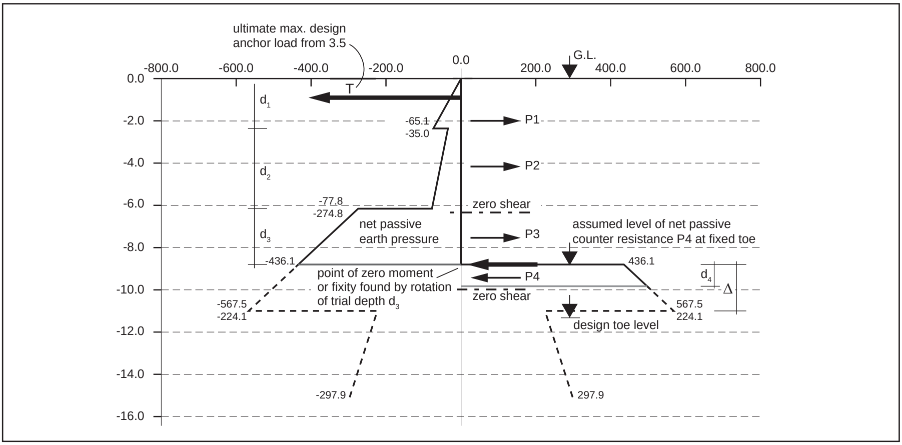


Fig. 12.9. Anchor pile net passive pressure diagram.

12.5.1.7. Calculation steps to check anchor pile length

1) Calculate horizontal net passive forces.
2) Find point of zero moment at trial depth d_3 where moment of anchor force T does not exceed moment of net passive resistance assuming pile rotates about point 0.

   Note: this is a recognized method of approximation for positioning an equivalent point of net passive resistance P_4 for cantilever walls.

3) Calculate theoretical depth d_4 to lowest point of zero shear where $P_4 = T - (P_1 + P_2 + P_3)$.

4) Minimum design toe level of pile = $d_1 + d_2 + d_3 + \Delta$
where Δ = 1.2 x d_4 to allow for approximations in [ii].

Step 2): from anchor pile net pressure diagram, try d_3 = 2.7 m.

Net horizontal pressure at d_3 = 274.8 + d_3 x (567.5 − 274.8) / (11.0 - 6.1) = 274.8 + 2.7 x 292.7 / 4.9 = 436.1 kN/m^2.

Check overturning by taking moments about point "0" level - ref anchor pile net pressures
Effective stress analysis - horizontal stresses - design values

Layer	Depth	*a*	*b*	*h*	*F*	*y*	*L*	*M*
	m	kN/m^2	kN/m^2	m	kN/m	m	m	kNm/m
					Soil			
I	0.0	0.0	-	2.40	78.1	0.80	7.20	562
	-2.4	-	65.1					
II	-2.4	35.0	-	3.70	208.7	1.62	4.32	901
	-6.1	-	77.8					
III	-6.1	274.8		2.70	959.7	1.25	1.25	1198
	-8.8	-	436.1					
Total								**2661**
					Anchor			
	-1.0				327.0		7.80	**2551**

Table 12.20. Calculation of moments about the support.

At 8.8 m depth overturning moment / restoring moment
= 2661 / 2551 = 1.04 > 1.0

Step 3): for fixed earth support net passive resistance force P_4 required to fix the toe of the pile

$P_4 = P_3 + P_2 + P_1 - T$ = 960 + 209 + 78 - 327 = 920 kN/m

To find depth d_4 to second point of zero shear
$P_4 = d_4$ x (436.1 + (436.1 + 59.7 x d_4)) / 2

Try d_4 = 1.88 m.

P_4 = 1.9 x (436.1 + (436.1 + 1.9 x 59.7)) / 2 = 925 kN/m > 920 kN/m

Step 4): calculate minimum depth for toe fixity required

Δ = 1.2 x d_4 = 1.2 x 1.9 = 2.28 m.

Minimum pile depth of toe required
= $d_1 + d_2 + d_3 + \Delta$ = 2.4 + 3.7 + 2.7 + 2.28 = 11.1 m.

12.5.2. Anchor pile bending moment calculation

The maximum bending moment is likely to occur at the first point of zero shear for the cantilever fixed anchor pile. There is also a lower point of zero shear and the bending moment should be checked in this position also or any position where soil may be aggressive for the long term section verification.

To find the first point of zero shear from pressure diagram:

d'_3 = depth of net passive resistance P_3' in layer III below 6.1 m

$T - (P_1 + P_2) = 327 - (78.1 + 208.7) = 40.2$ kN/m

$d'_3 \approx 40.2 / 274.8 \approx 0.15$ m

Try $d'_3 = 0.14$ m.

Net pressure at -6.24 m: 274.8 + (0.14 x (567.5 – 274.8) / 4.9) = 283.2 kN/m²

$P_3' = 0.14 \times (283.2 + 274.8) / 2 = 39.1 \approx 40.2$ kN/m

Zero shear is at depth 2.4 + 3.7 + 0.14 = 6.24 m.

Taking moments about point of zero shear at depth -6.24 m:

Check moments about zero shear point level (-6.24 m)
ref anchor pile net pressure diagram, see Fig. 12.9.

Layer	Depth	*a*	*b*	*h*	*F*	*y*	*L*	*M*
	m	kN/m²	kN/m²	m	kN/m	m	m	kNm/m
				Soil				
I	0.0	0.0		2.40	78.1	0.80	4.64	362
	-2.4		65.1					
II	-2.4	35.0		3.70	208.7	1.62	1.76	366
	-6.1		77.8					
III	-6.1	274.8		0.14	39.1	0.07	0.07	3
	-6.24		283.2					
Total								**731**
				Anchor				
	-1.0				327.0		5.24	**1713**

Table 12.21. Calculation of moments about zero shear point.

→ Maximum ultimate design bending moment on anchor pile
$M_{Ed} = 1713 - 731 = 982$ kNm/m.

12.5.3. Anchor pile section selection

For a design life of 50 years and considering the section thickness loss on each side is 0.6 mm, then the total thickness loss on the section is 1.2 mm.

From sections 12.4.3. - 12.4.6., AZ 26-700N in S 430 GP at 1.5 mm thickness loss has a bending moment capacity of 1157 kNm/m.

Therefore AZ 26-700N by inspection is adequate for the anchor pile:

$M_{c,Rd} = 1157 > 982 = M_{Ed}$ (values in kNm/m)

12.5.4. Location of anchorage

Note: For initial sizing and depth for the location calculation assume a cantilever anchorage depth of embedment of a minimum 2/3 length of pile below normal balanced anchorage failure plane. Balanced anchorage calculations are demonstrated in previous editions of the Piling Handbook. However once the location of the anchorage is established, checks of the global stability are carried out (e.g Kranz method) to check sliding failure of the system. If the stability is not satisfied then the anchor wall may require positioning further back or deeper, and the process re-checked.

PHB 9, 7.3.

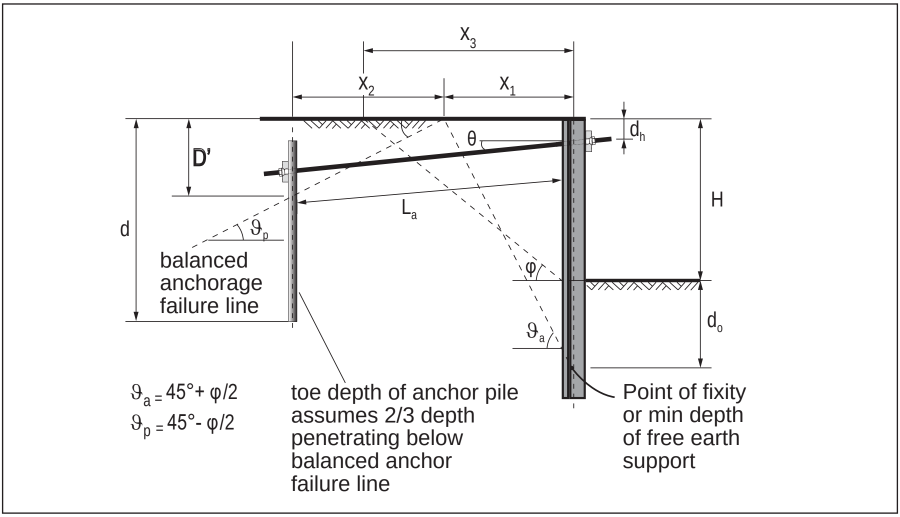


Fig. 12.10. Location of anchorage (cantilever anchor pile only).

Checking the location of the anchorage or minimum distance behind the main wall can be simply checked using the following empirical method from LEM analysis with the following provisions:

1) Soil strata between the sheet pile walls is assumed to have the value φ_d,mean for the check to ensure the soil is not intereacting between the two walls within the planes of rupture. FE analysis needs to be carried out if this is not the case.
2) System may require further checks for global stability at lower failure plane e.g. by Bishops method or sliding failure (outside the scope of this example). This entails locating the lowest point of zero shear in the anchor pile and checking the failure plane against sliding using Kranz's method (see section 7.4). This method is explained in detail in EAU 2012 (Chapter 8.5.).

From section 12.3.2., length of pile for free earth support = 15.0 m, H = 7.2 m, d_0 = 15.0 - 7.2 = 7.8 m.

Allow $\varphi_{d,\,mean} \approx 25°$ and d = depth = 11.1 m

$$x_1 = (H + _{d_0})\ \tan\left(45° - \frac{\varphi_{d,mean}}{2}\right) = 9.55\ m$$

Assuming $D' = d / 3$ = 11.1 / 3 = 3.7 m

$$x_2 = D'\ cot\left(45° - \frac{\varphi_{d,mean}}{2}\right) = 3.7 \times 1.57 = 5.81\ m$$

Minimum distance between main wall and anchor wall = 9.55 + 5.81 = 15.4 m.

12.5.5. Section recommendation

Anchor wall	AZ 26-700N	in crimped pairs[1)]
Steel grade	S 430 GP	
Pile length	10.6	m
Head level	-0.5	m
Toe level	-11.1	m
Anchor tie level	-1.0	m

[1)] *Crimping recommended, but not required by the design.*

12.6. Design of tie bars and fittings

1) Ultimate load is taken from main wall design analysis of ULS action effects (see section 12.3.5.). Tie bar assumed to be horizontal for the purposes of this calculation. Position of the tie bar fixing at anchorage is as close to depth -1.0 m as possible.
2) Temperature change effects are not considered to be significant as temperate climate prevails and system is largely buried.
3) $f_{y\ spec\ max}$ = 500 N/mm². *EC 3-5, 3.7.*
4) Tie bar design includes check against progressive failure of ties under ULS Accidental design situation in normal operating conditions.
5) Tie bars may be upset forged end type for benefit of durability at exposed end of tie through main sheet pile wall.
6) Tie bars fixed with a turnbuckle. No pre-stressing. Tie bars to have no articulated couplings at main wall so bending in thread to be allowed for in design i.e. k_t =0.6. Tie bars at anchorage may have spherical washer for articulation and facilitation for fixing small angular deviation. i.e. k_t = 0.9.

EC 3 - Part 5 UK NA

12.6.1. Option 1 - No waling - Design assumptions

1) Ties to be spaced at double AZ 26-700N centres i.e.1.4 m. No permanent waling ties to be directly connected to double crimped AZ sheets with bearing plate on outside exposed face of sheet piles.

12.6.1.1. Design ultimate tie bar load from Table 12.13.

12.6.1.1.1. Maximum ULS load from Case 2

Maximum tie load is 327 kN/m (from SGRM analysis), so no need to apply further model factor, and no inclination component to adjust as horizontal and normal angles assumed in relation to the sheet pile wall and anchorage.

For ties at 1.4 m centres, max ULS tie load from Case 2 :

F_{ed} = 327 x 1.4 = 458 kN.

12.6.1.1.2. Check case to prevent progressive failure of tie bars if one tie fails

Check ULS Case in normal operating conditions for maximum tie load for load to be taken by adjacent ties to a failed tie – (note the failure of a tie may not necessarily occur in the Accidental load case 1A). Also Case 2 does not apply to normal operating conditions because the event of highest possible permanent hydrostatic loading is not normal operational conditions. Therefore Case 1 applies.

Maximum tie load is 215 kN/m.

For ties at 1.4 m centres ULS max tie load in normal operating conditions from Table 12.13. Case 1:

F_{ed} = 215 x 1.4 = 301 kN

Load from 3 ties is taken by the 2 adjacent ties if one tie fails. In this case, max ULS tie load is:

F_{ed} = 3 x 301 / 2 = 451 kN < 458 kN

12.6.1.1.3. Maximum tie load

Tie bars to be designed for ULS maximum load F_{ed} = 458 kN.

12.6.1.2. Verification of the tie bar

12.6.1.2.1. Verification of the threaded end of tie bar at main wall

Ultimate tensile resistance of tie at threaded end at the sheet pile:

$$F_{tt,Rd} = \frac{k_t\, f_{ua}\, A_s}{\gamma_{M2}} \text{ with } \gamma_{M2} = 1.25$$

EC 3-5, 7.2.3.(2)

For tie bars without articulated joints and subject to possible bending at the thread due to either settlement of the anchorage system, fill or tie bar sagging the notch factor k_t is recommended in the UK National Annex: k_t = 0.6. *EC 3-5 UK NA*

Try upset ties with thread diameter M64, f_y = 500 N/mm^2, f_{ua} = 660 N/mm^2.

Allowing 50 years corrosion in the zone of high attack where end of ties may be exposed outside the sheet piles: loss of thickness = 3.75 mm per face for exposure to tidal water in zone of high attack. *EC 3-5 UK NA, Table 4.2.*

Total loss of diameter = 2 x 3.75 mm = 7.5 mm.

For M64 upset thread nominal thread area before corrosion A_s = 2676 mm². ASDO manufacturer, 2013

To find effective thread diameter:

$$d'^2 = \frac{4 \times 2676}{\pi} = 3407\,\text{mm}^2$$

→ d' = 58.4 mm

Therefore nominal effective diameter of upset end after corrosion d'_{red} = 58.4 - 7.5 = 50.9 mm (simplified approach).

Effective thread area after corrosion:

$$A_s = \frac{\pi \times 50.9^2}{4} = 2035\,\text{mm}^2$$

$$\boldsymbol{F_{tt,Rd}} = \frac{0.6 \times 660 \times 2035}{1.25} \times 10^{-3} = \mathbf{644\ kN} > 458\ \text{kN}$$

Utilisation factor $= \dfrac{458}{644} = 71\%$: 29% additional capacity in ULS.

12.6.1.2.2. Verification of the shaft of tie bar

$$F_{tg,Rd} = \frac{f_y A_g}{\gamma_{M0}} \text{ with } \gamma_{M0} = 1.00$$

EC 3-5, 7.2.3. (3)

Try M48 shaft. Where shaft is completely buried in natural fill surround at 50 years (assume surround is non compacted non aggressive sand), corrosion loss = 1.2 mm loss after 50 years, therefore diameter loss = 2.4 mm.

Effective diameter d'_{red} = 48 - 2.4 = 45.6 mm.

$$\rightarrow A_g = \frac{\pi \times 45.6^2}{4} = 1633\,\text{mm}^2$$

$$\boldsymbol{F_{tg,Rd}} = \frac{500 \times 1633}{1.00} \times 10^{-3} = \mathbf{816\ kN} > 458\ \text{kN}$$

12.6.1.2.3. Verification of the threaded end at anchor pile

Try same diameter as shaft: M48.

Effective stress area = 1473 mm².

Effective diameter $d'^2 = \dfrac{4 \times 1473}{\pi} = 1875\ \text{mm}^2$

→ d' = 43.3 mm

After 50 years corrosion in natural fill: d'_{red} = 43.3 - 2.4 = 40.9 mm.

Effective area after corrosion allowance $A_s = \dfrac{\pi \times 40.9^2}{4} = 1313\,\text{mm}^2$

$$F_{tt,Rd} = \frac{0.6 \times 660 \times 1313}{1.25} \times 10^{-3} = 416\ \text{kN} < 458\ \text{kN}$$

Add spherical washer at waling to allow rotation at threaded end → $k_t = 0.9$:

$$\boldsymbol{F_{tt,Rd}} = \frac{0.9 \times 660 \times 1313}{1.25} \times 10^{-3} = \mathbf{624\ kN} > 458\ \text{kN}$$

EC 3-5, 7.2.3. (4)

12.6.1.2.4. Final choice of tierods

M64 threaded upset ends ties at main wall connection and M48 shaft in 500 N/mm^2 yield strength with spherical washer at anchor wall.

12.6.1.3. Serviceabilty check

Serviceability checks and information and recommendations for consideration of elongation and settlement of the tie bar may require specialist advice from the manufacturer and is outside the scope of this example.

12.6.1.4. Option 1 - No waling - Summary detail

EC 3-5, Fig 7-5

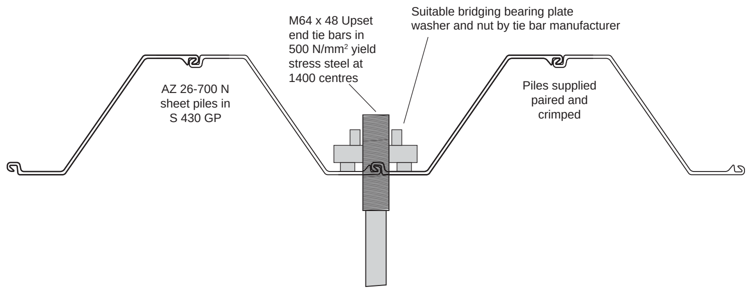


Fig. 12.11. Typical fixing detail - no waling.

Note: No allowance has been made in the calculations for installation stresses in the tie bars if prestressing or tension has been exerted on the ties if the sheet pile wall is adjusted in line or straightened by using the ties before final backfilling behind the wall. In this example there is no backfilling stage because the piles are driven behind an existing wall but nevertheless a cautious approach may allow a small percentage additional tensile load allowance for taking up slack and tightening to the anchorage system. By positioning the ties in this way it is possible to tighten the tie bars after filling behind the wall.

12.6.2. Option 2 - With waling - Design assumptions

1) Ties to be spaced at two pair AZ 26-700N centres i.e.2.8 m. Permanent waling behind both main wall and anchor pile wall;
2) Waling to be designed for SLS. Deformation to be considered for accidental loss of a tie and the adjacent ties designed to resist progressive failure.

12.6.2.1. Design ultimate tie bar load from Table 12.13.

12.6.2.1.1. Maximum ULS load from Case 2

Maximum tie load from Case 2 is 327 kN/m (from SGRM analysis). For ties at 2.8 m centres, maximum ULS tie load F_{ed} = 327 x 2.8 = 916 kN.

12.6.2.1.2. Check case to prevent progressive failure of tie bars if one tie fails

Check ULS case in normal operating conditions for maximum tie load for load to be taken by adjacent ties to a failed tie.

Load in normal operating conditions from Table 12.13. Case 1 is 215 kN/m.

For ties at 2.8 m centres ULS maximum tie load is:

F_{ed} = 215 x 2.8 = 602 kN

Load from 3 ties is taken by the 2 adjacent ties if one tie fails. In this case, maximum ULS tie load is:

F_{ed} = 3 x 602 / 2 = 903 kN < 916 kN

12.6.2.1.3. Maximum tie load

Tie bars to be designed for ULS maximum load F_{ed} = 916 kN.

12.6.2.2. Verification of the tie bar

12.6.2.2.1. Verification of threaded end of tie bar at main wall

Ultimate tensile resistance of tie at threaded end at sheet pile:

$$F_{tt,Rd} = \frac{k_t \, f_{ua} \, A_s}{\gamma_{M2}} \text{ with } \gamma_{M2} = 1.25$$ *EC 3-5, 7.2.3.(2)*

For tie bars without articulated joints and subject to possible bending at the thread due to either settlement of the anchorage system, fill or tie bar sagging the notch factor k_t is recommended in the UK National Annex: k_t = 0.6. *EC 3-5, UK NA*

Try upset ties with thread diameter M76, f_y = 500 N/mm², f_{ua} = 660 N/mm².

Allowing 50 years corrosion in the zone of high attack where end of ties may be exposed outside the sheet piles: loss of thickness = 3.75 mm per face for exposure to tidal water in zone of high attack. *EC 3-5, UK NA, Table 4.2.*

Total loss of diameter = 2 x 3.75 mm = 7.5 mm.

For M76 upset thread nominal thread area before corrosion = 3889 mm².

ASDO manufacturer, 2013

To find effective thread diameter: $d'^2 = \frac{4 \times 3889}{\pi} = 4952 \text{ mm}^2$

→ d' = 70.3 mm

Therefore nominal effective diameter of upset end after corrosion d'_{red} = 70.3 - 7.5 = 62.9 mm (simplified approach).

Effective thread area after corrosion:

$$A_s = \frac{\pi \times 62.9^2}{4} = 3107\,\text{mm}^2$$

$$\boldsymbol{F_{tt,Rd}} = \frac{0.6 \times 660 \times 3107}{1.25} \times 10^{-3} = \mathbf{984\ kN} > 916\ \text{kN}$$

Utilisation factor = $\frac{916}{984} = 93\%$: 7% additional capacity in ULS.

12.6.2.2.2. Verification of the shaft of tie bar

$$F_{tg,Rd} = \frac{f_y\, A_g}{\gamma_{M0}} \text{ with } \gamma_{M0} = 1.00$$

EC 3-5, 7.2.3. (3)

Try diameter Ø 60 shaft. Where shaft is completely buried in natural fill surround at 50 years (assume surround is non compacted non aggressive sand), corrosion rate = 1.2 mm loss after 50 years, therefore diameter loss = 2.4 mm.

Effective diameter d'_{red} = 60.0 - 2.4 = 57.6 mm.

$$A_g = \frac{\pi \times 57.6^2}{4} = 2606\,\text{mm}^2$$

$$\boldsymbol{F_{tg,Rd}} = \frac{500 \times 2606}{1.00} \times 10^{-3} = \mathbf{1303\ kN} > 916\ \text{kN}$$

12.6.2.2.3. Verification of the threaded end at anchor pile

Note: For Option 2 a spherical washer fitting is designed for the detail of the connection of the end of the anchorage to a waling behind the anchor wall. This enables a small degree of rotation of the tie at the anchor wall to eliminate risks of bending stresses transferred to the threaded part of the tie caused by settlement of the anchor system or filling materials.By using spherical head washers at the rear anchor fitting the notch factor k_t may be increased to 0.9.

Try same diameter as shaft: Ø 60

Effective stress area = 2362 mm^2.

Effective diameter $d'^2 = \frac{4 \times 2362}{\pi} = 3007\ \text{mm}^2$

→ d' = 54.8 mm

After 50 years corrosion in natural fill: d'_{red} = 54.8 - 2.4 = 52.4 mm.

Effective area after corrosion allowance

$$A_s = \frac{\pi \times 52.4^2}{4} = 2156\,\text{mm}^2$$

$$\boldsymbol{F_{tt,Rd}} = \frac{0.9 \times 660 \times 2156}{1.25} \times 10^{-3} = \mathbf{1024\ kN} > 916\ \text{kN}$$

12.6.2.2.4. Final choice of tierods

M76 threaded upset ends ties at main wall connection and Ø 60 shaft in 500 N/mm^2 yield strength with spherical washer at anchor wall.

12.6.2.3. Option 2 - Fixing details with walings

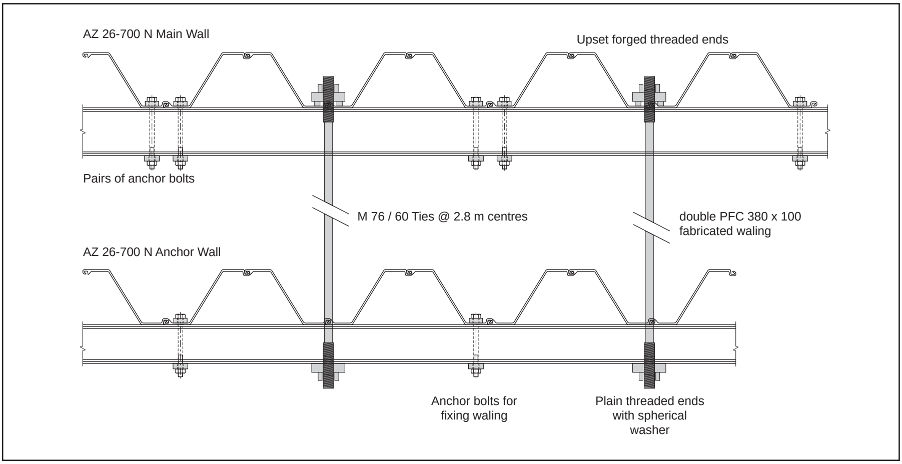


Fig. 12.12. Typical details with front and back walings.

12.6.3. Preliminary sizing of walings

Without a structural reinfocrced concrete (RC) capping beam it may be necessary to size the waling to limit deflections in the Serviceability Limit State in the Accidental design situation when one tie has failed. Because the ties have been detailed to anchor the piles directly by bearing to the outside flanges of the sheet piles the waling is not required to be designed to prevent the progressive collapse of the wall. If the ties were connected to the waling directly then the waling may be required to be designed to spread the load over twice the span, if a tie failed, to prevent progressive collapse. In this case the waling fittings and waling bolts would be subject to combined stresses due to large movements of the waling if a tie bar fails. This is a more complex structural verification procedure and outside the scope of the Piling Handbook. Where there is no RC capping beam detailed it is recommended to close the centres of the ties and fix the ties to the outside of the sheets directly as shown in this example.

12.6.3.1. Main wall waling

First of all the waling should be connected in continuous form using splicing plates which can be either bolted or welded. There are details in manufacturers technical brochures.

12.6.3.1.1. SLS verification, with one tie failing

For the SLS check on waling strength the bending moment is calculated from Case 1. The design tie force per running meter from Case 1 is 215 kN/m.

Hence, the characteristic tie force per running meter from Case 1 is:

p = 215 / 1.35 = 159 kN/m

The maximum SLS bending moment in the waling is:

$$M = \frac{pl^2}{10}$$

$$\rightarrow M = \frac{159 \times (2.8 \times 2)^2}{10} = 498 \text{ kNm}$$

For walings in S 430 steel grade, with f_y = 430 MPa, the section modulus required at yield is:

$$W_y = \frac{M}{f_y} = \frac{498}{430} \times 10^{-3} = 1158 \text{ cm}^3$$

Select PFC 380 x 100 x twin channel fabricated walings (108 kg/m nominal):

$W_y = 2 \times 791 = 1582 \text{ cm}^3$

Utilisation factor

$$= \frac{1158}{1582} = 73\%$$

Flange thickness t_f = 17.5 mm.

Loss of thickness due to corrosion in buried sand fill after 50 years
= 2 x 1.2 = 2.4 mm.

→ loss of strength ≈ 2.4 / 17.5 = 14 % << 26 %

Note: Assuming that loss of strength ≈ proportional to initial flange thickness. This is an empirical sizing! The reduced section modulus should be calculated allowing for corrosion and the section verified!

12.6.3.1.2. ULS verification

The maximum ULS bending moment from Case 2 with a tie load of 327 kN/m is:

$$M = \frac{327 \times 2.8^2}{10} = 256 \text{ kNm} \ll 495 \text{ kNm}$$

The verification of bending strength after corrosion considers 1.2 mm corrosion loss from each face of the channel sections.

From Manufacturer's data:
PFC 380 x 100, with W_{el} = 791 cm^3 and W_{pl} = 933 cm^3.

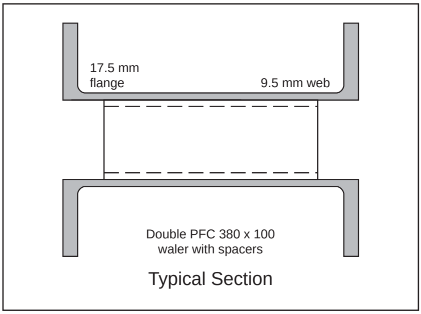


Fig. 12.13. Typical section on waling with spacer.

For twin channel waling as new
W_{el} = 2 x 791 = 1582 cm^3 and W_{pl} = 2 x 933 = 1866 cm^3.

After corrosion allowance check by CAD software for 2.4 mm thickness loss (1.2 mm on each face).

Single channel: $I_{y,red}$ = 12530 cm^4.

Double channel: $I_{y,red}$ = 2 x 12530 = 25060 cm^4.

$$W_{el,y,red} = \frac{25060}{37.67/2} = 1327 \text{ cm}^3$$

and

$W_{pl,y,red}$ = 2 x 769 = 1538 cm^3 (double channel, determined with CAD software).

Check channel section classification on compression flange:

EC 3-1, Table 5.2.

channel sections are Class 1 if $\frac{c}{t_f} \leq 9\,\varepsilon$

For steel grade S 355: 9 ε = 9 x 0.814 = 7.29

and

$$\frac{c}{t_f} = \frac{100 - 2.4}{17.5 - 2.4} = \frac{97.6}{15.1} = 6.46 < 7.29$$

Therefore channel section is Class 2 at least and it is allowed to use the plastic section modulus for bending resistance verification.

Bending resistance capacity for a double channel in class 1 or 2:

$$M_{c,Rd} = \frac{W_{pl}\ f_y}{\gamma_{M0}} \text{ with } \gamma_{M0} = 1.00$$

EC 3-1, 6.13.

$$M_{c,Rd} = \frac{1538 \times 355}{1.0} \times 10^{-3} = 546 \text{ kNm} > M \text{ (> 495 kNm and > 256 kNm)}$$

Note: The bending capacity after corrosion is higher than the ULS case for Option 2 and sufficient to resist the loss of a tie in the normal operational case.

Therefore twin PFC 380 x 100 (S 355) fabricated walings should be OK taking into account corrosion.

Note: Walings may be coated for protection and maintenance if required.

However the SLS deflection should also be checked for acceptability using the corroded section properties. The connection plates and fittings should also be verified if required taking into account combined forces, or the connections placed at positions of minimal moment and if not simple support with free ends may be assumed to verify the section after corrosion is taken into account. All the necessary structural checks are outside the scope of this example.

12.6.3.2. Anchor wall waling

Provided that the exposure conditions are the same then the rear anchor waling can be selected based on the main wall waling design with the arrangement as shown in 12.6.2.3.

It may be possible to accommodate a lighter waling if a SLS check is carried out taking into account support of the sheet piles and stiffness of soil in front of the anchor wall in the design situation if one tie fails. However it may not be worthwhile to carry out these design checks for a small weight saving in the steel waling.

12.6.3.3. Waling bolts

Waling bolts are necessary to connect the waling to the sheet piles.

The wall should be straightened first if necessary using the heavy ties and anchorage system rather than trying to use the bolts and waling. Packing plates are needed to accommodate any gaps between the waling and piles before anchor bolts are tightened up.

Waling bolts are designed and verified in a similar way to the anchor ties and similar design rules apply but the designer should take into account the worst exposure risk for the thread and shaft verification. Note it may be possible for the shaft under the bolt head to be exposed in a splash zone through any gaps in bolt holes and voids where water and oxygen may penetrate if fill is not compacted well surrounding the waling.

It is strongly recommended that anchor bolts are robustly designed. In former piling manuals an additional 25% load was recommended to be added to the design load.

From Table 12.11. ULS design anchor bolt load is 327 kN/m.

The anchor bolts are required to resist a total load between the ties at 2.8 m centres:

F_{ed} = 327 x 2.8 = 916 kN

Using 2 anchor bolts and a model factor of 1.25 for “tightening”

F_{ed} = 916 / 2 x 1.25 = 572 kN

12.6.3.3.1. Verification of the thread

Try M52 anchor bolts in steel grade S 355, with
f_y = 355 N/mm^2 and f_{ua} = 510 N/mm^2.
Effective stress area A_s = 1758 mm^2.

Effective diameter $d'^2 = \dfrac{4 \times 1758}{\pi} = 2237 \text{ mm}^2$

→ d' = 47.3 mm

After 50 years corrosion in natural fill: d'_{red} = 47.3 - 2.4 = 44.9 mm.

Effective area after corrosion allowance $A_s = \dfrac{\pi \times 44.9^2}{4} = 1583 \text{ mm}^2$

Assuming no bending in threads → k_t = 0.9

$$\boldsymbol{F_{tt,Rd}} = \frac{0.9 \times 510 \times 1583}{1.25} \times 10^{-3} = \mathbf{581\ kN} > 572 \text{ kN}$$

12.6.3.3.2. Verification of the shaft

$$F_{tg,Rd} = \frac{f_y\ A_g}{\gamma_{M0}} \text{ with } \gamma_{M0} = 1.00$$

Try M52 shaft, where shaft is subject to exposure to water and oxygen in void, allow 2 x 3.75 mm = 7.5 mm (Note: this is a very cautious approach).

Effective diameter d'_{red} = 52.0 - 7.5 = 44.5 mm.

$$A_g = \frac{\pi \times 44.5^2}{4} = 1555 \text{ mm}^2$$

$$\boldsymbol{F_{tg,Rd}} = \frac{355 \times 1555}{1.00} \times 10^{-3} = \mathbf{552\ kN} \approx 572 \text{ kN}$$

→ considering that 25% additional allowance for tightening has been taken into account. Alternatively protection could be considered rather than increasing steel grade or diameter further for anchor bolt.

12.6.3.3.3. Final choice of bolts

M52 anchor bolts in steel grade S 355 with forged heads sized in length to suit waling width and piling tolerances.

Note: Bolts may be coated with the same treatment as waling.

Note:
Any preliminary design for steel foundations, steel sheet piling structures, bearing piles, anchorage systems and accessories, as well as recommendations for pile installation (“designs”), provided by ArcelorMittal, or any of its subsidiaries or

group companies (collectively “ArcelorMittal”) are indicative only. ArcelorMittal does not warrant or guarantee that the designs will be error free, and will not be liable for any loss or damage arising from their use. Users of the designs do so at their sole discretion and risk and should satisfy themselves and verify that the information and recommendations provided are correct.

13 | Useful information

Chapter 13 - Useful Information

Contents

13.1. Discontinued U piles

The tables of values in this chapter apply to U piles when interlocked together to form a wall.

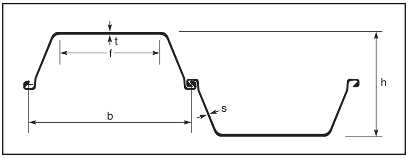


Fig. 13.1. Geometry for former U pile sections.

13.1.1. ArcelorMittal sections

Section	Width	Height	Thickness		Flat of pan	Sectional area	Mass		Moment of inertia	Elastic section modulus	Plastic section modulus
	b mm	*h* mm	*t* mm	*s* mm	*f* mm	cm^2/m	single pile kg/m	wall kg/m^2	cm^4/m	cm^3/m	cm^3/m
PU 6	600	226	7.5	6.4	335	97	45.6	**76.0**	6780	**600**	697
PU 7	600	226	8.5	7.1	335	106	49.9	**83.1**	7570	**670**	779
PU 8	600	280	8.0	8.0	318	116	54.5	**90.9**	11620	**830**	983
PU 9	600	280	9.0	8.7	318	125	58.8	**98.0**	12830	**915**	1083
PU 11	600	360	8.8	8.4	258	131	61.8	**103.0**	19760	**1095**	1336
PU 16	600	380	12.0	9.0	302	159	74.7	**124.0**	30400	**1600**	1878
PU 20	600	430	12.4	10.0	307	179	84.3	**140.5**	43000	**2000**	2363
PU 25	600	452	14.2	10.0	339	199	93.6	**156.0**	56490	**2500**	2899
L2S	500	340	12.3	9.0	275	177	69.7	**139.4**	27200	**1600**	1871
L3S	500	400	14.1	10.0	232	201	78.9	**157.8**	40010	**2000**	2389
L4S	500	440	15.5	10.0	244	219	86.2	**172.0**	55010	**2500**	2956
L5S	500	450	20.6	11.5	230	270	106.0	**212.0**	72000	**3200**	3783
JSP2	400	200	10.5	-	280	153	48	**120.0**	8740	**874**	971
JSP3	400	250	13.0	-	270	191	60.0	**150.0**	16800	**1340**	1487
JSP4	400	170	15.5	-	246	242	76.1	**190.0**	38600	**2270**	2618
GU12-500	500	340	9.0	8.5	262	144	56.6	**113.2**	19640	**1155**	1390
GU13-500	500	340	10.0	9.0	262	155	60.8	**121.7**	21390	**1260**	1515
GU15-500	500	340	12.0	10.0	262	177	69.3	**138.6**	24810	**1460**	1755
PU6R	600	280	6.0	6.0	323	90	42.2	**70.0**	8940	**640**	750
PU7R	600	280	6.5	6.3	323	94	44.3	**74.0**	9580	**685**	800
PU8R	600	280	7.5	6.9	323	103	48.7	**81.0**	10830	**775**	905
PU9R	600	360	7.0	6.4	296	105	49.5	**82.0**	16930	**940**	1115
PU10R	600	360	8.0	7.0	296	114	53.8	**90.0**	18960	**1055**	1245
PU11R	600	360	9.0	7.6	296	123	58.1	**97.0**	20960	**1165**	1370
PU13R	675	400	10.0	7.4	300	124	65.6	**97.0**	25690	**1285**	1515
PU14R	675	400	11.0	8.0	300	133	70.5	**104.0**	28000	**1400**	1655
PU15R	675	400	12.0	8.6	300	142	75.4	**112.0**	30290	**1515**	1790

Table 13.1. Dimensions and properties of discontinued ArcelorMittal U piles.

13.1.2. Corus sections

Section	Width	Height	Thickness		Flat of pan	Sectional area	Mass		Moment of inertia	Elastic section modulus	Plastic section modulus
	b	*h*	*t*	*s*	*f*		single pile	wall			
	mm	mm	mm	mm	mm	cm^2/m	kg/m	kg/m^2	cm^4/m	cm^3/m	cm^3/m
LX 8	600	310	8.2	8.0	250	116	54.6	**91.0**	12863	**830**	1017
LX 12	600	310	9.7	8.2	386	136	63.9	**106.5**	18727	**1208**	1381
LX 12 d	600	310	10.0	8.3	386	139	65.3	**108.8**	19217	**1240**	1417
LX 12 d 10	600	310	10.0	10.0	382	155	72.9	**121.5**	19866	**1282**	1493
LX 16	600	380	10.5	9.0	365	157	74.1	**123.5**	31184	**1641**	1899
LX 20	600	430	12.5	9.0	330	177	83.2	**138.7**	43484	**2023**	2357
LX 20 d	600	450	11.2	9.7	330	179	84.3	**140.5**	45197	**2009**	2380
LX 25	600	460	13.5	10.0	351	202	95.0	**158.3**	57656	**2507**	2914
LX 25 d	600	450	15.0	11.0	326	212	100.0	**166.7**	57246	**2544**	2984
LX 32	600	460	19.0	11.0	340	243	114.4	**190.7**	73802	**3209**	3703
LX 38	600	460	22.5	14.5	337	298	140.4	**234.0**	87511	**3805**	4460
GSP 2	400	200	10.5	8.6	266	157	49.4	**123.5**	8756	**876**	1020
GSP 3	400	250	13.5	8.6	270	191	60.1	**150.3**	16316	**1305**	1520
GSP 4	400	340	15.5	9.7	259	242	76.1	**190.3**	38742	**2279**	2652
6 (42)	500	450	20.5	14.0	329	339	133.0	**266.0**	94755	**4211**	4933
6 (122)	420	440	22.0	14.0	250	371	122.5	**291.7**	92115	**4187**	4996
6 (131)	420	440	25.4	14.0	250	396	130.7	**311.2**	101598	**4618**	5481
6 (138.7)	420	440	28.6	14.0	251	419	138.3	**329.3**	110109	**5005**	5924

Table 13.2. Dimensions and properties of discontinued Corus U piles.

Section	Width	Height	Thickness		Flat of pan	Sectional area	Mass		Combinea inertia	Elastic section modulus
	b mm	*h* mm	*t* mm	*s* mm	*f* mm	cm^2/m	single pile kg/m	wall kg/m^2	cm^4/m	cm^3/m
6W	525	212	7.8	6.4	333	109	44.8	**85.3**	6508	**711**
9W	525	260	8.9	6.4	343	124	51.0	**97.1**	11726	**902**
12W	525	306	9.0	8.5	343	147	60.4	**115.1**	18345	**1199**
16W	525	348	10.5	8.6	341	166	68.3	**130.1**	27857	**1601**
20W	525	400	11.3	9.2	333	188	77.3	**147.2**	40180	**2009**
25W	525	454	12.1	10.5	317	213	87.9	**167.4**	56727	**2499**
32W	525	454	17.0	10.5	317	252	103.6	**197.4**	70003	**3216**
1U	400	130	9.4	9.4	302	135	42.4	**106.0**	3184	**489**
2	400	200	10.2	7.8	270	156	48.8	**122.0**	8494	**850**
2B	400	270	8.6	7.1	248	149	46.7	**116.8**	13663	**1013**
2N	400	270	9.4	7.1	248	156	48.8	**122.0**	14855	**1101**
3	400	247	14.0	8.9	248	198	62.0	**155.0**	16839	**1360**
3B	400	298	13.5	8.9	235	198	62.1	**155.2**	23910	**1602**
3/20	508	343	11.7	8.4	330	175	69.6	**137.0**	28554	**1665**
4A	400	381	15.7	9.4	219	236	74.0	**185.1**	45160	**2371**
4B	420	343	15.5	10.9	257	256	84.5	**200.8**	39165	**2285**
4/20	508	381	14.3	9.4	321	207	82.5	**162.4**	43167	**2266**
4/20	508	381	15.7	9.4	321	218	86.8	**170.9**	45924	**2414**
5	420	343	22.1	11.9	257	303	100.0	**237.7**	50777	**2962**
10B/20	508	171	12.7	12.7	273	167	66.4	**130.7**	6054	**706**

Table 13.3. Dimensions and properties of discontinued British Steel U piles.

13.2. Discontinued Z piles

13.2.1. ArcelorMittal sections

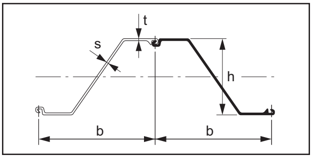


Fig. 13.2. Geometry for former Z-pile sections by ArcelorMittal.

Section	Width	Height	Thickness		Flat of pan	Sectional area	Mass		Moment of inertia	Elastic section modulus	Plastic section modulus
	b mm	*h* mm	*t* mm	*s* mm	*f* mm	cm²/m	single pile kg/m	wall kg/m²	cm⁴/m	cm³/m	cm³/m
AZ 12	670	302	8.5	8.5	360	126	66.1	**99**	18140	**1200**	1409
AZ 13	670	303	9.5	9.5	360	137	72.0	**107**	19700	**1300**	1528
AZ 14	670	304	10.5	10.5	360	149	78.3	**117**	21300	**1400**	1651
AZ 13-10/10	670	304	10.0	10.0	360	143	75.2	**112**	20480	**1350**	1589
AZ 17	630	379	8.5	8.5	348	138	68.4	**109**	31580	**1665**	1945
AZ 25	630	426	12.0	11.2	347	185	91.5	**145**	52250	**2455**	2875
AZ 28	630	428	14.0	13.2	347	211	85.0	**170**	45570	**2600**	3250
AZ 34	630	459	17.0	13.0	378	234	115.5	**183**	78700	**3430**	3980
AZ 36	630	460	18.0	14.0	378	247	122.2	**194**	82800	**3600**	4196
AZ 38	630	461	19.0	15.0	378	261	129.1	**205**	87080	**3780**	4417
AZ 36-700	700	499	17.0	11.2	427	216	118.5	**169**	89740	**3600**	4111
AZ 38-700	700	500	18.0	12.2	427	230	126.2	**180**	94840	**3800**	4353
AZ 40-700	700	501	19.0	13.2	427	244	133.8	**191**	99930	**4000**	4596
AZ 37-700	700	499	17.0	12.2	426	226	124.2	**177**	92400	**3705**	4260
AZ 39-700	700	500	18.0	13.2	426	240	131.9	**188**	97500	**3900**	4500
AZ 41-700	700	501	19.0	14.2	426	254	139.5	**199**	102610	**4095**	4745

Table 13.4. Dimensions and properties of discontinued ArcelorMittal Z-piles .

13.2.2. Corus sections

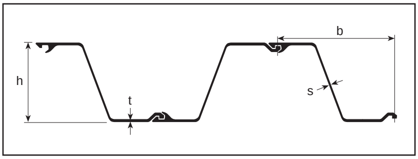


Fig. 13.3. Geometry for former Z-pile sections by Corus.

Section	Width	Height	Flange	Web	Mass		Elastic section modulus
	b mm	*h* mm	*t* mm	*s* mm	single pile kg/m	wall kg/m²	cm³/m
1 BXN	476	143	12.7	12.7	63.4	**133.2**	**692**
1 N	483	170	9.0	9.0	48.0	**99.4**	**714**
2 N	483	235	9.7	8.4	54.8	**113.5**	**1161**
3 NA	483	305	9.7	9.5	62.7	**129.8**	**1687**
4 N	483	330	14.0	10.4	82.7	**171.2**	**2415**
5	426	311	17.1	11.9	101.0	**237.1**	**3171**
1A	400	146	6.9	6.9	35.6	**89.1**	**563**
1B	400	133	9.5	9.5	42.1	**105.3**	**562**
2	400	185	8.1	7.6	47.2	**118.0**	**996**
3	400	229	10.7	10.2	61.5	**153.8**	**1538**
4	400	273	14.0	11.4	80.1	**200.1**	**2352**

Table 13.5. Dimensions and properties of discontinued Corus Z-piles .

13.3. The metric system

Linear measure

1 inch	= 25.4 mm	1 mm	= 0.03937 inch
1 foot	= 0.3048 m	1 cm	= 0.3937 inch
1 yard	= 0.9144 m	1 m	= 3.2808 feet or 1.0936 yds
1 mile	= 1.6093 km	1 km	= 0.6214 mile

Square measure

1 sq inch	= 645.16 mm^2	1 cm^2	= 0.155 sq in
1 sq foot	= 0.0929 m^2	1 m^2	= 10.763 sq ft or 1.196 sq yds
1 sq yard	= 0.8361 m^2	1 hectare	= 2.4711 acres
1 acre	= 0.4047 hectare		
1 sq mile	= 259 hectares	1 km^2	= 247.105 acres
1 hectare	= 10,000 m^2		

Cubic measurement

1 cubic inch	= 16.387 cm^3	1 mm^3	= 0.000061 cubic in
1 cubic foot	= 0.0283 m^3	1 m^3	= 35.3147 cubic ft or 1.308 cubic yds
1 cubic yard	= 0.7646 m^3		

Measure of capacity

1 pint	= 0.568 litre	1 litre	= 1.7598 pints or 0.22 gallon
1 gallon	= 4.546 litres		

Weight

1 oz	= 0.0284 kg	1 g	= 0.0353 oz
1 pound	= 0.4536 kg	1 kg	= 2.2046 lb
1 ton	= 1.016 tonnes or 1016 kg	1 tonne	= 0.9842 ton

Section modulus and inertia

1 $inch^3$	= 16.387 cm^3	1 cm^3	= 0.0610 $inch^3$
1 $inch^3$/foot	= 53.76 cm^3/m	1 cm^3/m	= 0.0186 $inch^3$/foot
1 $inch^4$	= 41.62 cm^4	1 cm^4	= 0.0240 $inch^4$
1 $inch^4$/foot	= 136.56 cm^4/m	1 cm^4/m	= 0.0073 $inch^4$/foot

13.4. Miscellaneous conversion factors and constants

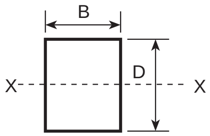
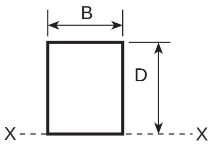
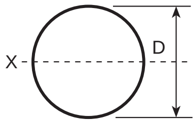
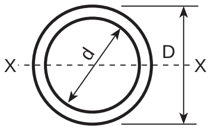
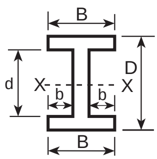
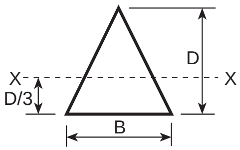
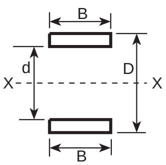
Linear measure

1 lb (f)	= 4.449 N
1 pound per linear foot	= 1.4881 kg per linear m
1 pound per square foot	= 4.883 kg per m^2
0.205 pound per square foot	= 1kg per m^2
1 ton (f) per linear foot	= 32.69 kN per linear m
1000 pound (f) per square foot	= 47.882 kN per m^2
1 ton (f) per square inch	= 15.444 N per mm^2
1 ton (f) per square foot	= 107.25 kN per m^2
100 pound per cubic foot	= 1602 kg per m^3
100 pound (f) per cubic foot	= 15.7 kN per m^3
1 ton (f) foot Bending Moment per foot of wall	= 10 kNm Bending Moment per metre of wall
1m head of fresh water	= 1 kg per cm^2
1m head of sea water	= 1.025 kg per cm^2
$1m^3$ of fresh water	= 1000 kg
$1m^3$ of sea water	= 1025 kg
1 radian	= 57.3 degrees
Young's Modulus, steel	= 210 kN/mm^2
Weight of steel	= 7850 kg/m^3
100 microns	= 0.1 mm = 0.004 inch

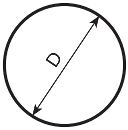
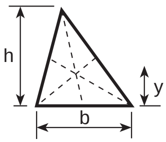
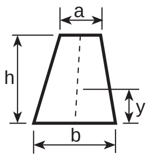
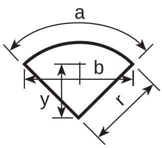
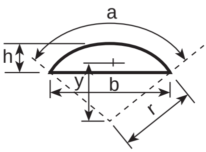
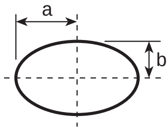
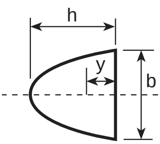
13.5. Bending moments in beams

Type	Total Load W bending moment	Maximum	Deflection
Cantilever	Concentrated at end	WL	$\frac{WL^3}{3\,EI}$
	Uniformly distributed	$\frac{WL}{2}$	$\frac{WL^3}{8\,EI}$
Freely supported	Concentrated at centre	$\frac{WL}{4}$	$\frac{WL^3}{48\,EI}$
	Uniformly distributed	$\frac{WL}{8}$	$\frac{5\,WL^3}{384\,EI}$
	Varying uniformly from zero at one end to a maximum at other end	$0.128\,WL$	$0.0131\,\frac{WL^3}{EI}$
One end fixed, other end freely supported	Concentrated at centre	$\frac{3WL}{16}$	$0.00932\,\frac{WL^3}{EI}$
	Uniformly distributed	$\frac{WL}{8}$	$0.0054\,\frac{WL^3}{EI}$
Both ends fixed	Concentrated at centre	$\frac{WL}{8}$	$\frac{WL^3}{192\,EI}$
	Uniformly distributed	$\frac{WL}{12}$	$\frac{WL^3}{384\,EI}$

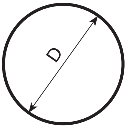
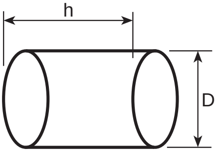
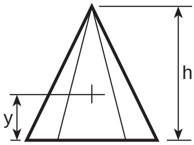
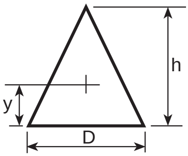
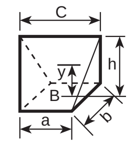
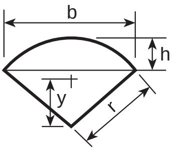
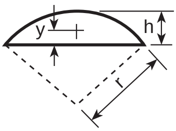
13.6. Properties of shapes

Section	Moment of inertia I_{xx}	Section modulus Z_{xx}	Radius of gyration r_{xx}
	$\frac{BD^3}{12}$	$\frac{BD^2}{6}$	$\frac{D}{\sqrt{12}}$
	$\frac{BD^3}{3}$		
	$\frac{\pi D^4}{64}$	$\frac{\pi D^3}{32}$	$\frac{D}{4}$
	$\frac{\pi (D^4-d^4)}{64}$	$\frac{\pi (D^4-d^4)}{32D}$	$\sqrt{\frac{D^2+d^2}{16}}$
	$\frac{BD^3-2bd^3}{12}$	$\frac{BD^3-2bd^3}{6D}$	$\sqrt{\frac{BD^3-2bd^3}{12(BD-2bd)}}$
	$\frac{BD^3}{36}$	$\frac{BD^2}{24}$	$\frac{D}{\sqrt{18}}$
	$\frac{B(D^3-d^3)}{12}$	$\frac{B(D^3-d^3)}{6D}$	$\sqrt{\frac{D^3-d^3}{12(D-d)}}$

13.7. Mensuration of plane surfaces

Figure	Descripton	Area	Distance "y" to centre of gravity
	Circle	$\frac{\pi D^2}{4}$	at centre
	Triangle	1/2 bh	$\frac{h}{3}$ at intersection of median lines
	Trapezoid or parallelogram	1/2 (a+b) h	$\frac{h\,(2a+b)}{3\,(a+b)}$
	Circular arc	–	$\frac{br}{a}$
	Circular sector	1/2 ar	$\frac{2br}{3a}$
	Circular segment	$\frac{ar}{2} - \frac{b}{2}\,(r-h)$	$\frac{b^3}{12 \times \text{area}}$
	Ellipse	π ab	at centre
	Parabolic segment	$\frac{2}{3}\,bh$	$\frac{2}{5}\,bh$

13.8. Mensuration of solids

Figure	Descripton	Surface area A and volume V	Distance "y" to centre of gravity
	Sphere	$A = \pi D^2$ $V = \pi/6\ D^3$	at centre
	Cylinder	Curved surface $A = \pi Dh$ $V = \pi/4\ D^2h$	at centre
 A = Base Area	Pyramid	$V = \frac{1}{3} Ah$	$\frac{h}{4}$ above base
	Cone	Curved surface $A = \frac{\pi}{4} D\sqrt{(4h^2 + D^2)}$ $V = \frac{\pi}{12} D^2h$	$\frac{h}{4}$ above base
 B is C. of G. of base	Wedge	$V = \frac{bh}{6}(2a+c)$	$\frac{h\,(a+c)}{2\,(2a+c)}$
	Spherical sector	Total surface $A = \pi r \times (2h+1/2b)$ $V = 2\frac{\pi}{3} \times r^2h$	$\frac{3}{4}\left(r - \frac{h}{2}\right)$
	Spherical segment	Spherical surface $A = 2\pi rh$ $V = \frac{\pi}{3} h^2 (3r-h)$	$\frac{h\,(4r-h)}{4\,(3r-h)}$

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