Design and Construction Manual for Steel Sheet Pile

ISSUED BY

NANJING GRAND STEEL PILING CO.,LTD.

THE BIGGEST SHEET PILING MANUFACTURER IN CHINA

Table of Contents

1 Overview

Hot-rolled steel sheet pile was initially used in Europe at the beginning of the 20th century and is now a proven technology in foreign countries. But in China, this technology is still at its early stage. With the rapid growth of Chinese economy, this environmentally friendly new building material will inevitably be widely used.

Depending on their production process, steel sheet pile products are divided into cold-bent and hot-rolled piles which bring out the best in each other and have their own advantages. In the past years, the market has given priority to hot-rolled steel sheet pile of which the processing method and information can be found everywhere that has been widely recognized. Cold-bent steel sheet pile is formed by cold-bending machine set through rolling depression, and its side interlock could be continually connected to form a steel structure of sheet-pile wall .Though produced by a different working method, the use of cold-bent steel sheet pile is similar to that of hot rolled steel sheet pile while their scopes of application are different to some extent. Throughout the years, cold bent steel sheet piles have been widely used, and broad operation experience has been accumulated.

Nanjing Grand Steel Sheet Pile Co., Ltd is a key enterprise of China being engaged the research & development, design, production and service of cold-rolled forming section. Grand developed and produced China's first piece of cold-bent steel sheet pile with a section modulus of 1360cm^3 in October 2006 and was then expanded to have established an annual production capacity of 400,000 tons of cold-bent steel sheet piles.

Grand has been engaged in the research & development of cold-bent steel sheet piles according to EN standards (EN 10249: non-alloy cold-bent steel sheet pile) and now ranks among leading cold-bent steel sheet pile manufacturers of the world at present. Furthermore, Grand has built up its own brand in the field of steel sheet pile, and its GP series cold-bent steel sheet piles have been used in a number of key projects at home and abroad. GP series products that feature outstanding section modulus and large moment of inertia can be widely used for both general retaining wall and basic structures and for ports, piers, river bank reinforcement and land and water foundation pit protection structures projects etc.

During its development in the past years, Grand has given full play to its own cold-rolled forming section development technology and improved the structures of lots of combined steel sheet piles, contributing to more rational project design and construction and more remarkable economic benefit; in the meantime, in order to better satisfy the requirements of individual customers, most cold-bent steel sheet piles of Grand are integrated with personalized design according to foundation conditions..

Grand can now produce GPU and GPZ series cold-bent steel sheet piles of over 80 specifications with the maximum section modulus of up to 6000 cm3/m and a maximum thickness of 16mm.. When the section modulus of our product is the same with that of hot-rolled steel sheet pile, the steel consumption per square meter would decrease by 10%~15%, and this means sharp reduction of construction cost; in this way, the concept of "environmentally friendly, energy-saving and high-efficiency new-style building material" is realized..

Nanjing Grand Steel Sheet Pile Co., Ltd has established an overall service system for complete and sophisticated design, production, technical support and construction. With the support from top-class research institutes and universities (colleges) in the field of engineering of China, our technology sections can provide all services that a leading cold-bent steel sheet pile manufacturer should be able to offer, and can provide technical support for any step during steel sheet pile engineering, offering targeted and effective technical service taking into account various needs of customers.

2 Grand (GP) Series Cold-bent Steel Sheet Pile

2.1 Specification of Grand (GP) Series Cold-bent Steel Sheet Pile

Cold-bent steel sheet pile is manufactured of hot (cold) rolled strip steel which passes though several pairs of forming blocks composed of forming roller sheet of different shapes and is formed through continuous roll-type bending process so as to achieve various sectional forms for various required steel sheet piles.

GP series cold-bent steel sheet pile products have the following characteristics:

(1) Thanks to the fact that cold-bent steel sheet pile is made of hot-rolled strip steel through continuous compaction, the design of sectional structure is provided with high flexibility for production of products with large section, small and equal wall thickness;

(2) GP series cold-bent steel sheet pile is suitable for large sized products with effective width, height and thickness not more than 16mm, having conformed (adapted) to the development trend of the world's cold-bent steel sheet pile products and satisfied the requirements of engineering application of cold-bent steel sheet pile;

(3) The maximum section modulus of U-mode products of GP series cold-bent steel sheet pile is up to 4260 cm^3/m^2, and that of Z-mode series is up to 5100 cm^3/m^2;

(4) Thanks to the rational section structure design and the state-of-art forming technology of GP series cold-bent steel sheet pile, the ratio of section modulus to product weight (also known as "mass coefficient") has been increasing continuously. This contributes to better economic benefit of application and broadens the application area of cold-bent steel sheet pile;

The use of large-sized cold-bent steel sheet pile facilitates the reduction of working load of pile sinking (driving) , the improvement of work efficiency and the reduction in quantity of water seal joints.

2.1.1 Specification of Grand Cold-bent Steel Sheet Pile

Grand Steel Sheet Pile Co., Ltd currently produces cold-bent steel sheet piles of the following specifications:

(1) U-shaped cold-bent steel sheet pile

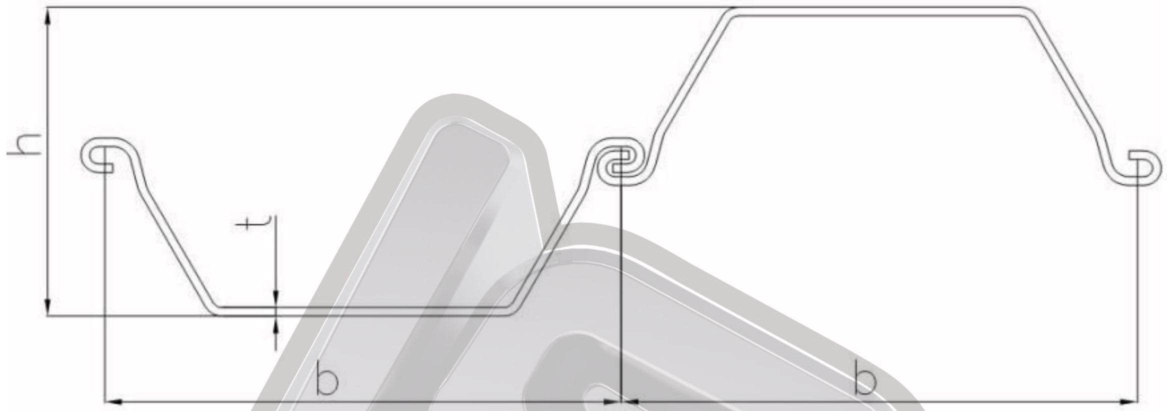


Fig. 2.1.1-1 GPU cold-bent steel sheet pile

Table 2.1.1-1 Technical parameters of GPU series products

Model	Effective width mm	Height mm	Wall thickness mm	Sectional area cm2	Weight Kg		Moment of inertia		Elastic section modulus cm3/m	Plastic section modulus	Radius of gyration cm
				Per linear meter	Per piece kg/m	Per linear meter kg/m2	cm4	cm4/m		cm3/m	
GPU11a	650	356	8.0	129.4	66.0	101.6	5032	20500	1152	1357	12.6
GPU12a	650	357	8.5	137.2	70.0	107.7	5339	21823	1223	1442	12.6
GPU13a	650	359	9.5	152.8	78.0	119.9	5950	24484	1364	1612	12.7
GPU14a	600	360	10.0	160.5	81.9	126.0	6254	25822	1435	1698	12.7
GPU18a	650	476	8.0	147.4	75.2	115.7	9724	42662	1793	2093	17.0
GPU19a	650	477	8.5	156.4	79.8	122.8	10315	45390	1903	2224	17.0
GPU20a	650	478	9.0	165.4	84.4	129.8	10904	48125	2014	2356	17.1
GPU21a	650	479	9.5	174.3	88.9	136.8	11491	50868	2124	2487	17.1
GPU22a	650	480	10.0	183.1	93.4	143.8	12075	53618	2234	2618	17.1
GPU23a	650	541	10.5	198.3	101.2	155.7	16709	62435	2308	2841	17.7

Table 2.1.1-1 Technical parameters of GPU series products

Model	Effective width mm	Height mm	Wall thickness mm	Sectional area cm2	Weight Kg		Moment of inertia		Elastic section modulus cm3/m	Plastic section modulus	Radius of gyration cm
				Per linear meter	Per piece kg/m	Per linear meter kg/m2	cm4	cm4/m		cm3/m	
GPU24a	650	542	11.0	207.4	105.8	162.8	17480	65470	2416	2976	17.8
GPU25a	650	543	11.5	216.5	110.5	170.0	18249	68512	2523	3111	17.8
GPU26a	650	544	12.0	230.0	117.3	180.5	19886	70979	2610	3245	17.6
GPU14b	750	476	8.0	128.0	75.3	100.4	9337	34498	1449	1742	16.4
GPU16b	750	478	9	143.5	84.5	112.6	10470	38909	1628	1960	16.5
GPU17b	750	479	9.5	151.7	89.3	119.1	11046	41480	1732	2086	16.5
GPU18b	750	478	9.0	149.3	87.9	117.2	11164	43965	1840	2152	17.2
GPU20b	750	480	10.0	165.4	97.4	129.8	12364	48985	2041	2392	17.2
GPU21b	750	480	10.0	168.3	99.1	132.1	12641	51214	2134	2478	17.5
GPU23b	750	540	10	170.5	100.4	133.8	15700	61718	2286	2710	19.0
GPU25b	750	540	10.0	178．3	105.0	139.9	16489	69237	2564	2974	19.7
GPU26b	750	561	10.5	191.9	113.0	150.7	20374	73019	2603	3090	19.5
GPU28b	750	562	11.0	203.1	119.6	159.4	21645	78809	2805	3313	19.7
GPU30b	750	564	12.0	225.0	132.5	176.6	24720	85879	3045	3622	19.5
GPU33b	750	604	12.0	229.9	135.4	180.5	28425	99175	3284	3925	20.8
GPU35b	750	606	13.0	252.6	148.7	198.3	31861	107531	3549	4264	20.6
GPU38b	750	608	14.0	271.3	159.7	213.0	34233	116028	3817	4591	20.7
GPU40b	750	610	15.0	308.2	181.4	241.9	42174	122056	4002	4929	19.9
GPU42b	750	544	16.0	327.9	193.0	257.4	44913	130430	4262	5257	19.9

Besides, other specifications are also available.

Table 2.1.1-2 Technical parameters of other products

Model	Effective width mm	Height mm	Wall thickness mm	Sectional area cm2	Weight Kg		Moment of inertia cm4	Elastic section modulus cm3/m cm4/m	Plastic section modulus cm3/m	Radius of gyration cm
				Per linear	Per piece kg/m	Per linear meter				
GPU10-450	450	360	8.0	148.9	52.6	116.9	18267	1015	1302	11.1
GPU11-450	450	360	9.0	166.1	58.7	130.4	20383	1132	1455	11.1
GPU12-450	450	360	10.0	183.9	64.9	144.3	22443	1247	1605	11.0
GPU11-575	575	360	8.0	133.9	60.4	105.1	19684	1094	1325	12.1
GPU12-575	575	360	9.0	149.9	67.6	117.6	21979	1221	1482	12.1
GPU13-575	575	360	10.0	165.6	74.8	130.0	24223	1346	1637	12.1
GPU11-600	600	360	8.0	131.7	62.0	103.4	19897	1105	1434	12.3
GPU12-600	600	360	9.0	147.4	69.4	115.7	22219	1234	1486	12.3
GPU13-600	600	360	10.0	162.9	76.7	127.9	24491	1361	1641	12.3
GPU18-600	600	350	12.0	220.3	103.8	172.9	32797	1874	2053	12.2
GPU16-650	650	480	8.0	139.5	71.2	109.5	39872	1661	1973	16.9
GPU18-650	650	480	9.0	156.1	79.6	122.5	44521	1855	2268	16.9
GPU20-650	650	540	8.0	153.7	78.4	120.7	56002	2074	2443	19.1
GPU23-650	650	540	9.0	172.1	87.8	135.1	62588	2318	2735	19.1
GPU30-700	700	560	11.0	216.6	119.0	170.1	83813	2993	3528	19.7
GPU32-700	700	560	12.0	236.2	129.8	185.4	90880	3246	3834	19.6
GPU7-750	750	320	5.0	72.7	42.8	57.0	11089	693	778	12.3
GPU8-750	750	320	6.0	86.7	51.1	68.1	13191	824	928	12.3
GPU9-750	750	320	7.0	100.7	59.3	79.0	15256	953	1076	17.5

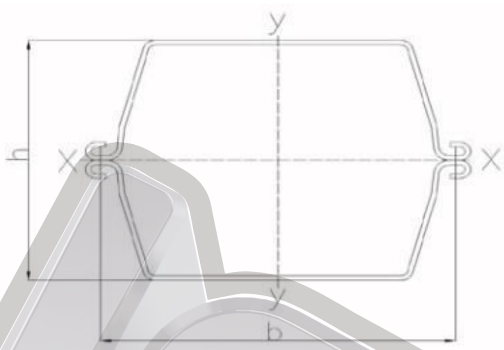


Fig. 2.1.1-2 GPU box pile

Table 2.1.1-3 Technical parameters of GPU series built-up piles

Model	b mm	h mm	Perimeter cm	Sectional area of section steel cm^2	Total area cm^2	Weight G Kg/m	Moment of inertia I		Section modulus		Radius of gyration cm	Wall width per	
							x-x cm^4	y-y cm^4	x-x cm^3	y-y cm^3		Wx cm^3	Gp kg
GPUC11a-2	650	411	240.7	168.2	2214.5	132.0	37108	92438	1806	2631	14.9	2778	203
GPUC12a-2	650	412	240.2	178.4	2221	140.0	39489	97779	1917	2774	14.9	2949	215.4
GPUC13a-2	650	414	239.2	198.6	2235	155.9	44274	108311	2139	3073	14.9	3291	239.9
GPUC14a-2	650	415	238.7	208.7	2241	163.8	46677	113502	2249	3220	15.0	3460	252.0
GPUC18a-2	650	531	270.1	191.7	2944	150.5	71361	110826	2688	3144	19.3	4135	231.5
GPUC19a-2	650	532	269.6	203.4	2951	159.6	75910	117288	2854	3327	19.3	4391	245.5
GPUC20a-2	650	533	269.1	215.0	2958	168.7	80468	123696	3019	3509	19.3	4635	259.6
GPUC21a-2	650	534	268.6	226.5	2965	177.8	85037	130051	3185	3689	19.4	4900	273.6
GPUC22a-2	650	535	268.1	238.1	2972	186.9	89617	136353	3350	3868	19.4	5154	287.5
GPUC23a-2	650	610	283.3	257.8	3083	202.4	108444	143171	3556	4005	20.5	5470	311.4

(2) Z-shaped cold-bent steel sheet pile

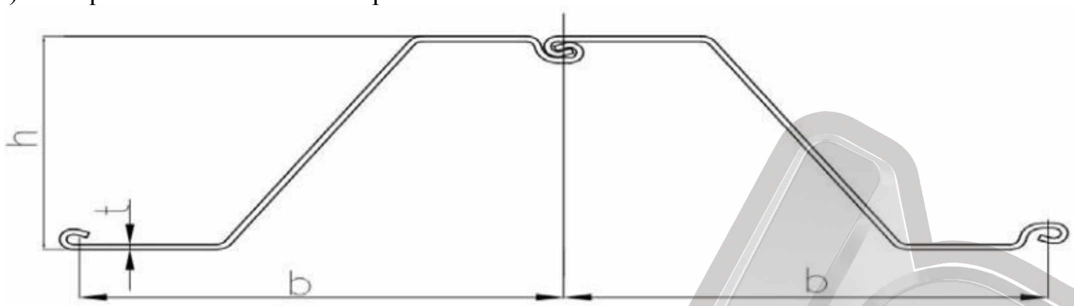


Fig. 2.1.1-3 GPZ cold-bent steel sheet pile

Table 2.1.1-4 Technical parameters of GPZ series products

Model	Effective width mm	Height mm	Wall thickness mm	Sectional area cm2	Weight Kg		Moment of inertia	Elastic section modulus cm3/m	Plastic section modulus	Radius of gyration cm
				Per linear	Per piece kg/m	Per linear meter	cm4	cm4/m	cm^3/m	
GPZ12b	750	318.5	8.5	120.1	70.7	94.3	19251	1209	1418	12.5
GPZ13a	700	416.5	6.5	99.5	54.7	78.1	27616	1326	1535	16.7
GPZ13b	700	319.5	9.5	133.8	78.8	105.0	21425	1341	1578	12.7
GPZ14a	700	417.0	7.0	107.0	58. 8	84.0	29671	1423	1649	16.7
GPZ14b	750	310.5	10.5	152.1	89.6	119.4	22329	1438	1722	12.1
GPZ15a	700	417.5	7.5	114.4	62.9	89.8	31715	1519	1763	16.7
GPZ16a	700	418.0	8.0	125.7	69.1	98.7	34706	1661	1937	16.6
GPZ17a	700	418.5	8.5	133.3	73.2	104.7	36793	1756	2054	16.6
GPZ18a	700	419.0	9.0	140.9	77.4	110.6	38871	1855	2170	16.6
GPZ18b	750	419.5	9.5	141.5	83.3	111.1	37982	1811	2140	16.4
GPZ19a	700	419.5	9.5	148.5	81.6	116.6	40939	1952	2286	16.6

Table 2.1.1-4 (continued) Technical parameters of GPZ series products

Model	Effective width mm	Height mm	Wall thickness mm	Sectional area cm2	Weight Kg		Moment of inertia cm4	Elastic section modulus cm3/m cm4/m	Plastic section modulus cm^3/m	Radius of gyration cm
				Per linear	Per piece kg/m	Per linear meter				
GPZ19b	750	420.5	10.5	159.0	93.5	124.8	40296	1916	2329	15.9
GPZ20a	700	420.0	10.0	156.1	85.7	122.5	42997	2047	2401	16.6
GPZ20b	700	448.0	8.0	133.5	73.3	104.8	44470	1985	2274	18.2
GPZ21a	700	448.5	8.5	141.6	77.8	111.1	47138	2102	2411	18.2
GPZ22a	700	449.0	9.0	149.7	82.3	117.5	49821	2219	2548	18.2
GPZ23a	700	449.5	9.5	157.7	86.7	123.8	52480	2335	2685	18.2
GPZ24a	700	450.0	10.0	165.7	90.1	130.1	55128	2450	2820	18.2
GPZ25b	750	451.0	11.5	181.6	106.9	142.6	56405	2498	2966	17.6
GPZ26a	700	450.5	10.5	180.0	98.7	141.3	59119	2625	3045	18.1
GPZ26b	750	452	12.0	193.3	113.8	151.8	59665	2640	3152	17.6
GPZ27a	700	451.0	11.0	187.9	103.3	147.5	61680	2735	3183	18.1
GPZ28a	700	451.5	11.5	196.2	107.8	154.0	64356	2851	3322	18.0
GPZ28b	750	453.0	13.0	211.7	124.6	166.2	63273	2794	3390	17.3
GPZ29a	700	452.0	12.0	206.6	113.5	162.2	66512	2943	3463	17.9
GPZ31a	700	452.5	12.5	216.7	119.1	170.1	70426	3113	3654	18.0
GPZ32a	700	489.0	11.0	198.6	109.1	155.9	78051	3192	3685	19.8
GPZ33a	700	489.5	11.5	207.3	113.9	162.7	81443	3328	3845	19.8
GPZ34a	700	490.0	12.0	219.1	120.4	172.0	85209	3478	4044	19.7
GPZ34b	750	491.0	13.0	224.5	132.2	176.3	84085	3425	4052	19.4
GPZ36a	700	490.5	12.5	227.9	125.2	178.9	88594	3612	4205	19.7
GPZ36b	750	492	14.0	240.1	141.3	188.5	89341	3632	4314	19.3
GPZ37a	700	491.0	13.0	239.4	131.5	187.9	91568	3730	4381	19.5

Table 2.1.1-4 (continued)   Technical parameters of GPZ series products

Model	Effective width mm	Height mm	Wall thickness mm	Sectional area cm2	Weight Kg		Moment of inertia cm4	Elastic section modulus cm3/m cm4/m	Plastic section modulus	Radius of gyration cm
				Per linear	Per piece kg/m	Per linear meter				
GPZ38a	700	491.5	13.5	248.2	136.4	194.8	94916	3862	4541	19.5
GPZ38b	750	522.0	15.0	268.1	157.8	210.5	100618	3855	4799	19.4
GPZ40a	700	492.0	14.0	257.0	141.2	201.7	98251	3994	4701	19.5
GPZ41a	750	520.0	13.0	241.0	141.9	189.2	106697	4104	4758	21.0
GPZ42a	750	550.0	13.0	240.5	141.6	188.8	116350	4231	4939	22.0
GPZ43a	750	521.0	14.0	256.5	245.1	201.3	112625	4323	5036	20.9
GPZ45a	750	551.0	14.0	258.3	152.0	202.7	124864	4532	5301	22.0
GPZ48a	750	520.0	15.0	292.8	172.4	229.8	124921	4805	5682	20.5
GPZ51a	750	521.0	16.0	311.4	183.4	244.5	132833	5099	6042	20.5

Additionally, there are also other specifications available

Table 2.1.1-5 Technical parameters of other products

Model	Effective width mm	Height mm	Wall thickness mm	Sectional area cm2	Weight Kg		Moment of inertia cm4	Elastic section modulus cm3/m cm4/m	Plastic section modulus	Radius of gyration cm
				Per linear	Per piece kg/m	Per linear meter				
GPZ18-635	635	380	8.0	140.6	70.1	110.3	34717	1827	2083	15.7
GPZ22-635	635	417	9.0	162.6	81.1	127.6	47225	2265	2603	17.0
GPZ25-635	635	418	10.0	162.6	81.1	127.6	47225	2265	2603	17.0
GPZ28-635	635	419	11.0	209.0	104.2	164.1	58786	2806	3298	16.8
GPZ30-635	635	420	12.0	227.3	113.3	178.4	63889	3042	3584	16.8
GPZ14-650	650	320	8.0	128.9	65.8	101.2	22312	1395	1602	13.2
GPZ34-675	675	490	12	224.4	118.9	176.1	84657	3455	4071	19.4
GPZ38-675	675	491.5	13.5	251.3	133.1	197.2	94699	3853	4555	19.4
GPZ18A-685	685	401	9.0	144.0	77.4	113.0	37335	1862	2163	16.1
GPZ20-685	685	402	10.0	159.4	85.7	125.2	41304	2055	2393	16.1

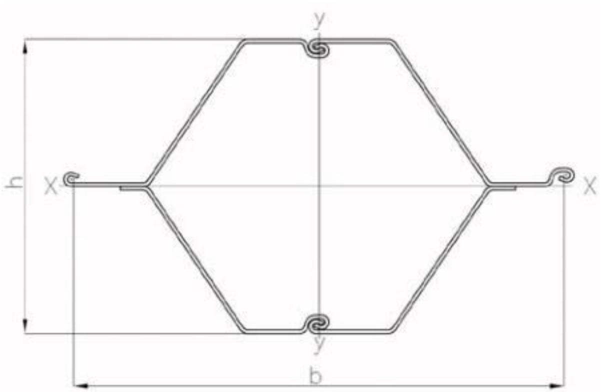


Fig. 2.1.1-4 GPZ box pile

Table 2.1.1-6 Technical parameters of GPZ built-up box piles

Model	b mm	h mm	Perimeter cm	Sectional area of section steel cm^2	Total area cm^2	Weight G Kg/m	Moment of inertia I		Section modulus		Radius of gyration cm	Wall width per	
							x-x cm^4	y-y cm^4	x-x cm^3	y-y cm^3		Wx cm^3	Gp kg
GPZC12b	1500	637	390.7	319.9	4917	251.1	148967	516673	4649	6620	21.6	3099	167.4
GPZC13a	1400	833	400	250.6	5958	196.7	198071	348579	4740	4799	28.1	3386	140.5
GPZC13b	1500	639	390.4	356.5	4948	279.9	166493	575716	5171	7353	21.6	3447	186.6
GPZC14a	1400	834	400	269.5	5958	211.6	213191	374765	5096	5161	28.1	3640	151.1
GPZC14b	1500	621	395.7	403.3	4833	316.6	176524	654660	5640	8308	20.9	3760	211.1
GPZC15a	1400	835	399.6	288.3	5973	226.3	228295	400862	5450	5521	28.1	3893	161.6
GPZC16a	1400	836	406.1	315.3	5991	247.5	250747	440714	5973	6030	28.2	4266	176.8
GPZC17a	1400	837	406.0	334.5	6007	262.6	266295	467488	6336	6397	28.2	4526	187.6
GPZC18a	1400	838	405.8	353.7	6022	277.7	281828	494173	6697	6763	28.2	4784	198.3

Table 2.1.1-6 (continued) Technical parameters of GPZ built-up box piles

Model	b mm	h mm	Perimeter cm	Sectional area of section steel cm^2	Total area cm^2	Weight G Kg/m	Moment of inertia I		Section modulus		Radius of gyration cm	Wall width per	
							x-x cm^4	y-y cm^4	x-x cm^3	y-y cm^3		Wx cm^3	Gp kg
GPZC18b	1500	839	414.9	383.5	6464	301.0	300498	625404	7133	8010	28.0	4755	200.7
GPZC19a	1400	839	405.5	372.9	6038	292.7	297344	520768	7057	7127	28.2	5041	209.1
GPZC19b	1500	841	415.6	433.9	6503	340.6	331303	728975	7832	9263	27.6	5221	227.1
GPZC20a	1400	840	405.4	391.9	6053	307.6	312846	547274	7415	7491	28.3	5296	219.7
GPZC20b	1400	896	433.5	330.9	6416	259.7	311856	448318	6933	6037	30.7	4952	185.5
GPZC21a	1400	897	433.3	351.1	6432	275.6	331184	475566	7354	6403	30.7	5253	196.9
GPZC22a	1400	898	433.1	371.3	6448	291.4	350493	502725	7774	6768	30.7	5553	208.1
GPZC23a	1400	899	433.0	391.3	6464	307.2	369782	529794	8192	7131	30.7	5851	219.4
GPZC24a	1400	900	432.7	411.4	6479	322.9	389050	556773	8609	7493	30.8	6149	230.7
GPZC25b	1500	903	436.2	486.9	6987	382.2	446363	782810	9830	9999	30.3	6553	254.8
GPZC26a	1400	901	440.3	442.7	6498	347.5	419995	602656	9269	8009	30.8	6620	248.2
GPZC26b	1500	904	441.1	516.4	7006	404.6	474516	833984	10429	10557	30.3	6953	269.7
GPZC27a	1400	902	440.1	463.2	6514	363.6	439810	629725	9695	8403	30.8	6925	259.7
GPZC28a	1400	903	440.0	483.6	6530	379.6	459606	657299	10119	8770	30.8	7228	271.1
GPZC28b	1500	906	441.3	567.2	7045	445.3	514601	933458	11260	11639	30.1	7507	296.9
GPZC29a	1400	904	439.9	509.7	6548	400.1	481028	698183	10570	9455	30.7	7550	285.8
GPZC31a	1400	905	443.7	533.1	6565	418.5	506972	726085	11127	9853	30.8	7948	298.9
GPZC32a	1400	978	466.1	488.1	7056	383.2	550497	654821	11199	8714	33.6	7999	273.7
GPZC33a	1400	979	465.9	509.6	7072	400.0	575246	683521	11689	9095	33.6	8349	285.7
GPZC34a	1400	980	468.2	537.9	7091	422.2	606078	725167	12294	9689	33.6	8781	301.6
GPZC34b	1500	982	471.0	593.8	7622	466.1	657192	940554	13304	11831	33.3	8869	310.7

Table 2.1.1-6 (continued) Technical parameters of GPZ built-up box piles

Model	b mm	h mm	Perimeter cm	Sectional area of section steel cm^2	Total area cm^2	Weight G Kg/m	Moment of inertia		Section modulus		Radius of gyration cm	Wall width per	
							x-x cm^4	y-y cm^4	x-x cm^3	y-y cm^3		Wx cm^3	Gp kg
GPZC36a	1400	981	468.0	559.6	7108	439.3	631056	754216	12787	10076	33.6	9133	313.8
GPZC36b	1500	984	468.5	636.4	7656	499.6	702742	1010148	14183	12772	33.2	9455	333.1
GPZC37a	1400	982	468.4	587.7	7127	461.3	659304	799733	13333	10617	33.5	9524	329.5
GPZC38a	1400	983	468.2	609.5	7143	478.5	684386	829236	13826	11007	33.5	9876	341.8
GPZC38b	1500	1044	473.6	719.8	8148	565.0	847322	1214902	16081	14837	34.3	10821	376.7
GPZC40a	1400	984	468.0	631.3	7159	495.6	709444	858647	14317	11395	33.5	10226	354.0
GPZC41a	1500	1040	509.0	629.7	8068	494.3	807839	968153	15435	11963	35.8	10290	329.5
GPZC42a	1500	1110	507.8	633.8	8523	497.5	893621	981373	16151	12302	37.5	10767	331.7
GPZC43a	1500	1042	503.6	671.9	8102	527.4	858884	1035574	16378	12865	35.8	10918	351.6
GPZC45a	1500	1102	507.4	681.0	8559	534.6	961511	1053873	17345	13207	37.6	11563	356.4
GPZC48a	1500	1040	522.9	759.7	8121	596.4	965970	1190055	18399	14472	35.7	12266	397.6
GPZC51a	1500	1042	522.4	808.6	8156	634.7	1029817	1265888	19576	15389	35.7	13051	423.7

(3) Straight cold-bent steel sheet pile

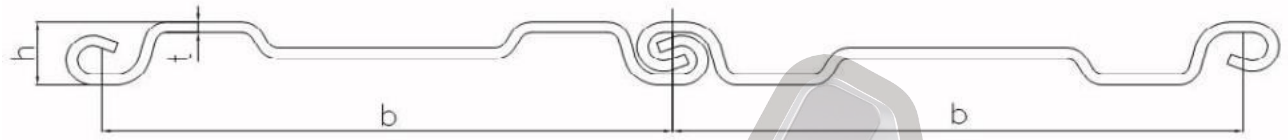


Fig. 2.1.1-5 GPX cold-bent steel sheet pile

Table 2.1.1-7 Technical parameters of GPX series products

Model	Effective width mm	Height mm	Wall thickness mm	Wall thickness mm	Sectional area cm^2		Moment of inertia	Elastic section modulus
				Per linear meter	Per piece	Per linear meter kg/m^2	cm^4/m	cm^3/m
GPX600-10	1200	69.5	10	152.1	71.6	119.4	594	171
GPX600-11	1200	70.5	11	166.6	78.5	130.8	651	185
GPX600-12	1200	71.5	12	180.9	85.2	142.0	711	199
GPX600-13	1200	72.5	12.5	188.0	88.5	147.6	738	205
GPX600-14	1200	79.0	14	215.8	101.6	169.4	1038	263

(4) Cold-bent steel sheet pile of other types

① L-shaped and S-shaped (light) cold-bent steel sheet pile

Fig. 2.1.1-6 L-shaped and S-shaped (light) cold-bent steel sheet pile

Table 2.1.1-8 Technical parameters of GPX series products

Model	Effective width mm	Height mm	Wall thickness mm	Weight Kg		Moment of inertia cm^4/m	Elastic section modulus cm^3/m
				Per piece kg/m	Per linear meter kg/m^2		
GPL1.5	700	100	3.0	21.4	30.6	724	145
GPL2	700	150	3.0	22.9	32.7	1674	223
GPL3	700	150	4.5	35.0	50.0	2469	329
GPL4	700	180	5.0	40.4	57.7	3979	442
GPL5	700	180	6.5	52.7	75.3	5094	566
GPL6	700	180	7.0	57.1	81.6	5458	606
GPS4	600	260	3.5	31.2	41.7	5528	425
GPS5	600	260	4.0	36.6	48.8	6703	516
GPS6	700	260	5.0	45.3	57.7	7899	608
GPS8	700	320	5.5	53.0	70.7	12987	812
GPS9	700	320	6.5	62.6	83.4	15225	952

② GPJ sawtooth shaped cold-bent steel sheet pile

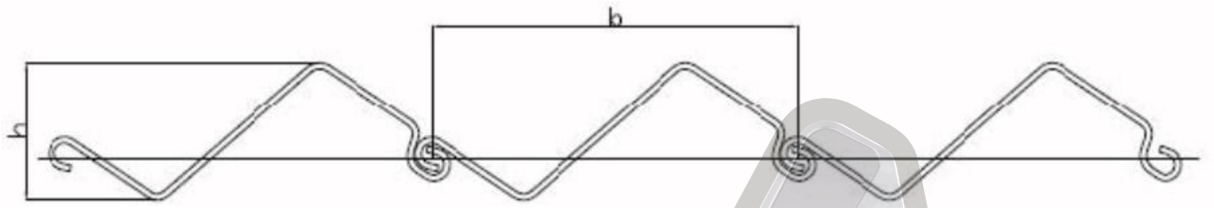


Fig. 2.1.1-7 GPJ cold-bent steel sheet pile

Table 2.1.1-9 Technical parameters of GPJ series products

Model	Effective width mm	Height mm	Sectional area cm^2/m	Weight kg/m^2	Moment of inertia cm^4/m	Elastic section modulus cm^3/m
GPJ12	812	187	111.0	86.4	3152	314
GPJ13	812	187.9	123.7	97.1	3506	349
GPJ14	808	196.3	141.2	110.9	5897	574
GPJ17	811	221	115.0	90.3	4900	398
GPJ18	855	204.1	124.2	97.5	4387	390
GPJ19	855	202.6	139.5	109.5	4363	420
GPJ25	870	232.3	156.7	123.0	7207	559
GPJ26	870	237.9	166.7	130.8	7833	598
GPJ28	870	229	182.5	143.2	7463	602
GPJ34	889	272.6	189.4	148.7	12204	782
GPJ36	889	268	202.5	159.0	12827	833
GPJ38	905	257.3	222.2	174.4	10647	801
GPJ45	922	340	210.0	164.9	21633	1132
GPJ48	904	266	242.9	190.7	22641	1168
GPJ50	904	340	258.4	202.8	23342	1216

③ GPG trench panel

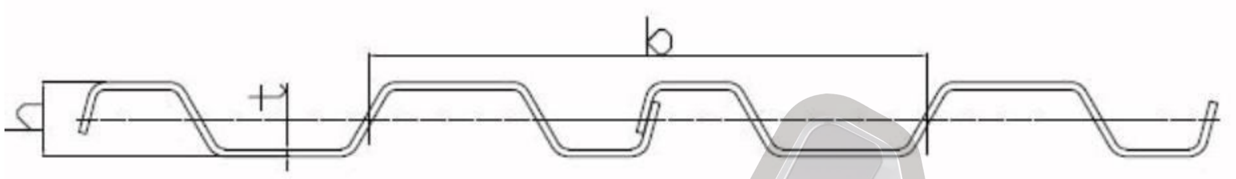
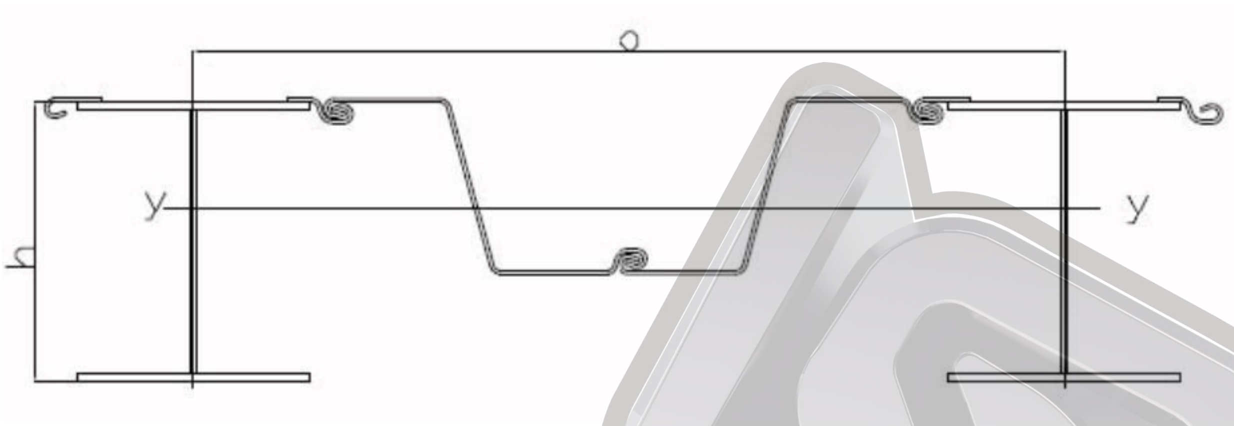


Fig. 2.1.1-8 GPG trench panel

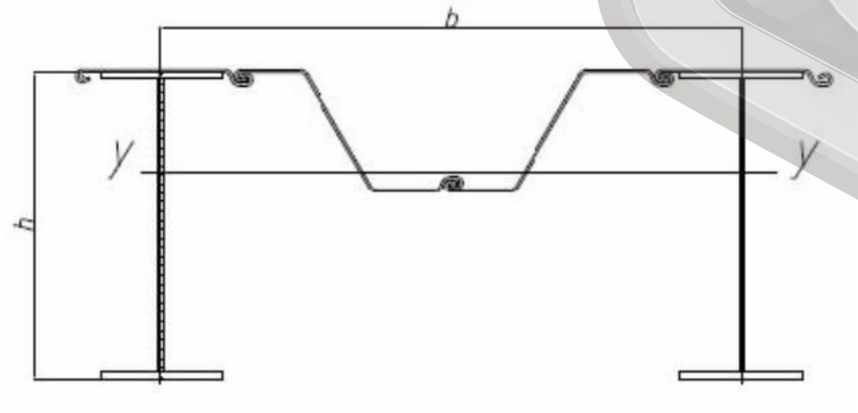
Table 2.1.1-10 Technical parameters of GPG series products

Model	Effective width mm	Height mm	Wall thickness mm	Sectional area cm^2/m	Weight Kg		Moment of inertia cm^4/m	Section modulus cm^3/m
					Per piece kg/m	Per linear meter kg/m^2		
GPG I-1	750	95	6	72.1	42.4	56.6	975	202
GPG I-2	750	96	7	84.2	49.6	66.12	1139	234
GPG I-3	750	8	8	96.4	56.8	75.7	1304	266
GPG II-1	650	80	6	79.5	40.5	62.4	758	190
GPG II-2	650	82	8	105.7	53.9	83.0	1013	247

④ GPH/GPZ18 built-up steel sheet pile



(a) GPH/GPZ18a



(b) GPH/GPZ18b

Fig. 2.1.1-9 GPH/GPZ18 built-up steel sheet pile

Table 2.1.1-11 Technical parameters of GPH/GPZ18 built-up steel sheet pile

Model	Effective width mm	Height mm	Sectional area cm^2/m	Weight kg/m^2	Moment of inertia cm^4/m	Section modulus cm^3/m
GPH600-14/GPZ18a	1870	600	221.1	173.6	112735	3020
GPH600-16/GPZ18a	1870	600	231.6	181.8	121767	3291
GPH600-18/GPZ18a	1870	600	245.1	192.4	131567	3594
GPH600-20/GPZ18q	1870	600	255.5	200.6	140248	3859
GPH800-17/GPZ18q	1870	800	252.7	198.3	228361	3859
GPH800-19/GPZ18q	1870	800	263.1	206.5	244778	4853
GPH800-21/GPZ18q	1870	800	281.6	221.1	265620	5339
GPH800-23/GPZ18q	1870	800	292.0	229.3	281428	5696
GPH1000-17/GPZ18q	1870	1000	275.8	216.5	382513	5973
GPH1000-19/GPZ18q	1870	1000	286.2	224.7	408639	6432
GPH1000-21/GPZ18a	1870	1000	306.9	240.9	443951	7090
GPH1000-23/GPZ18q	1870	1000	317.2	249.0	469201	7543
GPH600-14/GPZ18b	1890	600	205.5	161.3	98156	2664
GPH800-25/GPZ18b	1890	800	293.0	230.0	319753	5757
GPH1000-25/GPZ18b	1890	1000	302.6	237.5	424879	6683

2.1.2 Standard Rotation Angle of Cold-bent Steel Sheet Pile

When being connected, the standard rotation angle of Grand cold-bent steel sheet pile of the same model is as shown in the figure below:

(1) U-shaped cold-bent steel sheet pile

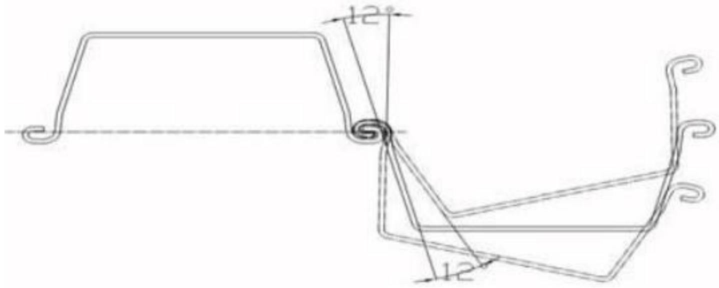


Fig. 2.1.2-1 Schematic diagram of rotation angle of GPU cold-bent steel sheet pile

(2) Z-shaped cold-bent steel sheet pile

Fig. 2.1.2-2 Schematic diagram of rotation angle of GPZ cold-bent steel sheet pile

(3) Straight cold-bent steel sheet pile

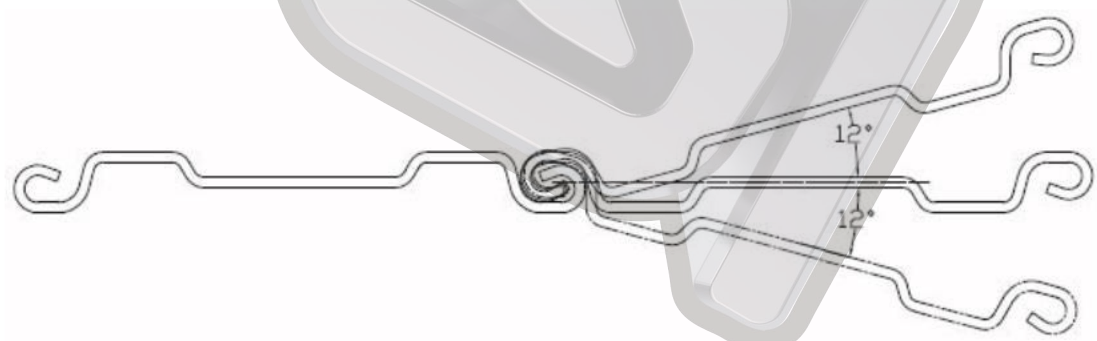


Fig. 2.1.2-3 Schematic diagram of rotation angle of GPX cold-bent steel sheet pile

2.1.3 Exchangeability of Cold-bent Steel Sheet Pile

The interlocks of Grand cold-bent steel sheet pile are designed professionally according to thickness of raw material and the requirement on quality and grade of material. The exchangeability of cold-bent steel sheet pile interlocks is available within the following scope:

(1) U-shaped cold-bent steel sheet pile

Fig. 2.1.3-1 Schematic diagram of rotation angle of GPU cold-bent steel sheet pile

(2) Z-shaped cold-bent steel sheet pile

Fig. 2.1.3-2 Schematic diagram of rotation angle of GPZ cold-bent steel sheet pile

(3) Straight cold-bent steel sheet pile

Fig. 2.1.3-3 Schematic diagram of rotation angle of GPX cold-bent steel sheet pile

2.2 Production Process

Cold-bent steel sheet pile is fabricated from qualified strip steel coil that gets into roll bending-forming block after the de-coiling, pre-straightening and butt welding procedures in the section of production preparation. The sectional form of product is achieved from strip steel blank after bending processing of several pairs of forming roller sheets, and then the finished product is straightened and cut in product lengths as stated. . After passing the rigorous final product inspection, the finished products are packed and put in storage.

Production line process of cold-bent steel sheet pile is as follows:

De-coiling -- leveling -- shearing and butt welding -- longitudinal shearing -- cold roll forming -- saw cutting -- inspection - collection and packing - warehousing

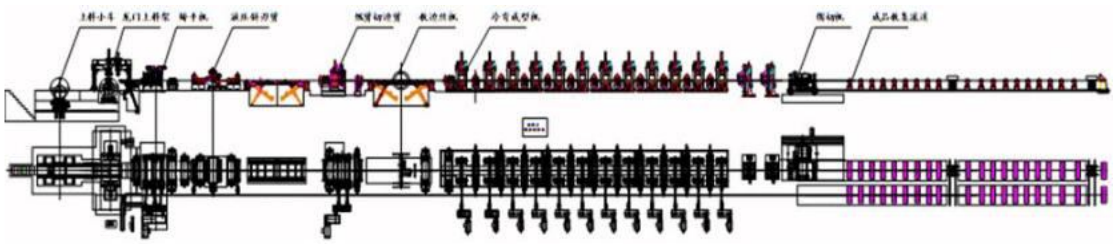


Fig. 2.2 Schematic diagram of cold roll bending

2.3 Material

The materials for fabrication of GP series cold-bent steel sheet pile are as shown in the table below:

Table 2.3-1 GP series cold-bent steel sheet pile material sheet

Steel grade	Chemical composition					Mechanical property			Work done by impact J
	C	Si	Mn	P	S	Yield strength MPa	Tensile strength MPA	Percentage elongation %	
Q235	≤0.18	≤0.30	0.35~0.80	≤0.040	≤0.040	≥235	375~460	≥26	≥27
Q345	≤0.20	≤0.55	1.0~1.6	≤0.035	≤0.035	≥345	470~630	≥22	≥34
MDB350	≤0.18	≤0.50	≤1.50	≤0.025	≤0.020	≥350	≥470	≥23	≥40
MDB380	≤0.18	≤0.50	≤1.60	≤0.025	≤0.020	≥380	≥480	≥22	≥40
MDB400	≤0.18	≤0.50	≤1.60	≤0.025	≤0.020	≥400	≥510	≥22	≥40
MDB420	≤0.18	≤0.50	≤1.60	≤0.025	≤0.020	≥420	≥520	≥21	≥40
MDB450	≤0.18	≤0.50	≤1.70	≤0.025	≤0.020	≥450	≥550	≥20	≥40

2.4 Product Quality Standard

Allowable deviation of outside dimensions of GP series cold-bent steel sheet pile products is as shown in the table below:

Table 2.4-1 Allowable deviation of shape and dimensions of GP series steel sheet pile

	Section shape		
U-shaped	Z-shaped		Straight model
Width	Per steel sheet pile ±2％ Sheet pile pairs connected through interlock: ±3％		
Height	h≤200 200＜h≤300 300＜h≤400 400＜h	±4 ±6 ±8 ±10	±4
Thickness	Comply with product standard of relevant strip steel as raw material		
Bending	Edgewise bend ≤0.25% of total length of section material; plane bending ≤ 0.25% of total length of section material		
Twisting	V≤0.2% of total length of section material, 100mm max.		
Length	Allowable deviation of length is ±50mm		
End verticality	In the athwartship measurement, the total deviation of the highest point of cut surface from the lowest point thereof shall not exceed 2% of section width.		
Angular deviation	When the length of short edge of sheet pile ≤10mm, this deviation shall be ±3°, and in other cases, shall be ±2°.		

2.5 Transport and Storage

2.5.1 Transport

The length of steel sheet piles produced according to the characteristics of roll bending-forming technology under

processing conditions of GP series cold-bent steel sheet piles is not limited. For the product length required by user, available means and route of transportation shall be taken into consideration.
Transportation of steel sheet pile is subject to various restrictions: traffic rules and actual road conditions shall be considered in case of road transport; for sea transportation, the conditions of unloading yard and machines and maritime meteorological conditions shall be investigated.

To prevent significant bending moment and deformation of steel sheet pile during transportation, the piles shall be provided with pad and sleeper and packed with steel packing strap. It should be notable that each piece of surface-treated steel sheet pile shall be separated by certain means, and that soft sealing pad shall be used with packing strap.

For transportation of ultra-long piece of steel sheet pile, transport vehicle shall be provided with at least five sleepers per 20m of length after the above-noted packing conditions are satisfied so as to prevent deformation; be sure to keep level in such a case.

2.5.2 Hoisting

Hoisting method of steel sheet pile is as shown in the figure below:

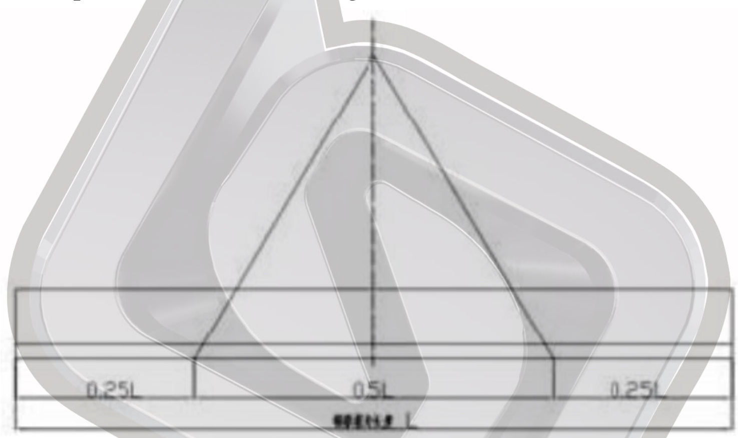
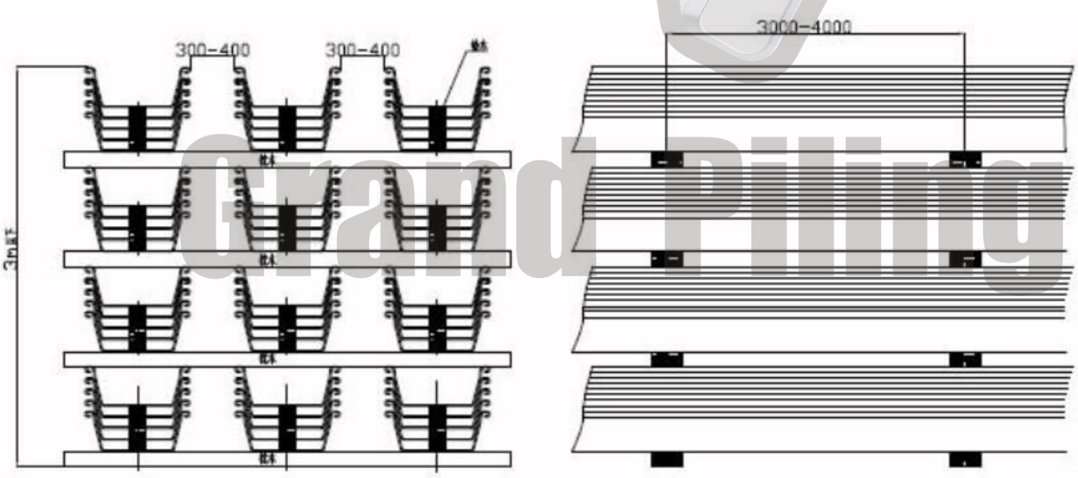


Fig. 2.5.2-1 Schematic diagram hoisting of steel sheet pile

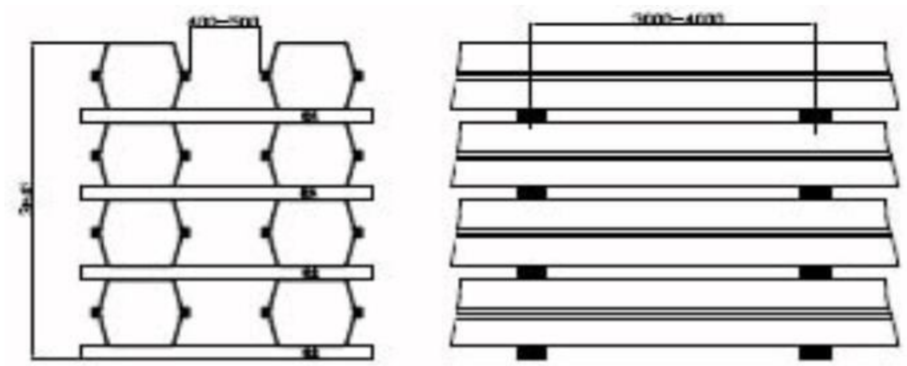
To increase friction force, dedicated lifting belt shall be used for hoisting of extra-long piece.

2.5.3 Storage

(1) U-shaped steel sheet pile



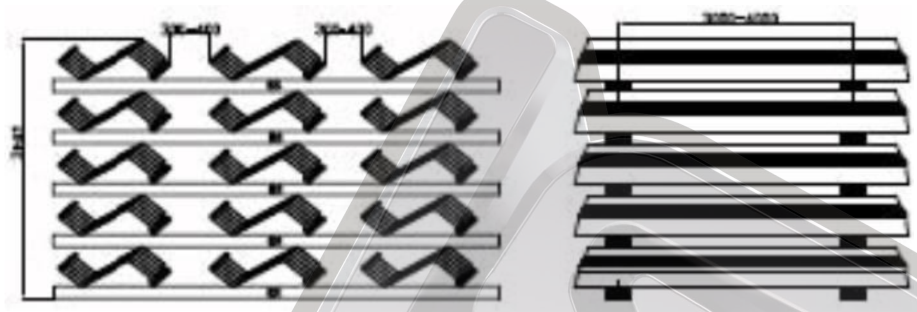
(a) GPU steel sheet pile



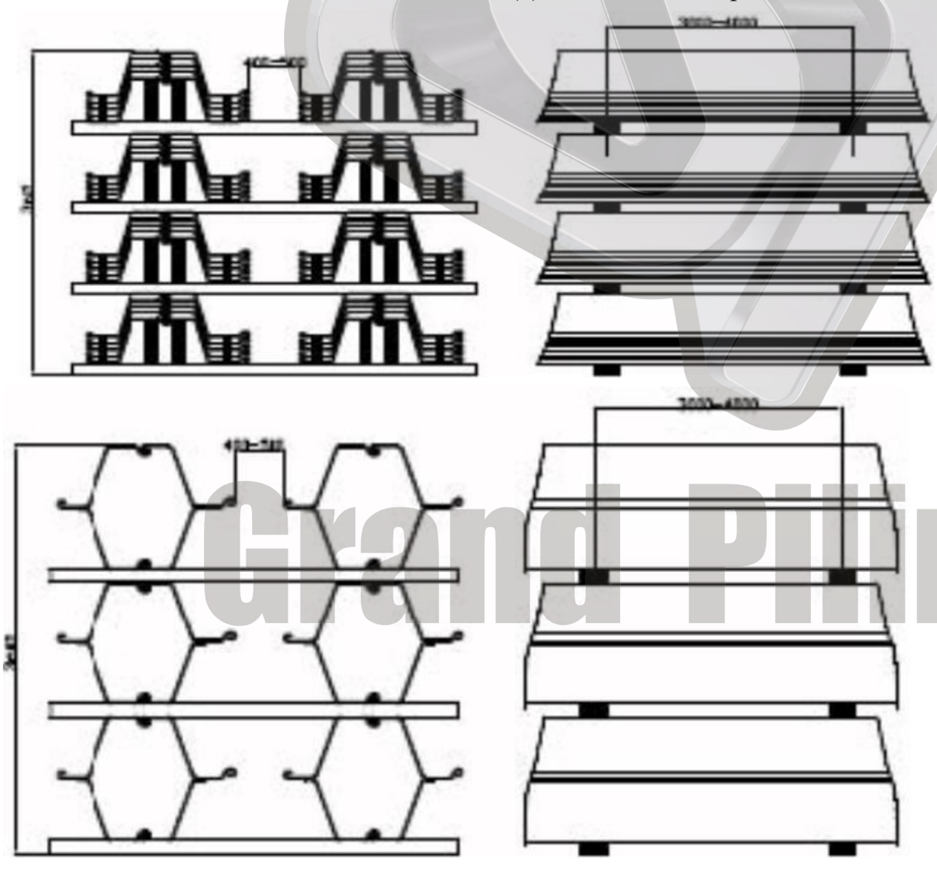
(b) GPU built-up pile

Fig. 2.5.3-1 Stacking of U-shaped steel sheet pile

(2) Z-shaped steel sheet pile



(a) GPZ steel sheet pile



(b) GPZ built-up pile

Fig. 2.5.3-2 Stacking of Z-shaped steel sheet pile

(3) Straight steel sheet pile

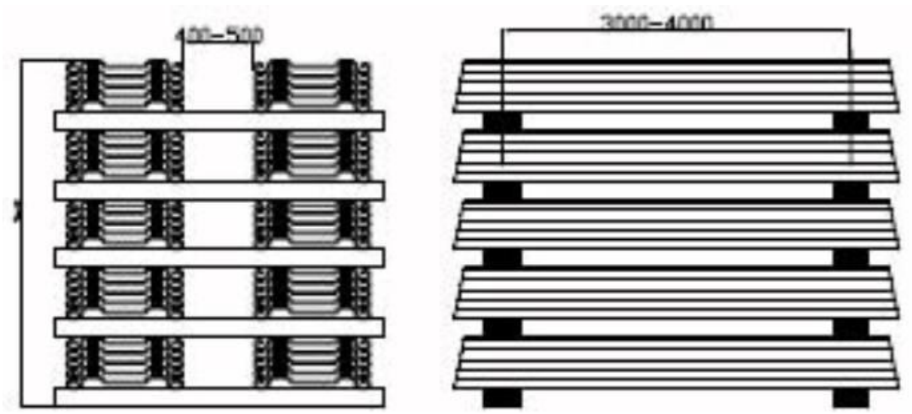


Fig. 2.5.3-3 Stacking of GPX steel sheet pile

3. Structural Design of Cold-bent Steel Sheet Pile

3.1 General Requirements

3.1.1 Scope of Application of Cold-bent Steel Sheet Pile

Construction of cold-bent steel sheet pile shows the following characteristics:

(1) The construction is simple and has no need of large-sized construction equipment;

(2) Rapid construction is possible; if that is the case, the construction period would be remarkably reduced;

(3) The section and length of cold-bent steel sheet pile can be changed according to foundation status, and this provides a possibility of high-efficiency and economical design;

(4) The relatively light wall body that is different from stand-alone type structure is highly conducive to aseismatic design.

Thanks to the above-noted advantages of cold-bent steel sheet pile, it has been widely used for waterfront construction project, bank-protection works and temporary works etc. as shown in Table 3.1.1:

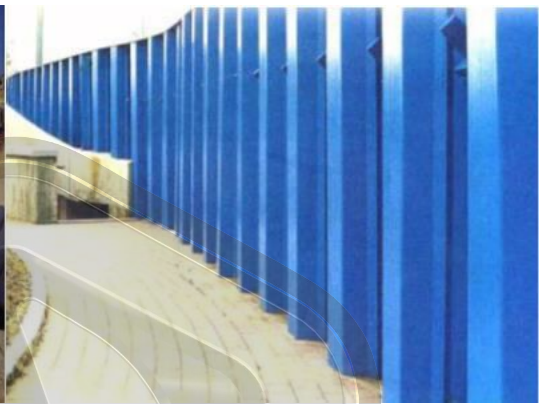
Table 3.1.1 Scope of application of cold-bent steel sheet pile

Application	Used for permanent structures	Water front and shipping docks
		Levee revetment
		Guard wall, retaining wall
		Wave breaker
		Diversion dike
		Anchorage spud
		Ship yard, dock yard
		Sluice, sluiceway
		Slope protection foot, resistance to local erosion, seepproof
	Used for temporary structures	Used for temporary structures
		Temporary bank revetment
		Cofferdam
		Temporary island building
	Emergency rescue and disaster relief	Emergency rescue and disaster relief
		Landslip and slump settlement prevention
		Resistance to drift sand

Figure 3.1.1 shows the examples of engineering applications of cold-bent steel sheet pile

(a) Dock (b) Ship yard

(c) Foundation pit (d) Road

Fig. 3.1.1 Application of cold-bent steel sheet pile

3.1.2 Classification of Cold-bent Steel Sheet Piles

The section of cold-bent steel sheet pile may be U-shaped, Z-shaped or straight.

(1) U-shaped cold-bent steel sheet pile

U-shaped cold-bent steel sheet pile that features large section modulus (W=600~3200 cm^3/m) is suitable for small and medium projects that bear relatively small earth (water) pressure, especially for temporary works. Depending on the different geological conditions, vibratory hammer with corresponding power is usually used for construction. In company with the progressive development of smelting technology, the width of single-piece U-shaped cold-bent steel sheet pile reached up to 750mm after the year 2002, which has brought about the acceleration of construction.

Grand U-shaped cold-bent steel sheet pile has the following merits:

①This product is wider than common hot-rolled steel sheet piles, and this means the reduction in time consumption of hoisting and pile sinking;

②The increase in width brings about the reduction of interlocks of walls per linear meter; In this way, the water sealing performance of wall body is directly improved, and the engineering cost is reduced;

③The increase in height and use of high grade ensures the outstanding statics characteristics and reduces the weight of wall per linear meter;

④The uniform thickness of section board ensures the favorable rigidity of pile sinking;

⑤The high-tensile steel and advanced production equipment insures the performance of cold-bent steel sheet pile;

⑥Dedicated interlock designed through finite element analysis software ensures the repeated use of cold-bent steel sheet pile;

⑦The symmetrical structure and the provided rotation angle is conducive to the correction of construction deviation

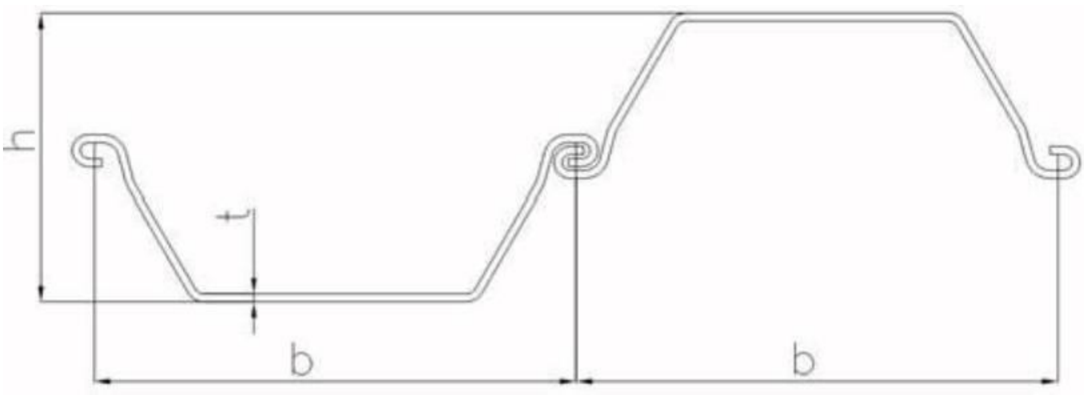


Fig. 3.1.2-1 Schematic diagram of U-shaped cold-bent steel sheet pile

(2) Z-shaped cold-bent steel sheet pile

Z-shaped cold-bent steel sheet pile has a large section modulus (W=1200~5015cm^3/m) and is suitable for large, medium and small projects that bear relatively large earth (water) pressure. Based on the characteristics of Z-shaped cold-bent steel sheet pile, two pieces are usually combined into one group for driving. Although its construction procedure is slightly longer and technical difficulty is higher than that of U-shaped cold bent steel sheet pile, its overall construction efficiency is higher since the group composed of two pieces of piles may have a width of 1160~1400mm, nearly 2~3 times the width of single piece of U-shaped cold-bent steel sheet pile; hence, Z-shaped cold-bent steel sheet piles have been widely used for a lot of dock projects of China that has a requirement for land-based area. Normally, the construction method is "pile pitching through vibration, and then pile sinking through hammering".

The most essential mechanical property of Z-shaped cold-bent steel sheet pile is the continuity of web and the symmetrical distribution of interlocks at specified positions at both sides of neutral axis, both have positive effect on section modulus of cold-bent steel sheet pile.

Grand Z-shaped cold-bent steel sheet pile has the following merits:

① Flexible design and resultant relatively high section modulus and mass ratio;

② Increase in moment inertia of section, improvement of bending rigidity of overall piling wall, and effective reduction of displacement deformation;

③ GPZ cold-bent steel sheet pile is wider than conventional cold-bent steel sheet piles, and this means the effective reduction of time consumption of hoisting and piling;

④ The use of high tensile steel remarkably improves the bending strength of cold-bent steel sheet pile wall;

⑤ The increase in section width brings about the reduction of interlocks of sheet-pile wall, which directly improves the water sealing performance of sheet-pile wall.

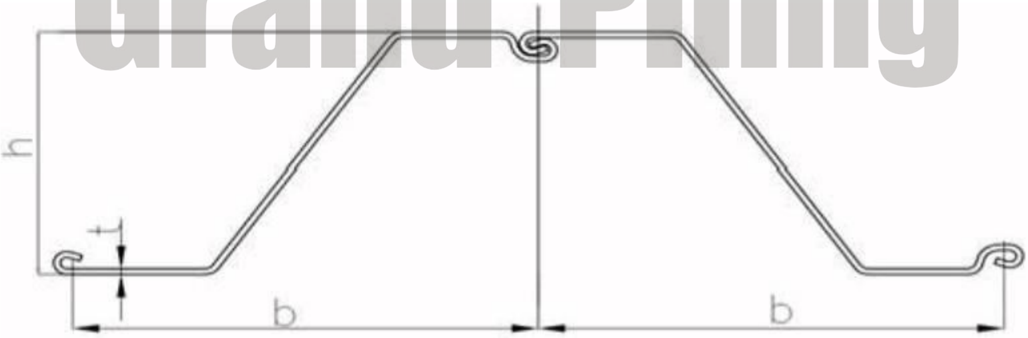


Fig. 3.1.2-2 Schematic diagram of section of Z-shaped cold-bent steel sheet pile

(3) Straight cold-bent steel sheet pile

Straight cold-bent steel sheet pile has a relatively small section modulus; however, its interlock has a strong resistance to horizontal tension that is up to 5500 kN/m; straight cold-bent steel sheet pile is applicable for large round built island cofferdam that bears horizontal tensile in horizontal direction and suitable for grid-shaped cold-bent steel sheet pile gravity wharf project; the construction of straight cold-bent steel sheet pile is very convenient.

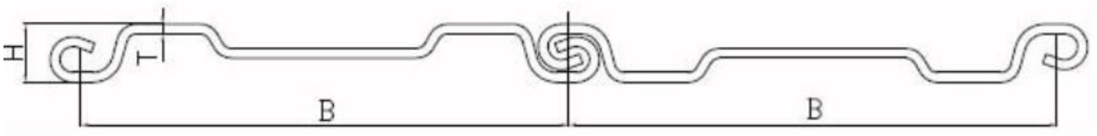


Fig. 3.1.2-3 Schematic diagram of section of U-shaped cold-bent steel sheet pile

(4) L-shaped and S-shaped (light) cold-bent steel sheet pile

The interlock of both L-shaped and S-shaped cold-bent steel sheet pile is located at one side of cold-bent steel sheet pile wall. Compared with U-shaped and Z-shaped cold-bent steel sheet piles, L-shaped and S-shaped light-duty cold-bent steel sheet piles feature light section, small space occupancy of piling wall, the same orientation of interlocks and convenient construction etc. and are applicable to small-range excavation projects such as municipal works.

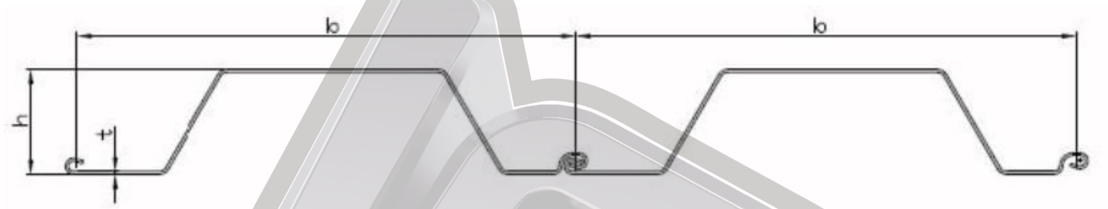


Fig. 3.1.2-4 Schematic diagram of section of L-shaped and S-shaped (light) cold-bent steel sheet pile

3.2 Calculation of Water Level and External Force Applied on Sheet-pile Wall Structure

3.2.1 Calculation of Water Level

The calculated water levels for design of sheet-pile wall are divided into design high water level, design low water level and extremely low water level; but under normal circumstances, only design low water level or extremely low water level are used for calculation since the lower the water level is, the higher the active earth pressure and surplus water pressure would be, and the more adverse effect would be on sheet pile wall.

In the design of sheet-pile wall as wave breaker, design high water level is the most unfavorable water level; however, this is not unconditional since anchor rod position may largely affect internal force calculation of sheet pile.

3.2.2 Surplus Water Pressure

(1) Hydrostatic pressure

When material that is more coarse than fine sand is back filled behind sheet pile wall and the sheet-pile wall is furnished with tidal influx resisting drainage hole behind which an inverted filter layer is fabricated, surplus water pressure could be left out of account; in case of backfilling of fine-grain material behind sheet-pile wall and the lack of drainage holes, residual head could be 1/3~1/2 of mean tide range.

Surplus water pressure distribution pattern is as shown in Fig. 3.2.2-1.

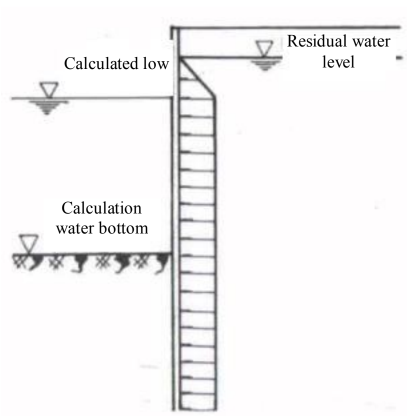


Fig. 3.2.2-1 Surplus water pressure

(2) Hydrodynamic pressure at earthquake

① When part or the whole of inner space of structure and facility in water is filled with water, the hydrodynamic pressure at earthquake is given by the following equation:

$$p_{dw} = \pm\frac{7}{8}k\gamma_w\sqrt{Hy} \qquad (3.2.2\text{-}1)$$

Where: p_{dw} means hydrodynamic pressure (kN/m^2);

k represents earthquake intensity;

γ_w means bulk density of water (kN/m^3);

H means water depth (m)

y means vertical distance between water surface and hydrodynamic pressure calculation point (m)

② The resultant force of hydrodynamic pressure and the position of its application point are given by the following equation:

$$\begin{cases} p_{dw} = \pm\dfrac{7}{12}k\gamma_w H^2 \\ h_{dw} = \dfrac{3}{5}H \end{cases} \qquad (3.2.2\text{-}2)$$

Where: p_{dw} means resultant force of hydrodynamic pressure (kN/m);

h_{dw} means distance between water surface and hydrodynamic pressure concurrence point (m)

Hydrodynamic pressure generated by water in hollow space of cuboid is as shown in the figure below:

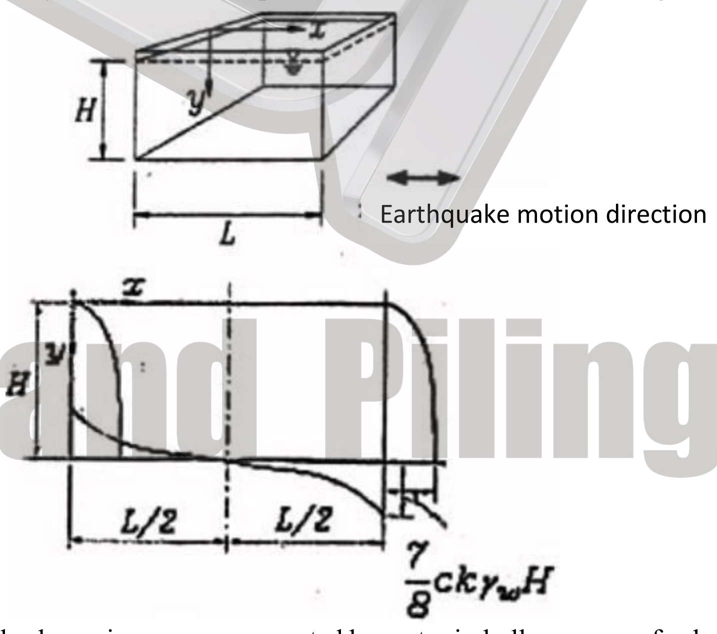
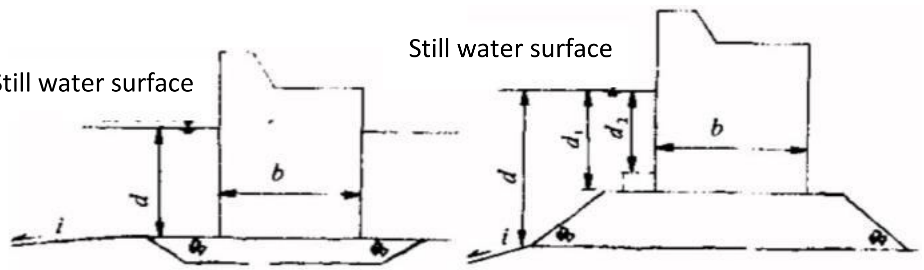


Fig. 3.2.2-2 Hydrodynamic pressure generated by water in hollow space of cuboid

In this figure, c means the correction coefficient. When $L/H<1.5$, $c = L/(1.5H)$; when $L/H\geq1.5$, $c=1.0$.

3.2.3 Wave Force

Waves that act on the straight-wall building as shown in Fig. 3.2.3-1 are divided into vertical wave, far broken wave and near broken wave. The wave forms could be identified as per Table3.2.3-1.



(a) Structure with concealed foundation bed and straight wall (b) Structure with open foundation bed and straight wall

Fig. 3.2.3-1 Structure with straight wall

The occurrence of vertical wave at structure with straight wall shall meet the requirements of Table 3.2.3-1 and meet the condition that "the line of wave crests is roughly parallel with structure, and the structure is longer than the length of a wave. In addition, attentions shall be paid to the following points:

(1) In case of a large steepness ($H/L > 1/14$) of progressive wave, broken vertical wave may occur in front of the wall;

(2) When the water depth d in front of structure with concealed foundation bed or low foundation bed and straight wall is less than $2H$ and bottom slope i is more than 1/10, near broken wave may occur in front of the wall; in such a case, model test shall be conducted to determine wave form and force;

(3) When open foundation bed is furnished with shoulder pad square of which the width is larger than wave height H, the water depth d_2 on foundation bed shall be replaced by that on square d_1 so as to determine the wave form and force.

Table 3.2.3-1 Wave form in front of structure with straight wall

Type of foundation bed	Occurrence condition	Wave form
Concealed foundation bed and low foundation bed d_1 2 d 3	T g d 8, d $2H$ T g d 8, d $1.8H$	Far broken wave
	T g d 8, d $2H$, i 1 10 T g d 8, d $1.8H$, i 1 10	Vertical wave
Medium foundation bed 1 d_1 3 2 d 3	d_1 $1.8H$	Near broken wave
	d_1 $1.8H$	Vertical wave
High foundation bed d_1 1 d 3	d_1 $1.5H$	Vertical wave
	d_1 $1.5H$	Near broken wave

Note: $\overline{T}$ means average wave period (s); H means the height (m) of progressive wave at which the structure is located; L is wave length (m); d means water depth(m) in front of structure; d_1 means water depth(m) on foundation bed; i means the slope at water bottom in front of structure

3.2.3.1 Vertical wave

The acting force of vertical wave on structure with straight wall could be determined by the following rules. In case of wave overtopping as shown in Fig. 3.2.3.1-1, the acting force of vertical wave could still be calculated respectively according to different conditions of *d/L*, but the wave force of the part over the top shall be deducted.

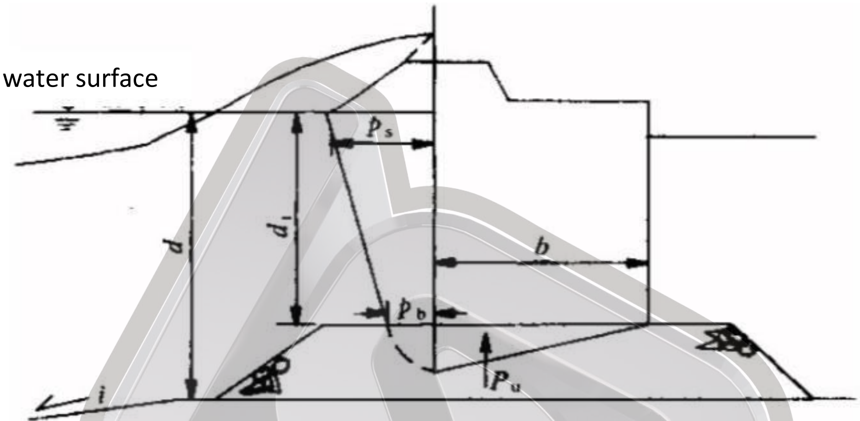


Fig. 3.2.3.1-1 Vertical wave pressure distribution in case of wave overtopping

(1) When $d \geq 1.8H$ and $d/L = 0.05 \sim 0.12$, the acting force of vertical wave under the effect of wave peak as shown in Fig. 3.2.3.1-2 is given by the following equation:

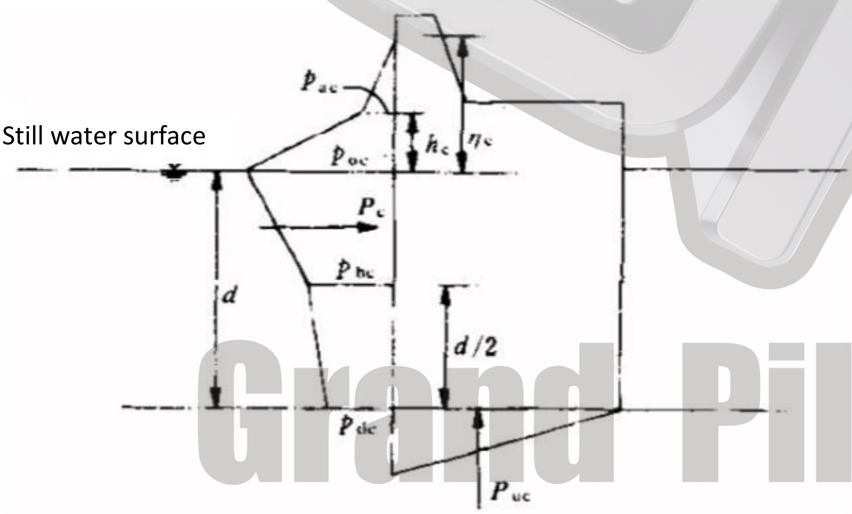


Fig. 3.2.3.1-2 Vertical wave pressure distribution under effect of wave peak

① Elevation of wave surface is give by the following equations:

$$\begin{cases} \dfrac{\eta_c}{d} = B_\eta (H/d)^m \\ B_\eta = 2.3104 - 2.5907 T_*^{-0.5941} \\ m = T_* /(0.00913 T_*^2 + 0.636 T_* + 1.2515) \\ T_* = \overline{T}\sqrt{g/d} \end{cases} \quad (3.2.3.1\text{-}1)$$

Where: η_c is wave surface elevation (m);

B_η and m mean coefficient

T_* is non-dimensional period (s).

② The pressure intensity of wave at wall surface h_c above still water surface is given by the following equation:

$$\begin{cases} \dfrac{h_c}{d} = \dfrac{2\eta_c / d}{n+2} \\ \dfrac{p_{ac}}{\gamma d} = \dfrac{p_{oc}}{\gamma d} \dfrac{2}{(n+1)(n+2)} \\ n = \max[0.636618 + 4.23264(H/d)^{1.67}, 1.0] \end{cases}$$

(3.2.3.1-2)

Where: h_c means the position (m) of acting point of wave pressure p_{ac} above still water surface;

n means the index of pressure distribution curve above still water surface, and its value takes the larger one from the two numbers in equation;

p_{ac} means the wall surface wave pressure (kPa) corresponding to h_c;

γ means specific weight of water (kN/m3);

p_{oc} is the wave pressure (kPa) at still water surface.

③ p_{oc} and the wave pressures at other characteristic points on wall surface are given by the following equation:

$$\frac{p}{\gamma d} = A_p + B_p (H/d)^q$$

(3.2.3.1-3)

Where: coefficients A_p, B_p and q are determined as per Table 3.2.3.1-1. When calculation is conducted according to Table 3.2.3.1-1, if p_{bc}>p_{oc}, it should assumed that $p_{bc} = p_{oc}$.

Table 3.2.3.1-1 Coefficients A_p, B_p and q

	Formula		A_1、B_1、a	A_2、B_2、b	、 、c
Wave peak	p_{oc} d	A A $A T$	0.02901	-0.00011	2.14082
	p_{bc} d		0.14574	-0.02403	0.91976
	p_{dc} d		-0.18	-0.000153	2.54341
Wave peak	p_{oc} d	B B $B T$	1.31427	-1.20064	-0.6736
	p_{bc} d		-3.07372	2.91585	0.11046
	p_{dc} d		-0.03291	0.17453	0.65074
	p_{oc} d		0.03765	0.46443	2.91698

	$\frac{p_{bc}}{d}$		0.06220	1.32641	-2.97557
	$\frac{p_{dc}}{d}$		0.28649	-3.86766	38.4195

④ The total horizontal wave force on wall body per unit length is given by the following equation:

$$\frac{P_c}{\gamma d^2} = \frac{1}{4}[2\frac{p_{ac}}{\gamma d}\frac{\eta_c}{d} + \frac{p_{oc}}{\gamma d}\left(1+\frac{2h_c}{d}\right) + \frac{2p_{bc}}{\gamma d} + \frac{p_{dc}}{\gamma d}] \quad (3.2.3.1\text{-}4)$$

Where: P_c means the total horizontal wave force on per unit length of wall body (kN/m)

⑤ Total horizontal wave moment on unit length of wall body is given by the following equation:

$$\frac{M_c}{\gamma d^3} = \frac{1}{2}\frac{p_{ac}}{\gamma d}\frac{\eta_c}{d}[1+\frac{1}{3}\left(\frac{\eta_c}{d}+\frac{h_c}{d}\right)] + \frac{1}{24}\frac{p_{oc}}{\gamma d}[5+\frac{12h_c}{d}+4\left(\frac{h_c}{d}\right)^2] + \frac{1}{4}\frac{p_{bc}}{\gamma d} + \frac{1}{24}\frac{p_{dc}}{\gamma d} \quad (3.2.3.1\text{-}5)$$

Where: M_c means total horizontal wave moment on per unit length of wall body (kN·m/m).

⑥ Wave lift at unit length of wall bottom face is given by the equation below:

$$p_{uc} = \frac{p_{dc} b}{2} \quad (3.2.3.1\text{-}6)$$

Where: p_{uc} means the wave lift at unit length of wall bottom;

b is the bottom width (m) of straight wall.

(2) When $d \geq 1.8H$ and $d/L = 0.05 \sim 0.12$, the acting force of vertical wave under the effect of wave trough as shown in Fig. 3.2.3.1-3 is given by the equation below:

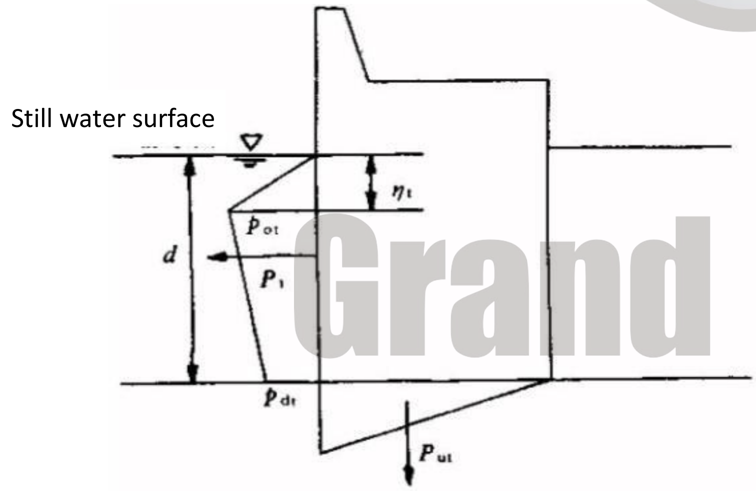


Fig. 3.2.3.1-3 Vertical wave pressure distribution under the effect of wave trough

① Elevations of wave trough and surface are given by the formula below:

$$\frac{\eta_t}{d} = A_p + B_p(H/d)^q \quad (3.2.3.1\text{-}7)$$

Where: η_t is elevation (m) of wave trough and surface

Coefficients A_p, B_p and q are determined based on values of $p_{ot}/\gamma d$ as shown in Table $p_{ot}/\gamma d$

Table 3.2.3.1-2 Coefficients A_p, B_p and q (under effect of wave trough)

		Formula	$A_1 B_1 a$	$A_2 B_2 b$	$\alpha\ \beta\ c$
Wave trough	$\frac{p_{ot}}{\gamma d}$	$A_p = A_1 + A_2 T_*^{\alpha}$	0.0397	-0.00018	1.95
$\frac{p_{dt}}{\gamma d}$		$A_p = 0.1 - A_1 T_*^{\alpha} e^{A_2 T_*}$	1.687	0.16894 -3.06115	-2.0195
Wave trough	$\frac{p_{ot}}{\gamma d}$	$B_p = B_1 + B_2 T_*^{\beta}$	0.98222		-0.2848
$\frac{p_{dt}}{\gamma d}$		-2.19707	0.92802	0.2350	
Wave trough	$\frac{p_{ot}}{\gamma d}$	$q = aT_*^{b} e^{cT_*}$ 20.1565	2.599	-0.8679	0.07092
$\frac{p_{dt}}{\gamma d}$			-1.9723	0.13329	

② Wave pressure at each characteristic point on water surface is given by the following equation:

$$\frac{p}{\gamma d} = A_p + B_p (H/d)^q \qquad (3.2.3.1\text{-}8)$$

Where : p means the wave pressure (kPa) at each characteristic point on wall surface;

Coefficients A_p, B_p and q are determined according to Table 3.2.3.1-2; when $p_{dt} > p_{ot}$, it should be assumed that $p_{dt} = p_{ot}$.

③ Total horizontal wave force per unit length of wall body is given by the equation below:

$$\frac{p_t}{\gamma d^2} = \frac{1}{2}\left[\frac{p_{ct}}{\gamma d} + \frac{p_{dt}}{\gamma d}\left(1 + \frac{\eta_t}{d}\right)\right] \qquad (3.2.3.1\text{-}9)$$

Where: p_t is the total horizontal wav force per unit length of wall body (kN/m).

④ Downward wave force per unit length of wall bottom face is given by the equation below:

$$p_{ut} = \frac{p_{dt} b}{2} \qquad (3.2.3.1\text{-}10)$$

Where: p_{ut} means the downward wave force per unit length of wall bottom face (kN/m)

(3) When $d \geq 1.8H$, $0.139 > d/L \geq 0.12$ and $8 < T_* \leq 9$, the wave force, wave moment, wave pressure intensity and wave surface elevation etc. are given by the equation below:

$$X_{T_*} = X_{T_*=8} - (X_{T_*=8} - X_{T_*=9})(T_* - 8) \qquad (3.2.3.1\text{-}11)$$

Where: X_{T_*} represents the value of wave force, wave moment, wave pressure intensity and wave surface elevation;

$X_{T_*=8}$ means the value calculated according to (4) and (5) by assuming that $T_* = 8$ and taking the actual wave condition H/d;

$X_{T_*=9}$ means the value calculated according to (1) and (2) by assuming that $X_{T_*} = 9$ and taking the actual wave condition H/d;

T_* means the $\overline{T}\sqrt{g/d}$ under actual condition of wave

(4) When $H/L \geq 1/30$ and $d/L = 0.139 \sim 0.2$, the acting force of vertical wave under the effect of wave peak as shown in Fig. 3.2.3.1-4 is given by the equation below:

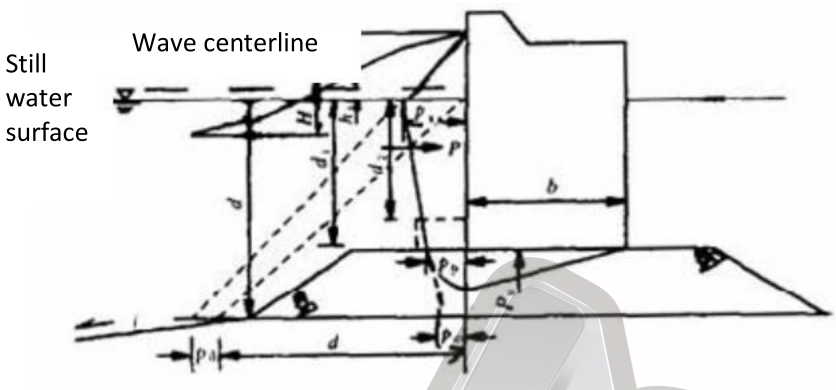


Fig. 3.2.3.1-4 Vertical wave pressure distribution under the effect of wave peak ($d/L = 0.139 \sim 0.2$)

① The height of wave centerline above still water surface (i.e. over-height) is given by equation 3.2.3.1-12 and could also be determined according to Fig. 3.2.3.1-5:

$$h_s = \frac{\pi H^2}{L} cth \frac{2\pi d}{L} \tag{3.2.3.1-12}$$

Where: h_s means the height of wave centerline above still water surface (m).

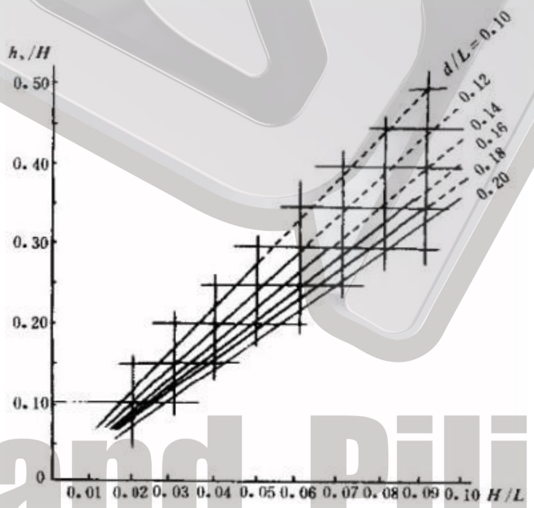


Fig. 3.2.3.1-5 Value of height h_s of wave above still water surface

② The wave pressure intensity at the point $h_s + H$ above still water surface is zero.

③Wave pressure intensity at water bottom is given by equation 3.2.3.1-13 and can also be determined according to Fig. 3.2.3.1-6:

$$p_d = \frac{\gamma H}{ch \frac{2\pi d}{L}} \tag{2.2.1-13}$$

Where: p_d means the wave pressure intensity (kPa) at water bottom;

γ means the specific weight of water (kN/m3)

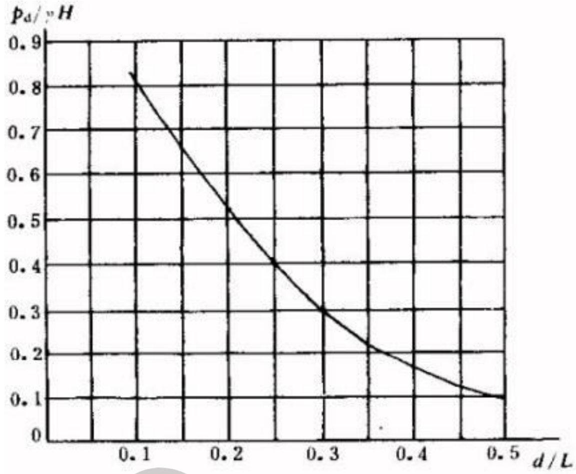


Fig. 3.2.3.1-6 Wave pressure intensity p_d at water bottom

④ Wave pressure intensity at still water surface is given by equation 3.2.3.1-14 and can also be determined according to Fig. 3.2.3.1-7:

$$p_s = (p_d + \gamma d)\left(\frac{H + h_s}{d + H + h_s}\right) \quad (3.2.3.1\text{-}14)$$

Where: p_s represents the wave pressure intensity (kPa) at still water surface.

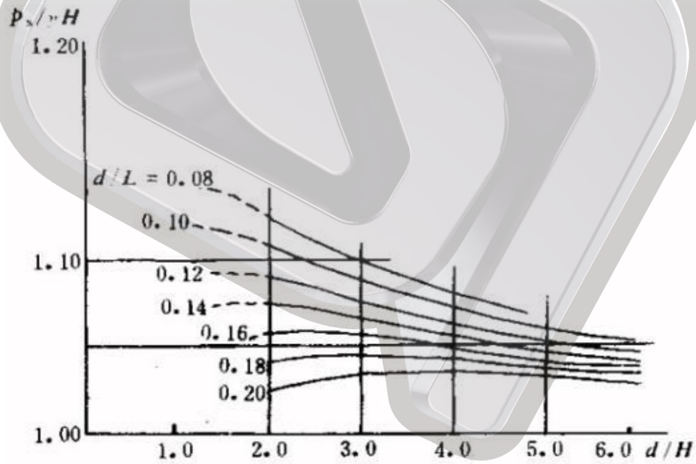


Fig. 3.2.3.1-7 Wave pressure intensity p_s at still water surface

⑤ Wave pressure intensity at wall bottom is given by the following equation:

$$p_b = p_s - (p_s - p_d)\frac{d_1}{d} \quad (3.2.3.1\text{-}15)$$

Where: p_b means the wave pressure intensity (kPa) at wall bottom

⑥ The wave pressure intensity is distributed in a linear manner below and above still water surface.

⑦ Total wave force per unit length of wall body is given by the following equation:

$$P = \frac{(H + h_s + d_1)(p_b + \gamma d_1) - \gamma d_1^2}{2} \quad (3.2.3.1\text{-}16)$$

Where: P is the total wave force per unit length of wall body (kN/m)

⑧ Wave lift at wall bottom surface is given by the following equation:

$$P_u = \frac{bp_b}{2} \quad (3.2.3.1\text{-}17)$$

Where: P_u means the wave lift at wall bottom surface (kN/m);

b is wall bottom width (m).

(5) When $H/L \geq 1/30$ and $d/L = 0.139 \sim 0.2$, the acting force of vertical wave under the effect of wave trough as shown in Fig. 3.2.3.1-8 is given by the equation below:

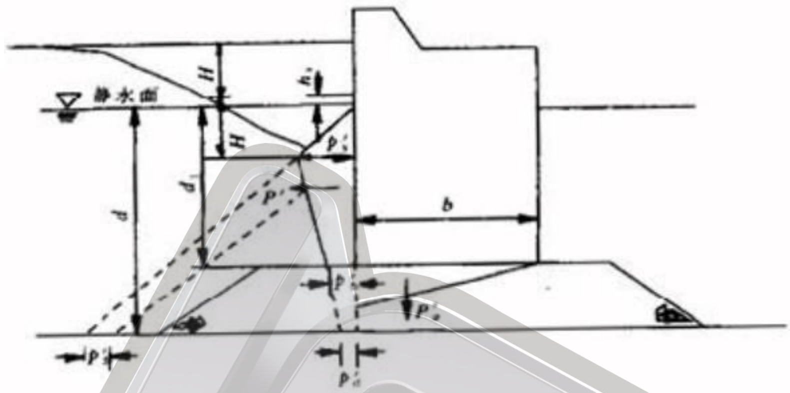


Fig. 3.2.3.1-8 Vertical wave pressure distribution under effect of wave trough

① Wave pressure intensity at water bottom is given by the following equation:

$$p_d' = \frac{\gamma H}{ch\frac{2\pi d}{L}} \quad (3.2.3.1\text{-}18)$$

Where: p_d' means wave pressure intensity at water bottom (kPa)

② Wave pressure intensity at still water surface is zero.

③ Wave pressure intensity at a depth of $H - h_s$ under still water surface is given by the equation below:

$$p_s' = \gamma(H - h_s) \quad (3.2.3.1\text{-}19)$$

Where: p_s' means the wave pressure intensity (kPa) at the depth of $H - h_s$ under still water surface.

④ Wave pressure intensity at wall bottom is given by the following equation:

$$p_b' = p_s' - (p_s' - p_d')\frac{d_1 + h_s - H}{d + h_s - H} \quad (3.2.3.1\text{-}20)$$

Where: p_b' is the wave pressure intensity (kPa) at wall bottom

⑤ Total wave force per unit length of wall body is given by the equation below:

$$P' = \frac{\gamma d_1^2 - (d_1 + h_s - H)(\gamma d_1 - p_b')}{2} \quad (3.2.3.1\text{-}21)$$

Where: P' means the total wave force (kN/m) per unit length of wall body.

⑥ Downward wave force per unit length of wall bottom surface is given by the following equation:

$$p_u' = \frac{bp_b'}{2} \quad (3.2.3.1\text{-}22)$$

Where: P_u' is the wave force (kN/m) per unit length of wall bottom surface.

(6) When $H/L \geq 1/30$ and $0.2 < d/L < 0.5$, the acting force of vertical wave under the effect of wave peak as shown in Fig. 3.2.3.1-9 is given by the equation below:

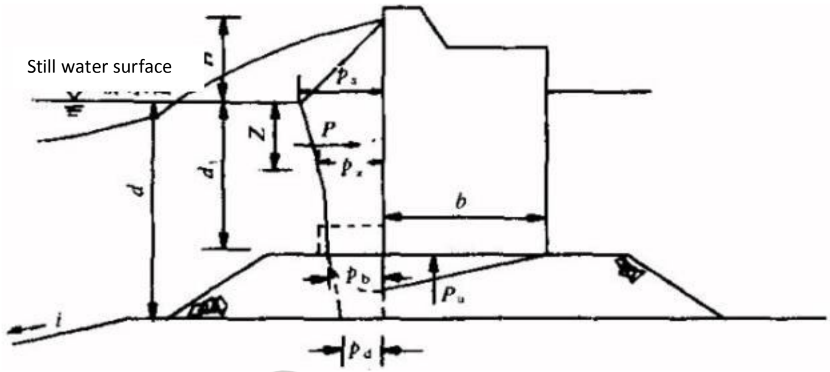


Fig. 3.2.3.1-9 Vertical wave pressure distribution under the effect of wave peak ($0.2 < d/L < 0.5$)

① Wave pressure intensity at the point H above still water surface is zero.

② Wave pressure intensity at still water surface is given by the following equation:

$$p_s = \gamma H \quad (3.2.3.1\text{-}23)$$

③ Wave pressure intensity is distributed in a linear manner above still water surface.

④ Wave pressure intensity at the depth of Z under still water surface is given by the equation below:

$$p_z = \gamma H \frac{ch\frac{2\pi(d-z)}{L}}{ch\frac{2\pi d}{L}} \quad (3.2.3.1\text{-}24)$$

Where: p_z means the wave pressure intensity (kPa) at the depth of Z under still water surface;

Z means the depth (m) under still water surface.

⑤ Wave pressure intensity at water bottom is given by equation 3.2.3.1-13.

⑥ Wave pressure intensity at wall bottom is given by the following equation:

$$p_b = \gamma H \frac{ch\frac{2\pi(d-d_1)}{L}}{ch\frac{2\pi d}{L}} \quad (3.2.3.1\text{-}25)$$

⑦ Total wave force per unit length of wall body is given by the equation below:

$$P = \frac{1}{2}\gamma H^2 + \frac{\gamma HL}{2\pi}\left[th\frac{2\pi d}{L} - \frac{sh\frac{2\pi(d-d_1)}{L}}{ch\frac{2\pi d}{L}}\right] \quad (3.2.3.1\text{-}26)$$

⑧ When drawing the diagram of wave pressure distribution at wall surface, pressure intensity values of at least five points could be used, including the three points at which the pressure is 0, p_s and p_b; in case of concealed foundation bed, p_b should be changed to p_d.

⑨Wave lift at wall bottom surface is given by equation 3.2.3.1-17.

(7) Acting force of vertical wave under effect of wave peak is calculated through relevant equations as stated in (4) and (5).

When $d/L \geq 0.5$, the wave pressure intensity at the depth of $Z = L/2$ under still water surface could be taken as zero; but $L/2$ should be taken in equation 3.2.3.1-24 under the effect of wave peak and equation 3.2.3.1-12 under the effect of wave trough.

3.2.3.2 Far broken wave

(1) Wave force under the effect of wave peak as shown in Fig. 3.2.3.2-1 could be given by the equation below:

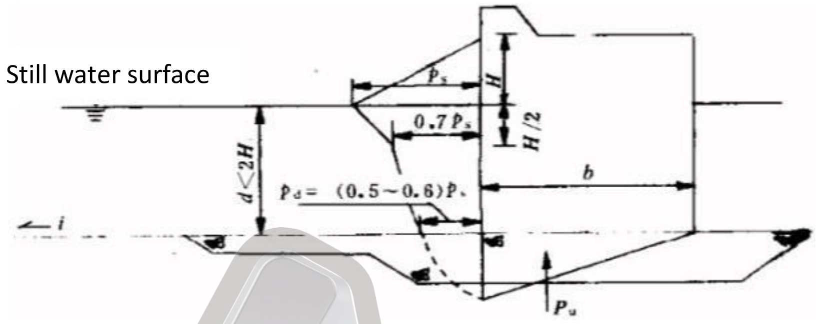


Fig. 3.2.3.2-1 Wave pressure distribution of far broken wave

① Wave pressure intensity at the point that is H above still water surface is zero.

② Wave pressure intensity at still water surface is given by the following equation:

$$p_s = \gamma K_1 K_2 H \quad (3.2.3.2\text{-}1)$$

Where: K_1 is a coefficient that is the function of underwater gradient i

K_2 is a coefficient that is the function of L/H

③ Coefficients K_1 and K_2 are used according to Tables 3.2.3.2-1 and 3.2.3.2-2 respectively.

Table 3.2.3.2-1 Coefficient K_1

Bottom slope $\bar{i}$	1/10	1/25	1/40	1/50	1/60	1/80	≤1/100
K_1	1.89	1.54	1.40	1.37	1.33	1.29	1.25

Note: bottom slope $\bar{i}$ may take the mean value within a certain range of distance in front of structure.

Table 3.2.3.2-2 Coefficient K_2

L/H	14	15	16	17	18	19	20	21	22
K_2	1.01	1.06	1.12	1.17	1.21	1.26	1.30	1.34	1.37
L/H	23	24	25	26	27	28	29	30	
K_2	1.41	1.44	1.46	1.49	1.50	1.52	1.54	1.55	

④ Wave pressure intensity changes in a linear manner above still water surface.

⑤ $Z = H/2$ Wave pressure intensity at the depth of $Z = H/2$ under still water surface

$$p_z = 0.7 p_s \quad (3.2.3.2\text{-}2)$$

⑥ Wave pressure intensity at water bottom is given by the equation below:

when $d/H \le 1.7$,

$$p_d = 0.6 p_s \quad (3.2.3.2\text{-}3)$$

When $d/H > 1.7$,

$$p_d = 0.5 p_s \quad (3.2.3.2\text{-}4)$$

⑦ Wave lift at wall bottom surface is given by the equation below:

$$P_u = \mu \frac{b p_d}{2} \quad (3.2.3.2\text{-}5)$$

Where: μ means the reduction coefficient of wave lift distribution diagram and takes 0.7

(2) Wave force under the effect of wave trough as shown in Fig. 3.2.3.2-2 is given by the following equation:

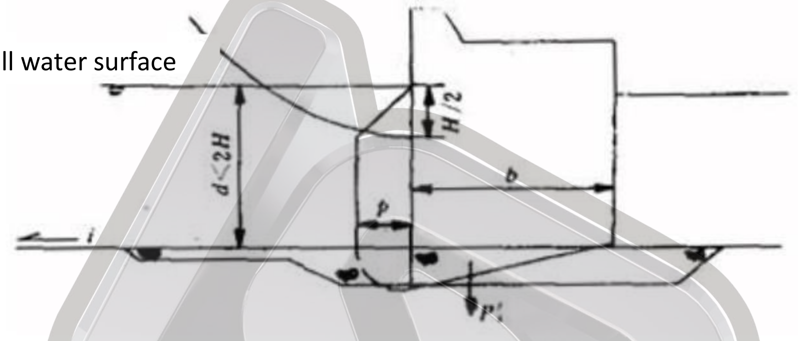


Fig. 3.2.3.2-2 Wave pressure distribution under the effect of wave trough

① Wave pressure intensity at still water surface is zero.

② The wave pressure intensity from the depth $Z = H/2$ to water bottom is given by the equation below:

$$p = 0.5\gamma H \quad (3.2.3.2\text{-}6)$$

③ Downward wave force at wall bottom surface is given by the equation below:

$$P_u' = \frac{bp}{2} \quad (3.2.3.2\text{-}7)$$

3.2.3.3 Near broken wave

When $d_1 \ge 0.6H$, the wave force of near broken wave on structure with straight wall as shown in Fig. 3.2.3.3 under the effect of wave peak could be determined according to the rules below.

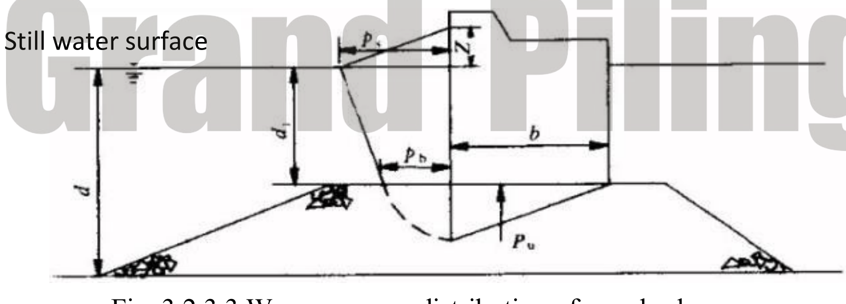


Fig. 3.2.3.3 Wave pressure distribution of near broken wave

① Wave pressure intensity at the point that is Z above still water surface is zero, and Z is given by the equation below:

$$Z = \left(0.27 + 0.53\frac{d_1}{H}\right)H \quad (3.2.3.3\text{-}1)$$

② Wave pressure intensity at still water surface is given by the equation below:

When $\frac{1}{3} < \frac{d_1}{d} \le \frac{2}{3}$:

$$p_s = 1.25\gamma H\left(1.8\frac{H}{d_1} - 0.16\right)\left(1 - 0.13\frac{H}{d_1}\right) \quad (3.2.3.3\text{-}2)$$

When $\frac{1}{4} \le \frac{d_1}{d} \le \frac{1}{3}$:

$$p_s = 1.25\gamma H\left[\left(13.9 - 36.4\frac{d_1}{d}\right)\left(\frac{H}{d_1} - 0.67\right) + 1.03\right]\left(1 - 0.13\frac{H}{d_1}\right) \quad (3.2.3.3\text{-}3)$$

③ Wave pressure intensity at wall bottom is given by the equation below:

$$p_b = 0.6\,p_s \quad (3.2.3.3\text{-}4)$$

④ Total wave force per unit length of wall body is given by the following equation:

When $\frac{1}{3} < \frac{d_1}{d} \le \frac{2}{3}$:

$$P = 1.25\gamma H d_1\left(1.9\frac{H}{d_1} - 0.17\right) \quad (3.2.3.3\text{-}5)$$

When $\frac{1}{4} \le \frac{d_1}{d} \le \frac{1}{3}$:

$$P = 1.25\gamma H d_1\left[\left(14.8 - 38.8\frac{d_1}{d}\right)\left(\frac{H}{d_1} - 0.67\right) + 1.1\right] \quad (3.2.3.3\text{-}6)$$

⑤ Wave lift at wall bottom surface is given by the following equation:

$$P_u = \mu\frac{b p_b}{2} \quad (3.2.3.3\text{-}7)$$

Where: μ is the reduction coefficient of wave lift distribution diagram and takes 0.7.

3.2.4 Earth Pressure

Earth pressure could be calculated according to earth pressure calculation charts of Coulomb, Rankine and Terzaghi. At present, most of the design standards recommend Coulomb earth pressure calculation formula as shown in Fig. 3.2.4-1:

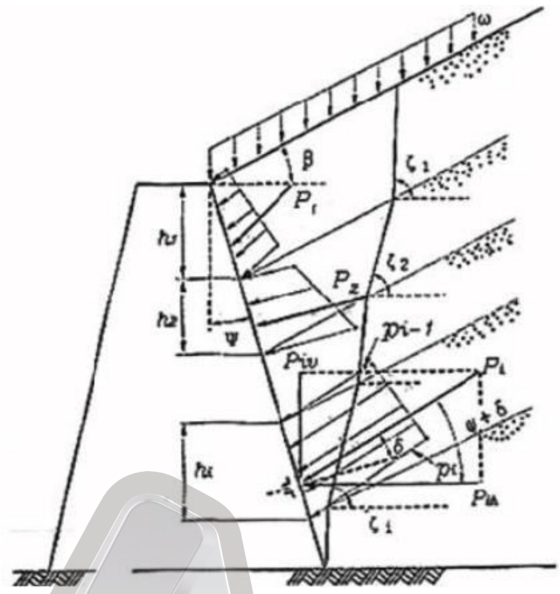


Fig. 3.2.4-1 Earth pressure calculation chart

(1) Earth pressure in normal times

① Pressure of sandy soil

a Active earth pressure

Earth pressure intensity on wall surface is given by equation 3.2.4-1, and the angle between failure surface and horizontal plane is calculated through equation 3.2.4-2.

$$P_{ai} = K_{ai}[\sum \gamma_i h_i + \frac{\omega \cos\psi}{\cos(\psi-\beta)}]\cos\psi \qquad (3.2.4\text{-}1)$$

$$\cot(\zeta_i-\beta) = -\tan(\phi_i+\delta+\psi-\beta) + \sec(\phi_i+\delta+\psi-\beta)\sqrt{\frac{\cos(\psi+\delta)\sin(\phi_i+\delta)}{\cos(\psi-\beta)\sin(\phi_i-\beta)}} \qquad (3.2.4\text{-}2)$$

$$K_{ai} = \frac{\cos^2(\phi_i-\psi)}{\cos^2\psi\cdot\cos(\delta+\psi)[1+\sqrt{\frac{\sin(\phi_i+\delta)\sin(\phi_i-\beta)}{\cos(\delta+\psi)\cos(\psi-\beta)}}]^2}$$

Where:

b Passive earth pressure

Earth pressure intensity on wall surface is given by equation 3.2.4-3, and the angle between failure surface and horizontal plane is calculated through equation 3.2.4-4.

$$P_{pi} = K_{pi}[\sum \gamma_i h_i + \frac{\omega \cos\psi}{\cos(\psi-\beta)}]\cos\psi \qquad (3.2.4\text{-}3)$$

$$\cot(\zeta_i-\beta) = \tan(\phi_i-\delta-\psi+\beta) + \sec(\phi_i-\delta-\psi+\beta)\sqrt{\frac{\cos(\psi+\delta)\sin(\phi_i-\delta)}{\cos(\psi-\beta)\sin(\phi_i+\beta)}} \qquad (3.2.4\text{-}4)$$

$$K_{pi} = \frac{\cos^2(\phi_i+\psi)}{\cos^2\psi\cdot\cos(\delta+\psi)[1-\sqrt{\frac{\sin(\phi_i-\delta)\sin(\phi_i+\beta)}{\cos(\delta+\psi)\cos(\psi-\beta)}}]^2}$$

Where:

$P_{ai}(P_{pi})$ is the active (passive) earth pressure intensity (kN/m^2) on wall surface below i layer;

ϕ_i is the internal friction angle of earth layer i;

γ_i is the bulk density (kN/m^3) of earth layer i;

h_i is the thickness of earth layer i;

$K_{ai}(K_{pi})$ is the active (passive) earth pressure coefficient of earth layer i;

ψ is the angle between wall surface and vertical direction;

β is the angle between ground surface and horizontal direction;

δ is the angle of wall friction;

ζ_i is the angle between failure surface of earth layer i and horizontal direction;

ω is upper load per unit area of ground surface (kN/m2).

② Earth pressure of clay

a Active earth pressure

Earth pressure intensity on wall surface is given by equation 3.2.4-5, while the negative earth pressure obtained through formula 3.2.4-5 is not taken into consideration.

(3.2.4-5) $$P_{ai} = \sum \gamma_i h_i + \omega - 2c$$

Where: c means cohesion.

b Passive earth pressure

Earth pressure intensity on wall surface is given by equation 3.2.4-6.

(3.2.4-6) $$P_{Pi} = \sum \gamma_i h_i + \omega + 2c$$

(2) Earth pressure at earthquake

① Pressure of sandy soil

a Active earth pressure

Earth pressure intensity on wall surface is given by equation 3.2.4-7, while the angle between failure surface and horizontal plane is given by equation 3.2.4-8.

(3.2.4-7) $$P_{ai} = K_{ai}[\sum \gamma_i h_i + \frac{\omega \cos\psi}{\cos(\psi - \beta)}]\cos\psi$$

(3.2.4-8) $$\cot(\zeta_i - \beta) = -\tan(\phi_i + \delta + \psi - \beta) + \sec(\phi_i + \delta + \psi - \beta)\sqrt{\frac{\cos(\psi + \delta + \theta)\sin(\phi_i + \delta)}{\cos(\psi - \beta)\sin(\phi_i - \beta - \theta)}}$$

$$K_{ai} = \frac{\cos^2(\phi_i - \psi - \theta)}{\cos\theta \cdot \cos^2\psi \cdot \cos(\delta + \psi + \theta)[1 + \sqrt{\frac{\sin(\phi_i + \delta)\sin(\phi_i - \beta - \theta)}{\cos(\delta + \psi + \theta)\cos(\psi - \beta)}}]^2}$$

Where:

b Passive earth pressure

Earth pressure intensity on wall surface is given by equation 3.2.4-9, and the angle between failure surface and horizontal plane is calculated through equation 3.2.4-10.

(3.2.4-9) $$P_{pi} = K_{pi}[\sum \gamma_i h_i + \frac{\omega \cos\psi}{\cos(\psi - \beta)}]\cos\psi$$

(3.2.4-10) $$\cot(\zeta_i - \beta) = \tan(\phi_i - \delta - \psi + \beta) + \sec(\phi_i - \delta - \psi + \beta)\sqrt{\frac{\cos(\psi + \delta - \theta)\sin(\phi_i - \delta)}{\cos(\psi - \beta)\sin(\phi_i + \beta - \theta)}}$$

$$K_{pi} = \frac{\cos^2(\phi_i + \psi - \theta)}{\cos\theta \cdot \cos^2\psi \cdot \cos(\delta + \psi - \theta)[1 - \sqrt{\frac{\sin(\phi_i - \delta)\sin(\phi_i + \beta - \theta)}{\cos(\delta + \psi - \theta)\cos(\psi - \beta)}}]^2}$$

Where:

θ is seismic combined angle, $\theta = \tan^{-1} k$ or $\theta = \tan^{-1} k'$

k means earthquake intensity

k' is apparent earthquake intensity;

The meanings of other symbols are the same with that in section 3.2.4(1)

② Earth pressure of clay

a Active earth pressure

Active earth pressure intensity at earthquake is give by equation 3.2.4-11, and the angle between failure surface and horizontal plane is calculated through formula 3.2.4-12.

$$P_a = \frac{(\sum \gamma \cdot h + \omega)\sin(\zeta_a + \theta)}{\cos\theta \cdot \sin\zeta_a} - \frac{c}{\cos\zeta_a \sin\zeta_a} \quad (3.2.4\text{-}11)$$

$$\zeta_a = \tan^{-1}\sqrt{1 - \left(\frac{\sum \gamma \cdot h + 2\omega}{2c}\right)\tan\theta} \quad (3.2.4\text{-}12)$$

Where: γ means the bulk density of earth (kN/m^3);

h is earth layer thickness (m);

ω is Upper load (kN/m^2) of per unit area

c is cohesion (kN/m^2)

θ is seismic combined angle, $\theta = \tan^{-1} k$ or $\theta = \tan^{-1} k'$

k means earthquake intensity;

k' is apparent earthquake intensity;

ζ_a means the angle between failure surface and horizontal direction

For the section under seabed, the earthquake intensity 10m under seabed shall be taken as zero when calculating earth pressure intensity; if the earth pressure intensity 10m below seabed is smaller than at seabed, the value at seabed shall be used.

b Passive earth pressure

Since there are too many unclear points regarding calculation of passive earth pressure of clay at earthquake, for the sake of simplicity, equation 3.2.4-6 is recommend.

(3) Specific weight γ, internal friction angle φ and cohesion c

γ, φ and c could be determined according to bore plug test data, and the indicators of non-cohesive filler could be used as per Table 3.2.4:

Table 3.2.4 Standard values of specific weight γ of filler and internal friction angle φ

填料名称	重度ɣ（3/mkN）		摩擦角ʋ（°）	
	Above water (moist specific weight)	Under water (float specific weight)	Above water	Under water
Find sand	18.0	9.0	30	28
Medium sand	18.0	9.5	32	32

Coarse sand	18.0	9.5	35	35
Grit	18.5	10.0	36	36
Gravel	17.0	11.0	38~40	38~40
Coal cinder	10.0~12.0	4.0~5.0	35~39	35~39
Block stone	17.0~18.0	10.0~11.0	45	45

Note: values regarding sandy soil in table are applicable when the content of fine particles with a size d<0 does not exceed 10%.

When calculating earth pressure, the specific weight of earth and filler could be used according to the following rules:

① In case of cohesive soil, float specific weight should be used below residual water level; Saturated unit weight should be used between residual water level and design high water level; natural specific weight shall be used above design high water level;

② In case of non-cohesive soil, float specific weight should be used below residual water level, and natural specific weight should be used above residual water level.

(4) Consideration of over-depth excavation at front edge of wall

Effect of over-depth excavation of at front edge of wall shall be taken into consideration in calculation of sheet pile wall; extra depth of excavation in front of wharf is normally 0.3~0.5m. In case of cohesive soil, the disturbing effect of dredging shall be taken into account: cohesion c of soil at mud surface takes 0; full value of c is taken more than 1m under mud surface; linear transition shall be performed between 0 and 1m.

(5) Apparent earthquake intensity

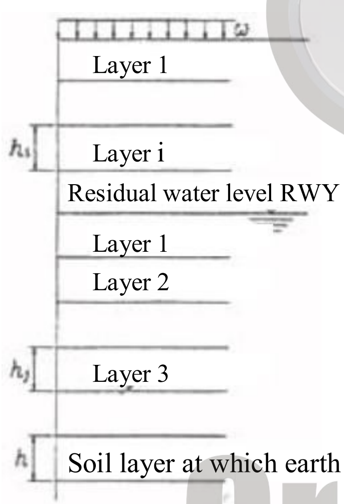


Fig. 3.2.4-2 Calculation diagram of apparent earthquake intensity

As shown above, the apparent earthquake intensity obtained using equation 3.2.4-13 for earth pressure underwater at earthquake.

$$k' = \frac{2(\sum \gamma_i h_i + \sum \gamma \cdot h_j + \varpi) + \gamma \cdot h}{2\{\sum \gamma_t h_i + \sum (\gamma - 10) \cdot h_j + \varpi\} + (\gamma - 10)h} k \quad (3.2.4\text{-}13)$$

Where: k' is apparent earthquake intensity;

γ_t means the bulk density of earth (kN/m3) at residual water level;

h_i is the soil layer thickness above residual water level (m);

γ is the bulk density (kN/m3) of saturated soil in air;

h_j means the thickness (m) of layer j above the layer at which earth pressure is calculated and below residual water level;

ϖ means the upper load (kN/m2) per unit area of ground surface;

h means the thickness of soil layer (m) for earth pressure calculation below residual water level;

k is earthquake intensity.

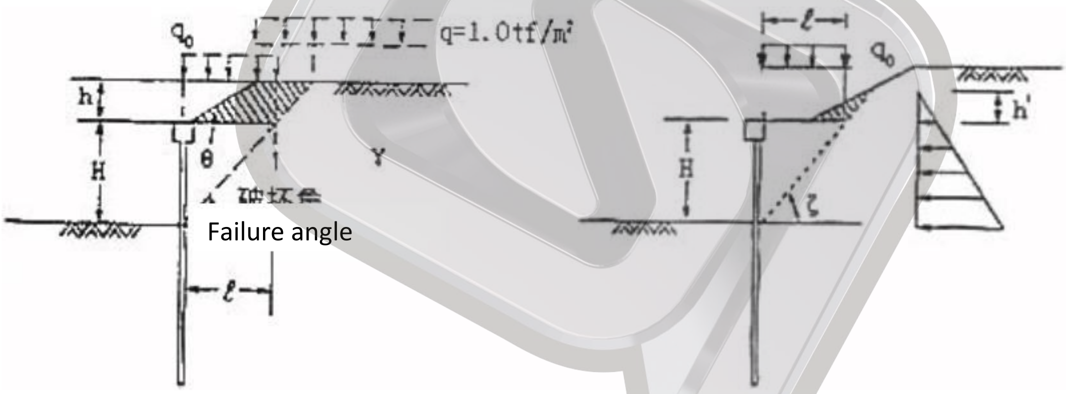
3.2.5 Upper Load

When designing cold-bent steel sheet pile, the upper load such as dead weight and charged load shall be taken into account as required as shown in the table below.

Table 3.2.5 Upper load

Upper load	Dead weight	Load of structure itself
Charged load	Heaped load	Load of sundry goods and bulk goods stacked on protecting sheet and in temporary warehouse and store; Snow cover on protecting sheet in snow-capped regions is taken as heaped load.
Movable load		Vehicle, train, handling machinery and other movable loads.

In case of relative complicated earth pile-up at back side, simple methods could be used to convert the earth into upper load as shown in the figure below.



(a) (b)

Fig. 3.2.5 Mound with special section shape

For the calculation of earth pressure at the section as shown in Fig. 3.2.5a, the earth load (the part marked with oblique lines) above failure angle could be approximately taken as uniformly distributed load (q_0) when $h \le H/3$, and as inclined earth pressure at bevel angle θ when $h > H/3$.

Figure 3.2.5b employs the same method as Figure 3.2.5a: the earth load of the part marked with oblique lines is converted into q_0, and the ground surface is taken as assumed plane.

(3.2.5) $q_0 = \gamma / l$

Where: γ is the weight of earth marked with oblique lines

3.3 Design Calculation of Sheet-pile Wall

3.3.1 Checking Calculation of Stability of Sheet-pile Wall "Skirting"

The minimum buried depth of sheet-pile wall in earth shall meet the requirements of "skirting" stability checking calculation; in other words, the kick-out force moment of standard value of active earth pressure (including wave force and surplus water pressure) against anchor point and the kicking resistance force moment of passive earth pressure standard value against anchor point shall meet the following formula:

(3.3.1) $M_p \ge \gamma_d \gamma_G M_A$

Where: M_p means the anti-kick stabilizing moment (kN·m) of passive earth pressure standard value against anchor point;

M_A means the kicking moment (kN·m) of active earth pressure standard value against anchor point;

γ_d is structural coefficient that should take values depending on foundation earth quality. When angle of internal friction $\gamma \leq 17°$, $\varphi=1.0$; when $\varphi \geq 17°$, $\gamma_d=1.15$;

γ_G is partial safety factor for action of earth pressure and $\gamma_G=1.35$.

3.3.2 Operating Status and Calculation Method of Steel-pile Wall

(1) Design calculation of sheet-pile wall is performed by different methods depending on operating status:

① Operating status of cantilever: in case of piling wall without anchor plate, the embedded part at lower section of sheet pile is completely built in.

② Operating status of free support and simply-supported beam: the lower end of single-anchor sheet-pile wall is in a "free support" operation status; it is similar to the condition in which sheet-pile wall is the free support beam (simply-supported beam) in vertical direction, the upper pivot is anchor point, and the lower pivot could be either calculated water bottom or the point of concurrence of passive earth pressure. This applies to the condition under which the earth in front of sheet-pile wall is relatively hard and the rigidity of sheet-pile wall is relatively high (rigidity per linear meter n>0.06).

③ Flexible building-in status means that the lower part of single-anchor or double-anchor sheet-pile wall is completely flexibly built into the earth, and is normally applicable for flexible sheet-pile wall (cold-bent steel sheet pile and timber sheet pile) or reinforced concrete sheet pile wall of which the rigidity per linear meter n ≤0.06.

④ An operation state between② and③.

(2) The calculation methods based on different operating status of sheet-pile wall are as follows:

① Operating status of cantilever: to be calculated according to cantilever by analytical method, graphical method or graphic analytic method.

② Free support status: to be calculated according to free support beam and imaginary beam by analytical method, graphical method or graphic analytic method; sheet-pile wall could also be directly taken as vertical beam on elastic foundation for calculation by vertical beam on elastic foundation method.

③ Flexible building-in status: according to the fact that the lower part or the whole of single-anchor or double-anchor sheet pile wall of which the bearing point is subjected to no displacement is flexibly built in the earth, the calculation could be performed by elastic line method (graphic analytic method) or vertical beam on elastic foundation method.

(3) Sheet-pile wall calculation method and graphic formula:

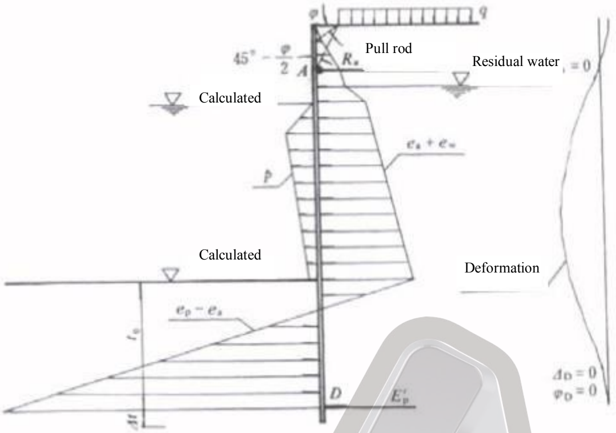


Fig. 3.3.2-1 Calculation chart for elastic line method

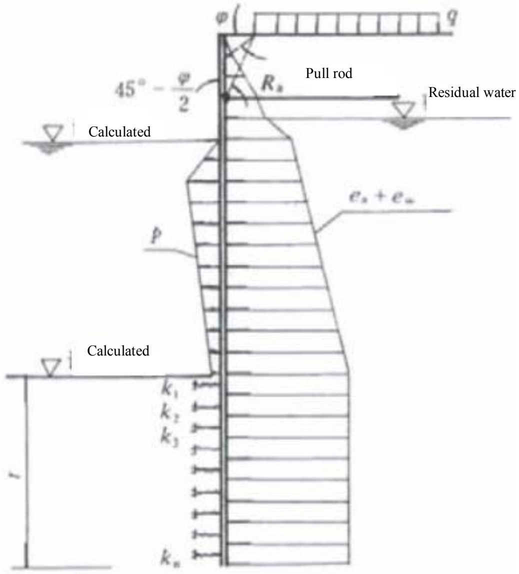
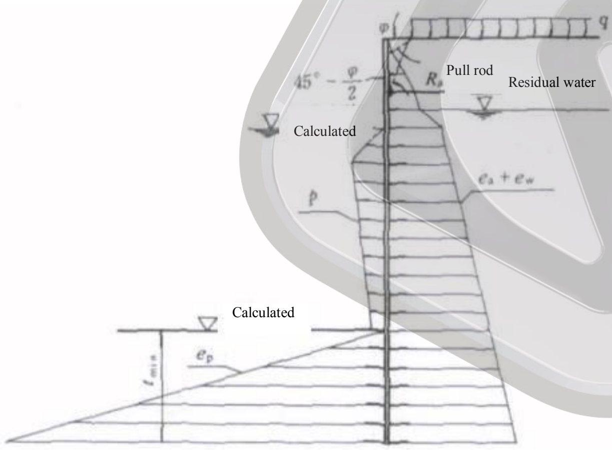


Fig. 3.3.2-2 Calculation chart for vertical beam on elastic foundation



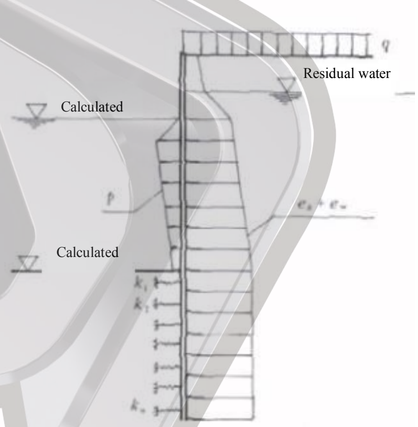


Fig. Calculation chart for free support method   Fig. 3.3.2-4 Calculation chart for piling wall without anchor plate

① The graphic formula of elastic line method is as shown in Fig. 3.3.2-1. The calculation is made based on the assumption that both the displacement of tie back anchor and the linear deflection of sheet-pile wall at bottom E_p is zero. Considering the fact that the earth pressure re-distribution behind the wall and the pull rod anchor point displacement may result in the reduction of mid-span bending moment of sheet-pile wall, the calculated maximum mid-span bending moment shall be multiplied by reduction coefficient ξ that should take 0.7~0.8. If the buried depth of sheet-pile wall obtained by this method is less than that given by equation 3.3.1, the latter shall be taken as the buried depth of sheet-pile wall.

② When calculation is made by vertical "beam on elastic foundation" method, the buried depth of sheet-pile wall shall be determined through equation 3.3.1.The internal force and deflection of sheet-pile wall could be determined by trussing finite-element method of which the calculation graphic formula is shown in Fig. 3.3.2-2.The part that occurs due to overload (ground load plus earth gravity) above calculated water bottom had better be considered for calculation of active earth pressure behind the wall of buried section.

This method may take into account the displacement of pull rod anchor point, which is composed of pull rod deformation under force and the displacement of anchor structure.

When pull rod anchor point displacement is taken into consideration, the calculated bending moment is not reduced; if pull rod anchor point displacement is left out of account, the calculated bending moment shall be reduced according to above-noted rules.

Internal force and deflection of sheet-pile wall could be determined by trussing finite-element method when "beam on elastic foundation" method is used for calculation. Elastic coefficient of elastic rod is determined by multiplying the reaction coefficient K of horizontal foundation by spacing. Reaction coefficient of horizontal foundation could be calculated by m method or other methods depending on property of underlying soil and design experience. The reaction coefficient of horizontal foundation is given by the following equation if m method is used:

$$K = mz \qquad (3.3.2)$$

Where: K means the reaction coefficient (kN/m^3) of horizontal foundation;

is the proportionality coefficient (kN/m^4) of reaction coefficient of horizontal foundation along with the increase in depth as shown in Table 3.3.2;

means the distance (m) between calculated point and calculated water bottom.

Table 3.3.2 Proportionality coefficient

Soil property of foundation	Values (kN / m4) m
Clay soil and silt with IL≥1	1000~2000
Clay soil and silty sand (1＞IL≥0.5)	2000~4000
Clay oil and medium & fine sand (0.5>IL≥0)	4000~6000
Clay soil and coarse sand (IL<0)	6000~10000
Gravel, gravelly sand, broken stone and pebble	10000~20000

Note: when horizontal displacement of sheet-pile wall at calculated water bottom is more than 10mm, the relatively small values shall be taken from table.

③ In case free support method is used for calculation, the buried depth of sheet-pile wall shall be given by equation 3.3.1. Calculation graphic formula of internal force of sheet-pile wall is shown in Fig. 3.3.2-3. Assuming that ultimate passive earth pressure occurs in front of sheet-pile wall within the range of minimum buried depth (t_{min}), t_{min} shall be obtained through equilibrium of force and moment. B ending moment of sheet-pile wall calculated based on this graphic formula is not to be reduced.

④ Calculation of sheet-pile wall without anchor: Buried depth of sheet-pile wall without anchor could be determined through equation 3.3.1.The displacement of its buried section and at calculated bottom could be calculated by "vertical beam on elastic foundation" method according to Fig. 3.3.2-4, and the calculated bending moment does not have to be reduced.

Calculation of sheet-pile wall could be performed using analytical method or graphic analytic method. Analytic method is short-cut and rational only under the simplest circumstances when the foundation and earth fill behind the wall has homogeneous property while the fill ground bears no load or the load is evenly distributed along the ground; otherwise, the calculation using analytic method would be extremely complicated; therefore, graphic-analytic calculation method is usually used; in other words, elastic line method is relatively convenient.

To achieve more accurate calculation results, earth pressure graphics had better be divided into concentrated forces with a height of 0.5~1.0m.To facilitate the graph plotting, the poles of force polygon had better be located on perpendicular line of starting point of horizontal pressure; if this is the case, the closed line of funicular polygon would also be perpendicular.

Finally, the closed line shall intersect with funicular curve on boundary of passive earth pressure graphics by cut-and-trial method. If closed line does not intersect with funicular curve, it may be because the buried depth of sheet-pile wall is not large enough; in such a case, pilot calculation shall be conducted after the buried depth of sheet pile is extended. If the intersection point between closed line and funicular curve is higher than the level of bottom boundary of passive earth pressure, it may be because the buried depth of sheet-pile wall is excessively large; in such a case, the depth shall be reduced

3.3.3 Buried Depth of Sheet-pile Wall

The minimum buried depth of sheet-pile wall: $t_{\min}$

$$t_{\min} = t_0 + \Delta t \quad (3.3.3\text{-}1)$$

Where: t_0 is the calculated buried depth of sheet-pile wall, and when graphic-analytic method is used for calculation, it means the buried depth of sheet pile in the case of coincidence of closed line and funicular curve at lower boundary of graphics of passive earth pressure; its value is obtained by successive approximation method;

$$\Delta t = \frac{E'_p}{2(e'_p - e'_a)} \quad (3.3.3\text{-}2)$$

Where: e'_p means the passive earth pressure at the depth of t_0 behind the wall;

e'_a means the active earth pressure at the depth of t_0 in front of the wall;

E'_p is measured in force polygon diagram and means the reverse passive earth resistance behind sheet-pile wall at lower boundary of passive earth pressure graphics.

3.3.4 Maximum Bending Moment and Bending Moment Reduction Coefficient of Sheet-pile Wall

Calculation for sheet-pile wall is performed using traditional earth pressure distribution method without considering displacement of anchor point or re-distribution of earth pressure caused by contortion of sheet-pile wall; the calculated maximum bending moment value of sheet pile could be reduced by 1/3 or multiplied by a reduction coefficient of 0.7~0.8, and the result may be taken as standard value of bending moment of sheet pile. However, no reduction is allowed under the following circumstances:

(1) Sheet-pile wall is furnished with no pull rod; or the weak anchorage of pull rod may result in displacement while pile toe is built in earth; in such a state of affairs, no re-distribution of earth pressure will occur;

(2) Behind the wall is unconsolidated clay soil;

(3) The earth behind wall does not reach the elevation of anchoring pull rod;

(4) The earth behind the wall from harbor basin to anchorage system is backfilled earth;

(5) Reinforced concrete sheet pile wall with relatively high rigidity

The so-called reinforced concrete sheet pile wall with relatively high rigidity should have a rigidity per linear meter n>0.06; in such a case, sheet-pile wall shall be taken as free end bearing beam for calculation, and the obtained sheet pile bending moment shall not be reduced. If n≤0.06, it is suggested sheet-pile wall should be regarded as flexible wall and deemed to have been built or semi-built in earth.

Rigidity per linear meter is given by the following equation:

$$\begin{cases} n = \dfrac{\delta}{t} \\ \delta = \sqrt{\dfrac{12J}{b+\Delta}} \end{cases} \quad (3.3.4)$$

Where: t is the buried depth (m) of sheet-pile wall;

n means the rigidity per linear meter of rectangular reinforced concrete sheet pile or reinforced concrete sheet pile that is equivalent thereto;

δ means the converted height (m) of rectangular wall section;

J is the inertia moment of reinforced concrete wall member (m^4);

b means the dimension (m) of wall member in direction of leading edge line;

Δ means the design gap (m) between wall members.

In case of reinforced concrete pipe pile, the dimension b is outside diameter D; in case of reinforced concrete trapezoidal sheet pile, $b = b_n$ is equal to the width of wing edge.

Both cold-bent steel sheet pile wall and timber sheet pile wall is looked upon as flexible wall.

3.3.5 Checking Calculation of Strength of Cold-bent Steel Sheet Pile

Cold-bent steel sheet pile could be plate shaped, basin shaped (Larsen shaped), Z shaped, I shaped, box shaped, of combined shape, composite shape or grid shaped depending on the requirement for calculated bending moment of sheet-pile wall. Strength per unit width of cold-bent steel sheet pile must meet the following equation:

$$\frac{\gamma_{GQ}}{1000}\left(\frac{N}{A}+\frac{M_{\max}}{W_z}\right)\le f_t \quad (3.3.5\text{-}1)$$

Where: N means the axial force (kN) per meter produced by standard effect value;

$M_{\max}$ is the maximum bending moment ($kN \cdot m$) of per meter of sheet-pile wall produced by standard effect value;

A means the cross-sectional area (m^2) of cold-bent steel sheet pile;

W_z is the elastic section modulus (m^3) of cold-bent steel sheet pile;

f_t means the designed strength value (MPa) of steel and should be used according to China national standard "Code for design of steel structures" (GBJ 50017-2003);

γ_{GQ} is a comprehensive breakdown coefficient that takes 1.35.

Special anti-rust measures need not be taken for cold-bent steel sheet piles in fresh water since the corrosion rate of cold-bent steel sheet pile is extremely low under mud level and at the side behind the wall and could be negligible. The surface of section above mud level in water is distributed uniformly and is subject to extremely weak corrosion.

Cold-bent steel sheet pile may suffer from serious corrosion in corrosive water and sea water, especially the areas in sea water with a lot of halobiotic shellfishes; however, the corrosion rate of sheet pile at shore side is extremely low, and the corrosion of the section under mud level could be negligible.

When rust resisting paint is applied, the starting time of corrosion may be put off by 5-10 years, and the degree of corrosion could be reduced. However, high cost is required since sandblast cleaning must be performed before the application of rust resisting paint. Therefore, sheet piling section with increased thickness is used under normal circumstances. Corrosion underwater could be prevented through catodic protection by means of impressed current or through sacrificial anode.

In case of harbor wharf, the elevation of parapet bottom surface could be properly reduced so as to minimize the corrosion surface of cold-bent steel sheet pile.

3.3.6 Calculation about Anchoring Pull Rod

Pulling force of anchoring pull rod is obtained through calculation about sheet-pile wall while taking into account uneven stress on anchoring pull rod; the pulling force is given by the following equation:

$$R_A = \xi_a R_a l_a \sec\theta \quad (3.3.6\text{-}1)$$

Where: R_A is the standard value of pulling force (kN) of pull rod;

ξ_a means the stress nonuniformity coefficient of pull rod; in case of pre-tension, ξ_a=1.35;

R_a is the standard value of pulling force of pull rod on per linear meter of sheet-pile wall (kN/m);

l_a means distance between pull rods (m);

θ is the angle (°) between pull rod axis and horizontal plane.

Steel pull rod could be designed as center tension member, and the pull rod diameter could be calculated according to the following equation:

$$d = 2\sqrt{\frac{1000R_A\gamma_{RA}}{\pi f_t}} + \Delta d \quad (3.3.6\text{-}2)$$

Where: d means the diameter of pull rod (mm);

R_A is the standard value of pulling force of pull rod (kN);

γ_{RA} means the breakdown coefficient of pulling force of pull rod and takes 1.35;

f_t is the designed value of steel strength (MPa);

Δd is the corrosion allowance (mm) and could take 2~3mm.

Hinged pivots for anchoring pull rod shall be set at both sheet-pile wall and anchoring board, and hinge pivot connecting plates shall be embedded on both launching nose of sheet-pile wall and the anchoring board.

The length of each segment of anchoring pull rod shall not exceed 10m; pull rod with a total length of more than 10m shall be divided into sections, and adjacent sections shall be hinged together. At the mid-section of pull rod should be fine-adjusting screw cap (rigging screw) for adjusting the tightness of pull rod. In case the anchoring pull rod may subside, supporting paxilla shall be inserted at the bottom of pull rod.

Anchoring pull rod and all its exposed ironworks such as hinge point and connecting plate etc. should be coated with asphalt or provided with other corrosion prevention measures, and the hinge joints shall be applied with lubricating butter.

3.3.7 Calculation about Nose Girder, Parapet and Capping Beam of Sheet-pile Wall

Nose girder at the top of sheet pile could be calculated as continuous 5-span girder with rigid support, and the support reaction is the pulling force of anchoring pull rod. The maximum bending moment of nose girder and at the suspended arm section of nose girder produced by standard value of pulling force of pull rod is given by the following equation:

$$\begin{cases} M_{\max} = \dfrac{1}{10}R_A l_a^2 \\ M_b = \dfrac{1}{2}R_A l_b^2 \end{cases} \quad (3.3.7)$$

Where: $M_{\max}$ is the maximum bending moment (kN·m) of nose girder produced by standard value of pulling force of pull rod;

R_A is the standard value of pulling force of pull rod on per linear meter of sheet-pile wall (kN/m);

M_b is the maximum bending moment (kN·m) of suspended arm section of nose girder produced by standard value of pulling force of pull rod;

l_a means distance between pull rods (m);

l_b is the length (m) of suspended arm section of nose girder.

An outreaching hinge point shall be furnished at rear edge (shore side) of reinforced concrete nose girder, and the node connection plate shall be embedded inside the girder.

Steel nose girder had better be produced with double-steel channel into para-abdomen shape, and the channel steel

had better be placed at shore side of sheet-pile wall; the rear edge of nose girder shall also be provided with a hinge point; the screw cap and setting plate of anchoring pull rod shall be placed outside sheet-pile wall but shall in no case stretch out of bulkhead line since this may hinder the berthing of vessel. In case of basin-shaped or Z-shaped sheet pile, the placement in recess part of sheet-pile wall is appropriate.
Reinforced concrete parapet is usually built so as to prevent the corrosion of cold-bent steel sheet pile in sea water, and the elevation of its bottom surface is lowered to neighborhood of low water level. In the case of simplex pull rod, parapet could be designed as vertical cantilever beam; when the pull rod is taken as fixed end, the primary load would be the active earth pressure behind the wall. In case pull rod is provided behind the top capping beam of parapet, the parapet shall be designed as simply-supported beam with the upper and lower pull rods as pivots; in such a case, the fixed end at pull rod at the top of sheet pile shall be calculated. To ensure the continuity and integrity of sheet-pile wall and bear the support bending moment of parapet at nose girder, sufficient joint strength shall be provided between cold-bent steel sheet pile or reinforced concrete sheet pile and reinforced concrete parapet. For the purpose of boat moorage, parapet shall be furnished with one or two rows of makefast mooring rings.

Reinforced concrete capping beam usually has adequate rigidity; if necessary, structural pull rod could be mounted so as to straighten up the bulkhead line. When required, one bollard shall be provided at the interval of 15~25m.The section of capping beam at which bollard is provided shall be sectionally enlarged, and the bollard pull rod and anchoring board shall be set separately in a splayed manner. Wheel apron devices above ground level that protect people and vehicles against running-out shall be provided at front edge of capping beam and shall in no case hinder the mooring of vessels.

Capping beam, parapet and nose girder shall be provided with berthing fender log for berthing cushion and provided with corresponding bolts that should be convenient for replacement; exposed part of ironworks shall be plated or coated with rust-proof layer.

3.4 Design Calculation of Anchoring Structure

3.4.1 The Most Rational Distance between Anchoring Wall (Board) and Sheet-pile Wall

The most rational distance (i.e. the minimum distance) between anchoring wall (board) and sheet-pile wall shall ensure that the earth slip surface in front of wall (board) that passes through the bottom side of anchoring wall (board) and the earth slip surface behind sheet-pile wall that passes through the maximum bending moment point in earth of sheet-pile wall squarely intersect at the ground. It is not necessary to further increase the distance between anchoring wall (board) and sheet-pile wall since this would not bring about the increase in passive earth pressure, but extend the length of anchor rod to no purpose (Fig. 3.4.1).

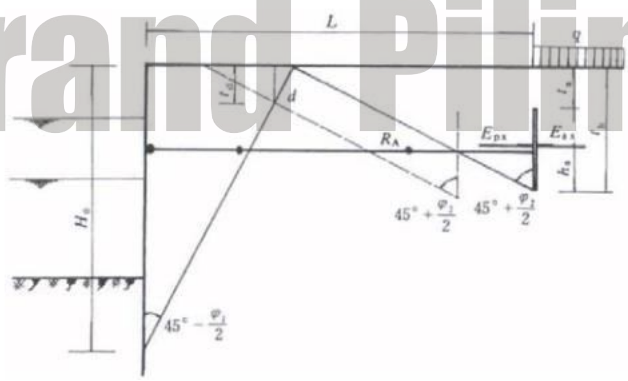


Fig. 3.4.1 Graphic formula for calculation of minimum distance between anchoring wall (board) and sheet pile

When filled up ground is level and the soil texture is uniform, the minimum distance could be calculated according to the following equation:

$$L = H_0 tg\left(45^\circ - \frac{\varphi_1}{2}\right) + t_h tg\left(45^\circ + \frac{\varphi_2}{2}\right) \quad (3.4.1\text{-}1)$$

Where: L is the minimum distance (m) between anchoring wall (board) and sheet-pile wall;

t_h is the buried depth of bottom of anchoring wall (board);

φ_1 and φ_2 are respectively the internal friction angle (°) of earth mass behind sheet-pile wall and in front of anchoring wall (board);

H_0 means the height of pyramid height at active fracture plane and takes the distance (m) between maximum hogging moment point and wharf surface when elastic line method is used.

If the distance between anchoring wall (board) and sheet-pile wall is less than above-noted minimum distance L for special reasons (in other words, the intersection point between passive fracture plane in front of anchoring wall (board) and the active fracture plane behind sheet-pile wall is under ground level), the passive earth pressure shall be reduced for checking calculation of shallow slide stability of anchoring wall (board), and the reduction value is given by the following equation:

$$\Delta E_{PX} = \frac{\gamma t_d^2 l_a K_p}{2} \quad (3.4.1\text{-}2)$$

Where: ΔE_{PX} means the horizontal component increment (kN) of standard value of passive earth pressure;

t_d is the distance from the intersection point between active fracture plane of earth mass behind sheet-pile wall and the active fracture plane of earth mass in front of anchoring wall (board) to the ground;

γ is earth weight density (kN/m3) within the range of

l_a means distance between pull rods (m);

K_p is the coefficient of passive earth pressure

3.4.2 Checking Calculation of Stability of Anchor

Checking calculation of stability of anchor includes:

(1) Checking of shallow sliding stability of anchoring wall (board);

(2) Checking of stability of anchoring wall (board) sliding along bottom surface of riprap pyramid;

(3) Checking of deep sliding stability of anchor (up-surge stability, stability of down-slide moving surface)

Above-noted checking calculations could be performed as per relevant norms.

Other matters needing attention:

(1) Position of anchor rod on anchoring wall (board): when anchoring wall (board) is freely supported on foundation, i.e. when the lower end of anchoring wall (board) is not built in foundation, anchor rod is preferably place at the middle of wall (board).

(2) If the cohesive strength of clay soil is taken into account in design calculation, the following requirements must be satisfied:

① Foundation soil must be undisturbed soil;

② Foundation must be under a constant cloak;

③ Soil must be kept in a saturated state constantly;

④ The soil has no thixotropic property

3.4.3 Calculation of Anchoring Structure Strength

When either cold-bent steel sheet pile and steel pile or reinforced concrete wall (board) and pile is used as anchoring structure, the strength calculation is the same with that of sheet-pile wall. In other words, in case of cold-bent steel sheet pile, the value obtained by multiplying calculated standard load value by comprehensive partial load factor shall be less than or equal to the design value specified in the Code. In case of reinforced concrete member, the comprehensive partial load factor for checking calculation of strength shall take 1.40; for cracking calculation and crack width calculation, comprehensive partial load factor shall take 0.85.

Anchoring wall usually employs a continuous structure. The calculation could be made by the method as used for continuous beam with multi-span rigid support by taking anchor rod pull force R_A as support reaction; continuous beam could be arranged at the middle (vertical) of wall. R_A here should be obtained by multiplying standard value of load by non-uniformity coefficient = ξ 1.35 of pull force of pull rod, i.e. $R_A = \xi R_A = 1.35RA$.Calculation about anchoring wall face could be made according to cantilever plates at both ends.

If anchoring board is in continuous form, the calculation would be the same with that of wall. If several pull rods share one board, the calculation about beam shall be made as continuous beam with multi-span rigid support, and the board surface shall be calculated according to cantilever plates at both ends. When one pull rod corresponds to one board, if nose girder is provided, the calculation of both girder and board shall be made as cantilever plate; in case there is no beam, checking calculation of punching shall be made besides the calculation made based on cantilever plate, and the accessories of pull rod hole shall be provided with strengthening rib.

Horizontal shift of anchoring wall or board could be implemented according to relevant specifications.

The calculation of length and internal force of anchored single pile and sheet pile could be made by the method as employed in calculation about sheet-pile wall without anchor under the effect of centralized horizontal force R_a.

When calculating internal force of anchored raking pile, it could be assumed both ends of pile are hinged, and the effect of surrounding soil mass on pile is not taken into consideration. Anchored raking pile shall be designed based on strength while its axial bearing capacity shall be calculated according to specification. Anchored raking pile must be located outside active fracture plane of soil mass behind sheet-pile wall, and the distance between pressure pile toe and sheet-pile wall shall not be less than 1.0m.

3.5 Checking Calculation of Overall Sliding Ability of Sheet-pile Wall

The checking calculation of overall sliding stability of sheet-pile wall could be made by the method for circular sliding surface.

The position of circular sliding surface could be selected using special points such as sheet pile toe etc.

If circular sliding surface passes through the area in front of anchoring wall (board), the effect of pull force of pull rod on stability could not be calculated.

If there is difficult ground or silt inter-layer that is not completely removed in the vicinity of areas above or below pile toe, it is necessary to check the condition of circular sliding surface or special sliding surface; in such a case, the sliding surface need to pass through difficult ground or silt inter-layer, but the beneficial effect of cutting-off force of sheet pile shall not be calculated.

For checking calculation of circular sliding surface, it is necessary to calculate the minimum resisting force partial factor in circular sliding surface of sheet pile toe, and its value shall be greater than or equal to specified value, i.e.

$$\frac{M_{RK}}{M_{sd}} \geq \gamma_d$$

Where: M_{sd} is the standard value (kN·m/m) of sliding moment acting on dangerous sliding surface;

M_{RK} means the standard value (kN·m/m) of anti-slide moment acting on dangerous sliding surface;

γ_R means the resisting force partial factor; shearing strength of earth normally employs consolidated quick shearing index, and the corresponding resisting force partial factor γ_R=1.2~1.4.

3.6 Structural Configuration and Computational Example of Sheet-pile Wall

3.6.1 Stand-alone Cold-bent Steel Sheet Pile Wall

Stand-alone cold-bent steel sheet pile wall is a structure that supports earth pressure, water pressure and other loads through the flexural rigidity of cold-bent steel sheet pile and the lateral resistance at pile penetration depth. Compared with anchor-type cold-bent steel sheet pile wall, stand-alone cold-bent steel sheet pile wall features simple construction, short construction period and the availability of construction in narrow area behind wall; therefore, stand-alone cold-bent steel sheet pile wall has been widely used for river revetment and agricultural water ditch revetment etc.

A computational example is used to clarify the method for calculation about stand-alone cold-bent steel sheet pile wall:

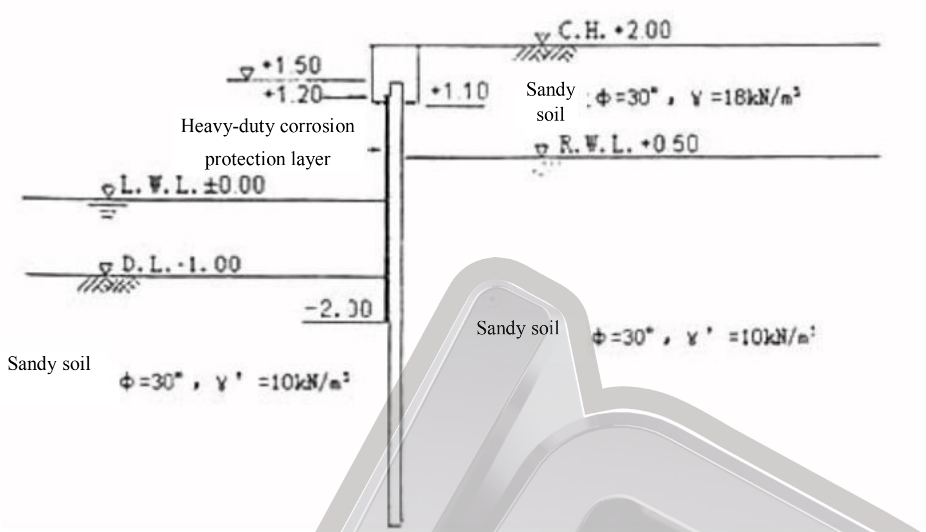


Fig. 3.6.1-1 Computational example for calculation about stand-alone cold-bent pile wall

(1) Design conditions

As shown above:

① Elevation

Crest height: C.H.=+2.00m

Design water depth: D.L.=-1.00m

Residual water level: R.W.L.= +0.50m

Low water level: L.W.L.= +0.00m

② Design earthquake intensity

Design earthquake intensity $k = 0.15$

③ Topside load

$q = 10kN/m^2$ (normal times), $q' = 5kN/m^2$ (earthquake)

④ Soil conditions

Back soil: internal friction angle $\varphi = 30^\circ$; bulk density $\gamma = 18kN/m^3$, $\gamma' = 10kN/m^3$ (in water)

⑤ Bulk density of sea water $\gamma_w = 10kN/m^3$

⑥ Wall friction angle

Active $\delta_a = +15^\circ$; passive $\delta_p = -15^\circ$

⑦ Admissible displacement

Crest 5cm in normal times, 10cm in case of earthquake

(2) Earth pressure intensity and residual water pressure intensity

Soil layer division is as shown in the figure below; earth pressure intensity and residual water pressure intensity at each soil layer boundary as shown in the figure is calculated.

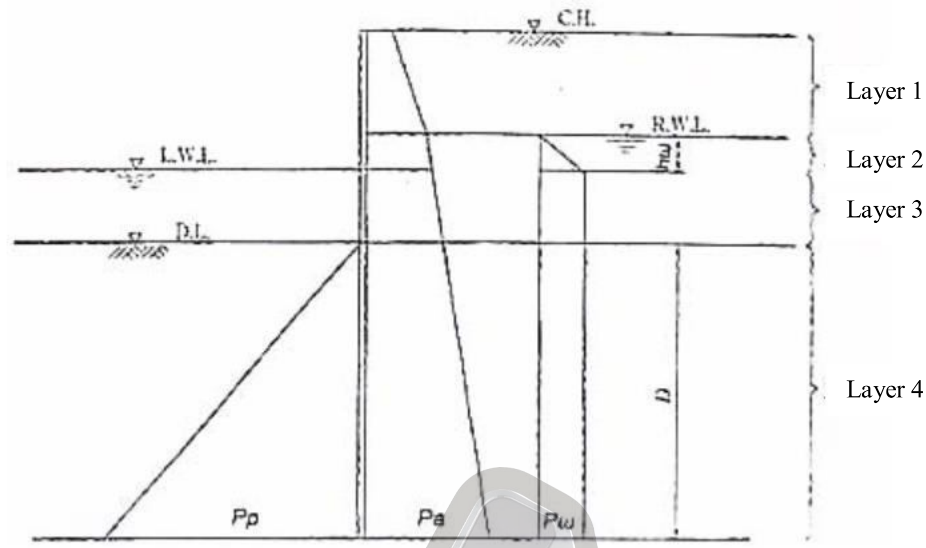


Fig. 3.6.1-2 Soil layer division

Table 3.6.1-1 Earth pressure intensity and residual water pressure intensity at active side in normal times

Layer No.	Position (m)	Load $\overline{\varpi}$ $\varpi = \sum \gamma_i h_i + q$ (kN/m2)	Earth pressure coefficient $K_a \cos\delta_a$	Earth pressure intensity $p_{ai} = K_a \cos\delta_a \varpi$ (kN/m2)	Residual water pressure p_{wi} (kN/m2)
Layer 1	+2.00	10.0	0.291	2.91	0.00
	+0.50	37.0		10.77	0.00
Layer 2	+0.50	37.0	0.291	10.77	0.00
	±0.00	42.0		12.22	5.05
Layer 3	±0.00	42.0	0.291	12.22	5.05
	-1.00	52.0		15.13	5.05
Layer 4	-1.00	52.0	0.291	15.13	5.05
	-20.00	242.0		70.42	5.05

Table 3.6.1-2 Earth pressure intensity at passive side in normal times

Layer No.	Position (m)	Load $\overline{\varpi}$ $\varpi = \sum \gamma_i h_i + q$ (kN/m2)	Earth pressure coefficient $K_p \cos\delta_p$	Earth pressure intensity $p_{pi} = K_p \cos\delta_p \varpi$ (kN/m2)
Layer 4	-1.00	0.0	4.807	0.00
	-20.00	19.0		913.33

Table 3.6.1-3 Earth pressure intensity and residual water pressure intensity at active side in case of earthquake

Layer No.	Position (m)	Seismic intensity	Load $\overline{\varpi}$ $\varpi = \sum \gamma_i h_i + q$ (kN/m2)	Earth pressure coefficient $K_a \cos\delta_a$	Earth pressure intensity $p_{ai} = K_a \cos\delta_a \varpi$ (kN/m2)	Residual water pressure intensity p_{wi} (kN/m2)

Layer 1	+2.00	0.15	5.0	0.393	1.97	0.00
+0.50			32.0	12.58		0.00
Layer 2	+0.50	0.18	32.0	0.417	13.34	0.00
±0.00			37.0	15.43		5.05
Layer 3	±0.00	0.18	37.0	0.417	15.43	5.05
-1.00			47.0	19.60		5.05
Layer 4	-1.00	0.27	47.0	0.504	23.69	5.05
-20.00			237.0	119.45		5.05

Table 3.6.1-4 Earth pressure intensity at passive side in case of earthquake

Layer No.	Position (m)	Seismic intensity	Load $\overline{\varpi}$ $\overline{\varpi}=\sum\gamma_i h_i+q$ (kN/m2)	Earth pressure coefficient $K_p \cos\delta_p$	Earth pressure intensity $p_{pi}=K_p \cos\delta_p \varpi$ (kN/m2)
Layer 4	-1.00	0.30	0.00	3.545	0.00
-20.00			190.0	673.55	

(3) Depth of assumed foundation plane

It is assumed that the foundation plane is located at the point where the sum of active earth pressure and residual water pressure is equal to passive earth pressure. The relationship between each layer and (active earth pressure + residual water pressure-passive earth pressure) is as shown in Table 3.6.1-5.

Table 3.6.1-5 Intensity in horizontal direction

Layer No.	Position (m)	In normal times				Earthquake				
		Active earth pressure p_{ai} (kN/m2)	Residual water pressure p_{wi} (kN/m2)	Passive earth pressure p_{pi} (kN/m2)	$p_{ai}+p_{wi}-p_{pi}$ (kN/m2)	Active earth pressure p_{ai} (kN/m2)	Flowing water pressure p_{dwi} (kN/m2)	Residual water pressure p_{wi} (kN/m2)	Passive earth pressure p_{pi} (kN/m2)	$p_{ai}+p_{dwi}-p_{wi}-p_{pi}$ (kN/m2)
Layer 1	+2.00	2.91	-	-	2.91	1.97	-	-	-	1.97
	+0.50	10.77	-	-	10.77	12.58	-	-	-	12.58
Layer 2	+0.50	10.77	0.00	-	10.77	13.34	-	0.00	-	13.34
	±0.00	12.22	5.05	-	17.27	15.43	-	5.05	-	20.48
Layer 3	±0.00	12.22	5.05	-	17.27	15.43	0.00	5.05	-	20.48
	-1.00	15.13	5.05	-	20.18	19.60	1.33	5.05	-	25.97
Layer 4	-1.00	15.13	5.05	0.00	20.18	23.69	-	5.05	0.00	28.74
	20.00	70.42	5.05	913.33	-837.86	119.45	-	5.05	637.550	-549.05

It is observed from the table that is in layer 4 both in normal times and in case of earth. If it is assumed that the position of $P_a + P_w - P_p = 0$ foundation plane is x, the calculation based on proportional distribution showed that $x = 0.45m$ in normal times, and $x = 0.95m$ in case of earthquake.

(4) Above-ground load

Partition calculation is made under the assumption that the load of earth pressure and residual water pressure above foundation plane is as shown in the figure below.

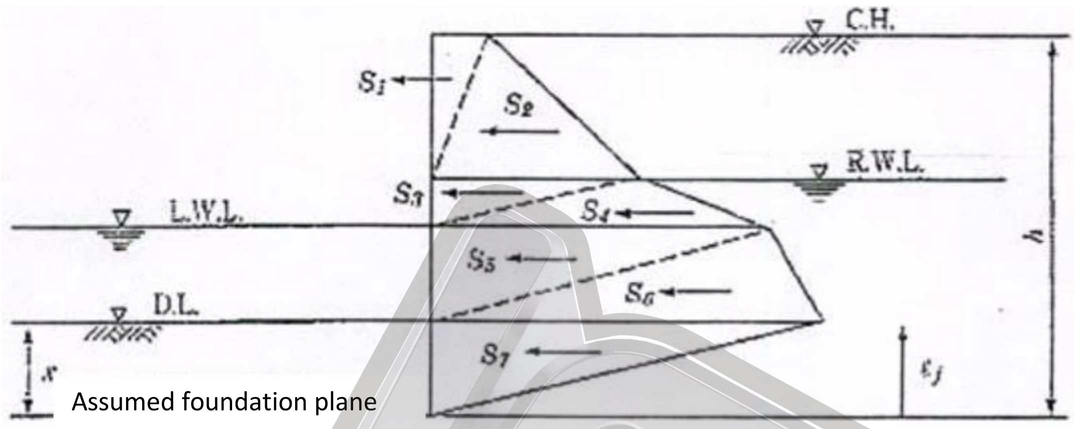


Fig. 3.6.1-2 Load diagram

Table 3.6.1-6 Horizontal force and bending moment

j			Normal times		Earth	
Horizontal force Sj (kN/m)	Distance lj (m)		Bending moment M= Sj·lj (kN/m/m)	Horizontal force Sj (kN/m)	Distance lj (m)	Bending moment M= Sj·lj (kN/m/m)
1	2.18	2.95	6.43	1.47	3.45	5.08
2	8.08	2.45	19.76	9.43	2.95	27.78
3	2.69	1.78	4.79	3.34	2.28	7.60
4	4.32	1.61	6.97	5.12	2.11	10.81
5	8.64	1.11	9.62	10.24	1.61	16.50
6	10.09	0.78	7.87	12.99	1.28	16.60
7	4.51	0.30	1.34	13.58	0.63	8.55
Total	Ho=40.50	-	M0=56.78	Ho =56.17	-	M0=92.93

The obtained height of resultant action point is *ho*:

$h_0 = 1.40m$ in normal times; $h_0 = 1.65m$ at earthquake

(5) Deformation coefficient

The table sets forth the deformation coefficient of crest displacement when above-ground part is designed as cantilever beam.

Table 3.6.1-7 Deformation coefficient

j		Normal times			Earthquake	
$\alpha_j = \frac{l_j}{h}$	$\zeta_j = \frac{1}{6}(3-\alpha_j)\alpha_j^2$	$Q_j = \zeta_j S_j$	$\alpha_j = \frac{l_j}{h}$	$\zeta_j = \frac{1}{6}(3-\alpha_j)\alpha_j^2$	$Q_j = \zeta_j S_j$	

1	0.855	0.261	0.570	0.873	0.270	0.398
2	0.710	0.192	1.553	0.747	0.209	1.974
3	0.516	0.110	0.297	0.578	0.135	0.449
4	0.468	0.092	0.399	0.535	0.118	0.603
5	0.323	0.047	0.402	0.409	0.072	0.738
6	0.226	0.024	0.239	0.324	0.047	0.608
7	0.086	0.004	0.016	0.160	0.012	0.164
Total	-	-	3.478	-	-	4.935

(6) Design of cold-bent steel sheet pile wall

Design is made under the assumption that cold-bent steel sheet pile model GPU11a is used.

① Displacement when it is designed as cantilever beam

In normal times: $$\delta_3 = \frac{Bh^3}{EI}\sum Q_i = \frac{1.0\times(3.0+0.45)^3}{2.0\times10^8\times205\times10^{-6}}\times3.478 = 0.0035m = 0.35cm$$

At earthquake: $$\delta_3 = \frac{Bh^3}{EI}\sum Q_i = \frac{1.0\times(3.0+0.95)^3}{2.0\times10^8\times205\times10^{-6}}\times4.935 = 0.0074m = 0.74cm$$

② Design of buried part

It is assumed that the N-value distribution of seabed foundation is calculated based on Figure 3.6.1-3.

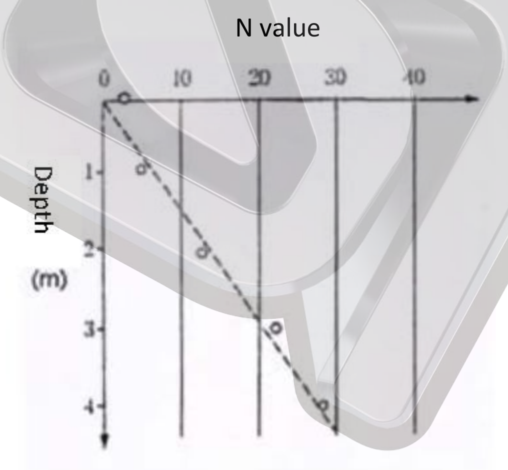


Fig. 3.6.1-3 N value distribution of seabed foundation

It is observed from Fig. 3.6.1-3 that the increasing rate of N is $\bar{N} = 7$, so ks=2.5×103 kN/m3.5. Hence the results as shown in Table 3.6.1-8 are achieved by comparing each element of standard pile with each element of cold-bent steel sheet pile wall based on the assumption that B=1m.

Table 3.6.1-8 Comparison between elements of standard pile and cold-bent steel sheet pile wall

Items	Standard pile				Cold-bent steel sheet pile wall	Ratio between elements
		$R = \frac{□□□墙}{□□□}$ 墙：wall			logR	
Acting height	Normal times	(h) s=1.00m		(h) p=1.40m	1.40	0.147
Earthquake	(h) s=1.00m			(h) p=l.65m	1.65	0.217

Rigidity	(EI) s=1.00kN·m2	(EI) p=41000kN·m2	4.10	0.613
Lateral resistance coefficient	(Bks) s=1000kN/m2.5	(Bks) p=2500kN/m2	2.50	0.398

And then calculate the ratio between elements according to results as shown in Table 3.6.1-8, and the results are as shown in Table 3.6.1-9.

Table 3.6.1-9 Ratio between elements

Symbols	Normal times	Earthquake
logRT= logRs	7×0.147-0.613+2×0.398=1.212	7×0.217-0.613+2×0.398=1.702
log*RM*	8×0.147-0.613+2×0.398=1.359	8×0.217-0.613+2×0.398=1.919
log*Ri*	9×0.147-2×0.613+2×0.398=0.893	9×0.217-2×0.613+2×0.398=1.523
logRi	10×0.147-2×0.613+2×0.398=1.040	10×0. 217-2×0.613+2×0.398=1.740

According to the equation Tp=BHo, logTs can be obtained using the following equation:

$$\log T_s = \log T_p - \log R_T$$

Normal times: logTs = log(1.0×40.50) -1.212 =0.395

Earthquake: logTs =log(1.0×56.17) -1.702=0.048

Elements of standard pile obtained through interpolation are shown in Table 3.6.1-10.

Table 3.6.1-10 Elements of standard pile

Symbols	Normal times	Earthquake
log(*M*max) s	0.539	0.176
log(lml) s	0.316	0.268
log(yo) s	-3.483	-3.946
log(io) s	-3.345	-3.756

Elements of cold-bent steel sheet pile wall obtained are as shown in Table 3.6.1-11.

Table 3.6.1-11 Elements of cold-bent steel sheet pile wall

K	Normal times			Earthquake
logK	*K*		logK	*K*
Mmax	0.539+1.359=1.898	79.07kN·m	0.176+1.919=2.095	124.45kN·m
lml	0.316+0.147=0.463	2.90m	0.268+0.217=0.485	3.05m
yo	-3.483+ 1.040=-2.443	0.36cm	-3.946+1.740=-2.206	0.62cm
*i*o	-3.345+ 0.893=-2.452	3.532×10-3rad	-3.756+1.523=-2.233	5.848×10-3rad

Hence, the displacement δ of the top of cold-bent steel sheet pile is as shown below:

In normal times:

$$\delta = y_0 + i_0(h+x) + \delta_3$$
$$= 0.36 + 3.532\times10^{-3}(300+45) + 0.35 = 1.93cm \le 5cm$$

At earthquake:

$$\delta = y_0 + i_0(h+x) + \delta_3$$
$$= 0.62 + 5.848\times10^{-3}(300+95) + 0.74 = 3.67cm \le 10cm$$

Bending stress intensity σ is as shown below:

In normal times:

$$\sigma = \frac{M_{\max}}{z} = \frac{79.07\times10^6}{1152\times10^3} = 68.6N/mm^2 \le 215N/mm^2$$

At earthquake: $\sigma = \frac{M_{\max}}{z} = \frac{124.45\times10^6}{1152\times10^3} = 108.0N/mm^2 \le 215N/mm^2$

Length l of cold-bent steel sheet pile is calculated assuming that the elevation of the top thereof is +1.50m. In such a case, the necessary buried depth of cold-bent steel sheet pile must be 1.5 times zero point 1 (lm1) of bending moment.

In normal times: lm1=2.90m, $l = 2.50 + 0.45 + 1.5\times2.90 = 7.3m$

At earthquake: lm1=3.05m, $l = 2.50 + 0.95 + 1.5\times3.05 = 8.0m$

According above-noted results, when the cold-bent steel sheet pile used is model GPU11a, l=8.0m.

3.6.2 Anchor-type Cold-bent Steel Sheet Pile Wall

As a structure that balances earth pressure, water pressure and other loads through the lateral resistance of buried part and the anchoring device connected with pull anchor, anchor-type cold-bent steel sheet pile wall is frequently used for relatively high quay wall and revetment of which the buried foundation is relatively weak.

A computational example is used to clarify the method for calculation about anchor-type cold-bent steel sheet pile wall:

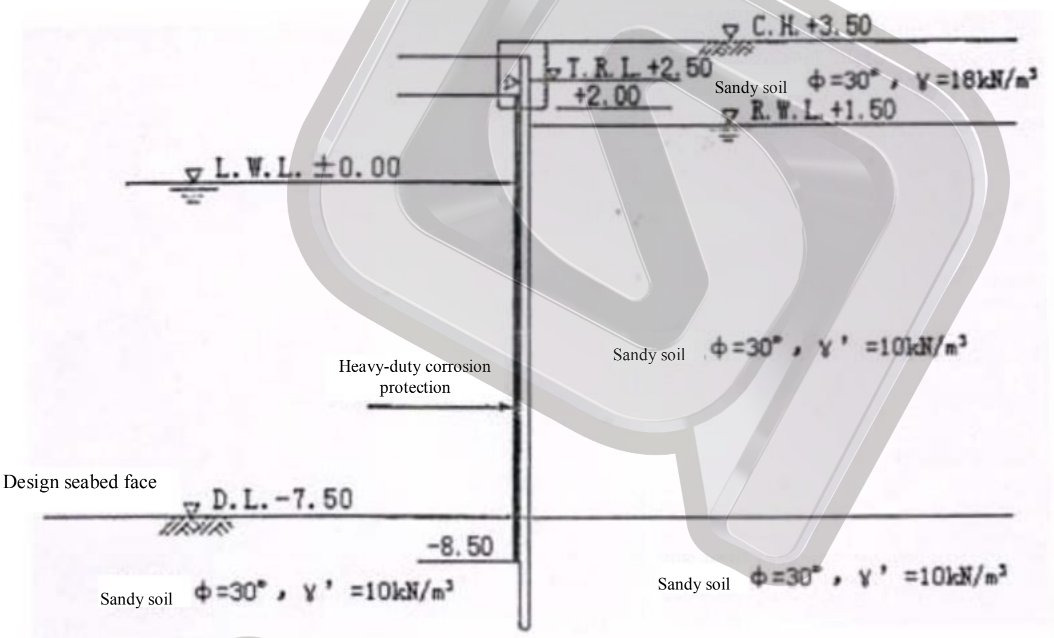


Fig. 3.6.2-1 Computational example of anchor-type cold-bent steel sheet pile

(1) 设计工况

As shown above:

① Elevation

Crest height: C.H.=+3.50m

Design water depth: D.L.=-7.50m

Residual water level: R.W.L.=+1.50m

Low water level: L.W.L.=±0.00m

Mount point of pull anchor: T.R.L.=+2.50m

② Topside load

$q = 20kN/m^2$ (in normal times), $q' = 10kN/m^2$ (at earthquake)

③ Soil conditions

Internal friction angle $\varphi = 30^\circ$; bulk density $\gamma = 18kN/m^3$, $\gamma' = 10kN/m^3$ (in water)

④ Bulk density of sea water: $\gamma_w = 10.1kN/m^3$

⑤ Design earthquake intensity

Land earthquake intensity: $k = 0.20$

Apparent earthquake intensity in water at Layer 2 (1.5m~-7.5m) $k' = 0.30$

Apparent earthquake intensity in water at Layer 3 $k' = 0.35$

⑥ Wall friction angle

Active $\delta_a = +15^\circ$; passive $\delta_p = -15^\circ$

⑦ Scope of heavy-duty corrosion protection: Front side until design seabed -1.0m (-8.50m) is provided with heavy-duty corrosion protection

(2) Earth pressure and residual water pressure

① Earth pressure coefficient

Table 3.6.2-1 shows the calculated earth pressure coefficient.

Table 3.6.2-1 Earth pressure coefficient

Internal friction angle φ	Wall friction angle		Earthquake intensity	Earth pressure coefficient		Failure angle	
	Active	Passive		Active $K_a \cos\delta_a$	Passive $K_p \cos\delta_p$	Active	Passive
30°	15°	-15°	0.00	0.291	4.807	56.9°	20.7°
			0.20	0.437	3.988	45.3°	18.5°
			0.30	0.543	3.545	37.8°	16.9°
			0.35	0.611	3.309	33.6°	15.9°
			0.40	0.693	3.058	28.8°	14.7°

② Earth pressure intensity, residual water pressure intensity and flowing pressure intensity

The calculation results of earth pressure intensity, residual water pressure intensity and flowing pressure intensity at each soil layer boundary surface and intensity variation point in normal times and at earthquake as shown in Fig. 3.6.2-2 through Fig. 3.6.2-3 are shown in Table 3.6.2-2 through 3.6.2-3.

In normal times:

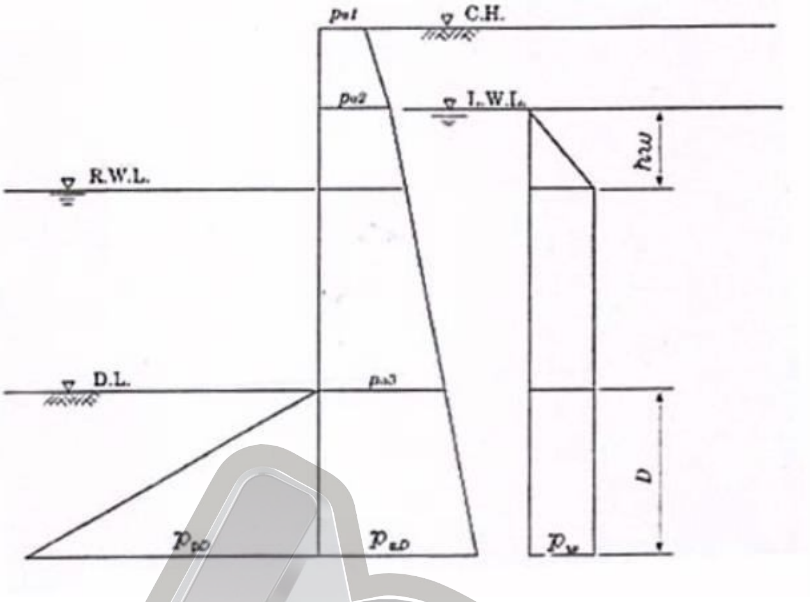


Fig. 3.6.2-2 Diagram of load distribution in normal times

Table 3.6.2-2 Earth pressure and residual water pressure intensity in normal times

i	Elevation (m)	Layer thickness (m)		Earth pressure coefficient Ki	Load ωi(kN/m2)	Earth pressure, residual water pressure, flowing pressure intensity pi= Kiωi, pw(kN/m2)
al	+3.50	-	0.291	20	5.82	
a2	+1.50	2.00	0.291	56	16.30	
a3	-7.50	9.00	0.291	146	42.49	
aD	-7.50-D	D	0.291	146+10D	42.49	2.91D
w				15.15	15.15	
pD	-7.50-D	D		4.807	10D	48.07D

At earthquake:

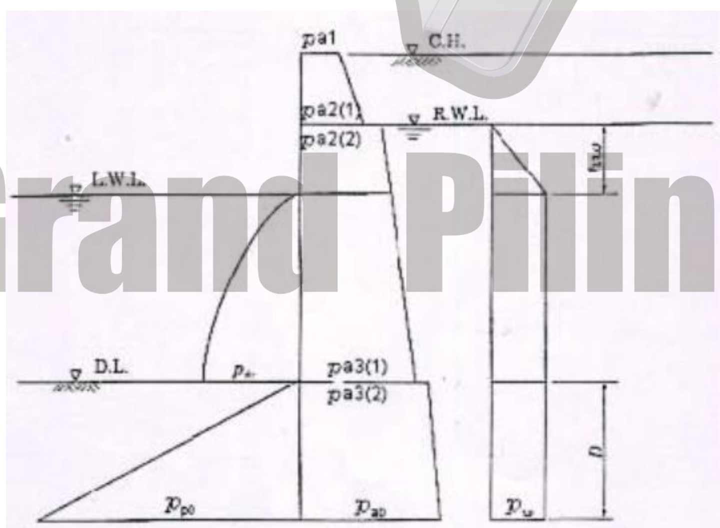


Fig. 3.6.2-3 Diagram of load distribution at earthquake

Table 3.6.2-3 Earth pressure, residual water pressure and flowing pressure intensity at earthquake

i	Elevation (m)	Layer thickness (m)	Earthquake intensity K(k′)	Earth pressure coefficient Ki	Load ωi(kN/m2)	Earth pressure, residual water pressure, flowing pressure intensity

						pi= Kiωi，pw，pdwi(kN/m2)	
a1	+3.50	-	0.20	0.437	10	4.37	
a2(1)	+1.50	2.00	0.20	0.437	46	20.10	
a2(2)	+1.50	9.00	0.30	0.543	46	24.98	
a3(1)	-7.50		0.30	0.543	136	73.85	
a3(2)	-7.50	9.00	0.35	0.611	136	83.10	
aD	-7.50-D	D	0.35	0.611	136+10D	83.10	+6.17D
w					15.15	15.15	
dw pD	-7.50 -7.50-D	 D	0.200 0.400		 3.058	13.26 10D	* 30.58D

③ Horizontal force and bending moment

Table 3.6.2-4 through 3.6.2-5 show the earth pressure and residual water pressure in normal times and at earthquake as shown in Fig. 3.6.2-2 through 3.6.2-3 and the bending moment caused by these forces in vicinity of pull anchor mount point.

In normal times:

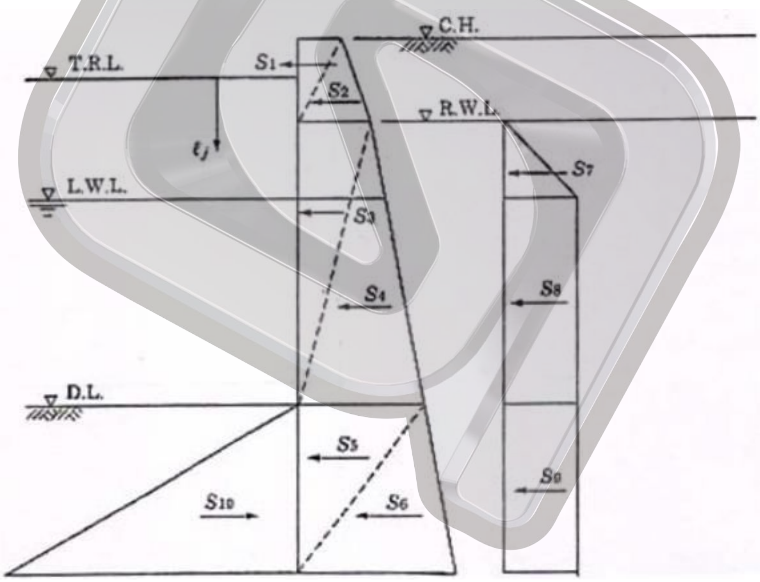


Fig. 3.6.2-4 Diagram of load in normal times

Table 3.6.2-4 Horizontal force and bending moment in normal times

j	Horizontal force Sj(kN/m)	Distance lj(m)	Bending moment Mj= Sjlj(kN·m/m)
1	5.82	-0.333	-1.94
2	16.30	0.333	5.43
3	73.35	4.00	293.40
4	191.21	7.00	1338.47
5	21.25D	10+1/3D	215.5D+7.08D2
6	21.25D+ 1.455D2	10+2/3D	215.5D+28.72D2+0.97D3
7	11.36	2.00	22.72
8	113.63	6.25	710.19
9	15.15D	10+1/2D	151.5D+7.58D2
10	24.04D2	10+2/3D	240.40D2+ 16.03D3

At earthquake:

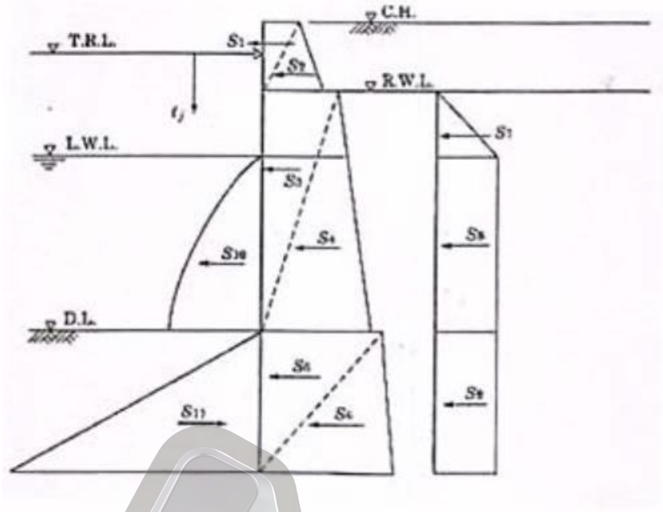


Fig. 3.6.2-5 Diagram of load at earthquake

Table 3.6.2-5 Horizontal force and bending moment at earthquake

j	Horizontal force Sj(kN/m)	Distance lj(m)	Bending moment Mj= Sjlj(kN·m/m)
1	4.37	-0.333	-1.46
2	20.10	0.333	6.69
3	112.41	4.00	449.64
4	332.33	7.00	2326.31
5	41.55D	10+1/3D	415.5+13.85D2
6	41.55D+3.055D2	10+2/3D	415.5D+58.25D2+2.04D3
7	11.36	2.00	22.73
8	113.63	6.25	710.19
9	15.15D	10+1/2D	151.5D+7.58D2
10	66.28	7.00	463.96
11	15.29D2	10+2/3D	152.90D2+10.19D3

(3) Design of cold-bent steel sheet pile wall

① Buried depth of cold-bent steel sheet pile wall

In normal times: 14.57D3+175.33D2-864.75D-3552.41≥0 ∴ D≥5.8m

At earthquake: 7.74D3+57.28D2-1179.00D-4773.66≥0 ∴ D≥11.2m

Therefore, if the elevation of top of cold-bent steel sheet pile is +3.0m, the length (l) of cold-bent steel sheet pile would be

l≥3.0+7.5+11.2=21.7m (l=22.0m)

② Support reaction

As shown in Fig. 3.6.2-6, support reaction is obtained by taking cold-bent steel sheet pile as freely supported beam supported by pull anchor mount point and the assumed hinge point set at seabed surface in normal times and or at earthquake.

In normal times:

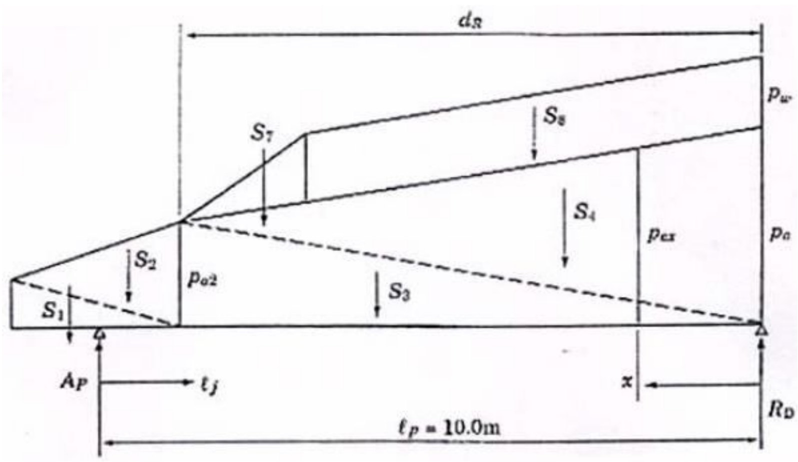


Fig. 3.6.2-6 Freely supported beam model in normal times

Resultant force above seabed surface as shown in Table 3.6.2-4 is shown as follows:

$$\sum S_j = \sum_1^4 S_j + \sum_7^8 S_j = 411.67kN/m$$

$$\sum M_j = \sum_1^4 M_j + \sum_7^8 M_j = 2368.27kN \cdot m/m$$

Thereby, support reaction is $R_D = \dfrac{\sum M_j}{l_D} = 237kN/m$, $A_p = \sum S_j - R_D = 175kN/m$.

At earthquake:

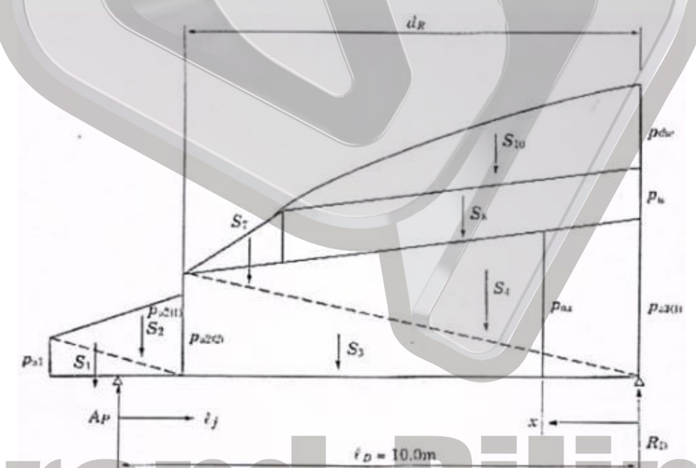


Fig. 3.6.2-6 Freely supported beam model at earthquake

Resultant force above seabed surface as shown in Table 3.6.2-5 is shown as follows:

$$\sum S_j = 660.18kN/m$$

$$\sum M_j = 3978.05kN \cdot m/m$$

Thereby, support reaction is $R_D = 398kN/m$, $A_p = 262kN/m$.

③ Maximum bending moment

In normal times:

Obtain $x = 4.66m$ by taking Qx=0 as the position at which the maximum bending moment is generated.

And then the active earth pressure intensity $p_{ax} = 28.93kN/m^2$ and bending moment $M_{max} = 527.65kN \cdot m/m$ are obtained.

At earthquake:

与平时一样，以Qx=0作为最大弯矩生成的位置，求出$x = 4.54m$和最大弯矩$M_{max} = 852.86kN \cdot m/m$。

Obtain $x = 4.54m$ and the maximum bending moment $M_{max} = 852.86kN \cdot m/m$ by taking Qx=0 as the position where maximum bending moment is generated as in normal times.

④ Correct the buried depth and sectional force of cold-bent steel sheet pile

Check the buried depth of cold-bent steel sheet pile determined by earth-base free support method; if buried depth is not adequate, it should be corrected. If that is the case, the maximum bending moment and reaction of binding rod mount point calculated by hinge point assumption method shall also be corrected.

a. Assumption of each unit of cold-bent steel sheet pile

It is assumed that cold-bent steel sheet pile model GPU35b is used:

Buried depth DF=115.50m; Young's modulus $E=2.0\times10^5 MN/m^2$; section inertia moment $I=10.75\times10^{-4} m^4/m$

b Foundation reaction efficient of cold-bent steel sheet pile wall

Get solution using $\varphi = 30^\circ$, $l_h = 27MN/m^3$.

c The distance between tie rod mount point and seabed surface HT =10.00 m

d Similarity coefficient and flexibility coefficient

$$\rho = H_T^4 / EI = 10.0^4 / (2.0\times10^5 \times 10.75\times10^{-4}) = 46.51$$

$$\varpi = \rho \cdot l_h = 46.51\times27 = 1256$$

Check the buried depth of cold-bent steel sheet pile:

DF/HT =1.15≥5.0916×ω-0.2-0.2591 = 0.9629

And hence, buried depth D=11.50m (total length L=22.00m) is sufficient.

e Correction of maximum bending moment

In normal times:

μN = 3.8625×ω-0.2+0.2255 =1.1525

MFN =μN×MTN = 1.1525×527.65 = 608.12kN ·m/m

At earthquake:

μS = 4.5647×ω-0.2+ 0.1329 = 1.2284

MFS =μS×MTS = 1.2284×852.86 = 1047.68kN· m/m

f Correction of reaction at binding rod mount point

In normal times:

μN = 1.8259×ω-0.2 + 0.6232 = 1.0614

MFN =μN×MTN =1.0614×175=185.75kN·m/m

At earthquake:

μS = 2.3174×ω-0.2 + 0.5514 = 1.1076

MFS =μS×MTS = 1.1076×262 = 290.18kN· m/m

⑤ Determine the section of cold-bent steel sheet pile

Obtain necessary section factor based on maximum bending moment.

In normal times:

$$Z \geq \frac{M_{max}}{\sigma_a} = \frac{608.12}{215\times10^3} = 2.83\times10^{-3} m^3 \leq 3.55\times10^{-3} m^3$$

At earthquake:

$$Z \geq \frac{M_{max}}{\sigma_a} = \frac{1047.68}{258\times10^3} = 4.06\times10^{-3} m^3 \leq 4.26\times10^{-3} m^3$$

Corrosion margin thickness obtained by assuming that the life cycle is 50 years:

tl=corrosion margin thickness at sea side: 0mm/year×50 years=0mm (heavy-duty corrosion protection)
t2=corrosion margin thickness at land side: 0.2mm/year×50 years=1.0mm
Accordingly, when cold-bent steel sheet pile model GPU35b is used, l=22.0m.

3.6.3 Unit-type Cold-bent Steel Sheet Pile Wall

Change most straight cold-bent steel sheet piles into circular or circular arc shaped piles and fill them with sand or grit to form a continuous wall; unit-type cold-bent steel sheet pile wall is a structure that resists external forces by creating a whole through thin steel shell and filling material. If there is a favorable foundation below seabed surface, the construction of stable structure is possible due to the fact that sufficient bearing capacity is expected from this structure when a small part of straight cold-bent steel sheet pile is buried in earth. Furthermore, the comparison of steel shell with U-shaped cold-bent steel sheet pile showed that straight cold-bent steel sheet pile with smaller unit weight realizes the optimal use of its tensile strength and features small consumption of steel and economic structure. However, its resistance to wave is relatively weak before the completion of filling, so adequate discussion shall be performed before construction.

A computational example is used to clarify the method for calculation about unit-type cold-bent steel sheet pile wall:

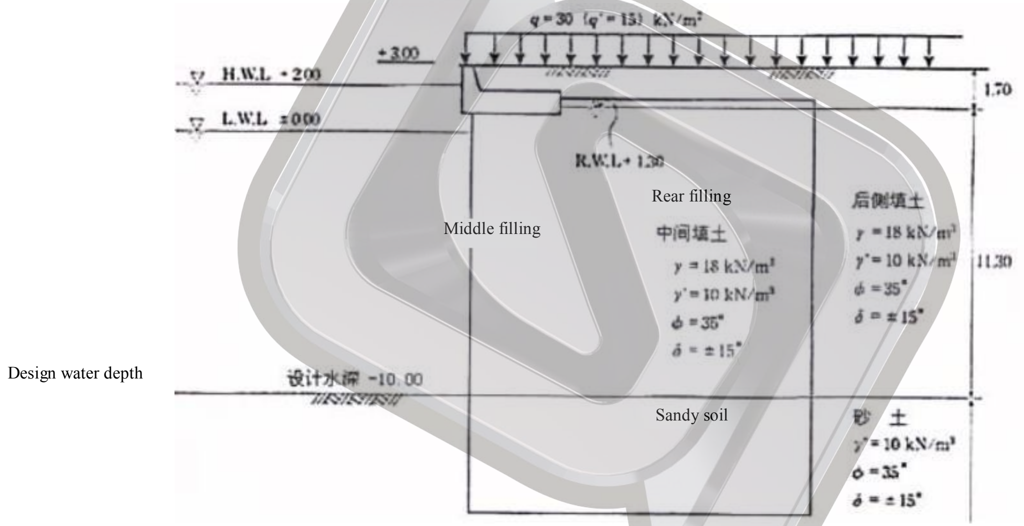


Fig. 3.6.3-1 Computational example of unit-type cold-bent steel sheet pile wall

(1) Design conditions
As shown above:

① Elevation

Crest height: C.H.=+3.00m
Design water depth: D.L.=-10.00m
Residual water level: R.W.L.=+1.30m
Low water level: L.W.L.=±0.00m

② Topside load

$q = 30kN/m^2$ (in normal times), $q' = 15kN/m^2$ (at earthquake)

③ Soil conditions

As shown in Fig. 3.6.3-1

Foundation reaction coefficient in horizontal direction: kh=20N/cm3
Foundation reaction coefficient in vertical direction: kv=kh=20N/cm3
Shearing rigidity coefficient: kS=kv/3=6N/cm3

④ Design earthquake intensity: $k = 0.15$

(2) Calculate converted wall width

In case of circular unit, T-shaped sheet pile connecting main body with circular arc shaped part is as shown in Fig. 3.6.3-2, and the direction angle θ_1=45°.At this point, the number of unit bodies must be integral multiple of the number "4".The radius of unit body is about 60% of converted wall width, and the center distance of unit is 1.1 times the diameter of unit body under most circumstances; each element of unit is assumed on this basis according to Fig. 3.6.3-2.

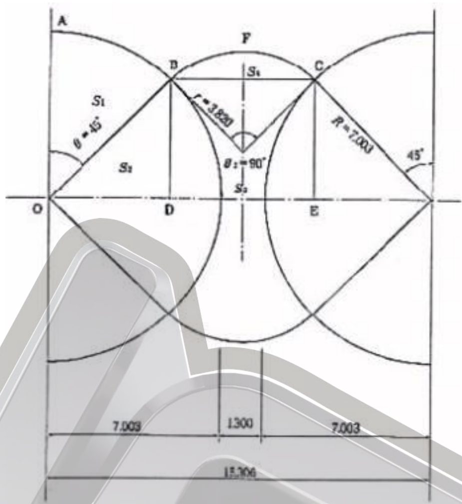


Fig. 3.6.3-2 Planar shape of unit

$R = 7.003m$, $r = 3.820m$, $L = 15.306$, $R = 7.003m$, $R = 7.003m$

Number of sheet piles of unit body: nR=88set

Number of sheet piles of circular arc shaped section: nr=24set

$$S_1 = \triangledown\ OAB \times 2 = \frac{\pi}{180°} \times 7.003^2 \times 45° = 38.52m^2$$

$$S_2 = \triangle\ OBD \times 2 = \frac{7.003^2}{2} \times \sin 90° = 24.52m^2$$

$$S_3 = \square\ BCED = 2 \times 7.003 \times 3.820 \times \cos 45° \times \sin 45° = 26.75m^2$$

$$S_4 = \frown \widehat{BFC} = \left(\frac{\pi \times 90°}{360°} - \frac{1}{2} \times \sin 90°\right) \times 3.820^2 = 4.165m^2$$

$$\sum S_{1\text{-}4} = 93.96m^2$$

$$2\sum S_{1-4} = 187.9m^2$$

$$\therefore \quad B = \frac{2\sum S_{1-4}}{L} = \frac{187.9}{15.306} = 12.28m$$

(3) Checking calculation of shearing deformation

① Load intensity distribution for calculation of displacement bending moment

The calculation of earth pressure acting on wall body is as shown in Fig. 3.6.3-3

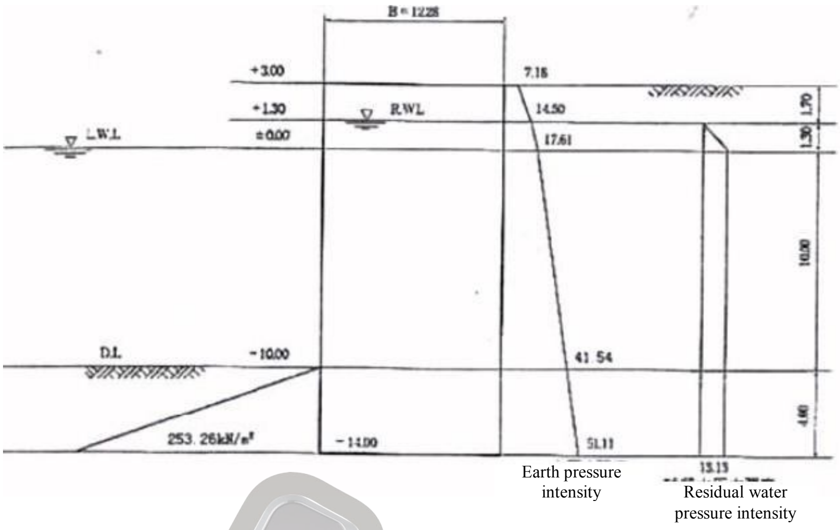


Fig. 3.6.3-3 Distribution of load intensity

② Result of checking calculation of shearing deformation

Checking calculation results relating to shearing deformation are shown in Table 3.6.3-1.

Table 3.6.3-1 Results of checking calculation of shearing deformation

	Seabed surface	Unit bottom surface
Deformation bending moment Md (kN·m/m)	2469.17	4156.52
Resisting moment of fill Mf(kN·m/m)	4758.09	3028.20
Interface friction resisting moment MS(kN·m/m)	0.00	2146.46
Mt=Mf+MS (kN·m/m)	4758.09	5174.66
Wall height H(m)	13.00	13.00
Buried depth D(m)	4.00	4.00
Coefficient α	0.00	1.00
R=1+ α·D/H	1.00	1.31
Resisting moment Mr=Mt·R(kN·m/m)	4758.09	6766.87
Safety coefficient Mr/Md	1.93	1.63
Minimum safety coefficient	1.20	1.20
Comment	OK	OK

(4) Checking calculation of stability of gravity wall

① Load acting on wall body

In normal times:

Horizontal load: calculate the horizontal component of earth pressure and the residual water pressure

Rotation center of bending moment is design seabed surface.

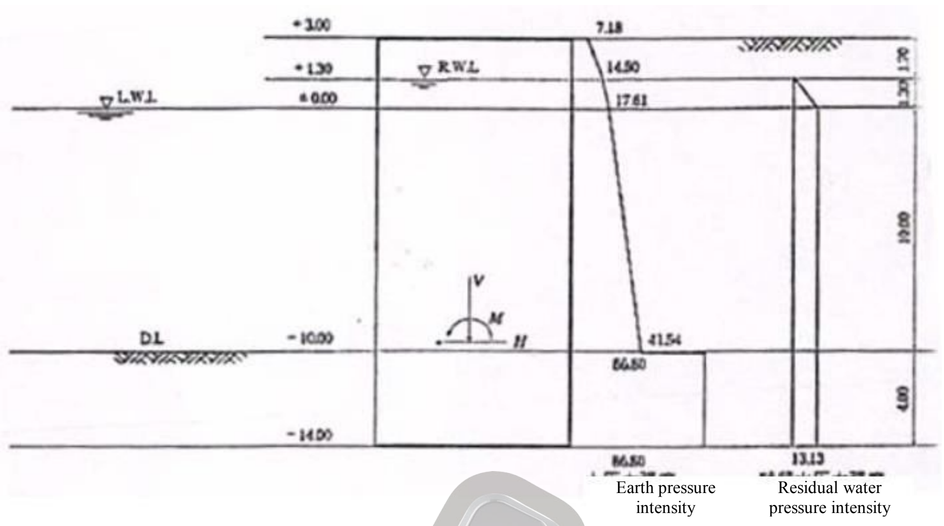


Fig. 3.6.3-4 Distribution of load intensity

At earthquake:

Horizontal load: calculate the horizontal component of earth pressure, the residual water pressure and the seismic force acting on wall body

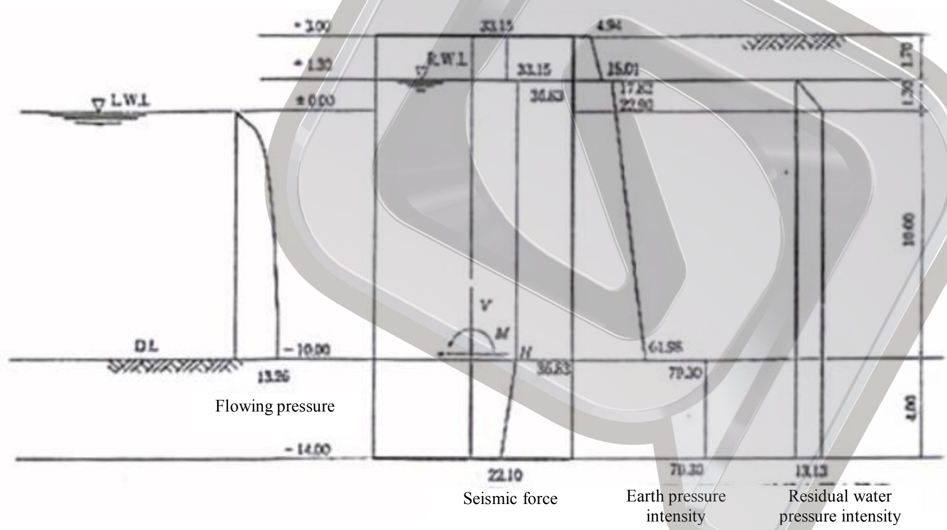


Fig. 3.6.3-5 Distribution of load intensity

Residual water pressure intensity

② Calculate foundation reaction and wall displacement

Calculate the foundation reaction by taking wall body as a rigid body elastically supported by foundation.

Horizontal foundation reaction does not exceed the passive earth pressure intensity determined by taking into account foundation yield.

The following results are obtained through convergence calculation.

In normal times:

$\theta = 0.00089rad$, $h = 11.10m$, $e = 10.63m$

At earthquake:

$\theta = 0.00698rad$, $h = 9.23m$, $e = -0.32m$

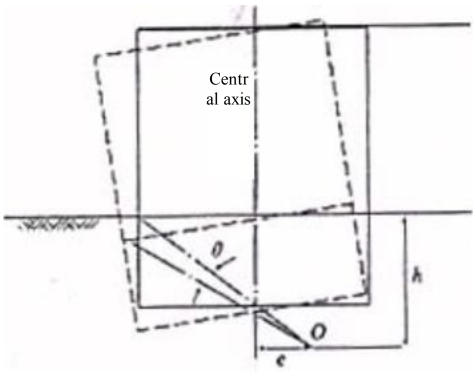


Fig. 3.6.3-6 Displacement mode of wall body

Table 3.6.3-2 Calculation results of bottom surface reaction (H.V)

	In normal times	At earthquake
Converted levee body width of unit B(m)	12.28	12.28
Design seabed face elevation (m)	-10.00	-10.00
Unit bottom surface elevation (m)	-14.00	-14.00
Buried depth of unit D(m)	4.00	4.00
Foundation reaction coefficient of unit bottom surface kv(kN/m3)	20000	20000
kh(kN/m3)	6000	6000
Rotation center h(m)	11.10	9.23
e(m)	10.63	-3.2
θ(rad)	0.00089	0.00698
Vertical reaction Displacement δvl (m)	0.0150	0.0406
δv2(m)	0.0040	-0.0451
Distribution length b(m)	12.28	5.816
Intensity q1(kN/m2)	299.10	811.52
q2(kN/m2)	80.08	0.00
Reaction Vh (kN/m)	2327.54	2359.92
Horizontal reaction Reaction Hb(kN/m)	466.73	1272.61

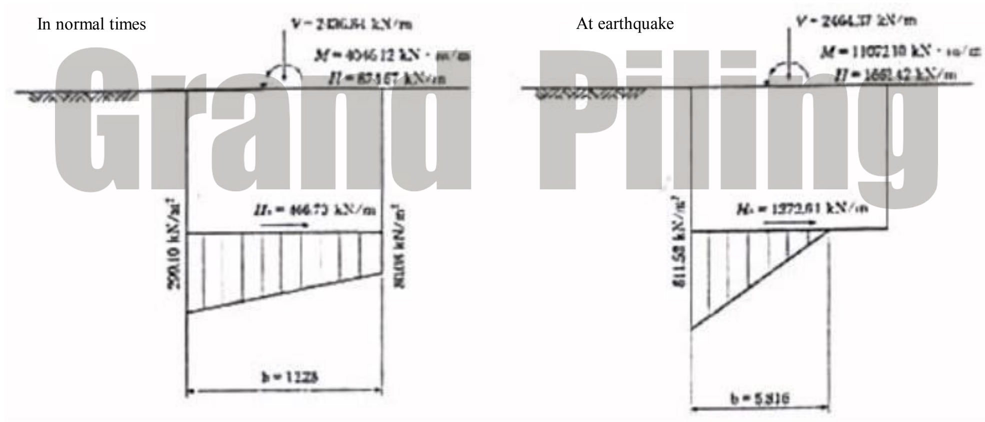
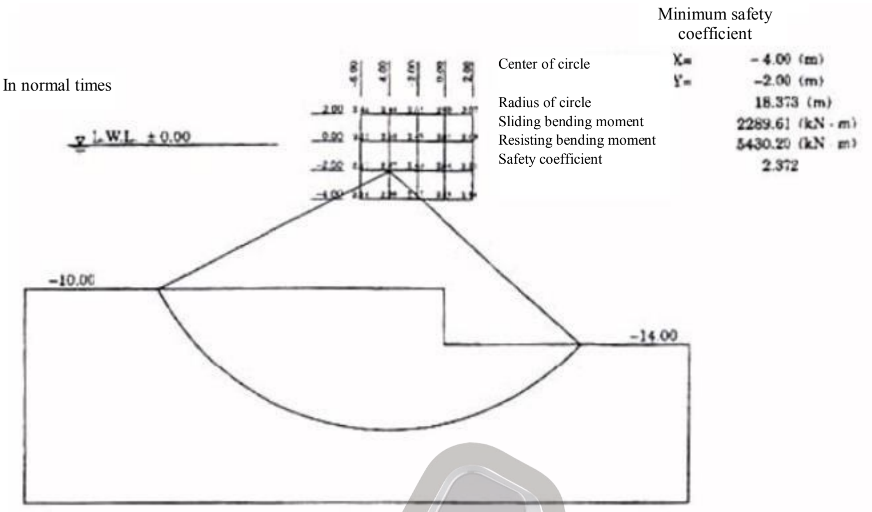
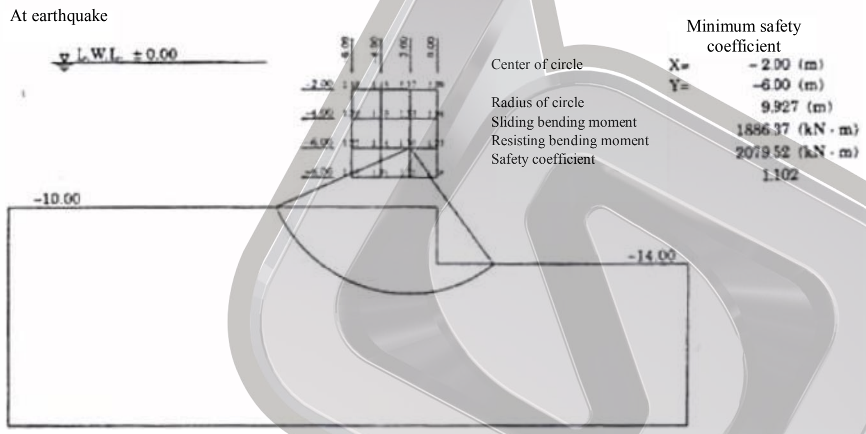


Fig. 3.6.3-7 Distribution of foundation reaction

③ Calculate the bearing capacity of foundation

To check the bearing capacity of foundation, the checking calculation of eccentric and inclined load shall be made by taking the foundation reaction obtained in ② as acting load





Calculation result:

Safety coefficient at normal times Fs=2.372≥1.20

Safety coefficient at earthquake Fs=1.102≥1.00

④ Calculate sliding shift of wall body

Safety coefficient in normal times:

$$F = \frac{2327.54 \times \tan 35^\circ}{466.73} = 3.49 > 1.2$$

Safety coefficient at earthquake:

$$F = \frac{2359.92 \times \tan 35^\circ}{1272.61} = 1.30 > 1.1$$

⑤ Checking calculation of displacement of revetment crest

In normal times:

Crest displacement $\delta = (13.00 + 11.10) \times 0.00089 = 0.021m$

$$\frac{0.021}{13.00} \times 100\% = 0.16\% < 1.5\%$$

At earthquake:

Crest displacement $\delta' = (13.00 + 9.23) \times 0.00698 = 0.155m$

$$\frac{0.155}{13.00} \times 100\% = 1.19\% < 1.5\%$$

(5) Checking calculation of pulling force of cold-bent steel sheet pile

Pulling force T of cold-bent steel sheet pile:
Main body

$$T = \{(18.0\times1.70 + 10.0\times11.30 + 30.0)\times0.700 + 10.1\times1.30\}\times7.003$$
$$=942.95kN / m < Ta = 1500kN / m$$

Circular arc shaped part

$$T = \{(18.0\times1.70 + 10.0\times11.30 + 30.0)\times0.350 + 10.1\times1.30\}\times3.820$$
$$=282.26kN / m < Ta =1500kN/m$$

(6) Standard section and plan

Standard section and plan are as shown in Fig. 3.6.3-8.

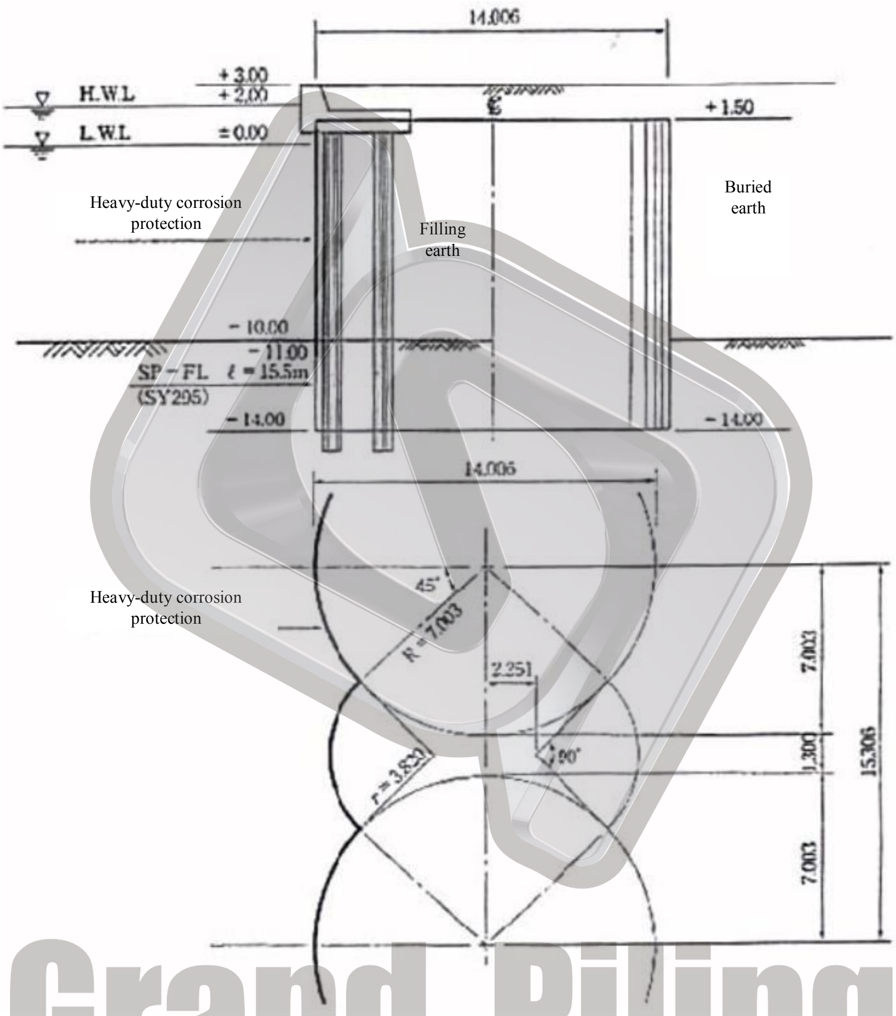


Fig. 3.6.3-8 Standard section and plan

3.6.4 Double-row Cold-bent Steel Sheet Pile Wall

Double-row cold-bent steel sheet pile wall is a structure in which the tops of double-row cold-bent steel sheet piles are connected using pulling anchor and high quality sandy soil is buried. In this structure, cold-bent steel sheet pile and sand fill works together to resist external force. Like stand-alone structure, this structure normally features high stability and perfect water retaining property; hence, it is frequently used for bank revetment, quay wall, wave breaker, diversion dike and temporary cofferdam etc.

The method for calculation of double-row cold-bent steel sheet pile wall is clarified below through a computational example:

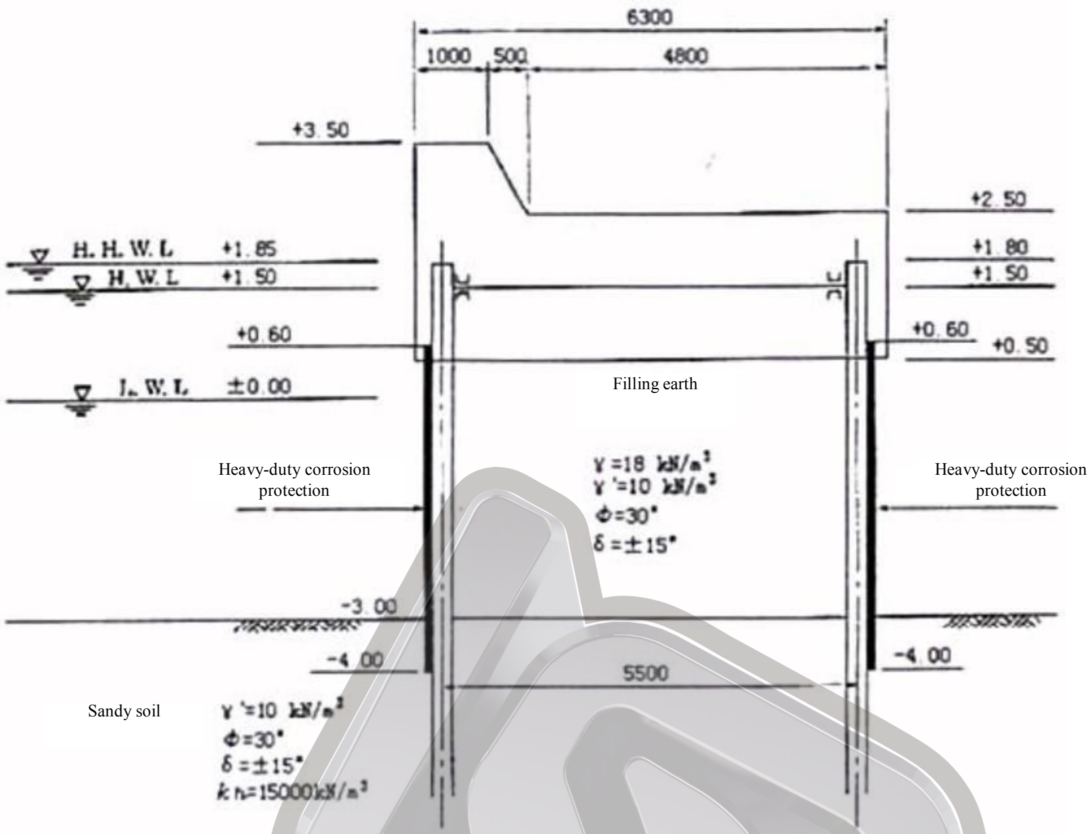


Fig. 3.6.4-1 Computational example of double-row cold-bent steel sheet pile wall

(1) Design conditions

As shown above

① Crest elevation

Crest elevation of wave breaker: +3.50m

Crest elevation at internal side of wave breaker: +2.50m

② Sea bed surface -3.00m

③ Design tide level

H.H.W.L. +1.85m

H.W.L. +1.50m

L.W.L. ±0.00m

R.W.L. +1.00m

④ Topside load 5kN/m2

⑤ Soil conditions

Earth fill:

Internal friction angle φ=30°

Bulk density γ= 18kN/m3，γ′= 10kN/m3

Wall friction angle δ=±15°

Basic foundation:

Internal friction angle φ=30°

Bulk density γ= 18kN/m3，γ′= 10kN/m3

Wall friction angle δ=±15°

Lateral foundation reaction coefficient kh=15000kN/m3

⑥ Design wave

Wave height H=2.00m
Cycle T=6.00sec

⑦ Allowable stress intensity

Cold-bent steel sheet pile σa=180N/mm2
Pulling anchor σa =94N/mm2
Steel channel σa=140N/mm2

⑧ Body width B=5.50m

(2) External force calculation

① Wave pressure intensity

H=2.00m, h=4.85m, hc=1.65m, L=37.71m

Thereof, $p_4 = 1.5\varpi H = 1.5 \times 10.1 \times 2.00 = 30.30 kN/m^2$

$$p_2 = \frac{\varpi H}{\cosh \dfrac{2\pi h}{L}} = \frac{10.1 \times 2.00^2}{1.3446} = 15.02 kN/m^2$$

$$\delta_0 = \frac{\pi H^2}{L} \coth \frac{2\pi h}{L} = \frac{3.14 \times 2.00^2}{37.71} \times 1.4959 = 0.50m$$

$$p_1 = (p_2 + \varpi h)\left(\frac{H + \delta_0}{h + H + \delta_0}\right) = (15.02 + 10.1 \times 4.85) \times \left(\frac{2.00 + 0.50}{4.85 + 2.00 + 0.50}\right)$$
$$= 21.77 kN/m^2$$

$$p_3 = \frac{H + \delta_0 - h_c}{H + \delta_0} p_1 = \frac{2.00 + 0.50 - 1.65}{2.00 + 0.50} \times 21.77 = 7.40 kN/m^2$$

② Resultant force and action point of wave pressure

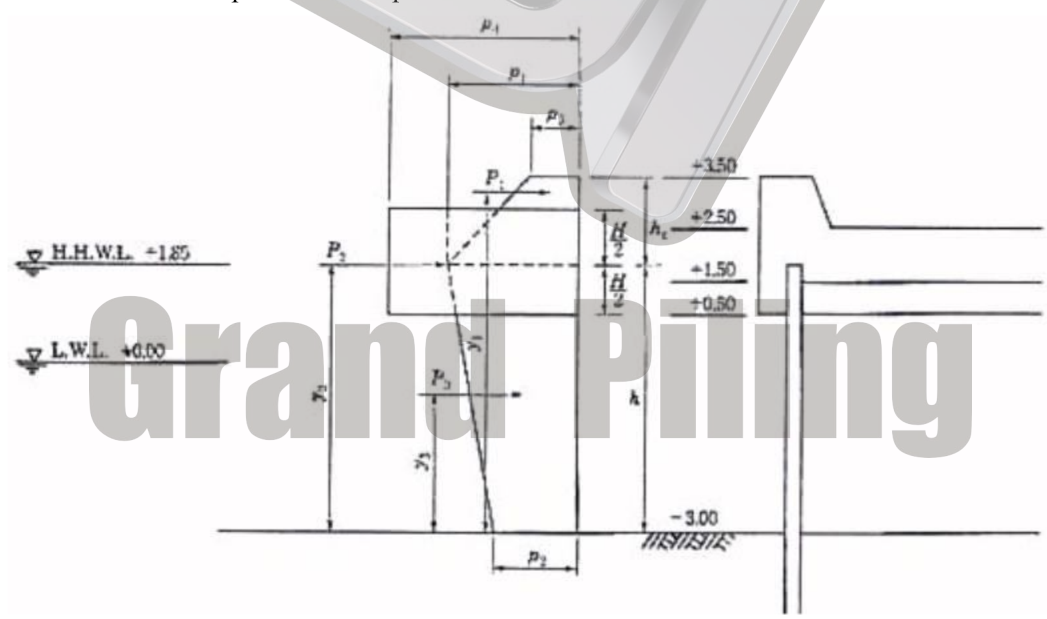


Fig. 3.6.4-2 Wave pressure acting on double wall

$$P_1 = 1/2 \times (7.40 + 13.06) \times 0.65 = 6.65 kN/m$$

$$P_2 = 2 \times 30.30 = 60.60 kN/m$$

$$P_3 = 1/2 \times (19.89 + 15.02) \times 3.85 = 67.20 kN/m$$

$$y_1 = \frac{2 \times 7.40 + 13.06}{7.40 + 13.06} \times \frac{0.65}{3} + 5.85 = 6.15m$$

$$y_2 = 4.85m$$

$$y_3 = \frac{2\times 19.89 + 15.02}{19.89 + 15.02} \times \frac{3.85}{3} = 2.01m$$

(3) Select the section of cold-bent steel sheet pile

The cold-bent steel sheet pile for checking calculation is model GPU20a.

Sectional modulus W=201×10-5m3/m

Inertia moment of section I=481×10-6m4/m

(4) Calculate effective dike height

① Elastic modulus Es of foundation

Thereof, $k_h = 15000kN/m^3 E_s = k_h \cdot B = 15000 \times 1.0 = 15000kN/m^2$

② Characteristic value β1

$$\beta_1 = \sqrt[4]{\frac{E_S}{4EI} \times 10^{-3}} = \sqrt[4]{\frac{15000}{4\times 2.0\times 10^5 \times 481\times 10^{-6}} \times 10^{-3}} = 0.444m^{-1}$$

③ Effective dike height

$$H = H_0 + \frac{1.5}{\beta_1} = (0.50 + 3.00) + \frac{1.5}{0.444} = 6.9m$$

(5) Calculate shearing deformation bending moment

Shearing deformation bending moment as a result of wave pressure is as follows:

$$\begin{aligned} M_0 &= \sum P_i \cdot y_i \\ &= 6.65\times(6.15+6.9-3.5) + 60.60\times(4.85+6.9-3.5) \\ &\quad + 67.20\times(2.01+6.9-3.5) \\ &= 927.01kN\cdot m/m \end{aligned}$$

(6) Calculate resisting bending moment of earth fill

Resistance bending moment as a result of shear resistance of earth fill is as shown below:

$$\begin{aligned} M_f &= \frac{1}{6}\gamma' \cdot H \cdot B^2\left(3 - \frac{B}{H}\cos\phi\right)\sin\phi \\ &= \frac{1}{6}\times 10\times 6.90\times 5.50^2 \times\left(3 - \frac{5.50}{6.90}\cos 30^\circ\right)\sin 30^\circ \\ &= 401.74kN\cdot m/m \end{aligned}$$

(7) Calculate the resistance bending moment of cold-bent steel sheet pile

The resistance bending moment acting on cold-bent steel sheet pile is as follows:

$$M_s = 1.2M_0 - M_f = 1.2\times 927.01 - 401.74 = 710.67kN\cdot m/m$$

(8) Stress intensity of cold-bent steel sheet pile

① Top displacement

$$Y_{TOP} = \frac{M_s \cdot H^2}{24EI}\times 10^{-3} = \frac{710.67\times 6.9^2}{24\times 2.0\times 10^5 \times 481\times 10^{-6}}\times 10^{-3} = 0.0147m$$

② Maximum bending moment of cold-bent steel sheet pile

$$M_{\max} = EI\beta_1^2 Y_{TOP} = 2.0\times 10^5 \times 481\times 10^{-6} \times 0.444^2 \times 0.0147\times 10^3 = 279kN\cdot m/m$$

③ Stress intensity of cold-bent steel sheet pile

$$\sigma = \frac{M_{\max}}{Z} \times 10^{-3} = \frac{279}{201 \times 10^{-5}} \times 10^{-3} = 139N/mm^2 \leq 215N/mm^2$$

(9) Calculate the buried depth of cold-bent steel sheet pile

$$\ell = \frac{5}{4} \cdot \frac{\pi}{\beta} = 8.84m$$

Thereof, if the crest elevation is +1.80m, the length (l) of cold-bent steel sheet pile should be
l = 1.80 + 3.00 + 8.84 = 13.64m→l= 14.00m

(10) Checking calculation of foundation bearing capacity

Take cold-bent steel sheet pile as a gravity-type structure of which the lower end is a bottom surface, and perform checking calculation of bearing pressure on foundation at a extremely small buried depth.

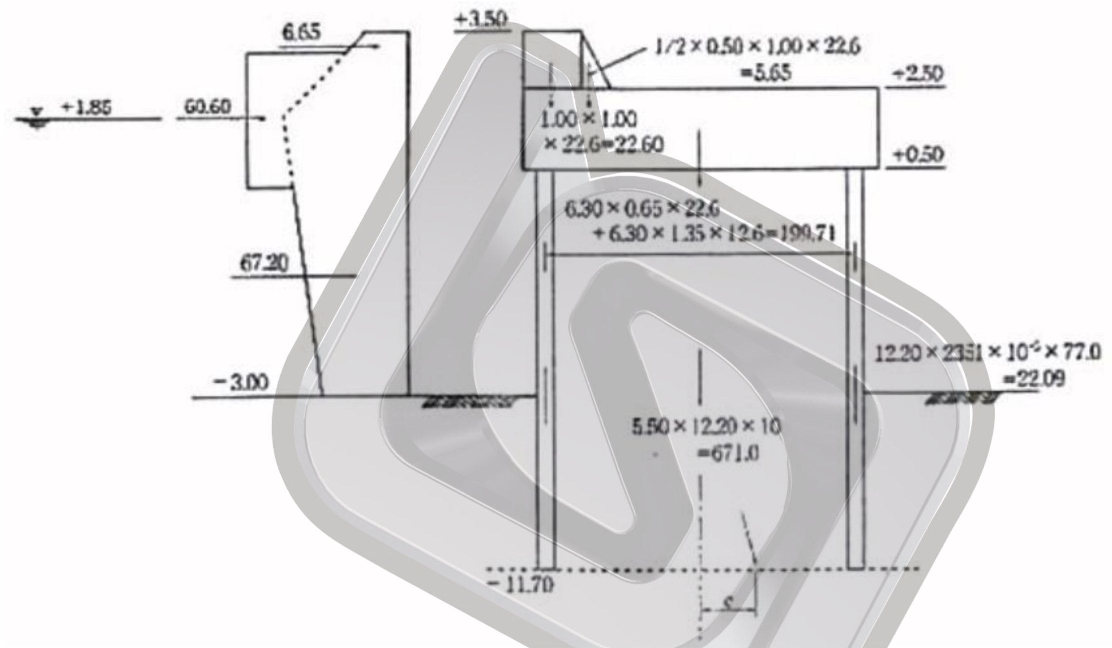


Fig. 3.6.4-3 Vertical resultant force of double wall

As shown in Fig. 3.6.4-3, the vertical resultant force V is:

$$\begin{aligned} V &= (1.00 \times 1.00 \times 22.6) + (1/2 \times 0.50 \times 1.00 \times 22.6) + (6.30 \times 0.65 \times 22.6) \\ &\quad + (6.30 \times 1.35 \times 12.6) + (5.50 \times 12.20 \times 10) + (12.20 \times 2351 \times 10^{-5} \times 77.0 \times 2) \\ &= 943.1kN/m \end{aligned}$$

Bending moment M at the center of bottom surface:

$$\begin{aligned} M &= 6.65 \times (6.15 + 8.70) + 60.60 \times (4.85 + 8.70) + 67.20 \times (2.01 + 8.70) \\ &\quad - 22.60 \times (6.30/2 - 1.0/2) - 5.65 \times (6.30/2 - 1.00 - 0.50/3) \\ &= 1568.5kN \cdot m/m \end{aligned}$$

Eccentric quantity e:

$$e = \frac{M}{V} = \frac{1568.5}{943.1} = 1.66mm$$

Since Bo=5.50m<e=1.66m here, the maximum foundation reaction p1 is:

$$p_1 = \frac{2}{3 \times \left(\frac{1}{2} - \frac{e}{B_0}\right)} \cdot \frac{V}{B_0} = \frac{2}{3 \times \left(\frac{1}{2} - \frac{1.66}{5.50}\right)} \times \frac{943.1}{5.50} = 576.8kN/m^2$$

If the "N" value near lower end of buried part of cold-bent steel sheet pile takes 20, the ultimate bearing capacity q_u and allowable bearing capacity q_a of shallow foundation are:

qu=βجγ1جBجNr+(bearing capacity increment as a result of lateral resistance)

=0.47×10×5.50×15+10×8.70×18+ (bearing capacity increment as a result of lateral resistance)
= 1953.8 + (bearing capacity increment as a result of lateral resistance)

qa =qu/F

= 1953.8/2.5 + (allowable bearing capacity increment as a result of lateral resistance)
= 1953.8 + (allowable bearing capacity increment as a result of lateral resistance)

Thereof, even if the increment of allowable bearing capacity as a result of q_a solely (without lateral resistance)

qa >p1

The bearing capacity of foundation will be secure

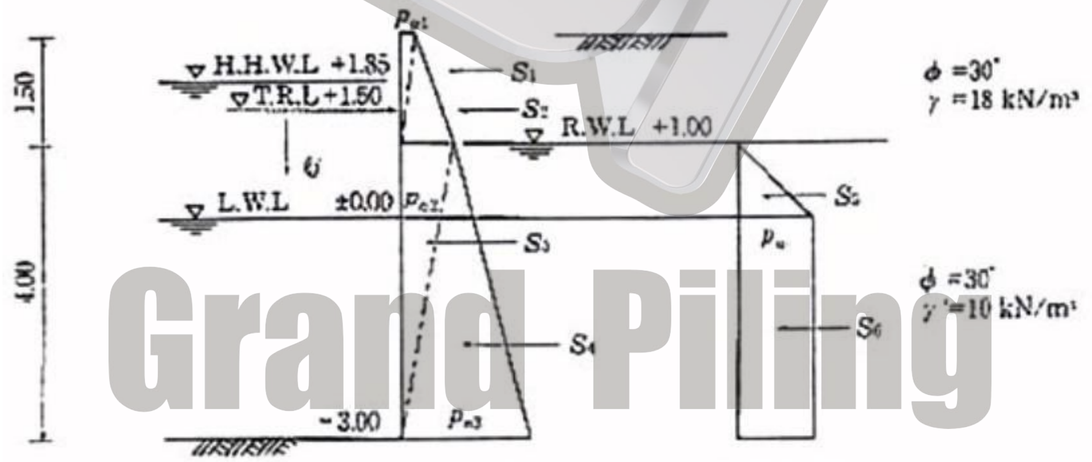
(11) Determine the section of pulling anchor

When φ=30° and δ=+15°, the coefficient of active earth pressure K_a • cosδ = 0.291

The earth pressure intensity and residual water pressure intensity at each soil layer boundary surface as shown in load diagram of Fig. 3.6.4-2 are listed in Table 3.6.4-1.

Table 3.6.4-1 Earth pressure intensity and residual water pressure intensity

i	Earth pressure coefficient Ki	Load ωi (kN/m2)	Earth pressure· Water pressure intensity pi= Ki·ωi(kN/m2)
al	0.291	5.0	1.46
a2	0.291	32.0	9.31
a3	0.291	72.0	20.95
w	-	10.1	10.0



Active earth pressure Residual water pressure

Fig. 3.6.4-4 Load diagram

Table 3.6.4-2 shows the pressure as shown in Fig. 3.6.4-2 and the residual water pressure induced bending moments at mount point of pulling anchor.

Table 3.6.4-2 Horizontal force and bending moment

j	Earth pressure+residual water pressure Sj(kN/m)	Distance lj(m)	Bending moment Mj= Sj·lj(kN·m/m)
1	1.10	-0.500	-0.55
2	6.98	0.000	0.00
3	18.62	1.833	34.13
4	41.90	3.167	132.70
5	5.05	1.167	5.89
6	30.30	3.000	90.90

Obtain reaction of support by taking cold-bent steel sheet pile as simply-supported beam supported by pulling anchor mount point and the assumed hinge point set at seabed face. According to Table 3.6.4-2, the resultant force above seabed surface is:

$$\sum P = \sum_{1}^{6} S_j = 103.95kN/m$$

$$\sum M = \sum_{1}^{6} M_j = 263.07kN \cdot m/m$$

Thereof, the reaction of assumed hinge point under seabed surface is:

$$R_D = \frac{\sum M}{l_D} = \frac{263.07}{4.5} = 58.64kN/m$$

$$A_p = \sum P - R_D = 103.95 - 58.46 = 45.49kN/m$$

Calculate necessary diameter assuming that the mounting interval of pulling anchor l_p=1.60m.Since there is protective concrete layer, corrosion margin thickness is not necessarily considered when calculating pulling force T.

$$T = A_p \cdot l_p = 45.49 \times 1.60 = 72.78kN$$

$$d = \sqrt{\frac{4T}{\pi \cdot \sigma_a}} = \sqrt{\frac{4 \times 72.78 \times 10^3}{3.14 \times 94}} = 31.4mm$$

Thereof, pulling anchor φ32mm is used.

(12) Determine the section of surrounding purlin

Steel channel combination is used as surrounding purlin.

The maximum bending moment on surrounding purlin:

$$M_{max} = \frac{1}{10} A_p l_p^2 = \frac{1}{10} \times 45.49 \times 1.60^2 = 11.65kN \cdot m$$

$$Z = \frac{M_{max}}{\sigma_a} = \frac{11.65}{140} \times 10^{-3} = 8.32 \times 10^{-5} m^3$$

Therefore, surrounding purlin employs 2[-125×65×6×8(Z=2×678×10-7m3) 。

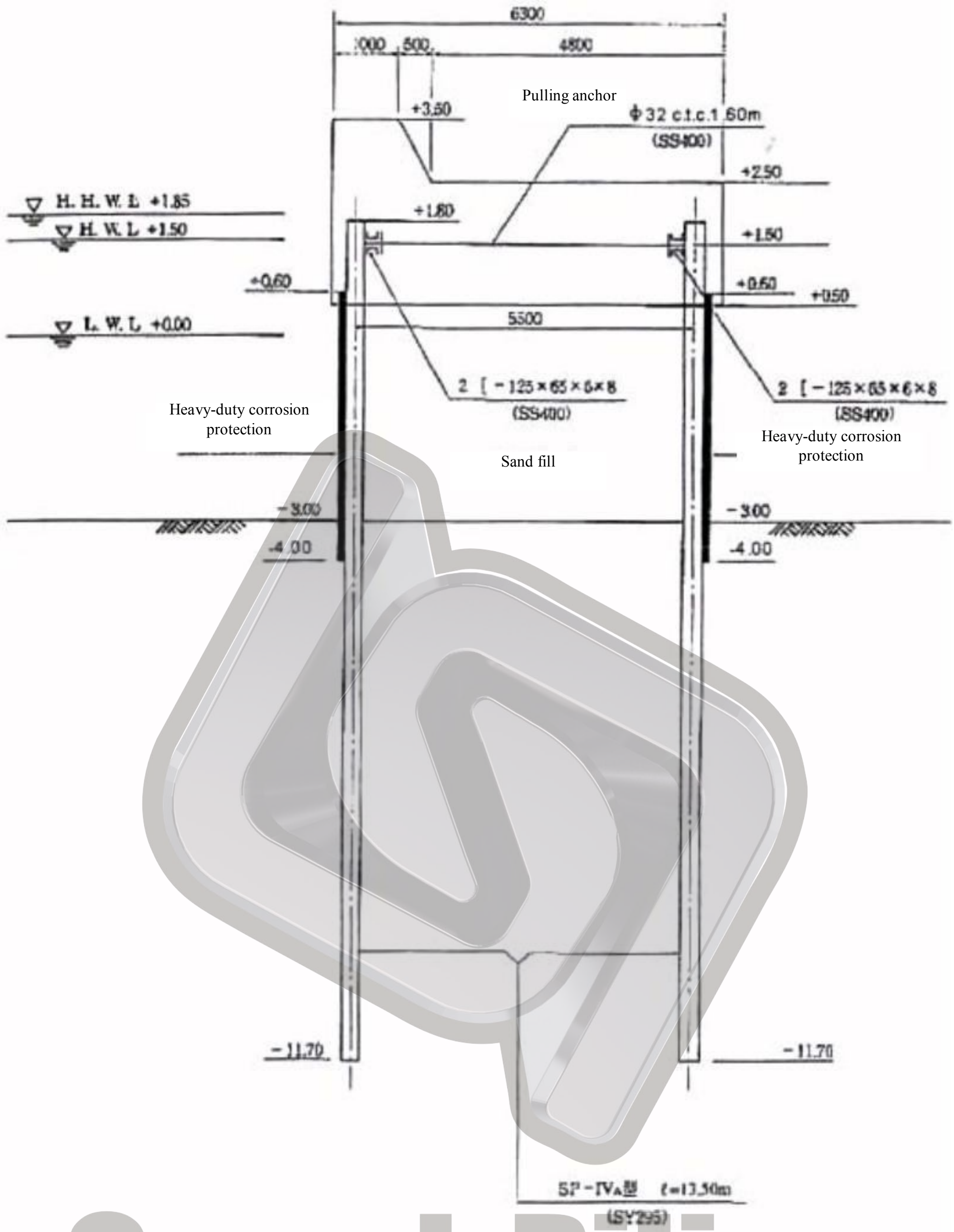


Fig. 3.6.4-5 Standard section

3.6.5 Temporary Structure

Cold-bent steel sheet pile is widely used as temporary structure and is divided into the following classes by purposes: retaining wall for excavation, retaining wall, temporary bank revetment, enclosed survey engineering, temporary built island, and seep-proof screen etc.

The method for calculation about temporary structure is clarified below through a computational example:

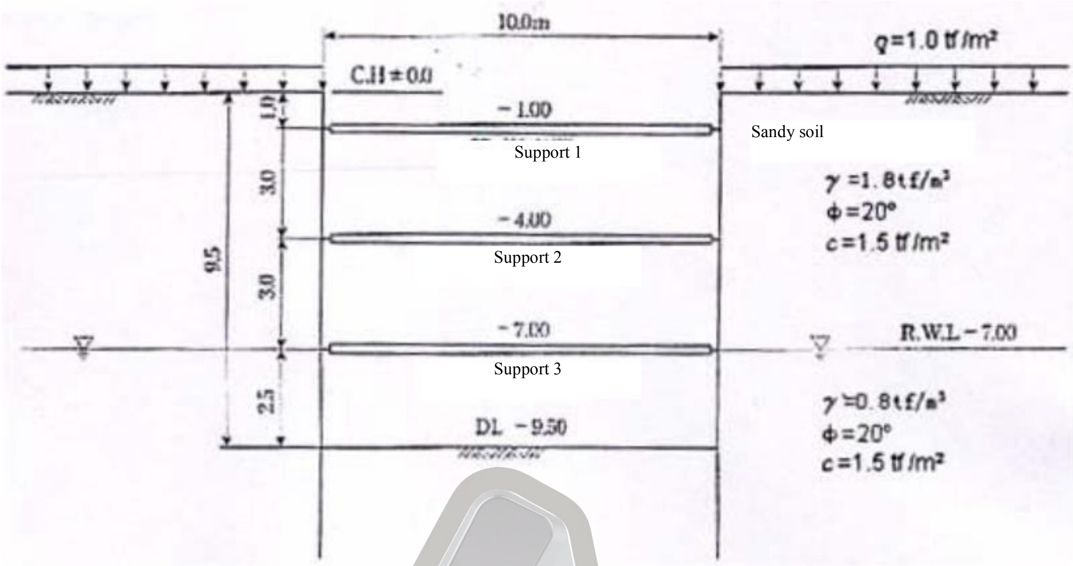


Fig. 3.6.5-1 Computational example of temporary structure (retaining water)

(1) Design conditions

As shown above:

① Elevation

Crest elevation: C.H.= ±0.00m

Excavation depth: D.L.=-9.50m

Residual water level: R.W.L.=-7.00m

② Topside load 10kN/m2

③ Soil conditions and supporting position

As shown above

④ Admissible displacement δa=30cm

⑤ Corrosion margin thickness and locking efficiency

Corrosion margin thickness is left out of account. The locking efficiency value for calculation of stress intensity is taken from the range 0.6~0.8 depending on restriction level. Locking efficiency takes 0.8 here.

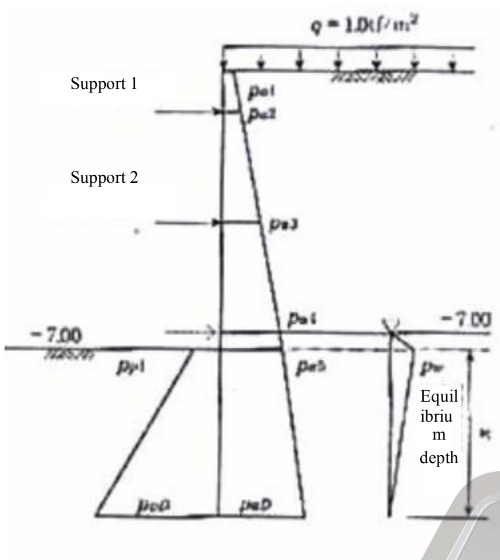
⑥ Lateral foundation reaction coefficient

When a displacement of about 30cm is assumed, the lateral foundation reaction coefficient k=20kN/cm3.

（2）Calculate equilibrium depth and assumed supporting point

① Earth pressure intensity and residual water pressure intensity

Fig. 3.6.5-2 is load distribution diagram before the configuration of table of the third support; Fig. 3.6.5-3 is the load distribution diagram at final excavation; Table 3.6.5-1 shows the load intensities.



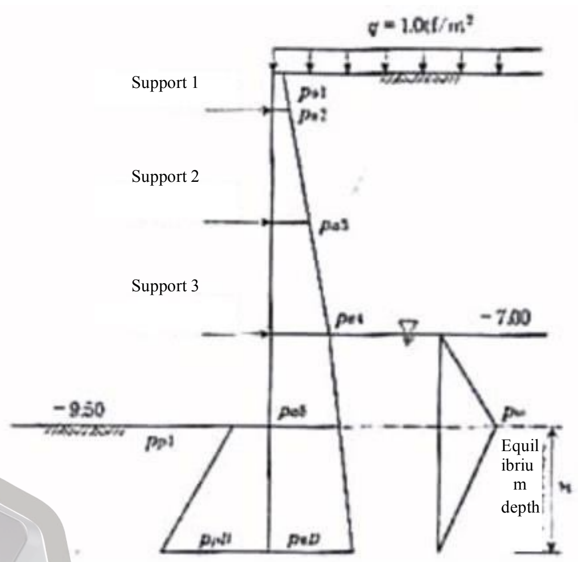


Fig. 3.6.5-2 Load distribution diagram of the third support configuration
Fig. 3.6.5-3 Load distribution diagram at final excavation

Table 3.6.5-1 Earth pressure intensity and residual water pressure intensity

i	αi=tan2(45º±φ/2)	βi=±2c×tan2(45º±φ/2)		Before the configuration of support 3		At final excavation
Load wi(kN/m2)		Earth pressure ·Water pressure intensity pi=αi·wi +βi		Load wi(kN/m2)		Earth pressure ·Water pressure intensity pi=αi·wi+βi
a 1	0.490	-21.01	10	-16.11	10	-16.11
a 2	0.490	-21.01	28	-7.29	28	-7.29
a 3	0.490	-21.01	82	19.17	82	19.17
a 4	0.490	-21.01	136	45.63	136	45.63
a 5	0.490	-21.01	140	47.59	156	55.43
a D	0.490	-21.01	140+8x	47.59+3.92x	156+8x	55.43+3.92x
w	-	-	5	5	25	25
pl pD	2.040 2.040	42.84 42.84	0 8x	42.84 42.84+ 16.32x	0 8x	42.84 42.84+ 16.32x

② Horizontal force and bending moment

Before the configuration of support 3:

The load as a result of earth pressure·residual water pressure below support 2 is subjected to partition calculation as shown in Fig. 3.6.5-4.

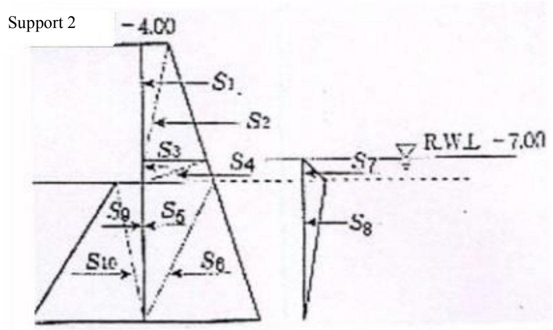


Fig. 3.6.5-4 Load diagram

Table 3.6.5-2 Horizontal force and bending moment

j	Horizontal force Sj(kN/m)	Distance lj(m)	Bending moment Mj=Sj·lj(kN·m/m)
1	28.76	1.0	28.76
2	68.45	2.0	136.90
3	11.41	3.167	36.14
4	11.90	3.333	39.66
5	$23.80x$	$3.5+x/3$	$83.30x+7.93x^2$
6	$23.80x+1.96x^2$	$3.5+2x/3$	$83.30x+22.73x^2+1.31x^3$
7	1.25	3.333	4.17
8	$2.5x$	$3.5+x/3$	$8.75x+0.83x^2$
9	$21.42x$	$3.5+x/3$	$74.97x+7.14x^2$
10	$21.42x+8.16x^2$	$3.5+2x/3$	$74.97x+42.84x^2+5.44x^3$

At final excavation:

Load as a result of earth pressure · residual water pressure below support 3 is as shown in Fig. 3.6.5-5; horizontal force and bending moment is shown in Table 3.6.5-3.

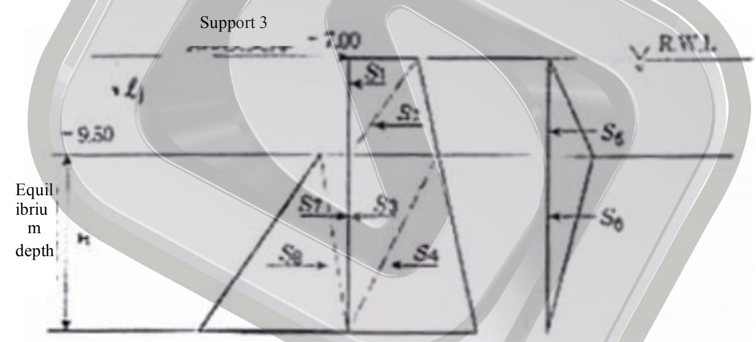


Fig. 3.6.5-5 Load diagram

Table 3.6.5-3 Horizontal force and bending moment

j	Horizontal force Sj(kN/m)	Distance lj(m)	Bending moment Mj=Sj·lj(kN·m/m)
1	57.04	0.833	47.51
2	69.29	1.667	115.51
3	$27.72x$	$2.5+x/3$	$69.30x+9.24x^2$
4	$27.72x+1.96x^2$	$2.5+2x/3$	$69.30x+23.38x^2+1.31x^3$
5	31.25	1.667	52.09
6	$12.5x$	$2.5+x/3$	$31.25x+4.17x^2$
7	$21.42x$	$2.5+x/3$	$53.55x+7.14x^2$
8	$24.12x+8.16x^2$	$2.5+2x/3$	$53.55x+34.68x^2+5.44x^3$

③ Horizontal force and bending moment

Before the configuration of support 3:

$0.413x^3+1.849x^2-2.541x-24.563=0 \quad \therefore x=3.22\text{m}$

At final excavation:

$0.413x^3+0.503x^2-6.275x-21.511=0 \quad \therefore x=4.59\text{m}$

④ Assumed supporting point

Take the application point of resultant force of passive earth pressure before equilibrium depth as assumed supporting point.

Before the configuration of support 3:

$$h_1 = \frac{M_9 + M_{10}}{S_9 + S_{10}} - 3.5 = \frac{118.264}{22.255} - 3.5 = 1.81m$$

At final excavation:

$$h_1 = \frac{M_7 + M_8}{S_7 + S_8} - 2.5 = \frac{189.872}{36.855} - 2.5 = 2.65m$$

(3) Buried depth of cold-bent steel sheet pile

The buried depth obtained according to bending moment equilibrium is 1.2 times the equilibrium depth.

Before the configuration of support 3:

D = 1.2x = 1.2×3.22 = 3.9m

Length of cold-bent steel sheet pile l = 7.5 + 3.9 = 11.4m→l=l1.5m

At final excavation:

D = 1.2x = 1.2× 4.59 = 5.5m

Length of cold-bent steel sheet pile l= 9.5 + 5.5 = 15.0m

Thereof, the length of cold-bent steel sheet pile obtained according to moment equilibrium is 15.0m.

(4) Calculate the stress at the section of cold-bent steel sheet pile

① Support reaction

Support reaction is calculated using earth pressure and water pressure used for section calculation as shown in Fig. 3.6.5-6; as shown in Fig. 3.6.5-7, support reaction is obtained based on simply-supported beam supported by supporting position and assumed supporting point.

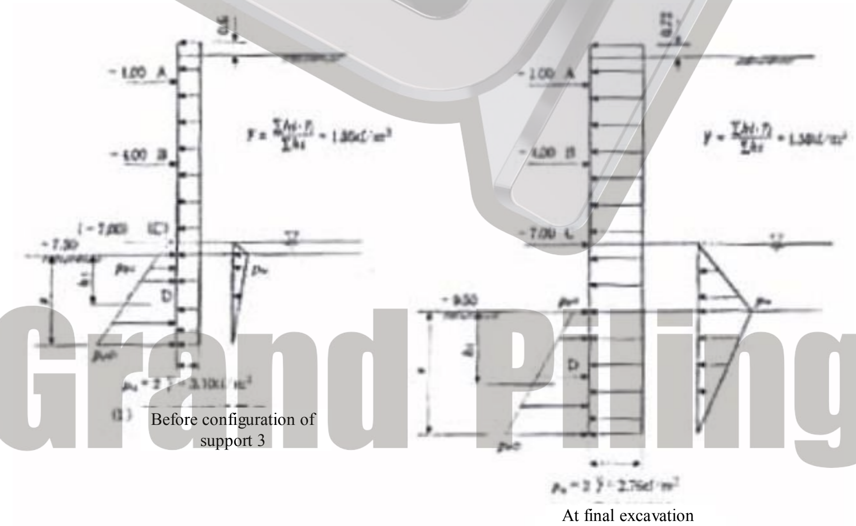


Fig. 3.6.5-6 Load distribution for calculation about section

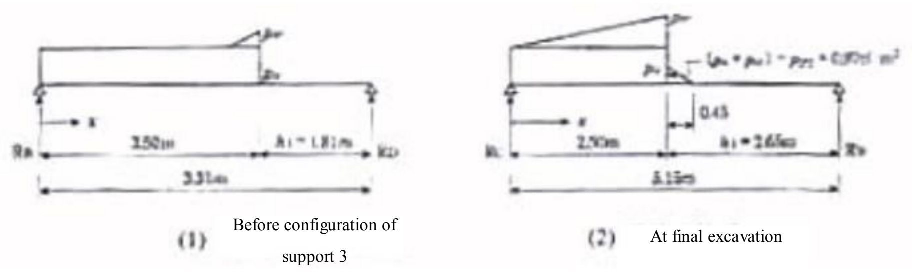


Fig. 3.6.5-7 Load between support and assumed supporting point

Before configuration of support 3：

$$R_B = 73.21kN/m R_D = 36.54kN/m$$

At final excavation:

$$R_C = 75.04kN/m R_D = 28.75kN/m$$

② Maximum bending moment

Before the configuration of support 3:

The position where shearing force Qx=0 is where the maximum bending moment occurs.

x=RB/pa=2.36 m

∴ Mmax=86.45kN·m/m

At final excavation:

Since Qx=0, x=2.00m

∴ Mmax=81.55kN·m/m

③ Determine the section of clod-bent steel sheet pile

The cold-bent steel sheet pile used is model GPU13a. Perform checking calculation of stress intensity that occurs.

$$\sigma = \frac{M_{max}}{W} = \frac{86.45\times10^5}{1360} = 63.6N/mm^2 \le 215N/mm^2$$

(5) Checking calculation of rigidity of cold-bent steel sheet pile

Fig. 3.6.5-8 shows the load at the time when displacement is calculated.

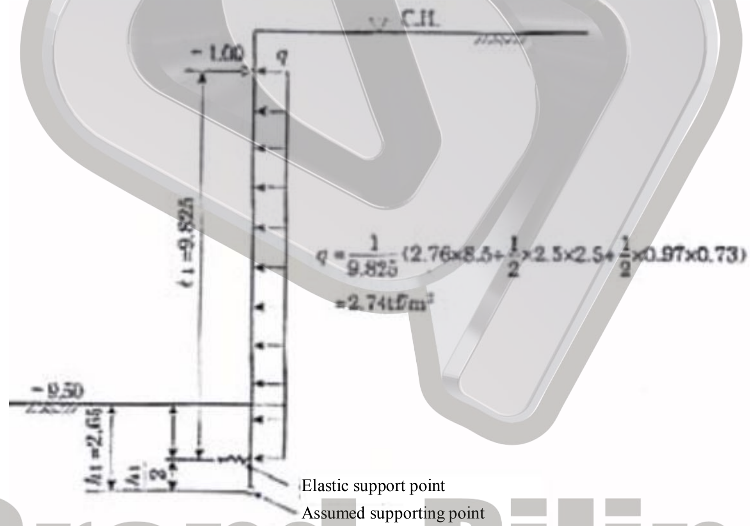


Fig. 3.6.5-8 Load at the time when displacement is calculated

①┌ Displacement as simply supported beam

The displacement of simply supported beam with the upmost support and 1/2 of assumed supporting point as supporting point:

$$\delta_1 = \frac{5ql^4}{384EI} = \frac{5\times27.4\times982.5^4}{384\times2.1\times10^6\times17400} = 9.10cm$$

② Effect of displacement of elastic supporting point

$$\delta_2 = 0.13cm$$

Thereof, the maximum displacement $\delta = \delta_1 + \delta_2 = 9.23cm < \delta_a = 30cm$

4 Construction of Cold-bent Steel Sheet Pile

4.1 Overview

Methods for cold-bent steel sheet pile construction are classified as follows.

(1) Vibration method using vibrating pile hammer;

(2) Pile jacking method using pile jacking machine;

(3) Pile jacking method using spiral drill;

(4) Piling method using pile hammer

For hard foundation, water jetting method could be used with.

Summary of construction methods is as follows:

(1) Vibration method using vibrating pile hammer

This is a construction technique that drives steel sheet pile into earth by transferring the vertical vibration generated by vibrating pile hammer. In default of hitting power, the head of steel sheet pile would not be damaged, which contributes to a relatively high construction efficiency; this technique is applicable for both driving and extraction of pile. However, the instantaneous current is extremely high in case of electric vibration, so large-scale electrical equipment is required; in case of hydraulic vibration, dedicated hydraulic unit is a must under most circumstances.

Recently, machines that are suitable for low-noise and low-vibration construction method and for construction under beam or in small space have been put in operational service.

(2) Pile jacking method using pile jacking machine

This is a construction technique that presses steel sheet pile into earth using the counter-force of drive-in steel sheet pile by clamping the intermediate section of steel sheet pile.

The main body of machine is very compact and applicable for construction under beam, but steel sheet piles need to be lifted with additional cranes.

This technique is suitable for soft foundation and can realize low-noise and low-vibration construction.

(3) Pile jacking method using spiral drill

With respect to this construction method, steel sheet pile is pressed in using hydraulic press during or after drilling; in this construction technique, both the drilling mechanism of spiral drill and the penetration mechanism of steel sheet pile is provided. Thanks to the use of sleeve, a high rigidity is realized during penetration, so this technique is suitable for hard foundation. Furthermore, this construction method can realize low-noise and low-vibration construction.

(4) Piling method using pile hammer

This construction method features large hitting power, mobility, high hitting frequency and operational abundance; however, feasible pile hammer must be selected so as to prevent compression failure of pile head during hitting.

Since this construction technique makes high noise and vibration, its use in surrounding areas of residential district, school and hospital etc. and designated areas is restricted; hence, it is rarely used in harbor district.

Pile hammer is classified into diesel engine driven pile hammer, hydraulic pile hammer, monkey and pneumatic pile hammer etc.

4.2 Construction Plan

4.2.1 Survey and Data Collection

(1) Investigation of Geography and Geology of Construction Area

① Land form and feature

Learn about the land feature and form (contour line or bathymetric chart) inside and outside pile foundation construction area; for water engineering, check the current situation of water front and bank slope, check if there is back fill on slope and if there is ballast or load reduction behind bank slope, and check the soil condition and thickness of back fill.

② Depth of water

Water depth is an important boundary condition for selection of ship equipment and is the basis for determination of construction scheme.

③ Vegetation cover

All vegetation covers (i.e. trees, bamboos, farm crops, frutex and weeds) within the range of land construction shall be removed, but it should be found out whether or not there are any varieties that need to be protected and must be transplanted as required.

④ Surface water and ground water

Surface water within the range of land construction shall be completely removed; hence, it is necessary to find out the source of surface water, the cause of generation and the contamination status etc.

⑤ Geological data

Since data of engineering geology and hydrological geology is among the most important data for engineering design and construction of pile foundation, there should be geologic report prepared by professional exploration agency, including drilling data, physical mechanics test data of earth, description or testing data of unfavorable geology, confined water permeability coefficient and other hydrological geology data.

(2) Investigate meteorological phenomena and sea conditions

As overhead operation accompanied by transport of heavy objects, the construction of steel sheet pile is dependent on meteorological conditions under most circumstances; it should be particularly noted that steel sheet pile construction may become very difficult due to the effect of waves during on-the-sea piling, and tidal stream and rivers may cause obstruction. The table below shows the natural conditions that have effect on operation.

Table 4.2.1-1 Meteorological and sea conditions for various operations

Operation classification	Operation designation	Operation content	Natural conditions that may influence the number of days in which operation is unavailable				
Meteorological condition			Sea condition				
Wind velocity (m/s)	Precipitation (mm/h)	Atmospheric temperature (℃)	Wave (m)	Tide (m/s)			
On the sea	Underwater operation	Underwater operation for removal of installation	More than 5			More than 0.3	More than 1.7
Piling operation (with pile driving barge)	Driving of steel sheet pile	More than 5	More than 10		More than 0.3	Piling operation (with pile driving barge)	
Concrete operation (with mixing ship and transport ship)	Template Mix concrete Transport concrete Pour concrete		More than 10 More than 3 More than 3 More than 3	Less than 4 Less than 4	More than 0.2 More than 0.3 More than 0.3 More than 0.3	Concrete operation (with mixing ship and transport ship)	
Grab type dredging	Wooden D50~80PS Steel D120~350PS	More than 5 More than 10			More than 0.3 More than 0.4	Grab type dredging	

Ground operation	Earth work	Labor for earth excavation Machinery for earth excavation		More than 10 More than 10		Ground operation	

Piling operation	Driving of steel sheet pile	More than 10	More than 10		Piling operation	
Concrete operation	Template Preparation and arrangement of reinforcement Mix concrete Transport concrete Pour concrete	More than 10	More than 10 More than 10 More than 3 More than 3 More than 3	Less than 4 Less than 4	Concrete operation	
Cut-off and welding operations	Field operation		More than 0		Cut-off and welding operations	
Corrosion resisting coat spraying operation	Field operation		More than 0 Atmospheric humidity is higher than 85%		Corrosion resisting coat spraying operation	

(3) Investigate operating environment

Collect data from surrounding area of project and analyze the effect of these external conditions on construction management. Accurately establish targeted construction organization design, technical proposal and countermeasures for ensuring engineering safety, quality and progress when preparing construction organization design. Develop targeted pre-plan for construction team management and the communication with community in every respect.

In construction preparation period, the Project Department shall collect environmental protection policies and regulations of local region, work out environmental protection measures and pre-plans and water & land transportation assurance related pre-plans and make report to relevant local government departments for recording.

4.2.2 Project Plan

Determine guiding material structure, steel sheet pile erection and driving method, select piling machine, determine the transportation method of materials and equipment, the manufacturing method of rolled section steel sheet pile and the construction method of anchorage, pulling anchor, surrounding purlin, back lining, land reclamation, excavation and top-side structure etc. based on above-noted investigation by taking into account the difference between land piling and sea piling.

Once construction method is determined, it would be possible to determine construction sequence and calculate the number of days required for each trade so as to arrange for project schedule using PERT (program evaluation and review technology), CPM (critical path method) and other techniques.

4.2.3 Material Plan

Since the time interval between order and delivery of steel sheet pile is about 1 month, be sure to ask steel sheet pile manufacturer about the time of delivery to whatever extent possible so as to make adequate schedule plan.

Pulling anchor and relevant accessories are usually not produced by steel sheet pile manufacturer, they must be ordered and purchased from pulling anchor manufacturer by other means. Delivery period is normally about 2 months. Considering the engineering requirements, pulling anchor and relevant accessories are ordered together with steel sheet pile.

4.2.4 Personnel Plan

In order to ensure the successful construction of steel sheet pile project, it is also very important to establish project team, designate responsible departments, define the command system and remove engineering fault. Individual skill differences may bring about variation of construction accuracy, which may easily affect the overall progress. So be sure to pay adequate attention to candidates of the Team.

4.2.5 Disaster Security Management

Construction safety management is a link of vital importance to the project. Safety mentioned here includes four aspects, i.e. personal safety, engineering safety, equipment safety and environmental safety. Since steel sheet pile project is based on mechanized construction and is involved with overhead operation of long heavy objects, it has a higher possibility than other construction works to bring about the following accidents:

(1) Collapse and damage of pile driver;

(2) Personal safety accident caused by swinging of steel sheet pile root during erection;

(3) Accident that occurs during interlock engagement overhead

Therefore, it is important to prevent labor injury to field operation personnel and pay adequate attention to the prevention of injuries to third parties in surrounding areas of construction site during construction of steel sheet pile. It should be particularly noted in temporary works that construction personnel should carry safety measures into execution from time to time when designated temporary facilities are not available; to ensure the completeness of method or equipment, adequate survey must be carried out and the agreement including project main body needs to be discussed depending on situation.

Appoint responsible persons for safety and hygiene before engineering construction of steel sheet pile and have them carry out the following works:

(1) Establish operation rules and ensure the correct implementation thereof in order to prevent labor disaster;

(2) Coordinate and adjust the operation between general contractor and subcontractors;

(3) Patrol operation field;

(4) Unitize the following operations and make thorough clarification to relevant surrounding personnel:

① Driving signal: signals relating to driving of cranes etc.;

② Marks: dangerous substance marks and "no entry" signs;

③ Alarm: alarm and rescue measures against fire, soil and sand collapse, water leakage and snow slide;

④ Measures for prevention of disasters relating to road, scaffold and mechanical equipment etc.: take measures for disaster prevention during the use of road, scaffold, pile driver, pile puller, crane, track device, alternating current arc welder and other equipment;

(5) Establishment of emergency response system and liaison network for accidents or disasters that occur during construction

4.3 Selection of Pile Sinking Technology and Equipment

4.3.1 Principle for Selection of Technology and Equipment

Pile foundation constructor shall, according to design documentation, hydrology, geology, meteorological phenomena and surrounding construction environment, analyze and optimize construction technology, perform comparative selection of constructional appliance in the aspect of economy and technology etc., and finally find an optimal technology and the best equipment mix plan that must following the following principles:

(1) State-of-art technology, rationality and feasibility;

(2) Reliable and easy-to-control quality;

(3) Low cost and high efficiency;

(4) Safe, reliable and handy;

(5) Compliance with relevant environmental protection regulations;

(6) Satisfaction of all requirements stated in contract

Optimization and selection of construction technology would directly relate with project construction period, quality, safety, economic benefit and other issues. Although "all roads lead to Rome", different construction technologies may make remarkable difference in the aspects of cost, technology and construction period etc. of the same pile foundation project.

Optimization and comparison of construction scheme has a lot to do with technical level, equipment level and technology innovation ability. Therefore, the study on and comparison of construction technology should be linked up with technology development and innovation so as to improve and promote the development of construction technology and technological level.

The selection of construction equipment changes depending on the change of technology: they correlate with each other and are closely linked. When equipment capacity and resources cannot meet technological requirements of a seemingly ideal technological program, it must be replaced by another one.

4.3.2 Selection of Construction Equipment

(1) Principle for Selection of Construction Equipment

Advanced and rational construction technology must be used with applicable construction equipment so as to obtain favorable benefit. Under normal circumstances, the selection of construction equipment shall follow the principles as follows:

① Meet design requirements;

② Comply with quality standards;

③ Adapt to conditions on construction site (including over-water construction areas);

④ Fulfill contractual commitment;

⑤ Adapt to geological conditions;

⑥ Match with other input equipment;

⑦ It is easy to purchase wear parts and fuel in the neighborhood;

⑧ Suitable machine shift cost, convenient operation, and outstanding performance;

⑨ Rating, function, power consumption and other built-in properties of equipment match project requirements or hardly ever go beyond engineering demand;

⑩ The equipment is in good condition and can ensure the normal construction of project.

(2) Basis for selection of construction equipment

The selection of construction equipment shall be based on geological conditions, field construction conditions (including over-water construction areas), pile material, pile length, possibility of defective materials, surrounding environment, transportation and other objective conditions and take comprehensive consideration of enterprise and market conditions.

4.3.3 Over-land pile sinking construction

Overland pile sinking is subject to limits of pile driving frame height and hoisting capacity and normally employs vertical pile extension technology.

Overland pile sinking may adopt hammering method, combined flushing and ramming method, vibration method and static pressure method.

4.3.4 Over-water Pile Sinking Construction

Over-water pile sinking is the major pile foundation construction method in open water area with wind and waves and is also a special operation using ships. Due to the influence of natural conditions, it is difficult to perform on-site pile extension as overland pile sinking since the operation is performed in floating pile driving barge. Hence, all piles driven over-water through pile driving barge are long piles of which the fabrication and sinking is suitable for over-water operation.

Since pile driving barge is the key part of over-water pile sinking through hammering, a multi-functional vessel group is required to assist in construction.

4.4 Driving and Pulling

4.4.1 Driving

(1) Inspection and rectification of steel sheet pile

Steel sheet pile is re-usable, so special attention shall be paid to the acceptance of steel sheet pile. Besides, special attention shall also be paid to distinguishing between different models of different types of steel sheet pile in the aspects of section dimensions and steel sheet thickness.

Due to its small rigidity of body, steel sheet pile is easily subjected to deformation during transportation and stacking; hence, steel sheet piles normally need to be modified and straightened, especially the straightness of pile body and interlock. Interlock smoothness is the primary precondition for successful driving of steel sheet pile, and interlock blockage is the "arch criminal" of common faults such as driving difficulty, adjacent pile linkage and pile body inclination etc. that may occur during driving of steel sheet pile. Therefore, people believe the success of driving relies 70% on straightening, and 30% on driving. For field inspection, a piece of more-than-two-meter long "interlock tester" with interlock that is cut off from pile body of the same model is usually used. This "interlock tester" can be comfortably lifted up by two persons and inserted into the interlock to be tested for lock unblocking testing. Steel sheet piles must be inspected one by one using "interlock tester". The "tester" must be able to smoothly pass through the interlocks at both sides of the whole length of pile, or the interlock and pile body should be straightened and rectified. Straightening and rectification is usually realized through oxyacetylene flame baking, hammer knocking plus cold water chilling until the "interlock tester" can smoothly pass through the treated interlock.

(2) Selection of pile driving machinery

Construction of steel sheet pile in water works normally employs hammering method and vibration method. The former is realized through diesel hammer, and the latter through vibratory hammer. The selection is made according to the type and length of steel sheet pile and geological conditions etc. Also, the two methods could be combined. Table 4.4.1 shows the situation to which the above-noted two types of hammers apply.

Table 4.4.1 List of hammer types

Hammer type		Geological conditions		Construction conditions			Merits and faults	
Silt, clay	Sand layer	Hard soil layer	Noise		Silt, clay	Sand layer		Hard soil layer
Diesel hammer	Suitable	Suitable	Available	Diesel hammer	Suitable	Suitable	Available	Diesel hammer
Vibrating hammer	Suitable	Available	Unavailable	Vibrating hammer	Suitable	Available	Unavailable	Vibrating hammer

(3) Technological flow diagram of steel sheet pile construction (see Fig. 4.4.1)

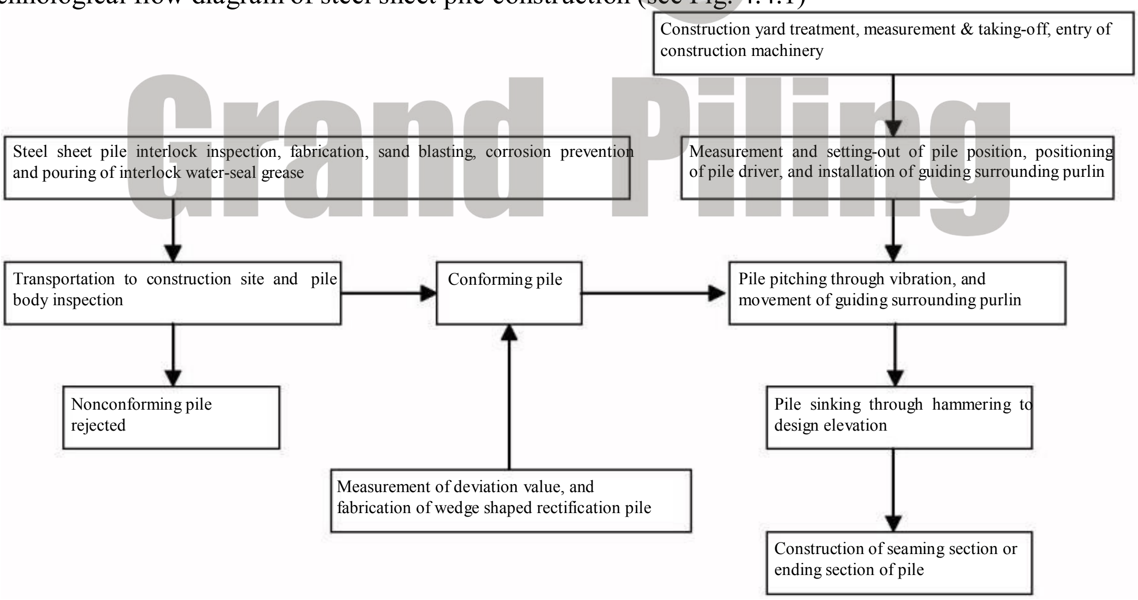


Fig. 4.4.1 Technological flow diagram of steel sheet pile construction

(4) Steel sheet pile sinking considerations

① Make first-phase preparations adequately and earnestly

a. Due to the small rigidity of its single piece, steel sheet pile is easy to deform during transportation, fabrication and stocking piling. It should be particularly noted that the deformation of pile extension, interlock and processing of re-used steel sheet pile shall be properly rectified and could be tested using interlock tester.

b. Make pre-phase preparation based on functions of steel sheet pile. It should be particularly noted that water sealing material for anti-seepage steel sheet pile shall be selected according to anti-seepage water head.

c. Fabrication, rust protection and corrosion prevention etc. shall be carried out in special plants (or shops) so as to meet the requirements on environment and fabrication accuracy. In particular, turning point pile and shaped piles shall be fabricated according to the requirements.

d. For measurement work, the axis, turning point, the first and last piles, and the perpendicularity between pile axis direction and normal axis shall be particularly controlled.

e. Obstacles especially underground obstacles shall be removed from construction axis of steel sheet pile.

② Special attention shall be paid to ground contact pressure of site in case of overland pile sinking; it should be particularly noted that ditches, creeks and wet areas (if any) shall be treated properly so as to protect piling machinery and transport vehicles against turnover.

③ The following points shall be paid attention to in case of over-water pile sinking:

a. Construction in shallow water: working platform using land driver could be established on water according to land form, water depth and operating conditions. Alternatively, the construction could be carried out at the surge of water using pile driving barge after dredging.

b. Over-water guiding surrounding purlin is generally provided with two-layer guide frame of which the upper layer shall be above construction water level.

c. Joining of steel sheet pile afloat shall be performed at flushing time to the best of our abilities, while the internal and external head difference of steel sheet pile shall be minimized; if necessary, holes shall be drilled so as to realize the water head equilibrium inside and outside.

d. Pay attention to water flow velocity, wave, wind strength and other natural conditions during over-water construction, and the construction shall be suspended when the said conditions fail to meet the requirements.

e. In case of over-water pile sinking, the sunk sheet pile shall be reinforced with surrounding purlin in a timely manner; in the mean while, the construction of top-side structure shall also be performed in a hurry-up manner so as to prevent damages caused by wind, waves, vessel attack and collision.

4.4.2 Pulling

When it is necessary to pull out steel sheet pile and re-drive the piles due to poor construction or when temporary facilities need to be pulled out after the completion of use, appropriate pulling method shall be taken by taking into full account the status of piling, the elapsed time after piling, the engagement status of interlock, and the soil conditions of piling area etc.

However, when large-sized pulling machinery is used, if damaged steel sheet pile cannot be pulled out or the pulling of steel sheet pile may cause looseness and settlement of surrounding foundation soil, it may be rational not to remove the pile or bury it completely.

(1) Extracting force

Extracting resistance is the sum of surrounding friction resistance of soil and the friction resistance of interlock and is extremely difficult to reckon. Under normal circumstances, as a matter for experience, extracting resistance is about 3 times the surrounding friction generated by static bearing capacity.

(2) Extracting method

The methods for pulling out steel sheet pile include the combination of pulley, wire rope, strut and hoist, the crane lifting, hydraulic puller and other static force based method and the dynamic method using vibrating hammer. Characteristics of various pulling methods are as shown in Table 4.4.2.

Table 4.4.2 Characteristics of various pulling methods

	Dynamic method		Static method
Vibrating hammer	Hoist, pulley		Hydraulic puller
Preparation: (1) Arrangement for working face (2) Required power supply capacity Work property: (1) Work difficulty level (2) Slope toe cutting capacity (3) Loss of crane lever (4) Construction speed Applicable soil texture	 Simple 60~200kW Simple Large Large Quick Sandy soil	 Relatively simple 40~60kW Simple Small None Slow Sandy soil, clay	 Simple 15~25kW Slow Medium None Slow Sandy soil, clay
Impact on environment:			
(1) Noise (2) Vibration	Small ~ medium Medium ~ large	Small Small	Small Small

4.4.3 Accessory Structure of Steel Sheet-pile Wall

(1) Construction sequence of pulling anchor type steel sheet-pile wall

Standard construction sequence of pulling anchor type steel sheet-pile wall is as shown in Fig. 4.4.3-1:

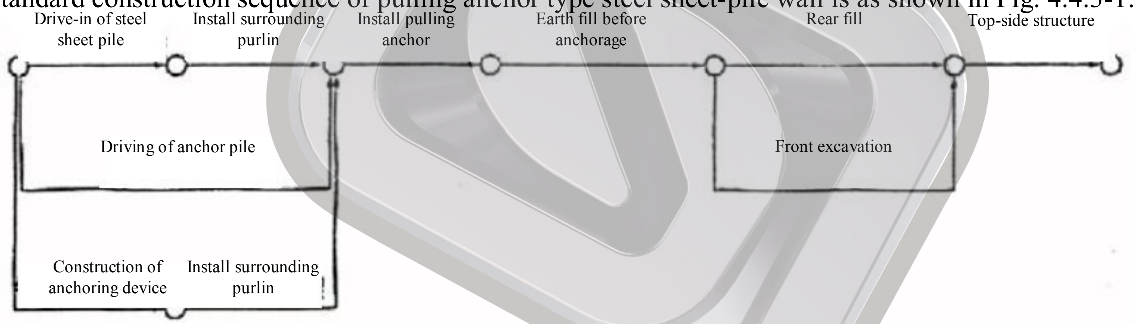


Fig. 4.4.3-1 Construction sequence of pulling anchor type steel sheet-pile wall

(2) Surrounding purlin structure

The purpose of surrounding purlin structure is to integrate the steel sheet-pile wall and rectify the roughness of steel sheet-pile wall, and it must be mounted immediately after the driving of steel sheet pile. Surrounding purlin is normally fabricated of channel steel, H beam steel and other steel sections, and the length of each piece is more than 4 times the spacing between pulling anchors. In case the concave-convex surrounding purlins of steel sheet-pile wall cannot be tightly bonded (but with an interval of more than 10mm), gasket or suitable metal parts shall be mounted and fastened adequately so as to prevent screw looseness. Estimate the tightness of bolt by knocking the head with a hammer.

Surrounding purlin could be mounted in front of or behind steel sheet pile. When surrounding purlin is directly arranged in front of steel sheet pile, the load of steel sheet pile acts directly on surrounding purlin; but when it is arranged behind steel sheet pile, since it is necessary to connect surrounding purlin to steel sheet pile using fastening bolts so as to bear the load of steel sheet pile, the construction of fastening bolt should be carried out with care.

(3) Anchoring device

Pulling anchor type steel sheet-pile wall shall be provided with anchoring device so as to form pulling anchorage. Anchoring device is usually arranged in the following way:

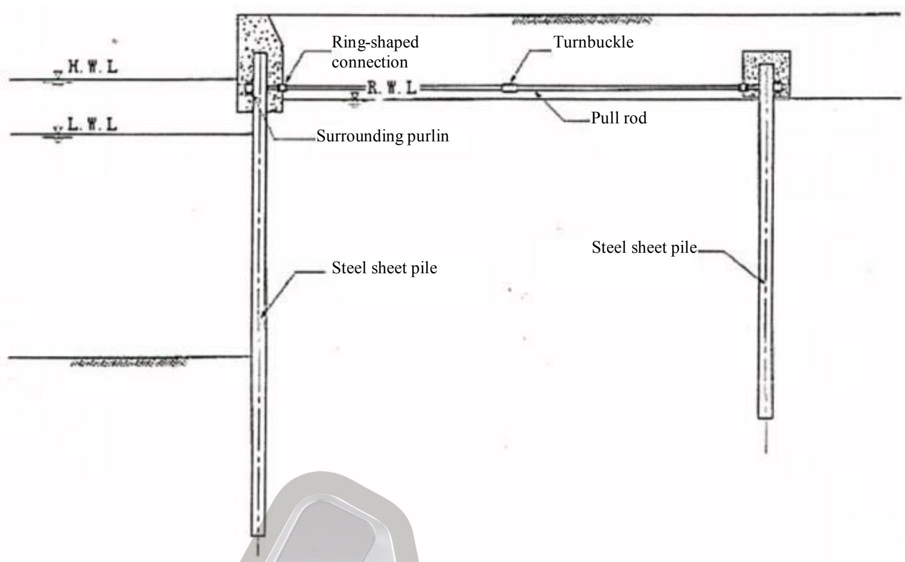


Fig. 4.4.3-2 Steel sheet pile anchoring device

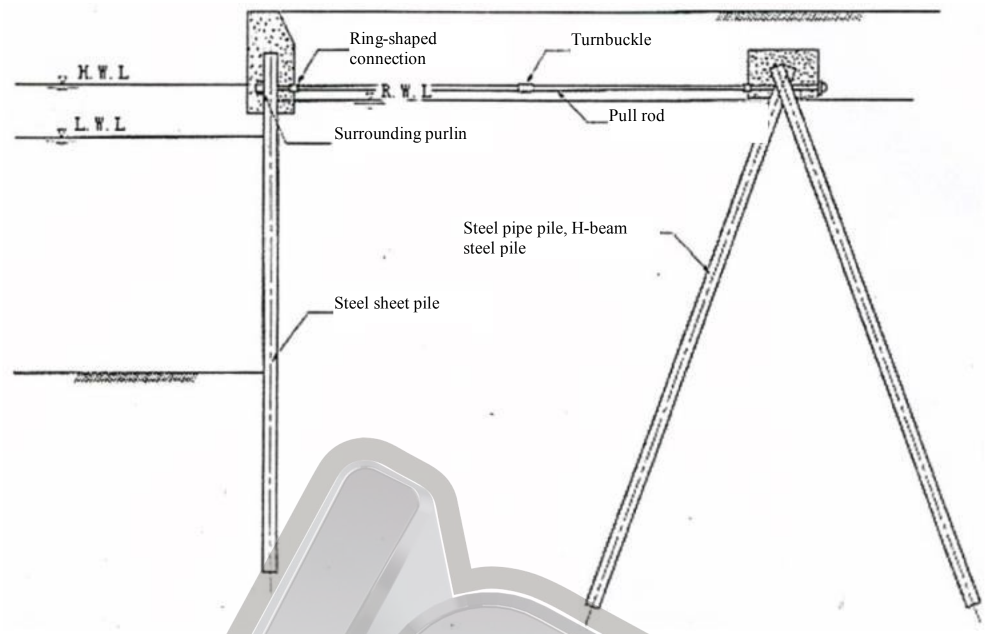


Fig. 4.4.3-3 Built pile anchoring device

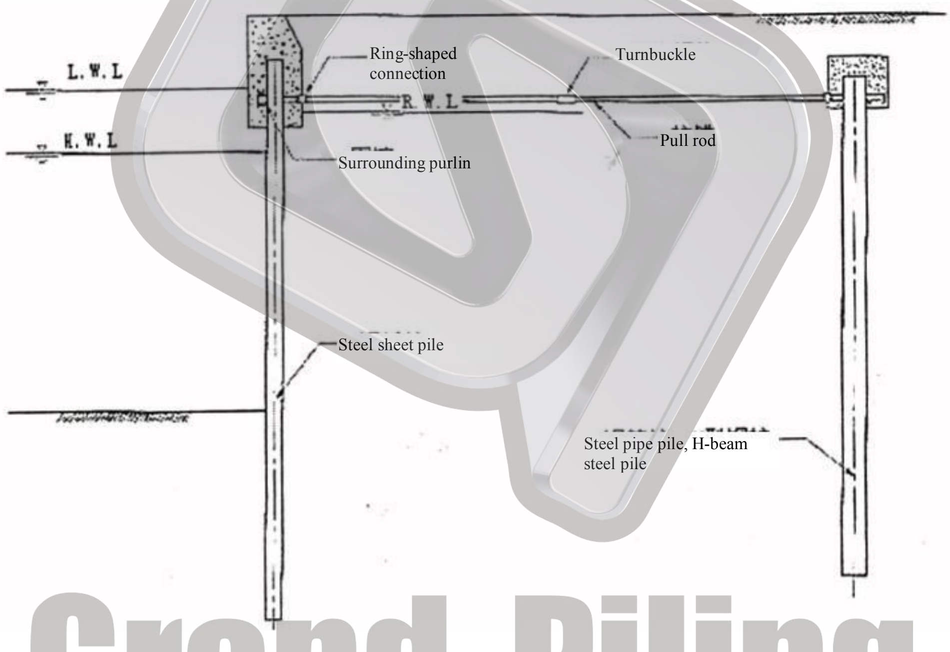


Fig. 4.4.3-4 Straight pile anchoring device

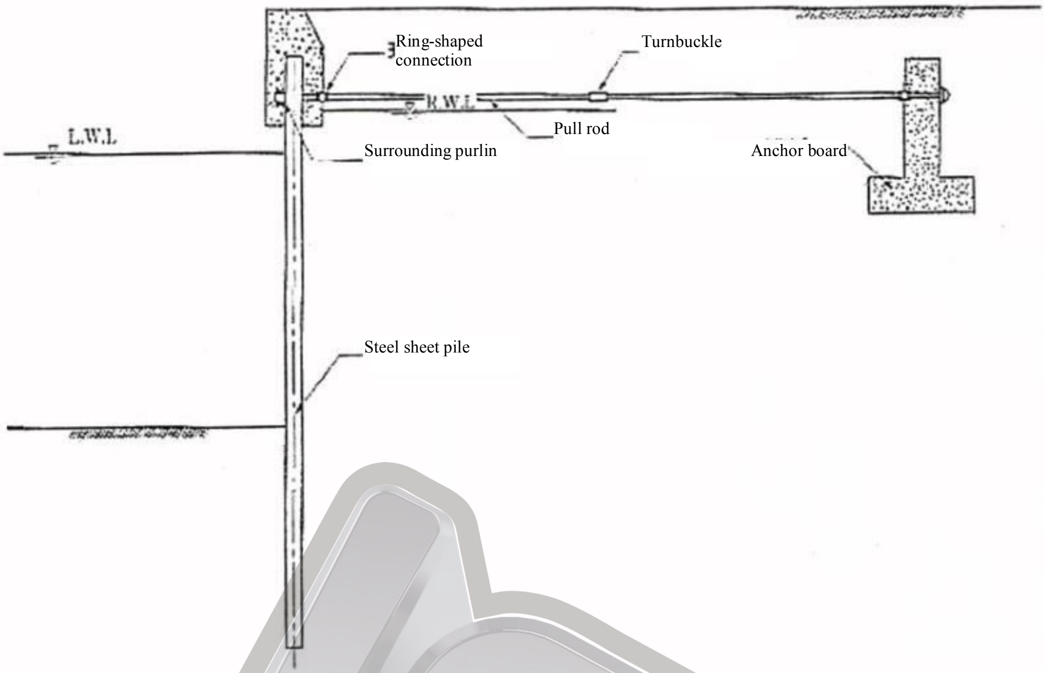


Fig. 4.4.3-5 Anchor board

The construction of anchoring device may vary under different construction conditions, and the stability of previous steel sheet-pile wall shall be taken into account when determining the driving period, sequence and method of steel sheet pile.

It should be particularly noted that pulling anchor installation and filling must be performed as soon as possible after the driving of previous steel sheet-pile wall in order to protect previous anchoring devices against the impact of wind and waves during over-water construction of steel sheet pile. However, when steel sheet pile must be constructed first and the impact of wave, tide and other factors are predicted before installation of pulling anchor, the following measures must be taken:

① Backfill half of the current foundation in advance

② Backfill the front area of steel sheet pile after the driving thereof and reinforce it with riprap.

(4) Anchorage fixing

Pulling anchor shall be mounted after the completion of erection of steel sheet pile, the configuration of anchoring device and the installation of surrounding purlin.

The standard installation of pulling anchor, ring-shaped connection and the method for use of turnbuckle are as shown in Fig. 4.4.3-6.

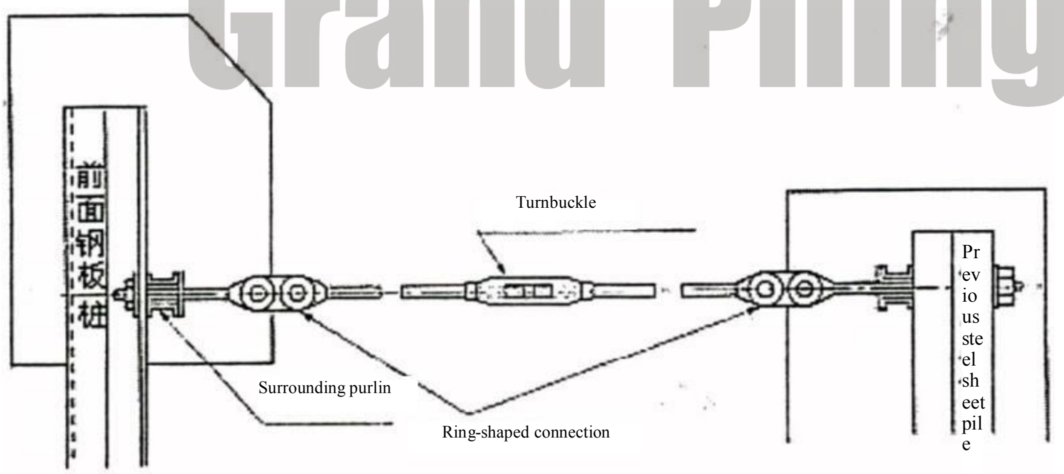


Fig. 4.4.3-6 Pulling anchor

① Install pulling anchor

Pulling anchor must be perpendicular to the normal of steel sheet pile and installed at specified angle or horizontally.

When pulling anchor is mounted on land, it is necessary to excavate to the height required for installation or perform earth-filling and level the ground off and then reinforce the foundation before the configuration. If the foundation is too soft to prevent aforehand settlement, piles at interval of several meters shall be erected and door-shaped frame shall be mounted before the configuration of pulling anchor.

② Install ring-shaped connection

Ring-shaped connection is normally mounted at steel sheet-pile wall side and the anchoring device side so as to prevent the earth pressure on pulling anchor or the filled up ground settlement bringing about bending stress. Ring-shaped connection must be correctly assembled and configured so as to realize smooth rotation at both upper and lower part.

Furthermore, the ring-shaped connection at the side of steel sheet-pile wall or anchoring device must be in close contact with wall in order to prevent top-side damage.

③ Fasten the pulling anchor

Upon the completion of pulling anchor installation, its whole length must be adjusted with nuts and turnbuckle; at the same time, appropriate pulling force shall be applied for fastening pulling anchor.

In principle, final fastening is performed through retaining nut at the side of turnbuckle or anchoring end.

When rectifying the deviation of normal of steel sheet-pile wall by tightening pulling anchor screws, the pull asymmetry of pulling anchor must fully act so as not to cause the reduction in stability of steel sheet-pile wall.

④ Pull cord

Pull cord is steel wire made of PC steel covered with polyethylene or hard steel wire. Since pull cord is furnished with metalworks at both ends, it could be fastened with nuts in the same way with pulling anchor.

The greatest strength of pull cord is its light weight and perfect toughness; furthermore, it requires no ring-shaped connection and turnbuckle.

(5) Back lining construction

When inputting back lining material, be sure to take into account the installation period of pulling anchor, the independence of steel sheet pile, meteorological phenomena and sea conditions.

When inputting back lining material before the installation of pulling anchor, be sure to take into account the impact of bending stress and top-side displacement of steel sheet pile on checking calculation of project.

When inputting back lining material after the installation of pulling anchor, be sure not to damage the pulling anchor.

Rubble ashlar, pit run gravel, furnace slag and other materials with large internal friction should be used as back lining so as to alleviate the earth pressure against steel sheet-pile wall

(6) Backfill construction

In principle, back-filling behind steel sheet-pile wall should be carried out after the completion of back lining. Backfill material is prepared from high quality earth and sand which is expected to be uniformly distributed along the total length of steel sheet-pile wall. In case the implementation is impossible due to engineering restrictions, backfill must be carried out immediately after the erection of steel sheet pile at each short interval, the configuration of anchor wall, the installation of surrounding purlin and pulling anchor and the input of back lining material.

In case of the risk of sliding failure of soil and sand during back-filling, the specified backfill height must not be exceeded even if short-term built-in operation is carried out. Be sure to avoid rapid backfill construction on weak ground. In order to prevent weak soil squeezing steel sheet pile from the side, it is necessary to perform gradual compaction while performing the earth filling upwards layer by layer. When pumping vessel is used for backfill, water shall be drained from time to time in order not to result in the increase in residual water level; it should be noted that there should be no local earth pressure acting on steel sheet-pile wall.

In principle, soil and sand on land shall be spread in mounting direction of pulling anchor. When it is necessary to spread soil in the direction that is perpendicular to pulling anchor, be sure not to bring negative effect onto pulling anchor. When the compaction direction is parallel with pulling anchor, the construction thickness of passive earth thereon is expected to be more than 1m; when the compaction direction is perpendicular to pulling anchor, the thickness is expected to be more than 1.5m.

(7) Excavation construction

Excavation in front of steel sheet pile is usually commenced upon the completion of erection of steel sheet pile, anchoring construction, surrounding purlin construction, pulling anchor construction and inner lining construction. The following points shall be paid attention to:

① Dredging construction shall be performed uniformly along the total length; in case of large-scale earthwork, the construction should be conducted in several layers.

② Be sure to reduce the passive earth pressure at deep cut section at planned water depth.

③ Excavation work may sometimes damage steel sheet-pile wall. It is recommended dredger with dredging bucket be used in the front areas less than 5m away from steel sheet-pile wall.

(8) Top-side structure

Construction of top-side structure must not be started until it is confirmed that the steel sheet-pile wall would not be subjected to displacement after the completion of input of back lining material, back-filling and the excavation work in front of steel sheet-pile wall.

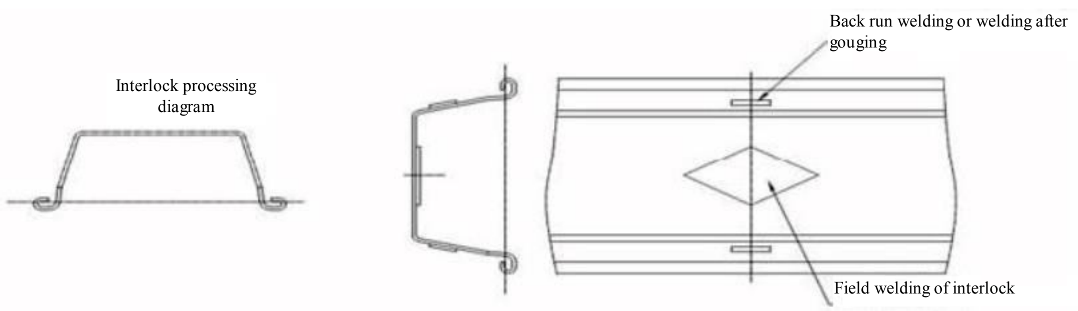
Facility joint shall left at appropriate interval for concrete of top-side structure, and its position shall not be the same with that of interlock of steel sheet pile.

If the normal of steel sheet-pile wall shows deviation during the pouring of concrete of top-side structure, the steel reinforcement protecting layer thickness or section shape could be changed so as to align the front normal. To mount bumper or bollard, it is essential that the buried hole of mounting bolt be set before pouring of concrete.

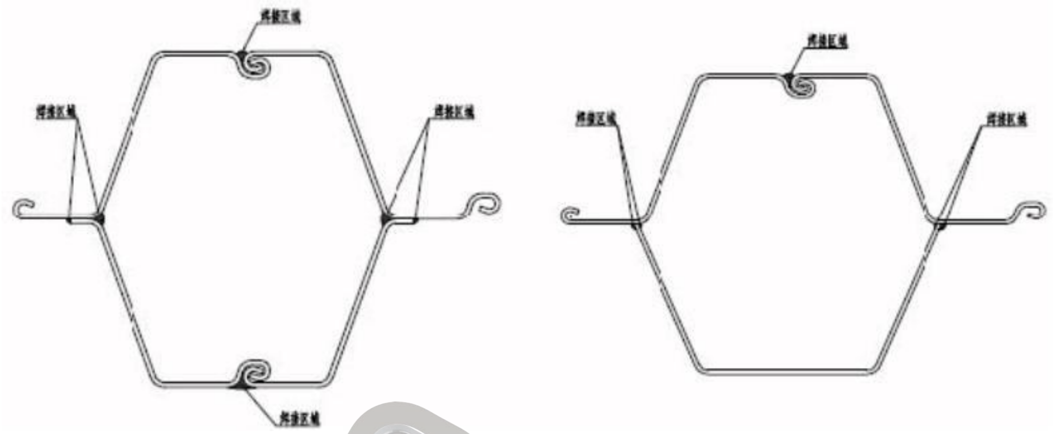
4.5 Welding

Welding has been widely used for steel sheet pile from the fabrication of various shaped steel sheet piles to longitudinal joint welding that ensures specified length. However, from the perspective of purpose, steel sheet pile relies on the use of economical high strength steel compared with welding. Furthermore, compared with steel with equal strength, this kind of steel contains more alloying element such as C, P and Cu etc. Hence, if the steel has a high C content, welding would result in the hardening and extension of heat affected zone; as a result, the steel sheet pile to be welded must be compared with steel used for other structures.

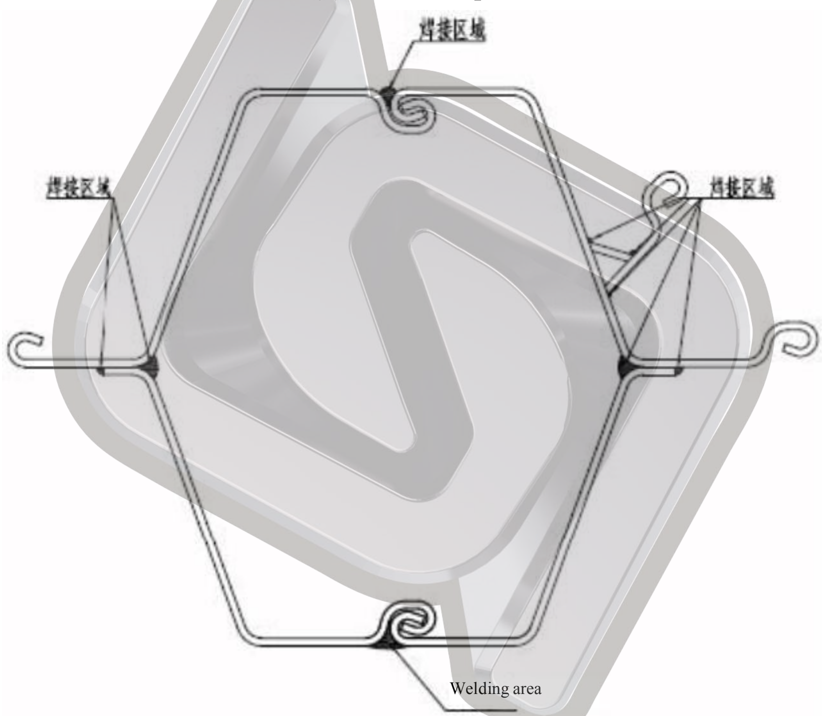
In principle, all materials for processing of steel sheet pile are weldable. In case of single-piece welding, it is to be noted that welding positions of steel sheet pile shall not be in the same plane during the use of steel sheet-pile wall; regardless of the length, single-piece steel sheet pile is allowed to weld only once, and the quantity shall not exceed 2% of total quantity; be sure to employ butt welding, welding chain, ultrasonic testing and make the reinforcement with steel sheet pile so as to ensure a strength that is not lower than base metal; when being welded, combined steel sheet pile shall be fixed with special fixture; in such a case, be sure to employ submerged arc welding, reinforcement of corner pile interlock through submerged arc welding, and ultrasonic testing.



(a) Single pile



(b) Combined pile



(c) Combined corner pile

Fig. 4.5 Welding diagram

4.5.1 Welding Method

GP series steel sheet piles are mostly subjected to manual electric arc welding, non-gas-shielded arc welding (semi automatic welding) and carbon-dioxide arc welding (semi automatic welding).

Welding operator shall prepare all equipment and tools necessary for welding before the welding; remove rust from area to be welded and polish it if necessary to a proper extent; the area to be welded shall be dry and free of moisture.

To ensure welding quality and avoid the negative effect of welding deformation, constructor shall fabricate welding jig and fixture according to practical situation of construction so as to keep welding deformation within the range of tolerance. Whether to use jig or not is dependent on practical situation. When jig is used, the gap between weldments shall be minimized under the precondition that the welding quality is ensured.

4.5.2 Electrode

The overall principle of deposition metal is that its chemical composition and mechanical properties are equivalent with that of base metal. It is recommended alkaline property of deposited metal be used, but not acid property. When selecting the type and diameter of deposition metal, it is necessary to take into account base metal wall thickness, material, welding structure, welding position and other factors.

In case of manual welding, if steel sheet pile is made of high tensile steel, low-H manual electric arc electrode is usually used. This kind of electrode minimizes the intrusion of Hydrogen that may exert adverse effect on welding metal; the flux is mainly composed of calcium carbonate and contains no organic matter. Hence, compared with iron based electrode, this electrode has less favorable operability, but weld metal has excellent mechanical properties, especially ductility and toughness. However, in order to minimize H content, electrode must be intensively dried (at 300~350℃ for about 1h) before the use so as to remove moisture absorbed by flux.

Non gas-shielded arc electrode contains flux which provides shielding gas for electric arc. Composed of gas generating agent, slag forming agent, alloying agent, deoxidizing agent and powdered carbon etc., this flux is decomposed under the effect of heat of electric arc, turning external gas into shielding gas while generating slag that isolates extraneous gas and protects molten metal. Besides, denitriding and deoxidizing materials may be added to flux to prevent nitriding and oxidation while preventing the degradation of mechanical property.

Electrode for carbon-dioxide arc welding is classified into solid wire electrode and cored electrode. In order to make sound welded area, solid wire electrode is fortified with deoxidizing elements Si and Mn; to ensure the necessary mechanical property and the stability and operability of electric arc, solid wire electrode is fortified with a property quantity of Ni, Cr, Ti and Al etc. Cored electrode features outstanding operability and good looking weld, but it is more expensive than solid wire electrode normally.

4.5.3 Welding Machine

Arc welding machine for manual welding is classified into direct current arc welding machine and alternating current arc welding machine. At the preliminary stage of welding, direct current arc welding machine is used for arc stability; and then, since AC welding machine that is less expensive and handier can also provide stable arc, alternating current arc welding machine is used at present in most cases.

With respect to non-gas-shielding semi automatic welding, AC arc welding machine is used when electrode is thick (3.2mm etc.), and DC welding machine is used when electrode is thin (2.0mm etc.). DC welding machine is usually used for carbon dioxide shielding semi automatic welding.

To perform welding in a proper manner, it is necessary to set corresponding electric current and voltage of welding machine, electrode and travel speed as shown in Table 4.5.3 according to welding content.

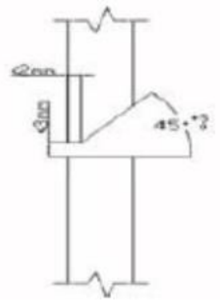
Table 4.5.3 Setting of welding machine

Welding condition	Welding diameter (mm)	Current (A)	Voltage (V)	Travel speed (cm/min)
Manual welding	4.0	140-190	20-35	15-20
5.0	190-250	25-30	20-30	
6.0	250-310	25-30	20-30	
Non-gas shielded semi automatic welding	2.4	200-420	24-27	15-25
3.2	350-480	25-28	20-30	

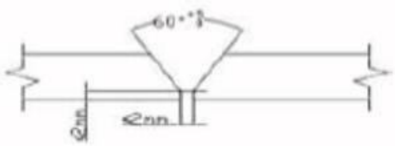
4.5.4 Welding Construction

(1) Opening shape

Longitudinal and horizontal interlocks of steel sheet pile usually employ butt welding with openings. The openings are normally K-shaped and V-shaped as shown in Fig. 5.5.4-1.



(a) K-shaped



(b) V-shaped

Fig. 4.5.4-1 Opening shapes

(2) Welding procedure

Butt welding of steel sheet pile is performed in the following steps:

Preheat - spot welding for erection - surface welding - grooving - back run welding

(3) Preheat

The properties of general steel products may change after heat treatment. Welding is equivalent with flashing and may bring about the hardening and the reduction of ductility in heat affected zone. To prevent this, the welding condition must be so determined that the maximum hardness Hv (10) of heat affected zone is kept less than 350. Carbon content of steel sheet pile is higher than in general steel, so low-H electrode is recommended and preheating is expected. Preheating is normally realized by heating adjacent area of welded zone through oxy-acetylene welding torch or allylene blowtorch. The temperature is usually measured with temperature choke.

(4) Note points of welding operation

The following points must be paid attention to during welding operation:

① Use welding equipment with adequate capacity and performance.

② Current and voltage is regulated in the neighborhood of welding area.

③ Since electrodes may be affected with damp when placed in atmosphere thus resulting in weld crack and pore etc., be sure to use adequately dry electrodes. Electrodes that have been affected by damp could be dried for about one hour at a drying temperature of 300~350℃.

④ In case of opening damage and other abnormalities, the normal shape shall be restored by hands.

⑤ Moisture, iron rust, iron scale and oil stains etc. that may cause pore, welding slag interfusion and weld crack etc. shall be removed using steel wire brush and rotary sander before welding.

⑥ Arc ignition may be coupled with remarkable shock cooling and result in hardened area, so be sure not to generate electric arc on base metal.

⑦ Like arc ignition, excessively short tack weld may result in the hardening of base metal and welded metal. Weld length shall be more than 40mm and avoid corner area and the starting point and ending point of main body welding.

⑧ The first butt welding layer is easily subject to insufficient welding, so gouging (grooving) back run welding is necessary.

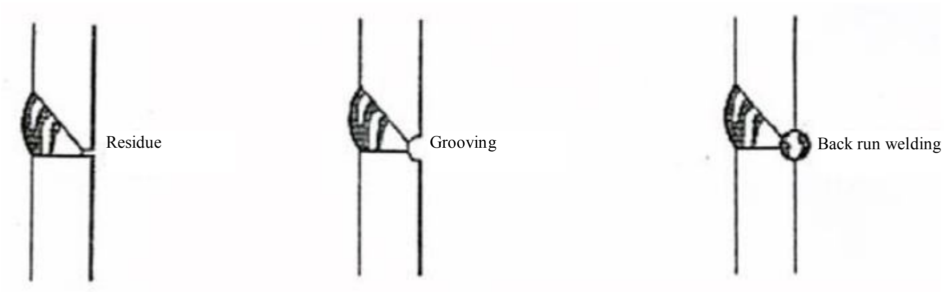
⑨ Try not to perform welding operation at an air temperature that is lower than 0℃ or in windy weather since these conditions may bring about high cooling velocity which would result in arc instability; if welding has to be performed under above-noted conditions, it is necessary to pre-heat the base metal in the area that is less than 100mm away from welded zone and take adequate draught-proofing measures.

(5) Reinforcement thickness

It should be noted that extreme reinforcement thickness may bring about stress concentration as a defect.

(6) Grooving

After the removal of residue generated during surface welding, there is a need to perform grooving so as to penetrate molten metal properly through back run welding. To achieve sound molten metal, the groove size is usually about 3~6mm.



(a) Surface welding (b) Grooving (c) Back run welding

Fig. 4.5.4-2 Grooving

Grooving methods are as shown below:

① Planing method: compressed air operated clink is used for planing.

② Gas grooving method: oxyacetylene flame and spray nozzle is used for the purpose of grooving; this method requires high proficiency level. This method would not make the noise like planing method.

③ Arc air gouging: grooving is made using electric arc generated through DC or AC welding machine with copper coated carbon rod that is partially black lead coated and using compressed air sprayed out of nozzle.

(7) Root welding method

Root welding electrode is used to perform complete fusion when welding is started from opening single-face so as to achieve artistic weld with no defect inside. This method requires no grooving but highly difficult welding operation. Only skilled welding operators may use this method.

4.5.5 Inspection

Welded zone is usually inspected as follows:

(1) Before welding: opening angle, route interval and cleaning of weld face etc.

(2) During welding: welding sequence, electrode diameter, electric current, arc length, and cleaning of areas between layers

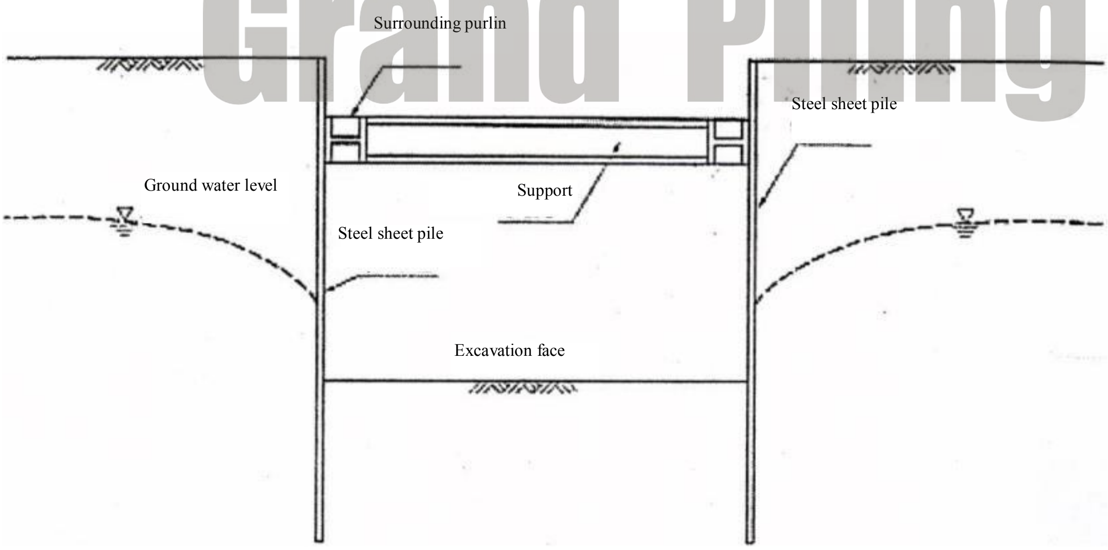
(3) After welding: visual inspection (undercut, lap joint, crack, reinforcement, and thickness etc.)

4.6 Water Sealing

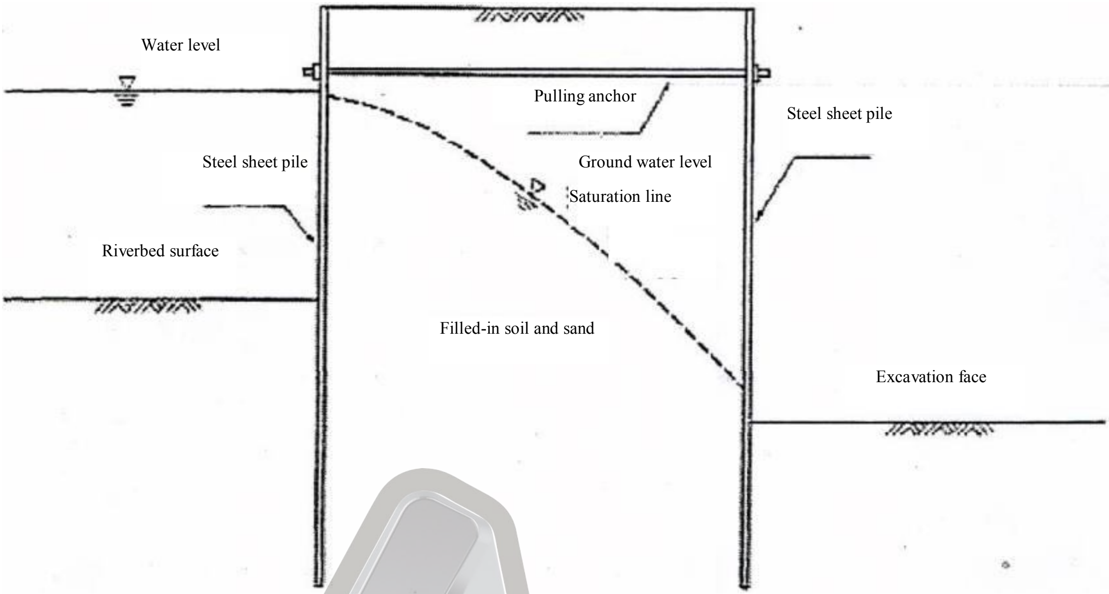
Steel sheet pile is generally used as structure component of quay wall and bank revetment or as water sealing wall or water barrier for embankment; also, steel sheet pile could be used as structure component and water sealing wall for various temporary structures.

4.6.1 Project Requiring Water Sealing

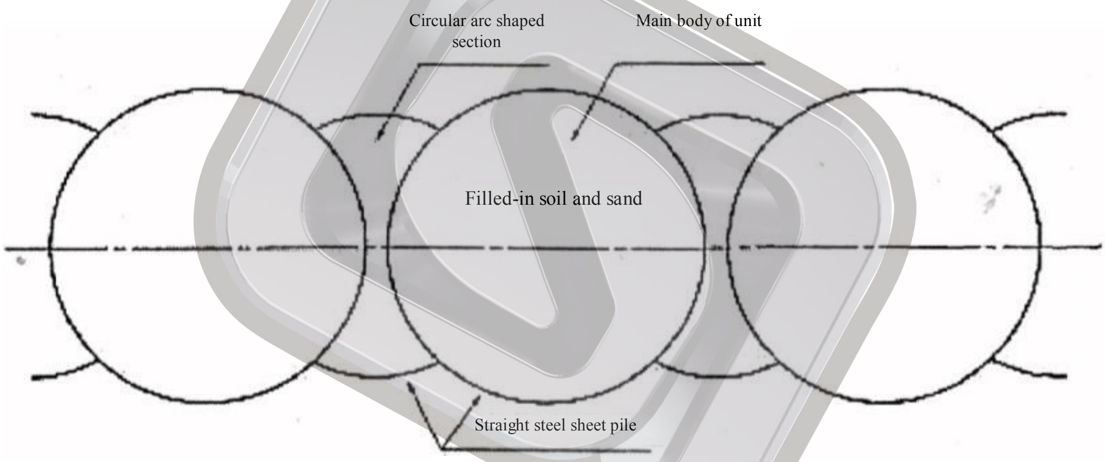
The works as shown below require steel sheet piles with water sealing property.



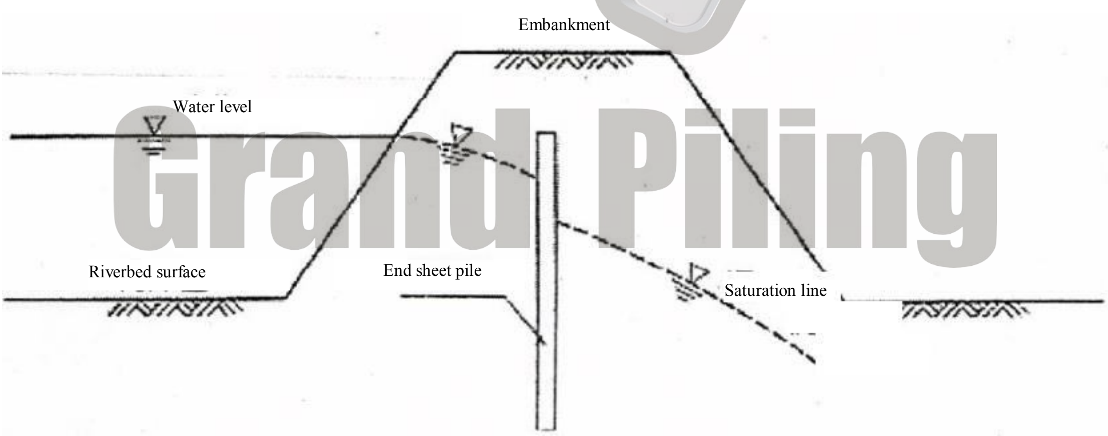
(a) Excavation works



(b) Cofferdam works using double-layer steel sheet-pile wall



(c) Cofferdam works using steel sheet pile unit



(d) Water barrier in embankment

Fig. 4.6.1 Works requiring water sealing

4.6.2 Water Sealing Property of Steel Sheet Pile

From the perspective of water sealing property, the most optimal state of interlock of steel sheet pile is gap-free engagement. But from the perspective of erection, some certain margin shall be reserved during manufacturing; therefore, water exclusion effect shall be realized through the sealing of interlock and the penetration depth of pile so as to ensure foundation pit safety.

Like hot-rolled steel sheet piles, the water sealing effect of GP series steel sheet piles is associated with various factors. Water sealing of steel sheet pile is classified into natural sealing and artificial sealing:

(1) Natural sealing means that interlock gap is blocked using suspended matters or soil, sand and other fine-particle substances at back side of steel sheet pile as shown in the figure below:

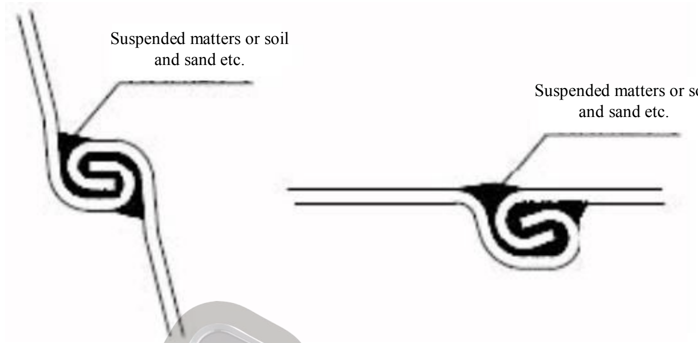


Fig. 4.6.2-1 Natural sealing

(2) Artificial sealing could be performed before or after the sinking of steel sheet pile

Materials for artificial water sealing are applied inside interlock before the driving and erection of steel sheet pile in the following ways:

① Mixture of ordinary asphalt, butter and sawdust etc. is uniformly applied to interior of interlock;

② Apply water-swollen sealing material uniformly into interlock and seal both ends and exterior of interlock and then remove the sealing during piling at construction site. This method has remarkable effect;

③ Various super artificial sealing

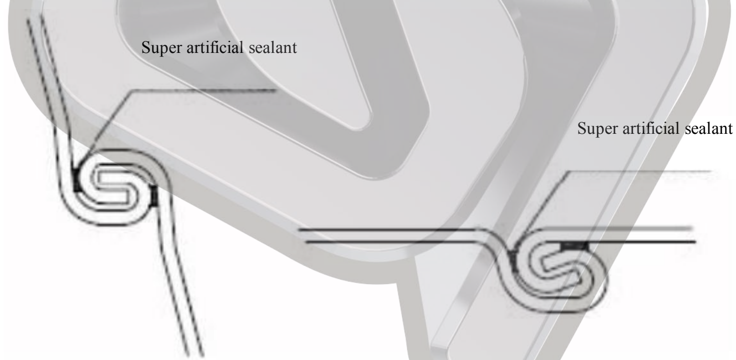


Fig. 4.6.2-2 Artificial sealing

Moisture-hardened liquid-state water sealing material with special resins from polyurethane and ethyl formate families as major component is directly poured into the interlock of steel sheet pile and dried naturally; this material swell gradually after being in contact with water and then plays the role of water sealing.

④ Magnetic sealant

This is a kind of soft rubber strip with super-strong magnetic force there-inside, of which the shape could be designed according to the shape of interlock. After the completion of construction, sealing strip is embedded into interlock connecting area of steel sheet pile at water surface. This sealing strip has remarkable effect on shoring foundation pit with small change of water pressure as shown in the figure below:

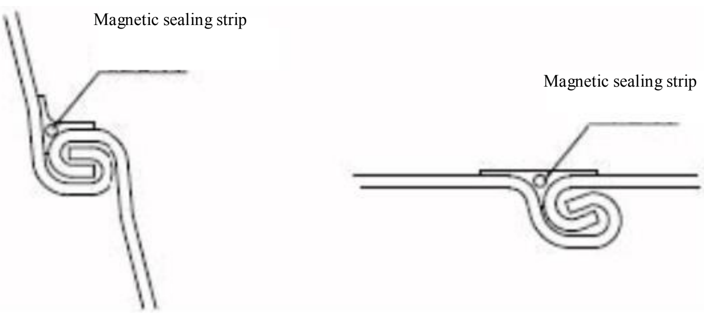


Fig. 4.6.2-3 Magnetic sealing

In all conscience, besides the vertical interlock sealing, water sealing shall be provided at steel sheet pile connecting area, the connection points between steel sheet pile and ring strip (or base plate) and between steel sheet pile and pull rod (anchor rod) etc. by welding of waterproof gasket or water stop (seal coat) or other technical measures for waterproof protection of underground works or node structure measures.

Water sealing effect of steel sheet pile under different conditions may vary. Here are some examples:

(1) Factors relating to steel sheet pile

① Corrosion degree of steel sheet pile;

② Bending and warping of steel sheet pile

(2) Construction environment related factors

① Water pressure;

② Water quality (turbidity);

③ Soil conditions (buried foundation and soil & sand at back side);

④ Degree of plugging of interlock by soil and sand

(3) Construction related factors

① Engagement state of steel sheet pile;

② Inclination and revolution of steel sheet pile

4.7 Corrosion Protection

Steel sheet pile is used in various environments such as atmosphere, earth, fresh water or strongly corrosive sea spray and sea water etc., so checking calculation of corrosion and corrosion protection is expected in case of long-term usage. Since no complete corrosion protection technique has been developed so far, the actually most cost-effective corrosion control shall be performed according to operating conditions.

4.7.1 Cause and Mode of Corrosion

4.7.1.1 Corrosion Mechanism

Steel corrosion is related to neutral water and oxygen, and its reaction formula is $Fe+H_2O+1/2O_2\rightarrow Fe(OH)_2$; the produced $Fe(OH)_2$ continues to react with oxygen and turns into "iron rust" composed of FeOOH, Fe_3O_4 and non-crystalline substances after the combination with or separation from H_2O.

The reaction through which steel is turned into "iron rust" proceeds under the effect of galvanic action. For various reasons, there are innumerable ultra-fine anodes (positive poles) and cathodes (negative poles) on the surface of steel that forms a local cell. Fe at anode of local cell is dissolved, and reduction reaction of oxygen occurs at cathode; therefore, corrosion reaction could continue.

4.7.1.2 Corrosion Mode

Corrosion mode is classified into general corrosion that almost uniformly occurs on all surfaces of metal, and local corrosion that is found at some certain part of metal surface. Local corrosion shows the following patterns:

(1) Pitting corrosion

Some certain part of steel surface under water is attached with corrosion products or porous attachment, under which the supply of dissolved oxygen is insufficient and the breathing property is poor; because of galvanic action, local corrosion occurs at this part.

(2) Stress corrosion

Stress corrosion occurs when metallic material is submerged in corrosive liquid that has special effect under stress. Since steel sheet pile is not used in such an environment, the said stress corrosion is left out of account under most circumstances.

4.7.1.3 Thickness of Iron Rust Layer

A fairly large proportion (more than 1/2 in most cases) of Fe lost due to corrosion contributes to the iron rust layer on steel surface; the remaining proportion remains on the surface of steel in the form of iron rust at the very start and is then stripped off as floating rust; another proportion does not turn into iron rust on surface but flows away in ionic condition.

Assuming that all Fe lost due to corrosion turn into iron rust layer on steel surface, the relation between the thickness of iron rust layer on steel surface and the decrease in board thickness could be expressed by the following equation:

$$y = \frac{D \times S \times x}{100A} \quad (4.7.1)$$

Where: y means the decrease (mm) in thickness of steel plate;

D is apparent specific gravity of iron rust;

A means the specific gravity of Fe (7.85);

x means the thickness (mm) of iron rust layer;

S means the content ratio (%) of Fe in iron rust layer.

The determination result of iron rust layer in general marine environment (atmosphere on the sea ~ in the sea) is $D = 2.01$ and $S = 52.4\%$; put them into above equation, and we get the result:

$$y = \frac{2.01 \times 52.4}{100 \times 7.85} x = 0.134x \qquad x / y = 7.46$$

I.e. The thickness of iron rust layer is about 7.5 times the decrease in sheet thickness; even if only about 50% of Fe lost due to corrosion turns into iron rust, iron rust layer thickness would be much more than (about 3~4 times) the decrease in sheet thickness as shown in the figure below. Hence, it should be noted that iron rust layer thickness is in no case equal to the decrease in sheet thickness.

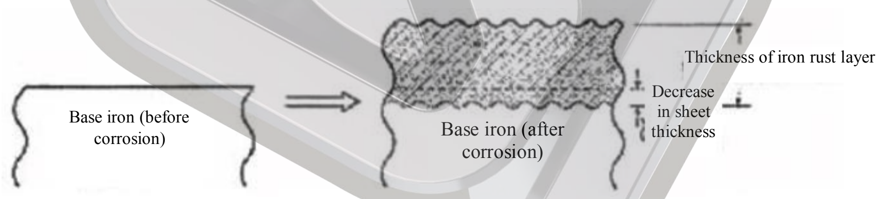


Fig. 4.7.1-1 Relation between iron rust layer thickness and the decrease in sheet thickness

4.7.1.4 Factors Affecting Corrosion

(1) Atmosphere

Corrosion of steel sheet pile in other environments is extremely low and does not play a leading role. Atmospheric exposure test result is as shown in figures below:

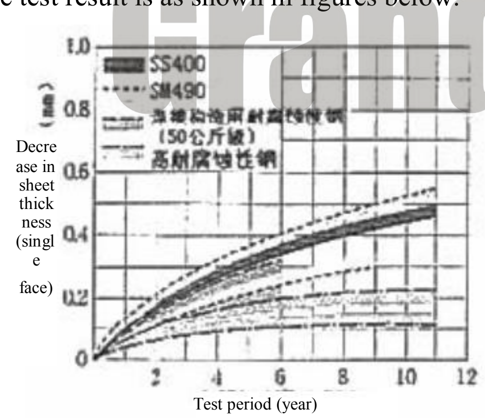


Fig. 4.7.1-2 Atmospheric exposure test in coastal industry zone

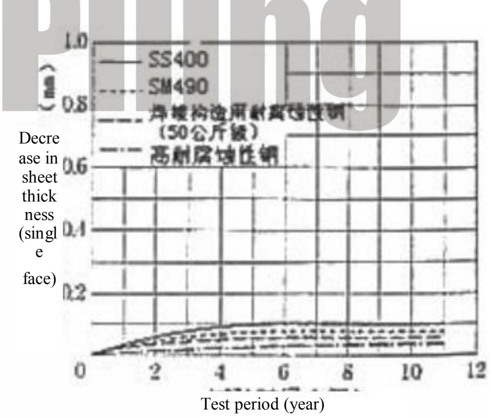


Fig. Atmospheric exposure test in inland city and rural zone

(2) Soil

Corrosion in soil environment is normally less serious than in water environment due to the following major factors:

① Air supply in water solution in soil is less complete than in pure water solution;

② Metal is surrounded by small particles in soil; since it is difficult for corrosion-generated water-soluble metallic compound to diffuse, the corrosion of metal body is suppressed.

In order to learn about the rough corrosion tendency of steel in soil, electric resistivity of soil is usually used. The higher the degree of water saturation is, the higher the concentration of electrolyte would be, and then the lower the resistivity would be; in case of a small electric resistivity, degree of corrosion would tend to increase. The table below shows the average yearly corrosion rate in soil.

Table 4.7.1-1 Average yearly corrosion rate in soil

Items		1~2m at earth surface and 30cm near ground water level(mm/y)	Conditions other than those shown in left table(mm/y)
Soil condition	Electric resistivity of soil(Ω*cm)		
High quality sand	6000	0.04～0.05	0.01～0.02
Sandy soil Gravelly soil Slity soil	1000～6000	0.05～0.07	0.01～0.03
Clay	1000	0.10～0.15	0.03～0.05

(3) Fresh water

Differently from sea water, river water contains less salty ions and thus has low electrical conductivity, which contributes to low corrosion rate.

Organism attachment conditions and corrosion modes are listed in the table below according to the corrosion status of steel sheet pile used for river bank revetment structure. It is observed from the table that the corrosion status in rivers with low salt concentration tends to be flat general corrosion with low mean corrosion velocity; furthermore, the maximum (average) local corrosion is less serious than in marine environment.

Table 4.7.1-2 Corrosion status in general rivers

River name	Attachment	Corrosion	Cl^- concentration in salt
The upmost water basin of river A (more than 15km away from river mouth)	Algae, water moss	Relatively flat general corrosion	(Less than 100)
River B	Oils, water moss	Relatively flat general corrosion; serious concavo-convex general corrosion detected under some certain feet	100 ~ 150
River C	Algae, water moss	Relatively flat general corrosion	55~65
River D	Thin water moss	Relatively flat general corrosion	40~80
River E	Water moss, lichen	Relatively flat general corrosion	50~90

Table 4.7.1-3 Corrosion rate and water quality of river

River name	Corrosion velocity (mm/y)		Water quality	
General corrosion (mean value)	Local corrosion (mean maximum value)	Dissolved oxygen (mg/l)	General corrosion (mean value)	Local corrosion (mean maximum value)

The upmost water basin of river A (more than 15km away from river mouth)	0.035	0.136	0.6~1.8	600	(Less than 100)
River B	0.054	0.101	1.05~1.70	600	100~150
River C	0.057	0.228	2.3~0.6	450	55~65
River D	0.076	—	0.55~1.93	400	40~80
River E	0.077	0.237	2.15~2.6	660	50~90

Corrosion rate in river is largely dependent on water quality: due to the small quantity of dissolved oxygen in rivers containing much oily wastes, almost no corrosion is detected at their bottom, in water and at surface thereof; in a seriously polluted river with low pH value, high flow velocity and active diffuse of O_2 supply, the corrosion may be unexpectedly serious; in rivers into which hot-spring water flows, acid corrosion may occur when pH value is less than 4.The effect of pH value on soft steel corrosion is as shown in the figure below.

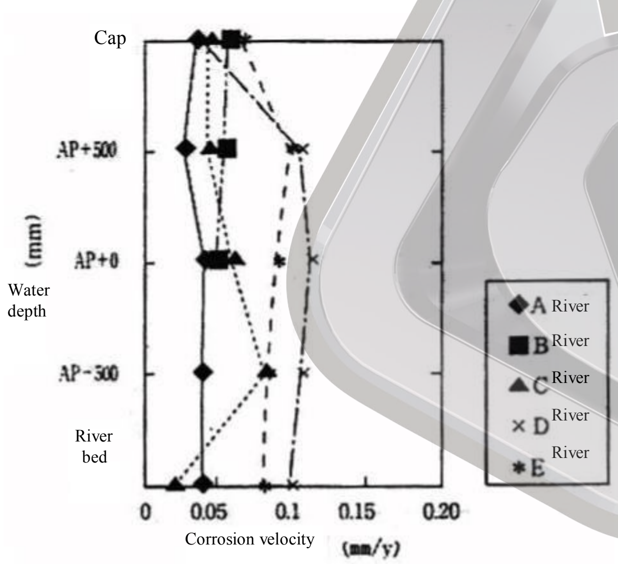


Fig. 4.7.1-4 Corrosion status in general rivers

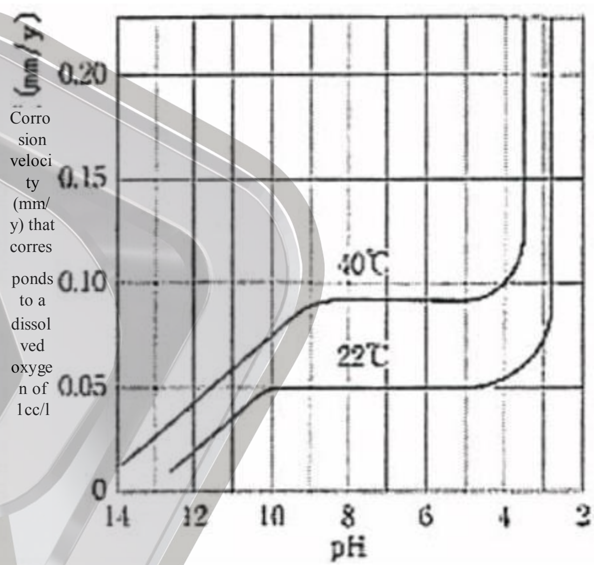


Fig. 4.7.1-5 Effect of pH value on steel corrosion

When the salt content of river is 0.01~0.03% and the average salt content of sea water (high seas) is about 3.5%, the salt concentration at river mouth would decrease. Generally speaking, low salt concentration corresponds to low corrosion velocity, but the corrosion velocity in the area where salt concentration is lower than in sea water may reach the peak as shown in the figure below. It is believed that the non-uniform mixture status as a result of specific gravity difference between river water and sea water results in various local corrosions.

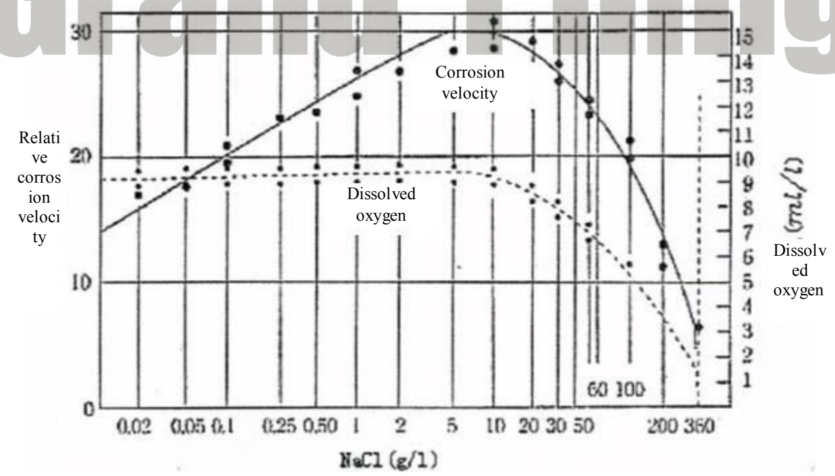


Fig. 4.7.1-6 Relation between salt concentration and corrosion velocity

(4) Marine environment

Distribution pattern of corrosion velocity of steel sheet pile in marine environment is as shown in the figure below:

① Due to the high content of sea salt particles (sea water spray sometimes) therein, marine atmosphere is more corrosive than land atmosphere. Furthermore, the corrosion would be more remarkable under the effect of sulphurous acid gas (if any).

② Due to the impact of sea water spray and tide, there is consistently thin moisture film on surface of steel in spray zone; because of the abundant oxygen supply through this film, the corrosion rate of steel is extremely high. Besides, the corrosion may accelerate due to the collision with floating wood, vessels and waves etc.

③ The steel in tidal zone is subject to cyclic repeated steeping caused by sea water, and its corrosion mode is similar to that in spray zone. But compared with spray zone, the corrosion velocity in tidal zone is extremely low. This is because the oxygen difference results in huge cell (ventilation difference cell) between tidal zone and the section that is close up thereto and the anode property of this section promotes the corrosion. Peak value of corrosion detected at the section close up to tidal zone as shown in the figure below reflects this phenomenon.

④ Under-water corrosion is next only to the corrosion in spray zone, but as already stated, in consideration of the increase in corrosion velocity at the section that is close up to tidal zone, the corrosion velocity in water depth direction at shallow sea is much the same. Furthermore, in the sea and tidal zone, the oxygen concentration cell generated by seaweed or attaching organism on steel surface or the organic matters and bacteria etc. produced from dead attaching organism would promote the corrosion. Sulfate-reducing bacteria grow remarkably in summer in contaminated sea area. As sulfate-reducing bacteria can promote cathodal reaction, the corrosion in summer is more serious.

⑤ The less contact with sea water in submarine soil results in less oxygen supply, so the slowest corrosion in marine environment is detected in submarine soil. But similarly to contaminated sea area, due to the effect of sulfate-reducing bacteria in areas where accumulated mud exists, the corrosion velocity is higher than in the sea.

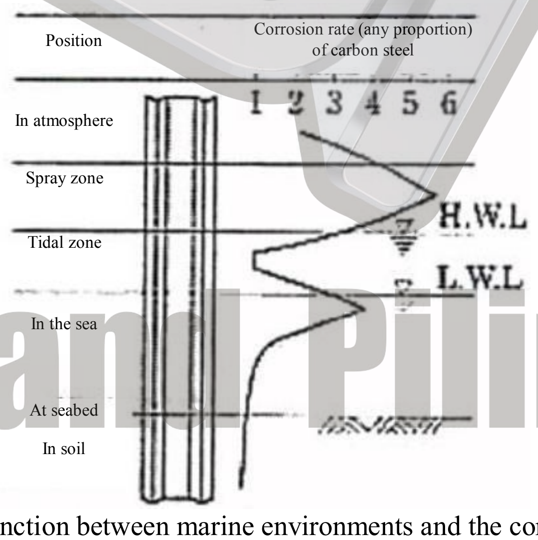


Fig. 4.7.1-7 Distinction between marine environments and the corrosion tendency

4.7.2 Corrosion Thickness Allowance

Design wall thickness of steel sheet pile shall be composed of effective thickness and corrosion thickness allowance. Effective thickness shall be calculated based on the strength and stability in service life and construction period. Corrosion thickness allowance means the corrosion thickness margin estimated and reserved during design and could be determined according to measured data of steel structure corrosion in similar environment. The yearly corrosion rate of steel sheet pile used as permanent supporting structure in corrosive environment may vary depending on atmosphere, water quality and other conditions: yearly corrosion rate of steel sheet pile (single face) is about 0.03-0.06 mm/a in freshwater environment, and about 0.05-0.5 mm/a in briny environment; in this case, the highest corrosion rate is found in water level fluctuation area and splash zone.

In case there is no measured data, corrosion margin thickness could also be calculated according to the following equation:

$$\Delta\delta = v[(1-P)t_1 + (t-t_1)] \quad (4.7.2)$$

Where: $\Delta\delta$ means the single-face corrosion thickness allowance (mm) required for steel sheet pile within the service life t;

v is the single-face average corrosion velocity (mm/y) of steel, which could be derived from Table 4.7.2-1 in case there is no measured data;

P means the protection efficiency (%) when other corrosion protection measures are taken, which could be derived from Table 4.7.2-2 when measured data is not available;

t_1 means the design service life (y) under the condition that other corrosion protection measures are taken;

t is the design service life (y) of protected steel sheet pile.

Table 4.7.2-1 Mean annual single-face corrosion velocity v of carbon steel for ocean engineering

Area	Mean annual corrosion velocity (mm/y)
Atmospheric zone	0.05~0.10
Splash zone	0.20~0.50
Water level fluctuation zone, underwater zone Under-mud zone	0.12~0.20 0.05

Note: mean corrosion velocity as shown in Table ① is suitable for environmental condition with pH=4~10 and should be increased properly for severely contaminated environment;

② When low alloy steel is used, values could be taken from the Table; however, the value shall be appropriately reduced for atmospheric zone;

③ The mean corrosion velocity of area that corresponds to estuary region with clear levels of water quality and salt content or the environment with high mean annual temperature, large wave and high flow velocity shall be appropriately increased;

④ For shore side of steel sheet pile, values could be taken by referring to under-mud zone.

Table 4.7.2-2 Cathodic protection efficiency P

Area	(%) $\bar{P}$
Above mean tide level	$0 \le \bar{P} < 40$
Between mean tide level and design low water level	$40 \le \bar{P} < 90$
Below design low water level	$\bar{P} \ge 90$

Note: in case of coating protection, the value of protection efficiency in design service life of coating could take 50%~95%; if coating protection and cathodic protection are combined, the mean protection efficiency could take 85%~95% under mean tide level, and could take the value according only to protection efficiency of coating above mean tide level.

4.7.3 Corrosion-resisting Coating

Comprehensive analysis and study on corrosion-resisting coating shall be made based on its design service life, environmental media, construction conditions, economy and other factors so as to determine the type, thickness and surface treatment grade of paint. Aluminum (zinc) spraying is a long-acting corrosion-resisting technique.

Aluminum (zinc) is sprayed onto surface of steel base and creates a electro-chemical corrosion resistant coating that can remarkably improve corrosion protection effect. Thanks to its favorable binding force and above-noted features, aluminum (zinc) spraying has been widely applied on steel structure surfaces in damp environment or areas with severe conditions such as ebb and flow zone (between river and ocean) and other environments with high content of chloride ion.

Surface treatment shall be performed before coating, and the quality of surface treatment has extremely large impact on coating quality. Different paints correspond to different steel rust removal quality requirements. Generally speaking, conventional oily paint with more favorable wetting property and permeability has slightly lower requirement on rust removal quality, while high performance paint such as zinc-rich paint requires high-quality rust removal.

Paint application shall be performed at a relative humidity of less than 85% and a temperature of more than 5℃; open-air construction is not allowable on rainy, snowy and foggy days, or coating adhesion would be adversely affected. Be sure to measure steel sheet temperature and relative humidity before coating. Only when surface temperature of steel sheet is more than 3℃ higher than dew point, can the construction be carried out.

Coating method shall be selected based on physical property of paint, construction conditions, requirements on coating and the condition of structure to be coated. Coating methods include brush coating, roller coating and high pressure airless spraying etc., and the coating could also be performed according to instructions of manufacturer. Metal spraying is classified into crucible spraying, powder spraying coating, wire spraying and plasma spraying depending on the type of spray gun used. Crucible spraying has been eliminated due to its inconvenience of use as the paint should be molten in crucible and then be sprayed out using compressed air. With respect to powder spraying, metal powder is fed into powder spray gun using powder feeder, molten with fuel gas and then sprayed out using compressed air. Since metal powder is hard to obtain, this method is seldom used at present. As the substitute for metal powder spraying, wire spraying is a common method in case of both arc melting and fuel gas melting. However, this method is applicable only to metals with relatively low melting point that can be turned into wires. The spraying of refractory metals and their oxides, carbides, silicides and borides etc. could employ plasma spraying method by which high-temperature plasma flame would be produced.

Examples of standard coating are as shown in the table below:

Table 4.7.3 Standard coating

Coating methods	Trades	Processing method and paint name	Rounds of coating	Number of coatings (g/m2/round)	Targeted film thickness (/round) μm
Organic zinc-rich paint + tar epoxy	Base treatment	Sand blasting or shooting method	-	-	-
Under-coating		Zinc-rich paint	1-2	200	200
Sur-coating		Tar epoxy	1	400-500	200
Tar epoxy series	Surface layer treatment	Sand blasting or shooting method	-	-	-
Tar epoxy			1	600	250

Coating inspection standard:

(1) Corrosion protection by coating

① Coating appearance standard: Coating appearance shall be uniform, flat and lustrous; defects such as lifting, crack and pinhole stripping etc. is not allowable. Slight local sag, brush mark, GPinkle and small quantity of particle dust that have no impact on performance are allowable.

② Coating thickness standard: The coating thickness measured with paint film thickness gauge must be able to meet design specified standard. Criterion of acceptability: coating of which the measured overall average thickness reaches 90% of nominal thickness is deemed qualified; in the calculation of mean value, the measurement point thickness that is 20% more than nominal thickness is calculated as 120% of nominal thickness.

(2) Corrosion protection by sprayed metal coating

① Coating appearance standard: Coating appearance shall be uniform and free of peeling, swelling, crack and chipping.

② Coating thickness standard: coating thickness must be able to meet design specified standard; the arithmetic mean value of thicknesses measured at measuring points using magnetic thickness tester shall be taken as the coating thickness at studied position; the testing position and quantity and the criterion of acceptability are determined based on design requirements.

③ Coating adhesion inspection standard: scribe grids in parallel with sharp cutter on coating with a spacing of 3~5mm, cohere with adhesive tape tightly and then pull it upwards vertically; the coating that does not fall off and show the base is deemed qualified.5~10 points shall be sampled randomly and uniformly on each piece of pile.

4.7.4 Corrosion Protection by Concrete Covering

Concrete or mortar is coated on the surface of steel. The large quantity of slaked lime generated through hydration or hydroclastic split of cement ensures the basicity of moisture environment of mortar, thus creating iron rust preventing mechanism.

Since corrosion protection by cathodic protection responds only to the section submerged in water, the top covers for revetment and quay wall of steel sheet pile are usually fabricated of concrete. When using this method, be sure to maintain adequate coverage thickness so as to prevent cracking and peeling.

4.7.5 Select Corrosion Resistant Type of Steel

The first thing designers have to think about is the selection of corrosion resistant steel based on the difference of corrosive medium. With progressive development of metallurgical industry, new types of special steels have been continuously developed; accordingly, it is possible to select corrosion resistant steel. For example, 16MnCu, 10CrMoAl and 10CrMoCuSi are used for seawater works; however, technical and economic evaluation shall be performed and other supporting protection measures shall be taken before the use.

4.7.6 Cathodic Protection

Cathodic protection is suitable for corrosion protection for steel pile below mean tide level of harbor work. Impressed-current protection system, sacrificial anodic protection system or the combination of these two systems is usually used for cathodic protection.

(1) Sacrificial anodic protection system

Sacrificial anodic protection is applicable for corrosion protection for steel structure below mean tide level of harbor work in sea water or brackish water of which the receptivity is less than 500Ω•cm (the use in sea water is recommended).Material of sacrificial anode shall have adequately negative electrode potential and be able to maintain surface activity during the use and to ensure uniform dissolution, the easiness of peeling of corrosion products, high capacitance, high current efficiency, processing and manufacturing easiness, sufficient source and low cost etc.

Construction of sacrificial anodic protection system:

① Sacrificial anode must be firmly mounted on steel structure to be protected and connected thereto in a short-circuit manner through welding (recommended) or fittings or cables;

② The size, weight, surface condition and iron core of sacrificial anode shall be inspected before installation, and the working surface shall not be stained with paint and oil stain; in addition, its mounting position shall adhere to design specification;

③ When sacrificial anode is fixed through underwater welding, welding shall be performed by underwater welding operators who have obtained certificate.

Quality inspection of sacrificial anodic protection system:

① Sacrificial anode shall be inspected according to design specification at delivery, and the manufacturer shall provide quality certificates regarding chemical composition, current efficiency, operating potential and open circuit potential in sea water, solubility property, dimensions, weight, and the contact resistance between anode body and iron core etc.

② Constructor shall perform construction according to design requirements and check the installation quality in a timely manner so as to ensure the reliable short-circuit connection between sacrificial anode and protected steel structure. When underwater welding is employed, the weld shall, to the greatest extent, be inspected through underwater photography or television, and the quantity of sacrificial anodes of which the weld is inspected shall not be less than 5%~10% of total number of sacrificial anodes. When nonconforming weld is detected, overall weld inspection shall be performed, and the nonconforming ones shall be repaired in a timely manner.

③ After the installation of sacrificial anode is over, constructor shall submit cathodic protection completion drawing indicating the actual number of anodes and their positions while measuring the potential at surface of protected steel structure. In case the potential fails to meet design requirements, remedial measures such as re-welding, replacement or supplement of sacrificial anode shall be taken in a timely manner.

(2) Impressed-current protection system

Impressed-current protection system normally includes auxiliary anode, direct supply (potentiostat or rectifier), reference electrode, testing equipment and cables.

Construction and quality inspection of impressed current cathodic protection system:

① When making electrical connection using welded steel reinforcement, be sure to set up marks and perform close inspection before concrete pouring; sold skip is not allowable. Connection points of reinforcement welding or cable connection in atmospheric zone or splash zone shall be protected through sealing.

② Auxiliary anode and its shielding board (tube) shall be installed according to the requirements of construction drawings. Metallic short circuit is not allowable between auxiliary anode and protected steel structure. There should be adequate allowance of the length of auxiliary anode cable under water.

③ Reference electrode shall be able to pass the inspection before installation and be operated according to the requirements of construction drawing. There should be certain allowance of the length of reference electrode cable under water. Ground point of reference electrode shall not be the same with or close to anode ground point.

④ DC power supply shall be mounted in a well-ventilated area where it is convenient to remove dust. DC power supply mounted outdoors in a dispersed manner shall be provided with well-ventilated drip-proof metal enclosure. Metal enclosure of DC power supply shall be grounded and its grounding resistance shall be less than 4Ω.When positive and negative electrodes of DC power supply are connected to corresponding anode cathode cables, polarity reversed is strictly prohibited.

⑤ Cable shall be arranged in steel pipe, PVC pipe and covered cable trench (bracket) and shall in no case be exposed to sun and the corrosion by strongly corrosive substances.

⑥ In case of segmented connection, cables shall be furnished with favorable sealing means and shall not be exposed to surrounding environment;

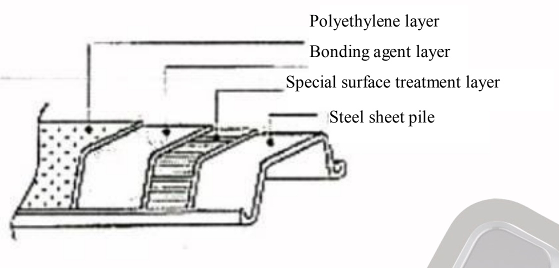
If necessary, they could be placed in junction box (case), and their anode connection shall in no case be in contact with metal enclosure of junction case (box).Try not to damage the sheath during the laying of cable; cable shall be repaired in case of partial damage, and be placed if serious damage occurs.

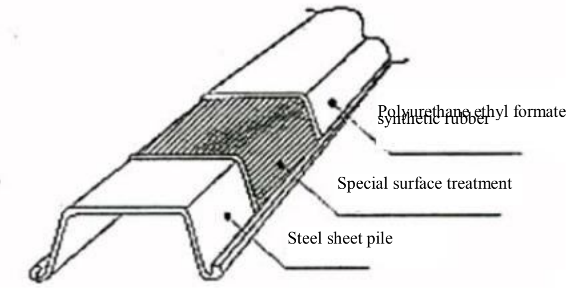
⑦ After the construction of cathodic protection system is completed, overall inspection shall be carried out for construction quality according to relevant norms and design requirements, and records shall be made.

4.7.7 Corrosion Protection of Cladding Layer

To realize long-acting corrosion protection, steel sheet pile manufacturer may cover its products with special HMWHDPE or polyurethane formic acid ethyl ester synthetic rubber in plant. Since the protecting material used is a kind of synthetic resin, electrical insulation is excellent. Furthermore, as corrosion control can adequately protect piles against high cohesive strength and stock-proof strength of the sea and other severe environments, it is ideal.

As is shown in Fig. 5.7.7-1, steel sheet pile from which the rust has been removed is applied with a layer of special surface conditioning agent (primer) in order to improve corrosion protection and bonding effect, and thereon, HMWHDPE or polyurethane formic acid ethyl ester synthetic rubber (two layers in case of polyethylene; the under-layer is bonding agent as shown in figure) is applied.





(1) Cladding polyethylene (2) Cladding polyurethane ethyl formate synthetic rubber

Fig. 4.7.7 Corrosion protection by cladding layer

Try not to damage protecting layer during construction of steel sheet pile with corrosion protection through cladding layer. It should be particularly noted that serious contact with guide frame and other materials during hoisting and driving may result in peeling, so crude assembly must be avoided.

China has not yet issued standard for inspection of quality of corrosion protection by cladding layer. The following requirements could be followed when specific design specification is unavailable:

(1) Cladding layer appearance standard: the appearance of cladding layer shall be flat, smooth and free of holes and exposed organic material. The number of bubbles with an area of less than $0.5\ cm^2$ shall not exceed 3 per square meter. Finishing layers should be provided with uniform paint film and free of serious sag and cockles.

(2) Cladding layer thickness standard: overall thickness of cladding layer could be measured with magnetic thickness tester. The testing position, number of test points and approval standard could be determined according to design requirements.

(3) Cladding layer withstand voltage test standard: withstand voltage test is conducted using special electrical equipment. Set test voltage according to design requirements and move the probe along the surface of tested cladding layer; if an alarm is given, it means that electrical spark breakdown occurs at his point, indicating that the cladding layer has defects such as thickness insufficiency, bubble, or failure of paint to fully percolate organic material. Carry out the test on a randomly selected surface area that accounts for 50% of total surface area along pile axis. The test is deemed to have been passed the number of alarm points does not exceed 5.

4.8 Construction Management

4.8.1 Overview

Pile foundation as the most important part of structure bears all static and dynamic loads of structure and transfers them to ground base so as to ensure the regular service of structure under safe conditions and the minimization of loss caused by unexpected disasters (such as earthquake and typhoon etc.).Therefore, construction management of pile foundation works is particularly important. Construction management means construction process management or control. Construction management of pile foundation work normally includes construction organization, planning, technology, quality, safety, equipment, material, finance, cost and civilized construction site etc., among which quality control is the link of the highest importance and must be attached great importance to by constructor, supervision unit, property owner and relevant authorities as it is the key to eliminate hidden quality troubles, reduce accident loss, avoid safety risk and preclude major accident.

4.8.2 Quality Control

Pile sinking mode is classified into dynamic sinking (hammering and vibration) and static sinking (i.e. pile jacking).

Before pile sinking, be sure to develop detailed construction organization plan, carry out close investigation on hydrological conditions, meteorological phenomena, geology, land form, existing building material or old buildings (or obstructions) in construction area, analyze construction difficulties and problems that may occur and take corresponding measures on a targeted basis while using appropriate constructional appliance and equipment so as to ensure the efficient and safe progress of construction.

(1) Over-water pile sinking

Special attention shall be paid to quality control of over-water pile sinking that features high difficulty and multiple influencing factors. Its control points are as follows:

① Provide on-board mechanical equipment of which the wind resistance (vessel), water flow resisting capacity, hoisting capacity and pile sinking ability (pile driving barge) meet the requirements. In addition, sufficient quantity of towing ships and tug boats with adequate capacity shall be provided.

② Be sure to attach importance to the analysis on geology and geomorphic information. It should be particularly noted that steep bank slope in pile sinking area may result in pile gliding, and thus bringing about increase in pile sinking plane deviation; on the other hand, land-slip may occur due to vibration and the increase in pore water pressure during pile sinking, so special attention shall be paid to this case.

③ Be sure to attach importance to the effect of wave force and drag force on process of and conditions after pile sinking. Wave force and drag force would not only affect the plane accuracy at pile position, but would also exert adverse impact on pitching of pile. Pile placement should be conducted in advance or in a delayed manner according to flow direction and velocity of water during pile placement. Be sure to prevent the impact of surge, wave, ship wave and heavy wind etc. during pile placement or sinking.

④ Try not to perform pile extension during over-water construction since the two pieces of piles above and below joint point may sway ceaselessly under the effect of water flow, wave and wind.

⑤ Technical personnel for pile foundation construction or survey personnel shall observe the pile sinking process at front section of pile driving barge or in front of pile driver and carry out corresponding treatment according to pre-plan in case of any exception.

⑥ Upon the completion of pile sinking, the pile shall be integrated with finished steel surrounding purlin in a timely manner so as to prevent the damage of single pile due to its repeated swaying under the effect of water flow and waves.

(2) In case of static jacking of pile, the management of the following points and the quality control shall be enhanced besides above noted relevant issues:

① During over-water pile jacking, it is necessary to adjust front and rear ballast water of vessel in a timely manner so as to assure the perpendicularity of pile during pile jacking; in the meantime, be sure to prevent the adverse effect of wave (especially surge), water flow and tidal level etc.

② In case of over-land pile jacking, there is a need to properly treat the foundation under pile jacking bracket so as to prevent the non-uniform settlement of foundation during pile jacking that may result in pile holder inclination. Adjust the equipment properly during pile jacking and ensure the perpendicularity of pile at the connection of upper and lower pieces of piles so as to avoid broken line intersection point at connection.

(3) Jacking equipment is recommended. Once holding equipment is used, it is necessary to check the fixture in order to ensure that the pile would not be damaged due to improper clamping.

4.8.3 Safety Management

To take safety measures, it is necessary to have an overall understanding of local hydrologic condition, meteorological phenomena, environment and natural disaster etc. and perform comprehensive analysis, take corresponding safety precautions and make safety pre-plan according to characteristics of this project.

(1) Hydrologic condition

Determine water flow velocity, direction, water element, tidal information, existence of surge, storm tide, slush, water depth, fluctuation of low water level and flood level, and analyze the cause of disasters and their potential impact on this project.

(2) Geology

Exploration bore hole, exploration data, land form (including over-land and under-water land forms), ground water level, confined water condition, pressure, distribution and layer height, and the existence of shifting sand, inflammable gases, underground river, erratic boulder, geological fracture zone, karst topography, underground obstruction, existing underground works and the erosion status in neighboring area of submerged structure etc.

(3) Weather condition

Collect meteorological data gathered by local weather station over the years, analyze the impact of wind, rain, snow, thunder, ice and fog etc. on engineering safety, and analyze the frequency and time period of disastrous weather such as typhoon, seasonal strong wind, thunderstorm and rainstorm etc.

(4) Environment conditions

Learn about the conditions, construction year and foundation type of buildings and structures in surrounding area of project and check if the said structures are constructed and reinforced and if there is precision instrument, meters and dangerous premises that are extremely sensitive to vibration; check if there is underground pipelines, dangerous goods storage tank and pool, HV line, under-water pipeline and hospital, school and residential area etc.

If necessary, be sure to supplement above-mentioned data in a timely manner and then make analysis and study on potential safety and quality troubles so as to enact corresponding measures. Develop pre-plan for unexpected accident that may occur and perform supervision from time to time according to relevant rules and regulations.

4.8.4 Equipment Management

During the process of construction, different types of construction equipment (including main piling equipment and auxiliary equipment) shall be used depending on pile foundation, specification, construction technology, geology, hydrologic data, meteorological phenomena and other factors. The name, dead weight, volume, energy support and consumption, power, and mechanical and electrical principle of these equipments may differ significantly. The fault of these equipments would have a strong impact on the progress, safety, quality, cost and environmental protection etc. of the whole project. Hence, the type selection, leasing, management, use, maintenance and repair of equipment shall be conducted by persons specially assigned during the overall process of construction.

4.8.5 Accident Prevention and Handling

(1) Over-land works

① Pay close attention to foundation and overhead pipelines

All pile foundation works are carried out on soft-soil foundation, so there is a need to check if the bearing capacity of foundation meets the requirements of constructional appliance on ground contact pressure, if there is hidden pitch or loosened soil for landfill of refuse so as to prevent constructional appliance overturn caused by inadequate local foundation strength. In the meantime, pay close attention to the distance between overhead pipelines and construction equipment so as to prevent electric shock or collision damage during equipment operation as a result of insufficient safe distance.

② Pay close attention to meteorological phenomena

During the periods of typhoon or monsoon, reliable draught-proofing measures shall be taken when stopping the service.

③ Pay attention to dangerous and old buildings in surrounding areas

In case there are dangerous and old buildings in the neighborhood, isolation means such as vibration-proof trench and anti-vibration wall etc. shall be provided so as to avoid the damage and collapse of dangerous and old buildings as a result of vibration and earth squeezing.

④ Informationized construction

Informationized construction shall be carried out for important buildings or works with complicated surrounding environment, i.e. to arrange observing points there-around, control the change in its plane and elevation, measure its pore water pressure, mount inclinometry pipe, measure the displacement of deep soil mass, and if necessary, to measure seismic wave or the stress and strain on important structures. Collect the said data in a timely manner so as to take corresponding measures.

⑤ Pay close attention to underground buildings and structures

Be sure to check upon underground buildings or structures before the commencement, and if necessary, work with relevant departments to find original completion data etc. Take particular care of urban water supply and drainage pipes, power cables, communication cables, coal gas and military facilities of which unnecessary loss may be caused due to long-term operation, poor data accuracy or inadequate completion information and great errors etc.

(2) Over-water works

The construction technique and management of over-water pile foundation works is more difficult due to the effects of factors such as water flow, wave, tide, strong wind, rain and fog etc.

① Matters needing attention during pile placement

a. With respect to over-water pile foundation positioning, the upstream-oriented allowance of pile placement should be determined based on the flow velocity and direction during pile placement; more attention is required in case of large water depth and high flow velocity. If the degree of impact of water flow on pile is unclear, pile placement trial could be conducted for fear that excessive pile foundation deviation would occur. Furthermore, the gradient of underwater topography also has great effect on pile placement position; hence, an allowance is reserved, and this allowance is associated with gradient and the hardness of surface soil conditions. Several trials could be conducted before formal pile placement if there is no experience that can be referred to.

b. For over-water pile foundation works, tidal difference is usually used for construction. Pile foundation in areas with low water level at bank side is usually constructed at the time of high water in the sea or tidal estuary so as to solve the problem of insufficient immersion depth of pile driving barge or inadequate height of pile driving frame; piles (especially concrete tubular pile) with cap elevation is usually driven at low water level; in order to protect pile body against the adverse effect of water hammer, pile sinking through hammering is not allowed when pile cap is submerged in water. Pile sinking shall be performed by taking the advantage of low-water season during construction in river or lake so as to create conditions for dry construction of top-side structure.

② Set up clear indications of navigation obstruction lamp etc.

Be sure to set up navigation obstruction lamp and other clear indications at pile cap so as to enable the personnel on ships can see over-water or under-water navigation obstructions both in the daytime and at night. Construction vessels shall be kept away from construction area at night so as to prevent dredging anchor breaking finished piles.

③ Clamp surrounding purlin for linkage in a timely manner

Upon the completion of one group of piles, they should immediately be connected using surrounding purlin which shall be designed and calculated according to operating conditions and have adequate rigidity and strength.

④ Make emergency pre-plan for protection against typhoon and flood

In order to ensure the safety of construction ship and personnel, emergency pre-plan shall be made against typhoon and flood according to data gathered by local weather station and oceanographic station over the years, and there is a need to arrange for shelter anchorage and towing ship on duty. In the meantime, completed works shall also be reinforced before typhoon and flood so as to ensure the safety. Marine operation personnel must receive information from local weather station and oceanographic forecasting station three times a day while collecting and analyzing medium-and-long-term weather and water level conditions so as to develop corresponding construction plan and countermeasures.

⑤ Pay attention to over-water structure, submerged structure and obstruction in construction area

Learn about the navigational clearance and clear width of over-water HV lines, bridges, pipe supports, submerged pipes, lines, fiber-optic cables, underwater obstacles, shipGPeck and old structures in construction area; when necessary, exploration or clearing shall be made in advance, while cooperating with maritime affairs authorities and pipeline authorities to handle relevant formalities and setting up proper over-water signs and marks.

⑥ Attach importance to operation on foggy days

Develop measures for operation and safe navigation on foggy days and mark the ships and pile foundations so as to prevent collision with other ships.

⑦ Formulate over-water operation safety system

It is essential that an operation system should be established for over-water operation safety management. Drinking is strictly prohibited during over-water operation. Be sure to wear life jacket. Staircase must be provided with safety net and life buoy, while corresponding medical and rescue facilities and networks shall be established. Relevant personnel shall receive trainings on over-water operation safety and obtain a certificate before going to work.

4.8.6 Intermediate Acceptance and Completion Documents

Handover of completed works to subsequent working process or property owner is an essential link of construction and an important part of construction management.

(1) Intermediate acceptance

Intermediate acceptance means the handling of handover procedure toward next process of pile foundation work or the handover to general contractor.

(2) Completion documents

As one of the important engineering documents, completion document shall be prepared according to file management standard and requirements of national or local government.

(3) Preparation of completion document

The essential purpose of preparation of completion document is to factually and comprehensively reflect relevant techniques and quality concerned during construction, and the documents include design document, drawings, design modification notice, construction related contact list, relevant meeting records & minutes, construction organization design document, record files relevant to construction, quality certificate and review report of material and semi-finished product, concrete related test report, qualification of welding procedure, report on handling of accident due to quality, builder's diary, economic contract and final account for completed project etc.

4.9 Environmental Assessment and Protection

4.9.1 Overview

Both design and construction of pile foundation works shall comply with national and local government's policy, decree, regulations and rules regarding environmental protection, and corresponding measures shall be taken to protect neighboring environment.

Constructor must, in construction preparation period, follow design document and relevant regulations of local government to establish environmental protection measures for pile foundation works and pre-plan for environmental monitoring and control design and submit them to relevant governmental departments for approval and then carry them out during construction. The approved environmental protection measures and environmental monitoring design shall be included in construction organization design of pile foundation works.

Pile foundation construction may adversely affect surrounding environment. For example, the noise, vibration and soil compaction effect generated during pile sinking by hammering and vibration method. It should be noted that over-land works may affect the working condition and daily life of businesses and residents around construction site and bring about adverse effect on surrounding buildings, flood control facilities, underground and ground pipes, trenches and lines of water, power and gas etc., various precision instruments and telecommunication instruments. To this end, the adverse effects of construction on environment shall be fully taken into account in design phase, and additionally, corresponding preventive measures shall be established and evaluation analysis shall be conducted.

4.9.2 Evaluation of Effect of Steel Sheet Pile Construction on Environment and Protection Measures

During pile foundation construction, be sure to implement Environmental Protection Law of the People's Republic of China, the Regulations for Prevention and Control of Environmental Noise of the People's Republic of China and other environmental protection related laws, regulations and rules and take measures to prevent and mitigate the impact of noise and vibration etc. generated during construction of pile foundation.

(1) Pile foundation construction noise and preventive measures

① Pile foundation construction noise

All pile foundation constructions will generate noise, especially the noise generated during pile sinking by hammering. Noise spreads in air in the form of plane sinusoidal wave and shows linear attenuation by the logarithm value of distance from noise source. In case of multiple noise sources, the noise at noise-affected point would be the combination of all noise sources. Noise intensity and the extent of harm is measured by dB (decibels). Noise level (dB) at noise source and noise-affected point could be measured with instrument.

The Standard of Environmental Noise of Urban Area (GB 3096) and the Measurement Method for Environmental Noise in Urban Area (GB 14623) have the following requirements on five environmental noise levels in urban area:

Table 4.9.2-1 Five environmental noise levels in urban area

Equivalent sound level LAeq: dB

Level	Daytime	Night
0	50	40
1	55	45
2	60	50
3	65	55
4	70	55

Note: sound level A: sound level A is the sound level measured using A-weighted network and is expressed by LA and in dB; equivalent sound level is the average energy value of A sound level within specified period of time and is also known as equivalent continuous sound level A that is expressed by LAeq and in dB.

2. The maximum value of burst noise at night shall not exceed 15dB as a standard value.

3. Areas to which the standards apply shall be defined by local government.

As shown above, standard 0 applies to resort area, senior villa zone, luxury hotels and other areas in particular demand for quiet atmosphere. Standard 0 plus 5dB shall be applicable to this kind of areas in suburban and rural areas.

Standard 1 applies to residential, educational and institutional areas and is suitable for rural living environment;

Standard 2 applies to areas where residence, commerce and industry are mixed;

Standard 3 applies to industrial areas;

Standard 4 applies to urban roads, traffic artery, areas at both sides of road, and areas at both sides of inland waterway that passes through urban area. Besides, this standard also applies to background noise (i.e. the noise level when no train passes) of areas at both sides of trunk line and auxiliary line of railway that passes through urban area.

② Measures for reduction of noise

Measures for reduction of noise include sound source control, insulation, protection facilities, construction time control and other basic protection methods as shown in the table below:

Table 4.9.2-2 Measures for reduction of noise

Methods	Measures	Protection effect	Remarks
Noise source control	1. Low-noise hammer 2. Low-noise or noise-free pile type and construction method 3. Reduce construction noise by improving pile cap, pile cushion material and pile chuck 4. Mount pile hammer type or integrated silencer cover on diesel pile and set sound deadening material at inner wall of cover.	With a protective cover, the noise of pile hammer can be reduced by 5dB~20dB; noise level A 30M away from piling zone can be kept less than 70dB	Despite the poor effect, silencer cover of pile hammer is cost-effective and easy-to-use.
Shielding wall	Design shielding wall according to criteria for noise control, attenuation and synthesis based on noise absorption coefficient of relevant materials	Noise can be reduced by 20dB~25dB	Shielding wall is preferably set 15M away from piling zone and shall not be higher than control height of noise source and noise affected point
Protective facilities	Sound attenuating chamber (such as operation control room of construction operating personnel and testing laboratory requiring sound isolation) made from materials with excellent noise damping effect	Reduce or avoid harm to human health and ensure safety in production	
Time control	Control pile sinking operation time, and get away from breathing space of residents and organizations in surrounding area	Ensure adequate rest and normal daily life of residents	Time control must be determined through multi-party coordination

(2) Impact of pile foundation construction and corresponding protection measures

① Impact of pile foundation construction induced vibration

a. Impact of pile foundation construction induced vibration on structures

Vibration caused by pile sinking by hammering and vibration may be transmitted to surrounding buildings. Once neighboring structures have resonance response to the vibration of pile foundation construction and some certain level of vibration duration or energy is reached, adjacent buildings would be damaged seriously.

b. Impact of pile foundation construction induced vibration on foundation soil mass

Vibration generated during pile sinking by hammering and vibration produces elastic waves that would spread to adjacent foundation soil mass, which brings about material additional change of foundation such as settlement, uplift, cracking, deflection and displacement etc. that may bring about adverse effect on daily life and work of neighboring organizations and residents. Vibration generated during over-water pile sinking by hammering may bring about bank slope instability and landslide accidents. So, be sure to fully take into account the interaction between soil and structure when paying attention to the impact of vibration generated during pile sinking by hammering and vibration on structures. To evaluate this interaction, it is necessary to measure on-site shear wave velocity and dynamic modulus in advance.

c. Impact of pile foundation construction on environment of urban area

When pile foundation construction is conducted in urban area, the vibration as a result of pile sinking by hammering and vibration will bring about effect on urban area. Vibration intensity (dB) is expressed by vertical vibration level Z. Measurement of vertical vibration level Z and the method for calculation of evaluation quantity should be in accordance with relevant provisions of China national standard GB10071.

② Measures for prevention of impact of pile foundation construction induced vibration

Vibration damage during pile foundation construction is mainly caused by pile sinking by hammering and by vibration, and the degree thereof is dependent hammering and vibration energy, the distance from vibration source, the geological conditions and other factors; furthermore, vibration hazards and noise hazards often co-exist. Therefore, the protection against noise and vibration hazards is usually taken into comprehensive account in actual construction. Protection against vibration during pile sinking construction is as shown in table below:

Table 4.9.2-3 Protective measures for construction induced vibration

Protection type	Methods and measures	Remarks
Selection of construction equipment and material	Pile cushion made of cushion material, bellevill spring pile cap, low-vibratory strength high-construction frequency pile hammer; pile body coated with materials for reduction of frictional resistance	
Selection of construction technology	Construction technologies such as pre-drilling method, dredging method, jetting method and static pressure method etc.	
Vibration attenuation wall	Build a wall (with a thickness of 50~60cm, a depth of 4~5m; soft soil layer is up to 15~16m thick) under ground at the point 5~10m away from pile-sinking zone between adjacent building, underground pipelines and ground pipe trench in pile-sinking zone.	Both empty trench type (for support or pouring of wall protecting mud) and solid wall type is available (underground continuous wall, cement mixing pile, jet grouting pile, plastic foam wall and asphalt wall are frequently used; alternatively, multi-layer mixture wall could be used)
Vibration attenuation hole	Earth holes with a diameter of 50~60cm and a depth of about 5m (hole could be extended to an appropriate extent if soft soil layer is relatively thick) is set at certain interval under ground at the point between pile sinking zone and neighboring building, underground pipelines, ground pipe trench, cable and wire 5~10m away from pile sinking zone.	Drill the hole using drilling pile machine. Slurry protection could be used.
Reinforcement (of dilapidated building)	1. Temporary underpinning reinforcement system 2. Temporary unloading and removal of wall body	

4.9.3 Development and Evaluation of Steel Sheet Pile Construction Environment Protection Program

(1) Measures for environmental protection of over-land construction

At construction preparation stage of over-land steel sheet pile construction, it is necessary to establish environmental protection program. To do so, the corresponding approach stated in previous section could be referred to.

(2) Environmental protection measures for over-land construction

Over-water construction is mainly conducted during hydraulic structure foundation construction. Over-water construction faces the same environmental problems with over-land construction, though the protective measures in some aspects are slightly different.

Ship is both the major equipment for over-water construction and the living facility for operating personnel. National laws and regulations expressly prohibit the over-standard emission of industrial and domestic pollutants produced during production.

(3) Development and evaluation of construction environment protection program

As an element of construction organization design, construction environment protection program shall be based on the basis, principle and procedure for construction organization design and comply with local government's environmental protection regulations, stipulations, norms and rules.

Construction environment protection program covers the following basic items:

① Environmental protection requirements of this project (including local government's environmental protection regulations and stipulations, EIA report requirements, and project characteristics etc.);

② Environmental protection design document of this project and the review comments of owner on environmental protection design;

③ Analysis on construction organization and technology used on environment;

④ Analysis on design and result of environmental protection measures (including monitoring and control) plan;

⑤ Construction plan of environmental protection measures (including monitoring and control);

⑥ Layout plan of environmental protection measure (including monitoring and control);

⑦ Budget statement of environmental protection measures (including monitoring and control);

⑧ Assessment comment on environmental protection program (including meeting minutes of assessment meetings organized by owner)

Environmental protection program (including monitoring and control) for pile foundation construction must be reviewed by relevant departments and professionals before approval. Constructor shall integrate the approved environmental protection program with pile foundation construction organization plan.

4.9.4 Implementation and Evaluation of Environmental Protection Program for Steel Sheet Pile Construction

Environmental protection program for construction shall be implemented before commencement. Environmental protection and monitoring and control facility is also an independent work which requires the preparation of construction scheme and expense budget and shall be separately conducted and settled; accordingly, an economic & technical comparison and effect valuation procedure shall be established for implementation of environmental protection and monitoring & control program.

(1) Pre-plan for monitoring, control and first aid repair of surrounding buildings (including underground structure) and underground pipeline during construction

Vibration due to pile sinking by hammering during construction would do harm to surrounding buildings. Affected buildings (including underground structures) and underground pipelines shall be monitored and protected according to approved environmental protection program. More specifically, settlement and displacement observation points shall be set on these facilities after excavating and removing the earth as shown in the table below:

Table 4.9.4-1 Monitoring and control of settlement and displacement of surrounding buildings caused by over-land pile sinking by hammering

Monitoring items	Monitoring method and means	Monitoring period	Early warning standard
Settlement and displacement of underground pipelines, wires and cables within affected range	Set fixed measuring point after excavation and perform conventional deformation measurement with gradienter and theodolite	Observe the change of settlement and displacement on a daily basis	Alarm is given when underground pipelines displacement reaches 1cm or the allowable curvature is close
Settlement and displacement of surface-layer soil and displacement of deep soil mass in surrounding area of pile zone	Set fixed measuring points at surface layer between building and pile zone. Use gradienter and theodolite to perform conventional deformation measurement. Observe the displacement of deep soil mass using inclinometer	When the rate increases, additional 2 to 4 times shall be performed; inclinometry and gap observation is normally performed once every 2~3 days, and should be performed more frequently when the change is significant.	
Change of excess pore water pressure in surrounding area of pile zone		Bury pore pressure gauge at boundary of pile zone and use it to determine pore water pressure	

Building foundation deformation Building crack observation	Establish settlement and displacement observation point in deformable area around building and measure the deformation using gradienter and theodolite. Make measuring mark at the top of building and use theodolite to measure the inclination of building.
	Make an observation cake of gypsum at crack and observe with crack magnifier

(2) Preplan for protection, monitoring, control and first-aid repair of coastal levee, wave prevention and flood control facilities during over-water pile sinking

Over-water pile sinking especially over-water pile sinking by hammering may bring about vibration effect. Therefore, pre-plan for first-aid repair after occurrence of accidents shall be formulated for protection of and safety monitoring and control of relevant coastal levee, wave prevention and flood control facilities during construction.

① To satisfy the requirements of ship draught during over-water construction, it is necessary to determine the safest over-water dredging scope when dredging operation for over-water construction is required at pre-construction stage. If near-shore draught can still not meet the requirements of ship for over-water pile foundation construction, piling shall be performed in a "closure gap suspension" manner. If the requirement can still not be satisfied, it is recommended the design be modified so as to ensure the safety of coastal levee and wave prevention and flood control facilities.

② During over-water construction, the ground anchor and mooring line of construction ship would collide with coastal levee and wave prevention and flood control facilities; Hence, feasible avoidance or reinforcement program should be formulated under the precondition that the function is not damaged and degraded; this program shall, at pre-construction stage, be submitted to local coastal levee and flood control authorities for approval before the implementation. The program must include pre-plan for first-aid repair.

③ Safety monitoring and control of coastal levee and wave prevention and flood control facilities must be conducted during over-water construction, especially during over-water pile sinking by hammering. See Table 4.9.4-1 for content.

Monitoring, control and protection of bank stability during over-water pile sinking

① Measures for ensuring bank stability during over-water pile sinking

a. Measures for ensuring bank stability shall be established in over-water pile foundation design Construction method and procedure that is propitious to bank stability shall be employed during over-water pile sinking construction. These measures may include gradient reduction, vertical drainage gallery, relieving platform and stage construction etc. that ensures the bank stability during construction period and service life.

b. If bank slope is not stable enough in construction period of over-water pile sinking, the observation shall be enhanced during over-water pile sinking construction period; once the evidence of collapse is detected, measures such as grading, toe ballast, crest load reduction, drainage by well points and erection of anti-skid pile etc. shall be taken immediately; during over-water pile sinking construction, perform the construction at high water level, stop the construction at low water level, reduce pile sinking rate, drive piles at intervals, drive the piles lightly with heavy hammer at low frequency. The two pile driving barges that are operating simultaneously must be kept away from each other by a certain distance.

c. For soft soil especially those with high sensitivity, the construction progress shall be slowed down so as to prevent soil mass collapse at bank slope. Effective drainage measures shall be taken when perform bargeed-in filling at crest of bank slope or behind quay wall during over-water pile sinking so as to prevent excessive head difference. When there is confined water that affects the stability at bottom of foundation pit, it is necessary to perform temporary pressure relief and prohibit the stock-piling of spoil and sundries at crest.

d. In order to ensure the stability of soil mass at bank slope during pile foundation construction, pore water pressure monitoring instrument or inclinometry tube together with displacement and settlement observation points shall be buried in bank slope within the range of pile foundation construction if necessary after computational analysis of rotational slip stability of bank slope. Be sure to perform observation and measurement from time to time during pile foundation construction, adjust and control the progress of pile foundation construction thereby and take corresponding technical measures so as to complete the construction of relevant pile foundation in a stable state of bank soil mass at bank slope. Pore water pressure monitoring instrument shall be buried within the calculated range of rotational slip. For pile sinking by hammering, the buried position of monitoring instrument shall be less than 30m away from pile sinking position

② Measures taken when bank slope is in a dangerous state

There is no definite value for early warning of dangerous state regarding earth slope stability. Under normal circumstances, the abrupt increase in pore water pressure and soil shift value is deemed the sign of hazardous state of bank stability in which measures must be taken. Under normal circumstances, pile sinking by hammering shall not be restored unless excess pore water pressure disappears and the soil displacement value is extremely close to zero.

4.10 Common Problems in Construction of Steel Sheet Pile and the Solutions thereof

4.10.1 Penetration is difficult due to excessive pile driving resistance

This problem may occur for two reasons. First, due to the deformation and corrosion of steel sheet pile connecting lock notch, steel sheet pile fails to move downwards along interlock smoothly; in such a case, steel sheet pile body and interlock of steel sheet pile shall be inspected and treated before piling; second, the piling resistance is excessively high in solid sand layer; if that is the case, it is necessary to perform detailed analysis of geology, make study on possibility of penetration and drive the pile while injecting water to facilitate the sinking.

4.10.2 Sector-shaped inclination of sheet pile in travel direction

This kind of inclination would largely decrease when "folding screen method for pile sinking" is employed. Because the resistance at interlock connection with previous sheet pile is higher than the resistance of soil mass around empty interlock to the pile at the other side during the travel of sheet pile, the sheet pile cap moves in the direction of travel. In such a case, be sure to keep the steel sheet pile interlock unhindered and apply grease inside interlock so as to reduce resistance; mount steel sheet pile "stop" at surrounding purlin for construction in order to limit the movement of steel sheet pile cap in travel direction.

After the formation of sector-shaped inclination of steel sheet-pile wall body, adjustment shall be performed at the earliest opportunity using a wedge-shaped pile (narrow top, wide bottom) fabricated according to measured inclination data (adjust the straightness and smoothness of this pile and interlock).

4.10.3 Neighboring sheet pile is dragged in

The top cause of this problem is the excessive resistance at interlock connection. This problem could be solved by taking measures as stated in the section named "sector-shaped inclination of sheet pile in travel direction". Once the tendency of "neighboring pile drag-in" is detected, the dragged pile shall be connected to finished pile through welding so as to prevent drag-in.

4.10.4 Twisting of pile body

Since steel sheet pile interlock employs hinged connection, twisting may occur during insertion and hammering. Hence, it is necessary stop and rectify the twisting, or the deflection of central axis of sheet-pile wall may occur. To prevent the twisting of pile body, a "stop" could be mounted on surrounding purlin in travel direction of pile and compose a "limit" together with surrounding purlin so as to lock the position of interlock at the other installation side of steel sheet pile that is sinking. It should be noted that this "stop" shall be firmly lapped with surrounding purlin and that grease shall be applied to notch groove of this "stop" in a timely manner so as to facilitate the sinking of pile body.

4.10.5 Interlock leakage

After the establishment of steel sheet-pile wall, slight leakage may occur at hinged connection of interlock and is not allowable for permanent structure requiring high-level leakage resistance. Fill interlock with the mixture of butter, asphalt and dry sawdust (equal volume) before pile sinking so as to improve leakage resistance and facilitate the driving of sheet pile. In recent years, polyurethane based water expandable putty has been widely used to fill interlock during steel sheet-pile wall construction at ship yard so as to improve the leakage resistance.

4.10.6 Interlock slip-off

When steel sheet pile interlock is damaged or driving is not stopped when obstacle is met with, interlock slip-off may occur. Hence, all interlocks shall be inspected strictly before construction. When piling meets with obstruction, be sure to find out the causes and stop the driving immediately. Besides, squashing of pile bodies that occur when "folding screen method" is used may bring about interlock slip-off. To solve this problem, the "stop" of terminal steel sheet pile and the stiffening sheet and clamping sheet of Z-shaped built pile could be removed before pile driving by "folding screen method" so as to relieve the compression force of sheet-pile wall.

4.10.7 Difficulty of pile pulling

The top cause of difficulty in pile pulling during removal of temporary steel sheet pile structures is interlock corrosion and deformation and the insertion of steel sheet pile into hard soil etc. To ensure the successful pile pulling, it is necessary to keep steel sheet pile and interlock smooth, straight and unhindered and to apply butter inside interlock during pile driving. If built pile is used during piling, the number and position of built pile shall be recorded; welding between pile bodies shall also be recorded for reference at pile pulling. When removing the first pile of wall body, be sure to turn aside the inter-connected pile bodies. Once the first pile is pulled out, the subsequent pile pulling would be smooth since the interlock resistance at one side disappears. Estimate pullout force of pile according to geological data collected before pile pulling and the degree of difficulty of piling and select corresponding pile pulling machine (vibratory hammer) and auxiliary equipment. The first pile is normally difficult to pull out. Knock the pile with hammer to make the interlock connection loose; you can also insert a steel tube at both sides of sheet pile and then inject water and air to destroy the side pressure of soil mass on pile to be pulled. Pile body can be pulled out in most cases after above-noted measures are taken.

References

From design to application of steel sheet pile (Nippon Steel)
Cold formed sheet piling of non alloy steels (EN10249-1)
Execution of special geotechnical work- Sheet pile walls (BS EN 12063:1999)
Piling Handbook
Pile foundation construction manual
Code for design of steel structures (GB50017-2003)