JTG

Industry Standards of
the People's Republic of China
中华人民共和国行业标准

JTG 3363—2019（EN）

Specifications for Design of Foundation of Highway Bridges and Culverts

公路桥涵地基与基础设计规范

（英文版）

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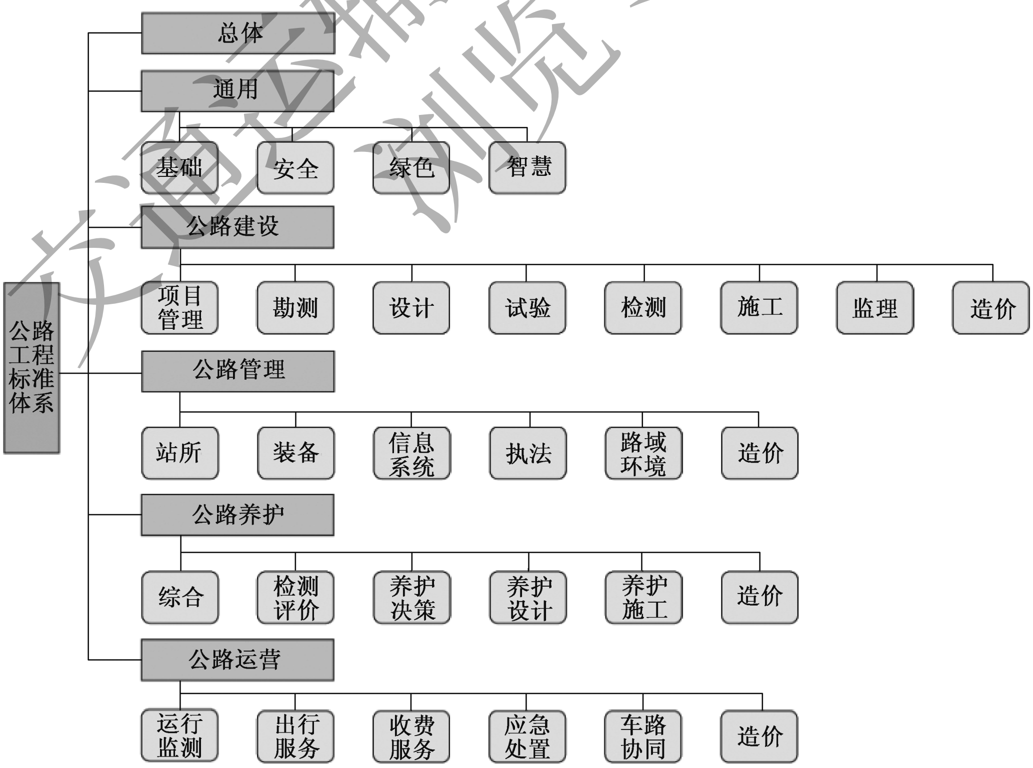
英文版编译出版说明

标准是人类文明进步的成果，是世界通用的技术语言，促进世界的互联互通。近年来，中国政府大力开展标准化工作，通过标准驱动创新、协调、绿色、开放、共享的共同发展。在丝绸之路经济带与 21 世纪海上丝绸之路，即“一带一路”倡议的指引下，为适应日益增长的全球交通运输发展的需求，增进世界连接，促进知识传播与经验分享，中华人民共和国交通运输部组织编译并发布了一系列中国公路行业标准外文版。

中华人民共和国交通运输部发布的公路工程行业标准代号为 JTG，体系范围涵盖公路工程从规划建设到养护和运营管理全过程所需要的设施、技术、管理与服务标准，也包括相关的安全、环保和经济方面的评价等标准。



中国政府历来高度重视交通基础设施建设,不断完善公路基础设施设计相关的标准规范。二十世纪八十年代,中国在原《公路工程技术标准》(JTJ01—81)基础上,开始制订公路路线、路基、路面、桥梁、涵洞等专业技术规范,并在1985年颁布实施了第一部《公路桥涵地基与基础设计规范》(JTJ 024—85),用于规范公路桥梁和涵洞的地基与基础设计。尔后,又经历了2007年的第一次修订(JTG D63—2007)和2019年的第二次修订(JTG 3363—2019)。经过四十多年的技术发展,现已建立了内容较为完整的公路桥梁和涵洞的地基与基础设计体系。

本次编译的《公路桥涵地基与基础设计规范》(JTG 3363—2019)中文版于2019年12月修订发布,并于2020年4月1日实施。

到2023年底,中国公路通车总里程已超540万公里,其中高速公路通车总里程超18万公里。公路桥梁总计已达107万座,其中特大桥1万余座。《公路桥涵地基与基础设计规范》(以下简称《规范》)一直是我国公路桥涵方面的强制性技术标准,对中国公路工程建设质量提供了重要保障。本规范在修订过程中开展了多项专题研究和调研工作,吸取了国内有关科研院所、高校、设计、检测等单位的研究成果和实际工程经验。

这些成果与经验在《公路桥涵地基与基础设计规范》(JTG 3363—2019)中得到了充分的体现。本英文版的编译发布便是希望将中国的工程经验和技术成果与各国同行进行交流分享,为其他国家的公路建设提供参考借鉴。

本英文版的编译工作由中华人民共和国交通运输部委托福州大学主持完成,并由中华人民共和国交通运输部公路局组织审定。

本英文版标准的内容与现行中文版一致,如出现异议时,以中文版为准。

感谢中文版主编袁洪先生在本英文版编译与审定期间给予的指导与支持。

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The People's Republic of China

Ministry of Transport

Public Notice

No.91

Public Notice on Issuing the *Specifications for Design of Foundation of Highway Bridges and Culverts*

The *Specifications for Design of Foundation of Highway Bridges and Culverts* (JTG 3363—2019) is hereby issued as one of the industry standards for highway engineering, to become effective on April 1, 2020. The former edition of the *Specifications* (JTG D63—2007) and its English version (JTG D63—2007 EN) named as *Code for Design of Ground Base and Foundation of Highway Bridges and Culverts* shall be superseded from the same date.

The general administration and final interpretation of the *Specifications for Design of Foundation of Highway Bridges and Culverts* (JTG 3363—2019) belong to Ministry of Transport, while the particular interpretation for application and routine administration of the *Specifications* shall be provided by the CCCC Highway Consultants Co., Ltd.

Comments, suggestions and inquiries are welcome and should be addressed to CCCC Highway Consultants Co., Ltd. (No. 85, Deshengmenwai Street, Beijing 100088, China). The feedbacks will be considered in future revisions.

It is hereby announced.

Ministry of Transport of the People's Republic of China
December 17, 2019

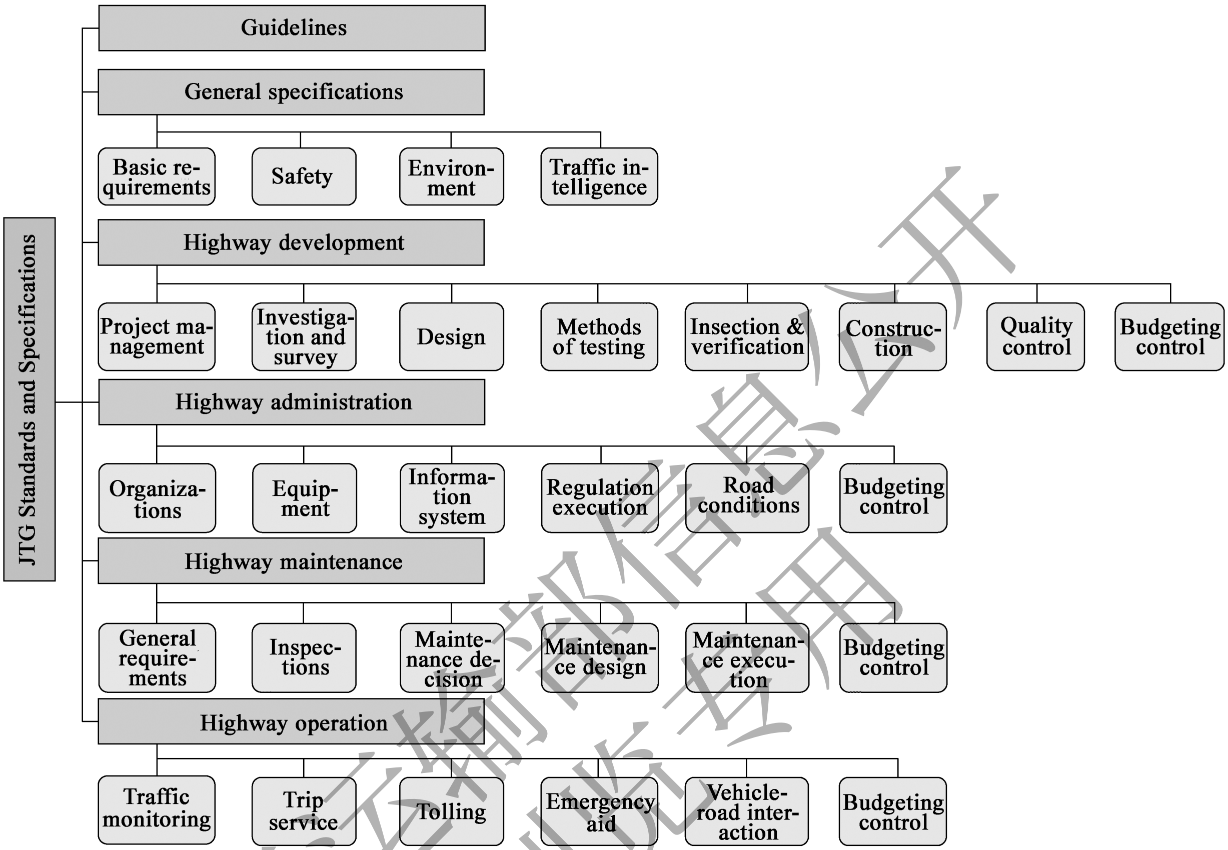
General Office of Ministry of Transport　　Printed on December 19, 2019

Introduction to English Version

Standards reflect the achievement of civilization, provide common languages for technical communications and improve global connectivity. In recent years, the Chinese government has been proactively implementing standardization to stimulate innovative, coordinated, greenness and opening up for shared development in China and worldwide. To align with the Belt and Road Initiative for mutual development, the Ministry of Transport of the People's Republic of China organized the compilation and publication of international version of Chinese transportation industry standards and specifications to meet the increasing demands for international cooperation in transportation, enhance global connectivity, promote knowledge dissemination and sharing of experience.

JTG is the designation referring to the standards and specifications of the highway transportation industry, issued by the Ministry of Transport of the People's Republic of China. Item compasses the entire lifecycle of highway engineering projects, from planning and construction to maintenance and operation management. It includes standards for the facilities, technologies, management, and services required throughout these processes, as well as standards related to safety, environmental protection, and economic evaluation.

In the highway standard system, it includes a number of standards for design, construction and maintenance of bridges, which have effectively supported the rapid development of highway bridges in China. The current bridge design specifications include: *General Specifications for Design of Highway Bridges and Culverts*, *Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts*, *Code for Design of Highway Masonry Bridges and Culverts*, *Specifications for Design of Highway Cable-stayed Bridge*, *Specifications for Design of Highway Suspension Bridge*, *Specifications for Design of Highway Concrete-filled Steel Tubular Arch Bridges*, *Specifications for Design of Highway Precast Concrete Bridges*, *Wind-resistant Design Specification for Highway Bridges*, *Specifications for Collision Design of Highway Bridges*, *Specifications for Landscape Design of Highway Bridges*, etc. As of the end of 2022, 1.033 million highway bridges with the total length of 85.765 million meters have been built in China, among which concrete bridges account for more than 90%.



In the 1980s, following the publication of the principal standard, *JTJ 01—81*: *Technical Standards for Highway Engineering*, a series of professional and vocational specifications were drafted and developed. These specifications involve in highway geometry, subgrade, pavement, bridges and culverts. The first edition of the standards of *Specifications for Design of Foundation of Highway Bridges and Culverts* (JTJ 024—85) was issued and implemented for design of foundation of highway bridges and culverts in 1985, followed by a first revision and second revision in 2007 (JTG D63—2007) and 2019 (JTG 3363—2019) respectively. For over four decades of technical development and continuous improvement, these Standards have effectively and successively served as guidelines for the design of highway bridges and culverts.

The Chinese version of *Specifications for Design of Foundation of Highway*

Bridges and Culverts was issued in December 2019 and has been implemented since April 1, 2020.

By the end of 2023, the total length of highways being operated in China about 5.10 million kilometers, of which the length of 180 thousand kilometers is for freeways. The *Specifications for Design of Foundation of Highway Bridges and Culverts* has always been a technical mandatory standard for construction of highway bridges and culverts, playing an important role in quality assurance of highway engineering construction.

During the revision, the editorial team conducted extensive special studies and investigations, reviewed the updated technical development and practical engineering experiences from research institutions, academies, design, and detection departments in China, and referred to relevant standards in China and other countries.

All these experiences and achievements have been incorporated and summarized in the *Specifications for Design of Foundation of Highway Bridges and Culverts.* The release of the English version of the Specifications aims to share with international professionals the engineering experience and technical achievements from China, to offer reference for highway construction in other countries.

The editing of the English version was conducted by Fuzhou University under the authorization of the Ministry of Transport of the People's Republic of China and approved by the Highway Department, the Ministry of Transport of the People's Republic of China.

The contents and numbering of the chapters, sections, clauses and sub-clauses in the English version are consistent with those in the Chinese version. In the event of any ambiguity or discrepancy between the English version and the Chinese version of the Specifications, the Chinese version shall prevail.

Gratitude is given here to Mr. Yuan Hong, the chief editor of the Chinese version, for the valuable guidance and comments during the review of the English version.

Feedbacks are welcome and will be taken into account in future editions. Please address them to the editing organization for English version in writing (Address:

No. 2, Wulongjiang North Avenue, Fuzhou University Town, Fuzhou, Fujian, China, Postal Code: 350108, E-mail: baochunchen@fzu.edu.cn).

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Foreword to Chinese Version

The revision of the *Code for the Design of Ground Base and Foundation of Highway Bridges and Culverts* (*JTG D*63—2007), pursuant to the "*Notice of Planning for Compilation and Amendment Program of Highway Engineering Standards in* 2015" (No. 312) issued by the Ministry of Transport of the People's Republic of China in 2015 was carried out by the chief editing organization of CCCC Highway Consultants Co., Ltd. The revision has been approved and issued as *Specifications for Design of Foundation of Highway Bridges and Culverts* (JTG 3363—2019) for implementation.

During therevision, the editorial team conducted extensive special studies and investigations, reviewed the updated technical development and design experience in China, and referred to the relevant domestic and international standards. Upon the completion of the first draft of the *Specifications*, the draft specification was circulated for comments from relevant experts and organizations involved in design, construction, maintenance, and administration, based on which several rounds of discussion, consultation, and updating were conducted before being finalized for approval.

This revised *Specifications* comprises 9 Chapters and 18 Appendixes. The content mainly includes: 1-General Provisions, 2-Terms and Symbols, 3-Basic Rules, 4-Geotechnical Classification, Engineering Characteristics and Bearing Capacity of Grounds, 5-Shallow Foundations, 6-Pile Foundations, 7- Caisson Foundations, 8-Underground Diaphragm Walls, 9-Special Grounds and Foundations.

The main revisions in thisversion include: addition of the stipulations on the calculation for the composite piles of concrete-filled steel tube; revision of the calculation formula on rock-socketed depth for the rock-socketed piles; revision of the relevant technical provisions on the treatment for soft soil were revised; supplementary requirements for design of the piles in the collapsible loess;

additional requirements for design of foundations in steep slopes and karst zone; supplementary provisions for the relevant design on the piles with expanded branches and plates.

Feedbacks are welcome and will be taken into account in future editions. Please address them to the chief editing organization for Chinese version (Li Xiaoti, Address: No. 85, Deshengmenwai Street, Beijing, CCCC Highway Consultants Co., Ltd., Postal Code: 100088; Tel: +86-010-82017041, Email: sssohpdi@163.com).

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Contents

1 General Provisions

1.0.1 The *Specifications* is developed for regulating the design of the foundation of highway bridges and culverts to ensure the quality of construction projects.

1.0.2 The *Specifications* is applicable to the design of foundations of highway bridges and culverts to be built on all classified highways.

1.0.3 The design of the foundations shall follow the principles of adjusting measures to local conditions, using local materials, and saving resources.

1.0.4 The type of foundations of highway bridges and culverts shall be reasonably selected based on geological, topographic, hydrological, and construction conditions, as well as the loads, materials, and types of superstructures and substructures, etc.

1.0.5 The foundations shall be designed based on the relevant survey data. The survey data shall accurately reflect the topography, landform, strata, adverse geology, physical and mechanical properties of rock and soil, underground water, etc.

1.0.6 Durability design shall be conducted for foundation structures in accordance with the requirements of the relevant specifications.

1.0.7 In addition to the *Specifications*, the design of foundations of highway bridges and culverts shall also comply with the provisions in the current relevant national and industrial standards.

2 Terms and Symbols

2.1 Terms

2.1.1 ground

The soil body or rock mass that bears the structural actions.

2.1.2 foundation

Components of substructure that transfer various actions of structure to the ground.

2.1.3 characteristic value of subsoil bearing capacity

The pressure value corresponds to the specified deformation inner the linear segment in the curve of soil pressure-deformation of ground measured by loading test. It is also known as ‘characteristic value of ground bearing capacity’.

2.1.4 joint

The crack or fissure without apparent displacement of the rock stratum between the two sides of the fracture plane of the rock mass.

2.1.5 bearing stratum

The stratum that directly bears the action of the foundation.

2.1.6 underlying stratum

The stratum with a certain depth below the bearing stratum, which is compressed or possibly shear-damaged.

2.1.7 gravity density

The gravity of rock/soil per unit volume, which is the product of the density of the rock/soil andthe gravitational acceleration, also known as ‘unit density’ or ‘unit weight’.

2.1.8 shallow foundation

A kind of foundation with abearing depth less than its width, for which the various resistances of soils at its sides are not considered in design.

2.1.9 seasonal frozen soil

The soil layer that is frozen in winter and fully thawed in spring/summer, and it is revised to 'seasonally frozen soil'.

2.1.10 permafrost

The soil layer that remains frozen for more than two years.

2.1.11 pile foundation

The foundation comprises one or several piles and pile cap or tie-beams.

2.1.12 negative friction

The downward resistance of friction generated by the soil to the surface of a pile when the soil around the pile has a settlement greater than that of the pile body, which is induced by soil self-weight consolidation, self-weight collapse, additional load on the ground, etc. It is also known as 'negative skin friction'.

2.1.13 foundation pile

The single pile in the pile foundation.

2.1.14 foundation of group-piles

The pile foundation comprises two or more piles.

2.1.15 open caisson foundation

A foundation of well-type structure with cutting edge and without top and bottom plates, which is constructed by lowering to the foundation bed through excavation of soil or rock inner the well or through other auxiliary measures for sinking, and finally the top and bottom of the caisson are sealed. It is also known as 'caisson foundation' or 'caisson'.

2.1.16 underground diaphragm wall

A diaphragm wall below the ground surface for water interception and impermeability, soil retaining, and load-bearing, and also known as 'diaphragm wall' for simple.

An engineering measurement to improve the bearing capacity, deformation property, or permeability of the ground soils.

2.1.18 pile with expanded branches and plates

A concrete cast-in-place pile with branches or plates that interact with the surrounding compacted soil, and alsoknown as 'under-reamed pile'.

2.1.19 tangential frost-heave

The force generated by the frozen expansion of the ground soils acting in a direction parallel to the side of the foundation. It is also known as 'tangential frost heave'.

2.2 Symbols

2.2.1 Symbols for resistances and material properties of ground

C_u—Nominal value of undrained shear strength of ground soil;

E—Modulus of elasticity of concrete;

E_{si}—Compressive modulus of the i^{th} soil layer;

f_a—Adjusted characteristic value of ground bearing capacity;

f_{a0}—Characteristic value of ground bearing capacity;

f_{aj}—Characteristic value of soil bearing capacity at the j^{th} branches and plates;

f_d—Design value of steel strength;

f_{rk}—Characteristic value of the uniaxial compressive strength of saturated rock;

q_r—Characteristic value of bearing resistance of soil at pile tip;

q_{rj}—Characteristic value of bearing resistance of soil at the end of the j^{th} branch or plate;

q_{rk}—Nominal value of bearing resistance of soil at pile tip;

R_l—Supporting force of the soil under the bottom plane and inclined surfaces of the cutting edge;

R_a—Characteristic value of axial compressive resistance of a pile;

R_b—Sum of the nominal values of bearing capacity of the ground soil under the cutting edge, the diaphragms, and the base beams of the caisson;

R_t—Characteristic value of axial tensile resistance of a pile.

2.2.2 Symbols for actions and action effects

$F_{fw,k}$—Nominal value of the buoyancy of water;

G_k—Nominal value of self-weight of caisson;

H—Horizontal force at the top surface of the bedrock;

H_i—Horizontal force caused by the combination of actions with nominal values or by the accidental combination of actions with nominal values;

M—Bending moment with respect to the gravity axis of the foundation base, induced by the actions of horizontal and vertical forces;

M_H—Bending moment at the top surface of the bedrock;

M_x, M_y—Bending moments around the X- and Y-axis of the foundation base, respectively, both are induced by the actions of horizontal and vertical forces;

N—Vertical force acting on the foundation base caused by the action combination;

P_i—Vertical force;

p—Compressive stress of the foundation base;

p_0—Superimposed compressive stress of the foundation base under the quasi-permanent combination of actions;

p_m—Average compressive stress in the replacement layer;

p_{max}—Maximum compressive stress of the foundation base;

p_{min}—Minimum compressive stress of the foundation base;

p_z—Compressive stress in the soft ground or soft soil layer;

p_{0k}—Superimposed compressive stress at the bottom of the replacement layer;

p_{gk}—Compressive stress at the bottom of the replacement layer induced by its self-weight;

p'_{0k}—Compressive stress of the foundation base;

p'_{gk}—Compressive stress of the foundation base induced by its self-weight;

q_{ik}—Nominal value of the skin friction at the i^{th} soil layer;

q_{pk}—Nominal value of critical load distributed along the perimeter of the pile;

q_{tk}—Nominal value of load combination;

R_f—Nominal value of total skin friction of the caisson wall;

R'_f—Nominal value of total skin friction of the caisson wall in the state under investigation;

S_{bk}—Effects of combination of actions with nominal values for stability of foundation;

S_{sk}—Effects of combination of actions with nominal values for instability of foundation.

2.2.3 Symbols for geometrical parameters

A—Area of foundation base;

A_l—Total supporting area of the diaphragms and base beams;

A_p—Sectional area of the pile tip;

A_{pj}—Area of the j^{th} branch or plate;

a, b—Side length of the foundation base; or the width of the lower plane in the cutting edge, and the level projection of inclined plane embedded into the soil in the cutting edge;

D_{rl}—Required relative density after the ground is compacted;

D_p—Equivalent transformed diameter of the prefabricated vertical drain;

d—Diameter of the pile body;

d_e—Effective diameter of the drainage cylinder;

d_w—Diameter of the sand drain well;

d_{min}—Minimum bearing depth of the foundation base;

e_o—Distance from the acting point to the centroid of the section;

h—Bearing depth;

h_0—Original height of the soil sample;

h_{max}—Allowable maximum thickness of frozen layer below the foundation base;

h_p—Height of soil sample after its settlement is stable;

h'_p—Height of soil sample after its additional settlement is completed;

h_r—Effective socketed depth of the pile in the bedrock;

h_z—Depth of replacement layer of sandy gravel;

I—Inertia moment of sectional area;

l—Length of rectangle foundation base;

l_i—Depth of the soil layer below the pile cap bottom or local scour line;

l_s—Center-to-center distance of sandy stone piles;

S—Settlement value of foundation;

S_0—Settlement value of foundation calculated by layerwise summation method;

S_{cu}—Compressive deformation value of the replacement layer itself;

S_s—Settlement value of the underlying layer;

t—Wall thickness of the steel pipe pile;

u—Perimeter of the pile;

V—Volume of drainage;

W—Section modulus of foundation base in eccentric direction;

W_x (W_y)—Section modulus against X-axis (Y-axis) in the edge of the eccentric direction of the foundation base;

z—Distance from the foundation base or pile tip to the top surface of soft ground or soft soil layer;

z_0—Nominal frost depth;

z_d—Design frost depth.

2.2.4 Symbols for analysis and others

c_1—Performing coefficient of tip resistance;

e—Natural void ratio of soil;

e_0—Void ratio of sand before ground treatment;

e_1—Required void ratio after the ground is compacted;

e_{max}, e_{min}—Maximum or minimum void ratio of soil;

k—Safety factor for stability of foundation structure;

k_0—Stability factor in resisting overturning of pier or abutment foundations;

k_1, k_2—Adjusting factors for the width and depth of the foundation base;

k_c—Stability factor against sliding of pier or abutment foundations;

k_{st}—Sinking factor;

m—Number of rock strata, or the proportionality factor for horizontal partial factor of subgrade in non-rock ground;

m_0—Bottom cleaning coefficient;

n—Number of soil layers;

α—Coefficient of superimposed compressive stress in the soil;

α_i—Influence coefficient of skin friction of the vibration-driven pile in each soil layer;

β—Modification coefficient considering the influence of the lateral extrusion or water immersion probability of ground soil;

β_0—Modification coefficient varied with the soil quality in different regions;

β_{si}—Enhancement coefficient of skin friction at the i^{th} layer of soil;

β_p—Enhancement coefficient of tip resistance;

γ—Unit weight of soil (Gravity density of soil);

γ_0—Importance factor of structure;

γ_1, γ_2—Transformed unit weight of soil layers within different depth;

γ_w—Unit weight of water (Gravity density of water);

γ_R—Partial factor;

δ_s—Collapsibility coefficient;

δ_{si}—Collapsibility coefficient of the i^{th} layer of soil from foundation base;

δ_{zs}—Collapsibility coefficient of soil self-weight;

δ_{zsi}—Collapsibility coefficient of soil self-weight for the i^{th} layer of soil;

θ—Pressure spreading angle of replacement layer;

λ—Adjusting factor for the characteristic value of ground bearing capacity;

λ_p—Soil plugging coefficient at pile tip;

μ—Coefficient of friction between the foundation base and the ground soil;

ζ_s—Performing coefficient of skin friction for the overlying layer;

φ—Inclination angle of the caisson during floating;

ψ_s—Empirical coefficient for settlement calculation;

ψ_{ze}—Coefficient of frost depth considering the influence of the environment;

ψ_{zf}—Coefficient of frost depth considering the influence of the foundation;

ψ_{zg}—Coefficient of frost depth considering the influence of the topographic aspect;

ψ_{zs}—Coefficient of frost depth considering the influence of the soil type;

ψ_{zw}—Coefficient of frost depth considering the influence of the frost heaving behavior of the soil.

3 General Rules

3.0.1 The bearing capacity and stability of the foundations of the highway bridges and culverts shall be checked, the settlement shall also be checked if necessary.

3.0.2 In checking the ultimate limit state of a foundation, the design safety level and importance factor of the structure shall be determined in accordance with the provisions in the current *General Specifications for Design of Highway Bridges and Culverts* (*JTG D60*).

3.0.3 In the design of a foundation, the requirements for construction and environmental protection shall be considered.

3.0.4 The materials of the foundation structures shall be designed to comply with the provisions of relevant structural design specifications.

3.0.5 The bearing depths of the foundations of highway bridges and culverts shall be determined by the types of foundations, and by the comprehensively consideration of the adverse influences of geology, hydrology, climate, and human activities during the construction and the service periods.

3.0.6 The vertical bearing capacity of the foundation or ground shall be checked in accordance with the following provisions:

1 Frequent combination or quasi-permanent combination of actions shall be considered. In the expression of the action combination, both the factor for frequent value and the factor for quasi-permanent value shall be taken as 1.0, and the impact factor shall be included in the vehicular loading.

2 The characteristic value of bearing capacity multiplied by the corresponding partial factor, γ_R, shall be greater than the corresponding effects of the combination of the actions.

3.0.7 The partial factors of ground bearing capacity, γ_R, may be taken from Table 3.0.7-1. The partial factors of compressive resistance of a single pile, γ_R, may be taken from Table 3.0.7-2.

Table 3.0.7-1 Partial factor of Ground Bearing Capacity, γ_R

Loading stage	Action combinations or ground condition		fa (kPa)	γ_R
In service	Frequent combination	Combination of permanent actions and variable actions	≥150	1.25
			<150	1.00
		Only structuralself-weight, preloading, soil self-weight, soil lateral pressure, vehicle load, and pedestrian load are considered	—	1.00
	Accidental combination		≥150	1.25
			<150	1.00
	Non-rock foundations of old bridge which have been compacted many years without damage		≥150	1.50
			<150	1.25
	Rock foundation of old bridge		—	1.00
During construction	Does not resist thrust from one direction		—	1.25
	Resist thrust from one direction		—	1.50

Note: f_a is the adjusted characteristic value of ground bearing capacity.

Table 3.0.7-2 Partial factor of Compressive Resistance of a Single Pile, γ_R

Loading stage	Action combinations		γ_R
In service	Frequent combination	Combination of permanent and variable actions	1.25
		Only structuralself-weight, preloading, soil self-weight, soil lateral pressure, vehicle load, and pedestrian load are considered	1.00
	Accidental combination		1.25
During construction	Construction load combination		1.25

3.0.8 The effect of the foundation base induced by the quasi-permanent combination of actions under the serviceability limit state shall be considered in the calculation of the foundation settlement. In the calculation, the concrete shrinkage, creep, and foundation displacement are not included in the permanent actions, and the variable actions only refer to the vehicle and pedestrian loads.

3.0.9 The stability of a foundation may be checked according to the following formula:

$$\frac{S_{bk}}{\gamma_0 S_{sk}} \geqslant k \tag{1.0.6}$$

where:

γ_0—importance factor of structure, taken as 1.0;

S_{bk}—effects of combination of actions with characteristic values for stability of foundation, which is calculated by the minimum effects induced by the fundamental combination and the accidental combination. The partial safety factor for action, frequent factor of action, and quasi-permanent factor of action in the expression of the combination of action are all taken as 1.0;

S_{sk}—effects of combination of actions with characteristic value for stability of foundation, which is calculated by the maximum effects induced by the fundamental combination and the accidental combination. The partial factor for action, factor for frequent value of action, and factor for quasi-permanent action in the combination of action are all taken as 1.0;

k—safety factor for the stability of the foundation structure.

4 Geotechnical Classification, Engineering Characteristics, and Bearing Capacity of Grounds

4.1 Geotechnical Classification of Grounds

4.1.1 The rock and soil in the ground of highway bridges and culverts may be classified as rock, gravelly soil, sandy soil, silty soil, cohesive soil, as well as special rock and soil.

4.1.2 The rock mass rating of strength shall be classified according to Table 4.1.2. When the test data are insufficient or unavailable, the rocks may be qualitatively classified according to Table A.0.1-1 in Appendix A.

Table 4.1.2 Rock Mass Rating of Strength

Rock mass rating of strength	Very strong rock	Moderately strong rock	Moderately weakrock	Very weak rock	Extremely weak rock
Nominal value of uniaxial compressive strength for saturated sample, f_{rk} (MPa)	$f_{rk} > 60$	$60 \geqslant f_{rk} > 30$	$30 \geqslant f_{rk} > 15$	$15 \geqslant f_{rk} > 5$	$f_{rk} \leqslant 5$

4.1.3 The weathering degree of rocks may be classified into five levels according to Table A.0.1-2 in Appendix A, namely: unweathered, slightly weathered, moderately weathered, highly weathered, and completely weathered.

4.1.4 According to the softening coefficient, rocks may be grouped into softening rock and unsoftening rock. When the softening coefficient is equal to or less than 0.75, it shall be classified as softening rock; when the softening coefficient is greater than 0.75, it shall be classified as unsoftening rock.

4.1.5 The intactness of the rock mass shall be classified according to Table 4.1.5. When the test data are unavailable, it may be classified according to Table A.0.1-3 in Appendix A.

Table 4.1.5 Classifications of Intactness of Rock Mass

Intactness classification	Very intact	Moderately intact	Moderately fractured	Veryfractured	Extremely fractured
Intact index	>0.75	(0.75 ~ 0.55]	(0.55 ~ 0.35]	(0.35 ~ 0.15]	≤0.15

Note: Intactness index is the square of the ratio of p-wave velocity in rock mass and in rock block.

4.1.6 The development degree of joint for rock mass shall be classified according to Table 4.1.6.

Table 4.1.6 Classifications of the Development Degree of Joint for Rock

Degree	Under-developed Joint	Developed Joint	Well-developed Joint
Spacing ofjoint (mm)	>400	(200 ~ 400]	≤200

4.1.7 When the rocks have a special composition, structure, or property, they shall be classified as the special rocks, such as soluble rocks, swelling rocks, dispersive rocks, or saline rocks.

4.1.8 Gravelly soil refers to the type of soil whose content of particles with a grain size greater than 2 mm exceeds 50% of the total mass. Gravelly soil may be classified according to Table 4.1.8.

Table 4.1.8 Classifications of Gravelly Soil

Soil name	Particle shape	Content of particle group
Boulder	Mainly round and sub-round	The content of particles with a grain size greater than 200 mm exceeds 50% of the total mass
Block stone	Mainly angular	
Cobble	Mainly round and sub-round	The content of particles with a grain size greater than 20 mm exceeds 50% of the total mass
Gravel stone	Mainly angular	
Roundgravel	Mainly round and sub-round	The content of particles with a grain size greater than 2 mm exceeds 50% of the total mass
Breccia	Mainly angular	

Note: The classification of gravelly soil shall be determined according to the content of the particle group from the largest to the smallest.

4.1.9 The density of gravelly soil may be classified according to Table 4.1.9 based on the blow count of the heavy dynamic cone penetration test, $N_{63.5}$. If the test data are unavailable, the density of the gravelly soil with an average grain size greater than 50 mm or with a maximum grain size greater than 100 mm may be determined according to Table A.0.2 in Appendix A.

Table 4.1.9 Density of Gravelly Soil

Blow count $N_{63.5}$	Density	Blow count, $N_{63.5}$	Density
$N_{63.5} \leqslant 5$	Loose	$10 < N_{63.5} \leqslant 20$	Medium dense
$5 < N_{63.5} \leqslant 10$	Slightly dense	$N_{63.5} > 20$	Dense

Notes: 1. This table is applicable to cobbles, gravel, round gravel and breccia, in which the average particle size is less than or equal to 50 mm and whose maximum particle size is less than 100 mm;

2. $N_{63.5}$ is the average value of the blow count number after modification.

4.1.10 The sandy soil refers to the soil whose content of particles with a grain size greater than 2 mm does not exceed 50% of the total mass and the content of particles with grain size greater than 0.075 mm exceed 50% of the total mass. Sandy soil may be classified according to Table 4.1.10.

Table 4.1.10 Classifications of Sandy Soil

Soil name	Content of particle group
Gravelly sand	The content of particles with a grain size greater than 2 mm counts for 25% to 50% of the total mass
Coarse sand	The content of particles with a grain size greater than 0.5 mm exceeds 50% of the total mass
Medium sand	The content of particles with a grain size greater than 0.25 mm exceeds 50% of the total mass
Fine sand	The content of particles with a grain size greater than 0.075 mm exceeds 85% of the total mass
Silty sand	The content of particles with a grain size greater than 0.075 mm exceeds 50% of the total mass

Note: The classification of sand shall be determined according to the content of the particle group from the largest to the smallest.

4.1.11 The density of sand may be classified according to Table 4.1.11 based on the blow count of normal penetration, N.

Table 4.1.11 Density of Sand

Blow count of nominal penetration, N	Density	Blow count of nominal penetration, N	Density
$N \leqslant 10$	Loose	$15 < N \leqslant 30$	Medium dense
$10 < N \leqslant 15$	Slightly dense	$N > 30$	Dense

4.1.12 The silty soil refers to the soil whose plasticity index, I_p, is no larger than 10, and the content of particles with a grain size greater than 0.075 mm does not exceed 50% of the total mass. The density and humidity of silty soil shall be classified according to Table 4.1.12-1 and Table 4.1.12-2, respectively.

Table 4.1.12-1 Density Classifications of Silty Soil

Void ratio, e	Density
$e < 0.75$	Dense
$0.75 \leqslant e \leqslant 0.90$	Medium dense
$e > 0.9$	Slightly dense

Table 4.1.12-2 Humidity Classifications of Silty Soil

Natural water content w (%)	Humidity
$w < 20$	Less wet
$20 \leqslant w \leqslant 30$	Wet
$w > 30$	Very wet

4.1.13 The cohesive soil refers to the soil whose plasticity index, I_p, is greater than 10, and the content of particles with a grain size greater than 0.075 mm does not exceed 50% of the total mass. The cohesive soil shall be classified according to Table 4.1.13 based on the plasticity index.

Table 4.1.13 Classifications of Cohesive Soil

Plasticity index, I_p	Soil name
$I_p > 17$	General cohesive soil
$10 \leqslant I_p \leqslant 17$	Silty cohesive soil

Note: The liquid limit and plastic limit are determined by a 76 g cone test, respectively.

4.1.14 The soft or hard state of cohesive soil may be classified according to Table 4.1.14 based on the liquidity index, I_L.

Table 4.1.14 Classifications for Soft or Hard States of Cohesive Soil

Liquidity index, I_L	Status	Liquidity index, I_L	State
$I_L \leqslant 0$	Extremely stiff	$0.75 < I_L \leqslant 1$	Semi liquid
$0 < I_L \leqslant 0.25$	Stiff	$I_L > 1$	Liquid
$0.25 < I_L \leqslant 0.75$	Plastic	—	—

4.1.15 The cohesive soil may be classified according to Table 4.1.15 based on the deposit age.

Table 4.1.15 Classifications for Deposit Age of Cohesive Soil

Deposit age	Classification of soil
Quaternary Late Pleistocene epoch (Q_3) and earlier	Old cohesive soil
Quaternary Recent epoch (Q_4)	Ordinary cohesive soil
After Quaternary Recent epoch (Q_4)	Newly-deposited cohesive soil

4.1.16 The compressibility of cohesive soil may be classified according to Table 4.1.16 based on the compressive factor, α_{1-2}.

Table 4.1.16 Compressibility Classifications of Cohesive Soil

Compressive factor value, α_{1-2} (MPa^{-1})	Classification of soil
$\alpha_{1-2} < 0.1$	Low compressibility soil
$0.1 \leq \alpha_{1-2} < 0.5$	Medium compressibilitysoil
$\alpha_{1-2} \geq 0.5$	High compressibility soil

4.1.17 Regional foundation soils with some special components, structures, or properties shall be classified as special soils, such as soft soil, swelling soil, collapsible soil, red clay, frozen soil, saline soil, plain fills, etc.

4.1.18 Fine-grained soil with high natural water content, large natural void ratio, and low shear strength, which is located in coastal areas, lakes, valleys, and river flats, and consistent with the provisions of Table 4.1.18, shall be classified as soft soil, such as mud, muddy soil, peat, peaty soil, etc.

Table 4.1.18 Identification Index of Soft Soil Foundation

Indexname	Natural watercontent, w	Natural void ratio, e	Internal friction angle for direct shear, φ	Vane-shear strength, C_u	Compressive factor, α_{1-2}
Index value	≥35 or liquid limit	≥1.0	Should be less than 5°	<35 kPa	Should be greater than 0.5 (MPa^{-1})

4.1.19 The cohesive soil with a natural water content larger than the liquid limit and with a natural void ratio no less than 1.5, which is deposited in the environment of still water or slow-flowing water and formed through biochemical action, shall be classified as mud. The cohesive soil or silty soil, whose natural water content is greater than the liquid limit while its natural void ratio is less than 1.5 but greater than or equal to 1.0, may be classified as muddy soil.

4.1.20 The cohesive soil with remarkable characteristics of water-swellable and losing-water shrinkage, whose clay is mainly composed of hydrophilic minerals and whose free swelling rate is greater than or equal to 40%, shall be classified as swelling soil.

4.1.21 Soil with the collapsibility coefficient greater than or equal to 0.015 that will have additional settlement after submerging into the water shall be classified as collapsible soil.

4.1.22 The high plastic clay with a liquid limit greater than 50 that is formed by lateralization of carbonate rocks, shall be classified as red clay. The red clay that retains its basic characteristics and has a liquid limit greater than 45 after being transported shall be classified as secondary red clay.

4.1.23 The soil with a soluble salt content greater than 0.3%, which has engineering characteristics of karst depression, salt swelling, and corrosion, shall be classified as the saline soil.

4.1.24 The fills may be classified into plain fills, compacted fills, miscellaneous fills, and hydraulically placed fills based on its components and genesis. The plain fills are composed of gravelly soil, sandy soil, silty soil, cohesive soil, etc. The compacted fills refer to the plain fill after compaction by means of mechanical tampers or vibratory compactors. The miscellaneous fills refer to the fills containing construction waste, industrial waste, domestic waste, and other debris. The hydraulically placed fills refer to the soil formed by hydraulic fillings.

4.2 Engineering Characteristics

4.2.1 The engineering characteristics of rock and soil may be indicated by the characteristic properties of compressive strength, shear strength, compressibility, collapsibility, blow count of dynamic cone penetration test, point resistance of static cone penetration test, bearing capacity of loading test, ground bearing capacity, skin friction, tip resistance of pile, etc.

4.2.2 The representative values of engineering characteristic properties of the ground, rock and soil may be indicated by the average value, nominal value, or characteristic value. The nominal value and average value shall be taken for the strength and the compressibility properties of rock and soil, respectively, and the characteristic value shall be taken for the properties of ground bearing capacity.

4.2.3 The shallow-layer plate loading test and deep-layer plate loading test of soil shall comply with the provisions in Appendices B and C. The loading test for the rock foundation shall comply with the provisions in Appendix D.

4.2.4 The compressibility properties of soil, such as compressive modulus, compressive factor, and deformation modulus, may be determined by laboratory compression test, in-situ shallow-layer or deep-layer plate loading test, lateral pressure test, etc.

4.2.5 In determining the engineering characteristic properties of the rock and soil in the highway bridge and culvert foundations, if the methods not clearly stipulated in the *Specifications* are used, they shall comply with the provisions in the current *Code for Highway Engineering Geological*

Investigation (JTG C20), and shall be in accordance with the analytical methods, the conditions of practical engineering loading, foundation drainage, etc.

4.3 Bearing Capacity of Grounds

4.3.1 The checks of the ground bearing capacity of bridges and culverts shall be controlled by the product of the adjusted characteristic value of ground bearing capacity, f_{a}, and the partial factor of ground bearing capacity, γ_{R}. It shall also comply with the following requirements:

1 In determining the adjusted characteristic value of ground bearing capacity, f_a, the characteristic value of ground bearing capacity, f_{a0}, shall be taken as the base value and revised in accordance with Clause 4.3.4 in the *Specifications* based on the bearing depth and width of the foundation as well as the type of ground.

2 The characteristic value of bearing capacity of soft soil may be determined in accordance with Clause 4.3.5 in the *Specifications*;

3 The partial factor of ground bearing capacity, γ_{R}, may be determined in accordance with Clause 3.0.7 in the *Specifications*.

4 The characteristic values of ground bearing capacity and partial factors for other special rocks and soils shall be determined according to the different regional experiences or corresponding standards.

4.3.2 The characteristic value of ground bearing capacity, f_{a0}, should be obtained by the loading tests or other in-situ test methods, and its value shall not be greater than half of the ultimate bearing capacity of the ground. For small or medium bridges and culverts, if it is not possible or extremely difficult to carry out the loading tests or other in-situ tests due to the site conditions, the characteristic value f_{a0} may also be determined by the following relevant provisions of Clause 4.3.3 in the *Specifications*.

4.3.3 If the characteristic values of ground bearing capacity, f_{a0}, are determined based on the type of rocks and soils, conditions, and characteristic indexes of physical-mechanical properties as well as engineering judgement, they may be taken from Table 4.3.3-1 to Table 4.3.3-7.

1 The characteristic value of ground bearing capacity for ordinary rock ground, f_{a0}, may be determined according to Table 4.3.3-1 based on the rock mass rating of strength and joint development degree. For complex rock strata (such as karst caves, faults, soft intercalated

layer, soluble rock, dispersive rock and softening rock, etc.), it may be comprehensively determined by various factors.

Table 4.3.3-1 Characteristic Value of Bearing Capacity of Rock Ground, f_{a0} (kPa)

Rock mass rating of strength	Development degree of joint		
	Under-developed joint	Developed joint	Well-developed joint
Very strong rock, moderately strong rock	>3000	3000 ~ 2000	2000 ~ 1500
Moderately weak rock	3000 ~ 1500	1500 ~ 1000	1000 ~ 800
weak rock	1200 ~ 1000	1000 ~ 800	800 ~ 500
Extremelyweak rock	500 ~ 400	400 ~ 300	300 ~ 200

2 The characteristic value of bearing capacity of gravelly soil ground, f_{a0}, may be determined according to Table 4.3.3-2 based on its type and density.

Table 4.3.3-2 Characteristic Value of Bearing Capacity of Gravelly Soil Ground, f_{a0} (kPa)

Soil name	Density			
	Dense	Medium dense	Slightly dense	Loose
Cobble	1200 ~ 1000	1000 ~ 650	650 ~ 500	500 ~ 300
Gravel	1000 ~ 800	800 ~ 550	550 ~ 400	400 ~ 200
Round gravel	800 ~ 600	600 ~ 400	400 ~ 300	300 ~ 200
Breccia	700 ~ 500	500 ~ 400	400 ~ 300	300 ~ 200

Note: 1. For gravelly soil composed of stiff rock and filled with sand, the higher value is taken; for gravelly soil composed of weak rock and filled with cohesive soil, the lower value is taken;
2. The f_{a0} for semi-cemented gravelly soil shall be increased by 10 ~ 30% based on the value of congeneric dense soil;
3. The loose gravelly soil is very rare in the natural riverbed, which need to take special attention to identify;
4. The f_{a0} for boulder and block stone may be taken by referring it as cobble and gravel, and the value may be appropriately increased.

3 The characteristic value of bearing capacity of the sand ground, f_{a0}, may be determined according to Table 4.3.3-3 based on the soil density and water level.

Table 4.3.3-3 Characteristic Value of Bearing Capacity of Sand Ground, f_{a0} (kPa)

Soil name	Humidity	Density			
		Dense	Medium dense	Slightly dense	Loose
Gravelly sand, coarse sand	Unrelated to humidity	550	430	370	200
Medium sand	Unrelated to humidity	450	370	330	150
Fine sand	Above water	350	270	230	100
	Underwater	300	210	190	—
Silty sand	Above water	300	210	190	—
	Underwater	200	110	90	—

4 The characteristic value of bearing capacity of the silty soil ground, f_{a0}, may be determined according to Table 4.3.3-4 based on the soil natural void ratio, e, and natural water content, w (%).

Table 4.3.3-4 Characteristic Value of Bearing Capacity ofSilty Sand Ground, f_{a0} (kPa)

e	w (%)					
	10	15	20	25	30	35
0.5	400	380	355	—	—	—
0.6	300	290	280	270	—	—
0.7	250	235	225	215	205	—
0.8	200	190	180	170	165	—
0.9	160	150	145	140	130	125

5 The characteristic value of bearing capacity of old cohesive soil ground, f_{a0}, may be determined according to Table 4.3.3-5 based on the compressive modulus E_s.

Table 4.3.3-5 Characteristic Value of Bearing Capacity of Old Cohesive Soil Ground, f_{a0} (kPa)

E_s (MPa)	10	15	20	25	30	35	40
f_{a0} (kPa)	380	430	470	510	550	580	620

Note: For old cohesive soil with E_s < 10 MPa, its characteristic value of bearing capacity, f_{a0}, shall be determined according to Table 4.3.3-6 by taking it as ordinary cohesive soil.

6 Thecharacteristic value of bearing capacity of ordinary cohesive soil ground, f_{a0}, may be determined according to Table 4.3.3-6 based on the liquidity index I_L and natural void ratio e.

Table 4.3.3-6 Characteristic Value of Bearing Capacity of Ordinary Cohesive Soil Ground, f_{a0} (kPa)

e	I_L												
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
0.5	450	440	430	420	400	380	350	310	270	240	220	—	—
0.6	420	410	400	380	360	340	310	280	250	220	200	180	
0.7	400	370	350	330	310	290	270	240	220	190	170	160	150

continued

e	I_L												
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
0.8	380	330	300	280	260	240	230	210	180	160	150	140	130
0.9	320	280	260	240	220	210	190	180	160	140	130	120	100
1.0	250	230	220	210	190	170	160	150	140	120	110	—	—
1.1	—	—	160	150	140	130	120	110	100	90	—	—	—

Note: 1. The f_{a0} value may be increased accordingly if the particles with a grain size greater than 2 mm in the soil exceed 30% of the total mass.

2. When $e < 0.5$, $e = 0.5$ is taken; when $I_L < 0$, $I_L = 0$ is taken. In addition, $f_{a0} = 57.22E_s^{0.57}$ is taken for ordinary cohesive soil with the values of e and I_L exceeding the range listed in the table.

3. If the characteristic value of bearing capacity of ordinary cohesive soil ground, f_{a0}, is taken as greater than 300 kPa, it shall have in-situ test data as its basis.

7 The characteristic value of bearing capacity of newly-deposited cohesive soil ground, f_{a0}, shall be determined according to Table 4.3.3-7 based on the liquidity index I_L and natural void ratio e.

Table 4.3.3-7 Characteristic Value of Bearing Capacity of Newly-deposited Cohesive Soil Ground, f_{a0} (kPa)

e	I_L		
	≤ 0.25	0.75	1.25
≤ 0.8	140	120	100
0.9	130	110	90
1.0	120	100	80
1.1	110	90	—

4.3.4 The adjusted characteristic value of the ground bearing capacity, f_a, may be determined by Eq. (4.3.4). If the underwater foundation is based on the impervious stratum, the value f_a may be increased by 10 kPa per meter according to the water depth measured from the average normal water level to the ordinary scour line.

$$f_a = f_{a0} + k_1\gamma_1(b-2) + k_2\gamma_2(h-3) \tag{4.3.4}$$

where: f_a—adjusted characteristic value of ground bearing capacity (kPa);

b—minimal width at the bottom of a foundation (m). When $b < 2$ m, $b = 2$m is taken; when $b > 10$ m, $b = 10$ m is taken;

h—bearing depth of a foundation base measured from the natural ground (m). When there is scouring, it is measured from the scour line. When $h < 3$ m, $h = 3$ m is taken; when $h/b > 4$, $h = 4b$ is taken;

Table 4.3.4 Adjusting Factors of the Width and Depth of the Foundation Base for the GroundBearing Capacity (k_1, k_2)

Factors	Cohesive soil				Silty soil	Sand								Gravelly soil			
	Old cohesive soil	Ordinary cohesive soil		Newly deposited cohesive soil	—	Silty sand		Fine sand		Medium sand		Sandy gravel, coarse sand		Gravel, round gravel		Cobble	
		$I_L \geqslant 0.5$	$I_L < 0.5$		—	Medium dense	Dense	Medium dense	Dense	Medium dense	Dense	Medium dense	Dense	Medium dense	Dense	Medium dense	Dense
k_1	0	0		0	0	1.0	1.2	1.5	2.0	2.0	3.0	3.0	4.0	3.0	4.0	3.0	4.0
k_2	2.5	1.5		2.5	1.0	2.0	2.5	3.0	4.0	4.0	5.5	5.0	6.0	5.0	6.0	6.0	10.0

Note: 1. For sand and gravelly soil that are slightly dense and loose, their k_1 and k_2 values may be taken as 50% of that in the table.

2. For highly weathered and completely weathered rocks, the factors may be taken with reference to the types of soil that the rocks would be weathered into; no modification is needed for the factors of rocks in other states.

k_1, k_2—adjusting factor of the width and depth of the foundation base, respectively; determined according to Table 4.3.4 based on the type of soil in the bearing stratum of the foundation base;

γ_1—natural unit weight of soil of bearing stratum in the foundation base (kN/m^3). If the bearing stratum is under the water surface and permeable, the buoyant unit weight shall be taken;

γ_2—weighted average unit weight of soil above the foundation base (kN/m^3). In the transforming, if the bearing stratum is under the water surface and impermeable, the γ_2 shall be taken as the saturated unit weight regardless of the water permeability of the soil above the foundation base; in case of water permeable, the buoyant unit weight shall be taken for the part of soil stratum in water.

4.3.5 The characteristic value of bearing capacity of soft ground, f_{a0}, shall be determined by the following provisions:

1 The characteristic value of bearing capacity of soft ground, f_{a0}, shall be obtained by loading test or other in-situ tests. When it is difficult to conduct a loading test or other in-situ test, the adjusted characteristic value of bearing capacity of untreated soft ground for culverts, small or medium bridges, f_a, may be determined by the following two methods:

1) The characteristic value of bearing capacity of soft ground, f_{a0}, is taken according to Table 4.3.5 based on the natural water content of undisturbed soil, w; then the adjusted characteristic value of ground bearing capacity, f_a, is computed according to Eq. (4.3.5-1):

$$f_a = f_{a0} + \gamma_2 h \tag{4.3.5-1}$$

Table 4.3.5 Characteristic Value of Bearing Capacity of Soft Ground, f_{a0} (kPa)

Natural water content w (%)	36	40	45	50	55	65	75
f_{a0} (kPa)	100	90	80	70	60	50	40

2) The adjusted characteristic value of bearing capacity of soft ground, f_a, is determined by the strength properties of the undisturbed soil:

$$f_a = \frac{5.14}{m} k_p C_u + \gamma_2 h \tag{4.3.5-2}$$

$$k_p = \left(1 + 0.2\frac{b}{l}\right)\left(1 - \frac{0.4H}{blC_u}\right) \tag{4.3.5-3}$$

where: m—adjusting factor for resistance, it may be selected between 1.5 ~ 2.5 by considering the sensitivity of the soft soil, the length-to-width ratio of the foundation, etc.;

C_u—nominal value of undrained shear strength of ground soil (kPa);

k_p—coefficient;

H—horizontal force caused by action (characteristic value) (kN);

b—foundation width (m), it is taken as (b-$2e_b$) when there is eccentric action;

l—foundation length perpendicular to b side (m), it is taken as (l-$2e_l$) when there is eccentric action;

e_b, e_l—eccentricity in transverse and longitudinal directions, respectively.

2 The characteristic value of bearing capacity of soft ground treated bydrained consolidation measures, f_{a0}, shall be determined by loading test or other in-situ tests; the characteristic value of bearing capacity of soft ground treated by composite foundation measures, f_{a0}, shall be determined by loading test. Then, the adjusted characteristic of bearing capacity of soft ground, f_a, is calculated according to Eq. (4.3.5-1).

5

5.1 Bearing Depth

5.1.1 The bearing depth of the foundation base of the highway bridge/culvert shall comply with the following provisions:

1 The safety values of the bearing depth of the foundation base for the pier and abutment foundations of bridges on the non-rocky riverbed should not be less than the values listed in Table 5.1.1.

Table 5.1.1 Safety Value ofBearing Depth of Foundation Base (m)

Bridge type	Total scour depth				
	0	5	10	15	20
Large, medium and small bridges (without riverbed pavement)	1.5	2.0	2.5	3.0	3.5
Super-large bridge	2.0	2.5	3.0	3.5	4.0

Note: 1. The total scour depth is the sum of the depths of natural scour, general scour, and local scour that is measured from the riverbed surface.

2. If accurate data on the degradation of the riverbed are not available or the accurate data of the design flow, water level, and original cross-section are unsure, the values in this table should be increased accordingly;

3. If there are some existing bridges in upstream and/or downstream of the bridge to be built, it is necessary to investigate the super-flood scour of the existing bridges. The bearing depth of the foundations of the piers or abutments of the bridge to be built should not be less than the scour depth of the existing bridges, and it is suggested to increase a safety value if necessary.

4. If there is apavement layer on the riverbed, the foundation base should be set deeper than 1 m under the top surface of the pavement layer.

2 The minimumbearing depth for the foundation bases of the pier and abutment on the rock

riverbed may be determined with reference to the provisions in the current *Hydrological specifications for Survey and Design of Highway Engineering* (JTG C30).

3 For an abutment locatedin the river channel, the elevation of its foundation base shall be the same as that of the pier foundation base even if its maximum scour depth is less than that of the pier. For the abutment located in the river shoal, the elevation of its foundation base shall be the same as that of the pier foundation base if the river is an unstable one; and the elevation may be determined in accordance with the calculated scour depth of the abutment if the river is a stable one.

4 For a culvert foundation, thebearing depth shall be no less than 1 m below ground or riverbed if the foundation (excluding rock ground) is located in the place without scour. If the foundation is located in the place with scour, the bearing depth of the foundation base shall be set no less than 1 m below the local scour line. If there is a pavement layer on the riverbed, the foundation base should be set no less than 1 m below the top surface of the pavement layer.

5.1.2 If the ground is a frost heaving soil layer, the bearing depth of the pier/abutment foundation base of the bridge/culvert shall comply with the following provisions:

1 If the superstructure is statically indeterminate, the foundation base shall be located no less than 0.25 m below the depth of frost potential.

2 If the foundation is allowed to be set in theseasonal frost heaving soil layer, the minimum bearing depth may be calculated by the following equations:

$$d_{\min} = z_d - h_{\max} \tag{5.1.2-1}$$

$$z_d = \Psi_{zs} \Psi_{zw} \Psi_{ze} \Psi_{zg} \Psi_{zf} z_0 \tag{5.1.2-2}$$

where:

$d_{\min}$—minimal bearing depth of the foundation base (m);

z_d—design frost depth (m);

z_0—nominal frost depth (m). When the data is unavailable, it may be adopted according to Appendix E;

Ψ_{zs}—influence coefficient of frost depth by soil type, which is taken from Table 5.1.2-1;

Ψ_{zw}—influence coefficient of frost depth by frost heaving behaviors of soil, which is taken from Table 5.1.2-2. The classification of seasonal frost heaving soil can be found in E.0.2;

Ψ_{ze}—influence coefficient of frost depth by environment, which is taken from Table 5.1.2-3;

ψ_{zg}—influence coefficient of frost depth by topographic aspect, which is taken from Table 5.1.2-4;

ψ_{zf}—influence coefficient of frost depth by foundation, ψ_{zf} = 1.1 is taken;

h_{max}—allowable maximum depth of frozen layer below the foundation base (m), which is taken from Table 5.1.2-5. The classification of seasonal frost heaving soil is given in E.0.2 in the *Specifications*.

Table 5.1.2-1 Influence Coefficient of Frost Depth by Soil Type, Ψ_{zs}

Soil type	Ψ_{zs}	Soil type	Ψ_{zs}
Cohesive soil	1.00	Medium sand, coarse sand and gravelly sand	1.30
Fine sand, silty sand, silty soil	1.20	gravelly soil	1.40

Table 5.1.2-2 Influence Coefficient of Frost Depth by Frost Heaving Behavior of Soil, Ψ_{zw}

Type of frost heaving behavior of soil	Ψ_{zw}	Type of frost heaving behavior of soil	Ψ_{zw}
Non-frost heave	1.00	Strong frost heave	0.85
Weak frost heave	0.95	Extra strong frost heave	0.80
frost heave	0.90	—	—

Table 5.1.2-3 Influence Coefficient of Frost Depth by Environment, Ψ_{ze}

Ambient environment	Ψ_{ze}	Ambient environment	Ψ_{ze}
Village, town, field	1.00	Urban area	0.90
Inner suburb	0.95	—	—

Note: If the urban area has a population of 0.2 ~ 0.5 million, the influence coefficient shall be taken as that for the inner suburb; if the urban area has a population greater than 0.5 million but less than or equal to 1 million, the influence coefficient shall be taken as that of the urban area; if the urban area has a population more than 1 million, the influence coefficient is taken as that for the urban area, and the influence coefficient of the area inner 5 km from the urban area is taken as that of the inner suburb.

Table 5.1.2-4 Influence Coefficient of Frost Depth by Topographic Aspect, ψ_{zg}

Topographic aspect	Flat	Adret	Ubac
ψ_{zg}	1.0	0.9	1.1

Table 5.1.2-5 Allowable Maximum Depth of Frozen Layer below the Foundation Base, h_{max}

Type of frost heaving behavior of soil	Weak frost heave	frost heave	Strong frost heave	Extra strong frost heave
h_{max}	0.38 z_0	0.28 z_0	0.15 z_0	0.08 z_0

3 If a culvert foundation is set in the ground with seasonally frozen soil, the following requirements shall be satisfied:

1) The bearing depth of the culvert foundation base in its outlet/inlet and its body within the range of 2 m ~ 6 m (or a segment of the culvert with a length no less than 2 m) from the outlet/inlet may be determined by the Eq. (5.1.2-1).

2) The bearing depth of the foundation at the middle part of the culvert may be determined based on regional experience.

3) In extremely cold regions, if the bearing depth at the middle part of the culvert differs greatly from that at the outlet/inlet, a transition segment shall be set between them.

4) In areas with deep of frost potential, the foundation soil from the foundation base to the depth of frost potential may be replaced by coarse-grained soil (including gravelly soil, gravelly sand, coarse sand, and medium sand, but the silty clay content shall not exceed 15%, or the particles with grain size smaller than 0.1 mm shall not exceed 25%).

4 If the pier/abutment foundation base is set in the non-frost heaving soil layer, the bearing depth of the foundation base in design may be unrestricted by the depth of frost potential.

5.1.3 The elevation of the top surface of the pier or abutment foundation should be determined comprehensively according to the conditions of the bridge site, the difficulty of the construction, aesthetics as well as the holistic of the entire structure.

5.2 Check for Ground Bearing Capacity and Eccentricity of Foundation Base

5.2.1 In checking the ground bearing capacity of piers and abutments, various actions that may occur during construction and in service shall be considered, and the following requirements shall be met:

1 If the height of the abutment backfill is larger than 5 m, the superimposed vertical compressive stresses of the abutment foundation base induced by the abutment backfill shall be considered, and the stresses may be analyzed according to Appendix F in the *Specifications*.

2 For soft soil or soft ground, if the distance between the adjacent abutments and/or piers is less than 5 m, the superimposed vertical compressive stresses of soft soil or soft ground induced by the adjacent piers or abutments shall be considered.

3 For the abutment foundation with adverse ground soil behind the abutment, the stability against sliding of abutment and embankment shall be checked concomitantly.

5.2.2 If the socket effect is neglected, the bearing capacity of rock and soil at the bottom of the foundation may be checked by the following formulas:

1 For a foundation base subjected to only axial load:

$$p = \frac{N}{A} \leqslant f_a \tag{5.2.2-1}$$

where:

p—average compressive stress of foundation base (kPa);

N—vertical force at foundation base induced by the action combination specified in Clause 3.0.6 of the *Specifications* (kN);

A—area of foundation base (m^2).

2 For a foundation base subjected to uniaxial eccentric compression, besides the provisions of Item 1 in this Clause, the following condition shall also be satisfied:

$$p_{\max} = \frac{N}{A} + \frac{M}{W} \leqslant \gamma_R f_a \tag{5.2.2-2}$$

where:

$p_{\max}$—maximum compressive stress of foundation base (kPa);

M—bending moment with respect to the gravity axis of the foundation base, induced by the horizontal and vertical forces on the pier or abutment under the action combination specified in Clause 3.0.6 of the *Specifications* (kN.m);

W—section modulus of the foundation base in the eccentric direction (m^3).

3 For a foundation base subjected to biaxial eccentric compression, besides the provisions of Item 1 in this Clause, the following condition shall also be satisfied:

$$p_{\max} = \frac{N}{A} + \frac{M_x}{W_x} + \frac{M_y}{W_y} \leqslant \gamma_R f_a \tag{5.2.2-3}$$

where:

M_x, M_y—bending moment on the base about X- and the Y-axis (kN.m), induced by the horizontal and vertical forces acting on the abutment or the pier, respectively;

W_x, W_y—section modulus for the extreme fiber of foundation base with respect to the eccentric direction of *X*- and *Y*-axis (m^3), respectively.

5.2.3 For a pier or abutment foundation on a bedrock subjected to uniaxial eccentric load with an eccentricity e_0 larger than the core radius of the cross-section, ρ (as shown in Eq. (5.2.5-2)), only the compression resistance of the foundation base (no tension resistance, as illustrated in Fig. 5.2.3) should be taken into account in calculating its maximum compressive stress. The maximum compressive stress of the foundation base with rectangular cross-section, $p_{\max}$, may be calculated by the following formula:

$$p_{\max}=\frac{2N}{3\left(\frac{b}{2}-e_0\right)a}\leqslant\gamma_R f_a \tag{5.2.3}$$

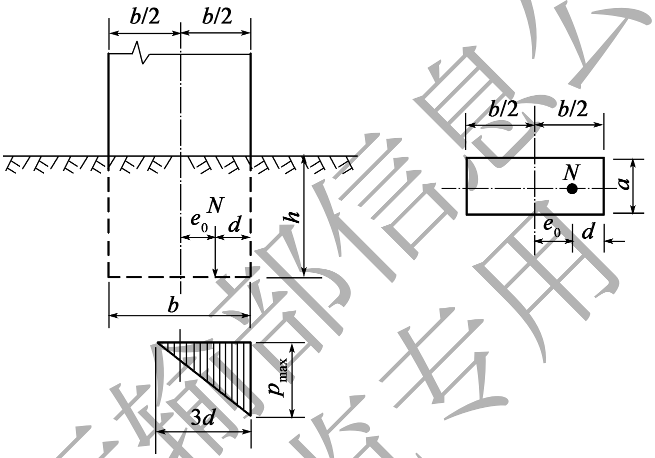
where:

b—side length of the foundation base in the eccentric direction (m);

a—side length of the foundation base perpendicular to the side with a length of b (m);

e_0—distance from acting point of eccentric load N to the centroid of the section (m);

N—uniaxial eccentric load on the foundation of the pier or abutment (kN).



a)Foundation elevation　　b)Foundation plan

Fig. 5.2.3　Diagram of Stress Re-distribution of Foundation Base on the Bedrock with a Rectangular Cross-section under Uniaxial Eccentric Compression

5.2.4　For a pier or abutment foundation on a bedrock subjected to biaxial eccentric load with an eccentricity e_0 larger than the core radius of the cross-section, ρ (as shown in Eq. (5.2.5-2)), only the compression resistance of the foundation base (no tension resistance) should be taken into account in calculating its compressive stress. The maximum compressive stress of the foundation base with rectangular or circular cross-section, p_{max}, may be determined in accordance with the provisions in the Appendix G of the *Specifications*.

5.2.5　The resultant eccentricity on the foundation base of the pier or abutment shall be checked, and the following provisions shall be satisfied:

1　The allowable value of the resultant eccentricity of the foundation base of the pier or abutment, $[e_0]$, shall conform to the provisions in Table 5.2.5.

Table 5.2.5 Allowable Value of the Resultant Eccentricity of Foundation Base of Pier or abutmen, $[e_0]$

Action cases	Type of grounds	$[e_0]$	Remarks
Combination of only the permanent actions with characteristic values	Non-rock ground	Pier, 0.1ρ	For arch bridges and rigid frame bridges, the design shall be carried out in such a way that the resultant force acts as close as possible to the centroid of the foundation base of the piers.
		Abutment, 0.75ρ	
Combination of permanent actions with characteristic values, or combination of accidental actions with characteristic values	Non-rock ground	ρ	For robust piers (single direction thrust piers) in arch bridges, the eccentricity is not limited, but the safety factor of stability against overturning failure shall conform to the provisions specified in Table 5.4.3 of the Specifications
	Moderate fractured to extremely fractured rock ground	1.2ρ	
	Intact and moderately intact rock ground	1.5ρ	

2 The eccentricity of the acting point of resultant external force with respect to the gravity axis of the foundation base, e_0, may be calculated according to Eq. (5.2.5-1):

$$e_0 = \frac{M}{N} \leqslant [e_0] \tag{5.2.5-1}$$

where:

M—bending moment about the centroid of the foundation base induced by all external forces (vertical forces and horizontal forces) (kN. m);

N—vertical acting on the foundation base (kN).

3 The cross-sectional core radius of the foundation base subjected to uniaxial or biaxial eccentric compression, ρ, may be calculated by the following equations:

$$\rho = \frac{e_0}{1 - \frac{p_{\min} A}{N}} \tag{5.2.5-2}$$

$$p_{\min} = \frac{N}{A} - \frac{M_x}{W_x} - \frac{M_y}{W_y} \tag{5.2.5-3}$$

where:

$p_{\min}$—the minimal compressive stress on the foundation base, which is tensile stress when the value is negative (kPa).

5.2.6 In design of a foundation with soft ground or soft soil layer underlying the foundation base, the bearing capacity of the soft ground or soft soil layer shall be checked by the following formula:

$$p_z = \gamma_1 (h + z) + \alpha (p - \gamma_2 h) \leqslant \gamma_R f_a \tag{5.2.6}$$

where:

p_z—compressive stress on the soft ground or soft soil layer (kPa);

h—bearing depth of the foundation base (m). It is measured from the general scour line if the foundation is scoured; it is measured from the natural ground level if there is no scour; it is measured from the surface of excavation if the foundation is located on an excavation area;

z—distance from the foundation base to the top surface of the soft soil layer or of the soft ground (m);

γ_1—transformed unit weight for each soil layer within the depth of $h+z$(kN/m^3);

γ_2—transformed unit weight for each soil layer within the depth of h(kN/m^3);

α—coefficient of superimposed compressive stress in the soil, see Clause J. 0. 1 in the *Specifications*;

p—compressive stress of the foundation base (kPa). If $z/b>1$ (where b is the width of the rectangular foundation base), p is taken as the average compressive stress of the foundation base; if $z/b \leqslant 1$, p is taken as the compressive stress at the position with a distance of $b/3 \sim b/4$ to the maximum compressive stress from the graph of the compressive stress of foundation base (the compressive stress at the $b/4$ position is adopted if the compressive stresses in the front and back edges of trapezoid graph are significantly different; otherwise, the compressive stress at the $b/3$ position is adopted);

f_a—characteristic value of bearing resistance of soil on the top surface of the soft soil layer or of the soft ground (kPa), which is taken according to the provisions of Clause 4.3.4 or A4.3.5.

5.3 Check for Settlement

5.3.1 The settlement shall be calculated in case the abutments or piers are built on grounds with complex geological conditions, non-uniform soil texture, or of poor bearing capacity, in case the abutments or piers are built on a ground with an underlying layer of thick soft clay that has great compressibility, in case the differential settlement is needed to be calculated due to the lengths of the adjacent spans have obvious difference, or in case the settlement is needed to be considered in advance for the overpass bridge to ensure its vertical clearance.

5.3.2 In settlement calculation, the action effect transmitted to the foundation base shall be computed in accordance with the provisions in Clause 3.0.8 of the *Specifications*.

5.3.3 The settlement requirements of abutments or piers shall be in accordance with the following provisions:

1 The additional longitudinal gradient (to form a bevel) on the bridge deck induced by

settlement difference between adjacent abutments and/or piers (excluding the settlement during construction) shall not larger than 2‰.

2 For an external statically indeterminate structure, the settlement difference between adjacent abutments and/or piers shall not cause the structure stresses not meet the requirements.

5.3.4 The final settlement of an abutments/pier foundation may be calculated by the following equations:

$$s = \psi_s S_0 = \psi_s \sum_{i=1}^{n} \frac{p_0}{E_{si}} (z_i \bar{\alpha}_i - z_{i-1} \bar{\alpha}_{i-1}) \tag{5.3.4-1}$$

$$p_0 = p - \gamma h \tag{5.3.4-2}$$

where:

S—final settlement of the foundation (mm);

S_0—settlement value of the foundation calculated by layerwise summation method (mm);

ψ_s—empirical coefficient for the settlement calculations. It may be determined by the data of regional settlement observation or by the empirical judgment of engineering. If the data of settlement observation or empirical judgment of engineering is unavailable, it may be determined according to the Clause 5.3.5 of the *Specifications*.

n—number of the divided soil layers within the investigated depth in the calculation of the foundation settlement (Fig. 5.3.4);

P_0— superimposed compressive stress on the foundation base subjected quasi-permanent combinations of actions (kPa);

E_{si}—compressive modulus of the i^{th} soil layer under the foundation base (MPa). It shall be calculated by taking the compressive stress segment of soil from 'the soil dead weight compressive stress' to 'the summation of soil dead weight compressive stress and superimposed compressive stress';

Z_i, Z_{i-1}—distance from the foundation base to the i^{th} soil layer base and to the $(i\text{-}1)^{th}$ soil layer base (m), respectively;

$\bar{\alpha}_i$, α_{i-1}—coefficient for the average of superimposed compressive stress within the range from the measured point of the foundation base to the i^{th} soil layer base and to the $(i-1)^{th}$ soil layer base, respectively, which may be taken by Clause J.0.2 of the *Specifications*;

p—compressive stress of foundation base (kPa). If $z/b > 1$ (where b is the width of rectangular foundation base), p is taken as the average compressive stress of foundation base; if $z/b \leqq 1$, p is taken as the compressive stress at the position with a distance of $b/3 \sim b/4$ to the maximum compressive stress from the diagram of the compressive stress of foundation base (if the difference of the compressive stress in the front and back edges of trapezoid graph is relatively large, the compressive stress at the $b/4$ position is adopted; otherwise, the compressive stress at the $b/3$ position is adopted);

h—bearing depth of foundation base (m). It is measured from the general scour line if there is scoured in the pier or abutment; it is measured from the natural ground level if there is no scoured; it is measured from the excavation base if the foundation is located in an excavation area;

γ—unit weight of soil within the depth h (kN/m^3). When the foundation is based on water-permeable ground, buoyant unit weight shall be taken for the soil under the water level.

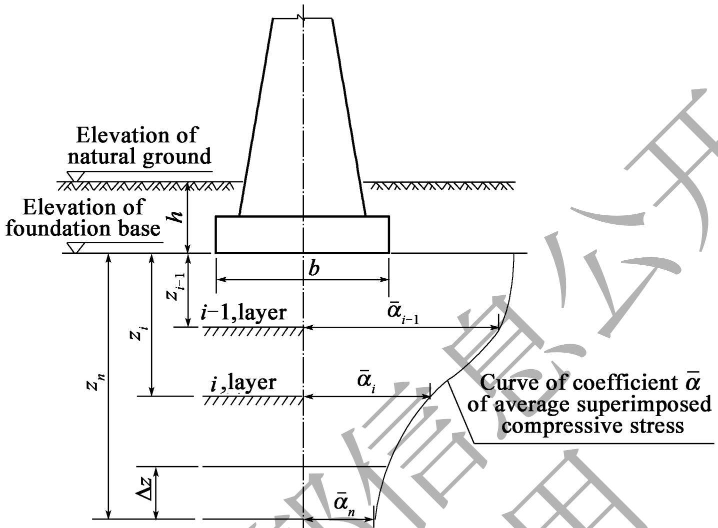


Fig. 5.3.4 Layering Diagram for Calculation of Foundation Settlement

5.3.5 Empirical coefficient for settlement calculations, ψ_s, may be determined according to Table 5.3.5. The equivalent value for compressive modulus within the stratum range in the settlement calculation, $\bar{E}_s$, may be calculated by the following formula:

$$\bar{E}_s = \frac{\sum A_i}{\sum \frac{A_i}{E_{si}}} \tag{5.3.5}$$

where:

A_i—the integral value along the soil layer depth for the superimposed compressive stress on the i^{th} layer of soil.

Table 5.3.5 Empirical Coefficient for Settlement Calculatio n, ψ_s

Superimposed compressive stress of foundation base	$\bar{E}_s$ (MPa)				
	2.5	4.0	7.0	15.0	20.0
$p_0 \geqslant f_{a0}$	1.4	1.3	1.0	0.4	0.2
$p_0 \leqslant 0.75 f_{a0}$	1.1	1.0	0.7	0.4	0.2

5.3.6 In the calculation of ground settlement, the investigated depth, z_n, shall be taken in accordance with the requirement of Eq. (5.3.6). If there is still soft soil layer below the investigated depth, the assumed depth shall be increased to include the soft soil layer.

$$\Delta s_n \leqslant 0.0025 \sum_{i=1}^{n} \Delta s_i \tag{5.3.6}$$

where:

Δs_n—the computed settlement of the soil layer with a thickness of ΔZ upwards from the bottom of the investigated depth, z_n, where ΔZ is referred to Fig. 5.3.4 and its value is taken according to Table 5.3.6;

Δs_i—the computed settlement of the i^{th} soil layer within the range of the investigated depth (mm).

Table 5.3.6 Value of ΔZ

Foundation base width, b (m)	$b \leqslant 2$	$2 < b \leqslant 4$	$4 < b \leqslant 8$	$b > 8$
ΔZ(m)	0.3	0.6	0.8	1.0

5.3.7 For a foundation with a base width in the range of 1 m ~ 30 m and without influence on its settlement from adjacent loads, its investigated depth for the ground settlement at the center of the foundation base, z_n, may also be calculated by the following simplified equation:

$$z_n = b(2.5 - 0.4\ln b) \tag{5.3.7}$$

where:

b—foundation width (m);

Z_n—the investigated depth for the ground settlement at the center of the foundation base (m). If there exists a bedrock within the investigated depth, the z_n may be measured from the foundation base to the surface of the bedrock; if there is a stiff clay layer with relatively large depth, which has a void ratio less than 0.5 and compressive modulus greater than 50 MPa, or there is a dense sandy cobble layer with relatively large depth, which has compressive modulus greater than 80 MPa, the z_n may be measured from the foundation base to the surface of the soil layer mentioned.

5.4 Check for Stability

5.4.1 The stability against overturning of pier or abutment foundations shall be calculated by the following equations (Fig. 5.4.1):

$$k_0 = \frac{s}{e_0} \tag{5.4.1-1}$$

$$e_0 = \frac{\sum P_i e_i + \sum H_i h_i}{\sum P_i} \tag{5.4.1-2}$$

where:

k_0—factor of safety against overturning of abutment or pier foundations;

s—distance from the section centroid to the calculated overturning axis, which is on the extension line measured from the section centroid to the acting point of the resultant force (m);

e_0—eccentricity of the resultant force of all external forces, R, with respect to the foundation base gravity axis of the acting point of the checked cross-section (m);

P_i— vertical force caused by fundamental or accidental combinations of actions with characteristic values, in which the partial safety factors and combination factors are not considered (kN);

e_i—lever arm of vertical force P_i to the centroid of the checked section (m);

H_i—horizontal force caused by the fundamental combination of actions with characteristic values or by the accidental combination of actions with characteristic values (kN), in which the partial safety factor and combination factor are not considered;

h_i—lever arm of horizontal force to the checked section (m).

Note: 1. Positive or negative signs shall be taken for the bending moment according to the revoluting directions about the gravity axis of its checked section.

2. For the rectangular foundation with concave gap, the enveloping side of the base section shall be taken as its overturning axis.

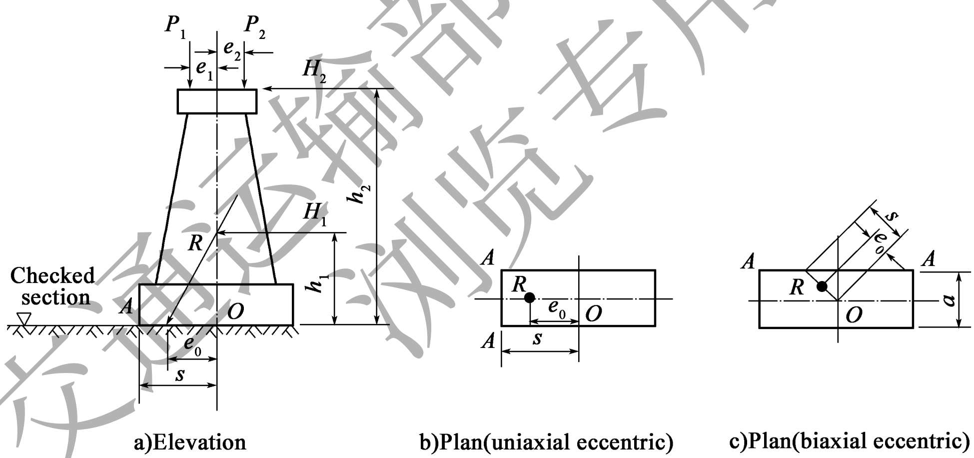


Fig. 5.4.1 Schematic Diagram for Checking the Stability of Pier and Abutment Foundation

O-Centroid of the section, R-Acting point of the resultant force: A - A - Overturning axis in checking

5.4.2 The factor of safety against sliding of pier or abutment foundations, k_c, shall be calculated by the following equation:

$$k_c = \frac{\mu \sum P_i + \sum H_{ip}}{\sum H_{ia}} \tag{5.4.2}$$

where:

k_c—factor of safety against sliding of pier or abutments foundations;

$\sum P_{\mathrm{i}}$—sum of vertical forces;

$\sum H_{\mathrm{ip}}$—sum of horizontal forces in resisting sliding;

$\sum H_{ia}$—sum of horizontal forces causing sliding;

μ—coefficient of friction between the foundation base and the ground soil. It shall be determined by test. If the test data are unavailable, it may be taken according to Table 5.4.2.

Note: $\sum H_{\mathrm{ip}}$ and $\sum H_{\mathrm{ia}}$ are the sum of horizontal forces in two opposite directions, respectively. The one with a larger absolute value is the sum of sliding horizontal force $\sum H_{\mathrm{ia}}$, and the other one is the sum of stabilizing force against sliding $\sum H_{\mathrm{ip}}$. The $\mu \sum P_i$ is the stabilizing force in resisting sliding.

Table 5.4.2 Friction Coefficient in the Foundation Base

Classification of foundation soil	μ
Cohesive soil (liquid ~ stiff), silty soil	0.25 ~ 0.35
Sandy soil (silty sand ~ gravelly sand)	0.30 ~ 0.40
gravelly soil (loose ~ dense)	0.40 ~ 0.50
Weak rock (extremely weak rock ~ moderately weak rock)	0.40 ~ 0.60
Stiff rock (moderately strong rock, very strong rock)	0.60 ~ 0.70

5.4.3 In checking the stability of piers and abutments against overturning or sliding, the safety factors for stability shall not be less than the values specified in Table 5.4.3.

Table 5.4.3 Minimum Safety Factors for Stability against Overturning or Sliding

Action combinations		Checked items	Minimum safety factors for stability
Service stage	Combination of permanent loads (excluding concrete shrinkage and creep, buoyancy force) and vehicle, pedestrian load, expressed bycharacteristic values	Overturning	1.5
		Sliding	1.3
	Combination of various actions withcharacteristic values	Overturning	1.3
		Sliding	1.2
Combination of actions in construction withcharacteristic values		Overturning	1.2
		Sliding	

5.4.4 If the foundation is located in seasonally frozen soil or permafrost soil layer, the stability against frozen uplift shall be checked. The calculation method may refer to the Appendix H in this *Specifications*.

6 Pile Foundations

6.1 General

6.1.1 The pile foundation shall be designed according to not only the provisions of relevant specifications but also the following provisions:

1 Checking the compression, uplift, and horizontal resistance of the pile groups or single piles based on the functional use and load characteristics of the pile foundations.

2 Checking the overall stability under the most adverse combination of actions for the pile foundations located on the slope or shoreside.

6.1.2 Piles may be classified according to the following provisions:

1 Classified by load-bearing properties:

1) Friction pile: the load on the pile head is mainly taken by the skin friction (side resistance along its surface), and the tip resistance of the piles is also considered.

2) End-bearing pile: the load on the pile head is mainly taken by the tip resistance at the pile tip, and the skin friction of the piles is also considered.

2 Classified by piling methods:

1) Non-soil-compaction piles, including cast-in-place piles drilled (bored) by dry-operation method, cast-in-place under-reamed piles, cast-in-place drilled piles using slurry to protect the wall, and cast-in-place drilled piles using a sleeve to protect the wall, etc.;

2) Partially-soil-compaction piles, including pre-drilled driven piles, prestress concrete pipe piles with open-end, steel pipe piles with open-end, and cast-in-place drilled root piles, etc.;

3) Soil-compaction piles, also called driven piles, including precast piles, prestressed concrete pipe piles with close-end, steel pipe piles with close-end driven by hammering, static loading or vibration, etc.

6.1.3 The elevation of the pile cap bottom shall comply with the following requirements:

1 In the regions of seasonal frost heaving soil, if the pile cap bottom is located in soil, its bearing depth shall be in accordance with the relevant provisions of Clause 5.1.2;

2 In a river with drift ice, the elevation of the pile cap bottom shall be no less than 0.25 m lower than the lowest bottom of the ice layer;

3 If a pile foundation is prone to be collided by drift rafts, other drifting objects, or ships, the elevation of its pile cap bottom shall be determined to ensure the piles will not be collided directly.

6.1.4 For piles in the regions of seasonally frozen soil, if it is necessary to set the lateral bracings between piles, the location of the bracings shall keep away from the frost heaving layer.

6.1.5 In a same pile group foundation, friction piles and end-bearing piles should not be used together; piles with different diameters, different materials, or with too large different elevations of pile tips should also not be used together too.

6.1.6 For piles used in large and super-large bridges with the following circumstances, the compressive resistance of a single pile shall be determined through the static loading test.

1 The bearing depth of a pile is far deeper than that of common piles.

2 It is difficult to determine the compressive resistance of the pile due to the complex geological conditions.

3 The foundation is a new-type pile foundation or a pile foundation constructed by new technology.

4 The pile foundation has other special requirements.

6.2 Detailing Requirements

6.2.1 The dimensions of the concrete piles should meet the following detailing requirements:

1 The diameters of the drilled piles should be designed no less than 0.8 m;

2 The diameters or minimum side width of the bored piles should not be less than 1.2 m;

3 The diameters of the concrete pipe piles may be 0.4 m to 1.2 m, their minimum wall thicknesses should not be less than 80 mm.

6.2.2 Concrete piles shall meet the following detailing requirements:

1 Concrete strength class for the pile bodies shall not be less than C25. When reinforcing steel with a nominal strength value of 400 MPa or above is used, the concrete strength class shall not be less than C30. The filled concrete for pipe piles shall not be less than C20.

2 Reinforced concrete-driven piles shall be reinforced by the reinforcing steel along its full length according to the stress requirements at various stages, including transportation, driving into soil, and in service. The stirrups or spiral reinforcing bars at both ends of the pile and the pile splicing zone shall be densified with reduced spacing, the spacing may be taken as 40 mm ~ 50 mm.

3 The reinforcing bars in a drilled (bored) pile may be arranged by segments according to the internal forces along the pile body. Even if no reinforcing bar is needed from the analysis results of the internal forces, the piles shall still be reinforced according to the detailing requirements in the pile top with a length of 3.0 m ~ 5.0 m. Piles shall be reinforced according to the following requirements:

1) The diameter of the primary reinforcement in piles shall not be less than 16 mm; the number of main reinforcing bars in each pile shall not be less than 8; the clear distance between the main bars shall not be less than 80 mm, or shall not be greater than 350 mm.

2) If the number of the reinforcing bars is large, reinforcing bars may be used in bundles. The diameter of an individual bar in the bundles shall not be larger than 36 mm. The number of the individual bars in a bundle shall not be more than 3 if the diameter of the individual bar

is no larger than 28 mm, and the number shall be 2 if the diameter is larger than 28 mm.

3) The cover thickness of the reinforcing bars shall meet the requirements in the *Specification for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTG 3362).

4) The diameter of the closed stirrup or spiral reinforcement shall not be less than 8 mm and 1/4 times the diameter of the primary reinforcement. Their central distance shall not be 15 times larger than the diameter of the primary reinforcement and shall not be larger than 300 mm.

5) One stiffening stirrup with a diameter of 16 mm ~ 32 mm shall be arranged in the skeleton of the reinforcement cage with an interval of 2.0 m ~ 2.5 m.

6) Positioning concrete blocks shall be set around the reinforcement cages, or other positioning measures shall be used.

7) The primary reinforcement at the bottom of the reinforcement cages should be slightly bent inward.

4 The segment length of precast reinforced concrete piles shall be determined according to construction conditions, and the number of connectors shall be minimized. The strength of the connectors shall not be less than that of the pile body. The connector flange shall not project outside the pile body, and the connectors shall not be loose or cracked in service or during the driving process.

5 For the prestressed concrete pipe piles with open-end whose pile tip is socketed into the highly weathered rock in non-saturation, effective measures shall be taken to prevent the bearing stratum at the pile tip from softening induced by water seepage.

6.2.3 Steel pipe piles shall be designed in accordance with the following detailing requirements:

1 The welded connectors for steel pipe piles shall adopt the butt welding with equal strength.

2 The end forms of the steel pipe piles shall be determined comprehensively according to the characteristics of the soil layers the pile penetrated, the properties of bearing stratum at the pile tip, pile size, soil squeezed effect, and other factors. The end forms of steel pipe piles may be in one of the following forms:

1) Open end with stiffeners (with or without internal diaphragms), open end without stiffeners

(with or without internal diaphragms);

2) Closed end with flat tip or cone tip.

3 The diameter and wall thickness of the steel pipe piles should be designed in accordance with the following requirements:

1) The ratio of diameter-to-wall thickness should not be greater than 100.

2) The minimum wall thickness required for resisting hammering may be determined by engineering judgment or the following formula:

$$t = 6.35 + \frac{d}{100} \tag{6.2.3}$$

where:

t—wall thickness of steel pipe pile (mm);

d—diameter of steel pipe pile (mm).

6.2.4 The steel tube structure of concrete-filled steel tube composite piles shall comply with the requirements of Clause 6.2.3 of the *Specifications*, and its ratio of diameter-to-wall thickness, d/t, may be calculated according to Eq. (6.2.4):

$$\frac{d}{t} = (20 \sim 135)\frac{235}{f_d} \tag{6.2.4}$$

where:

f_d—value of design strength of steel (MPa).

6.2.5 The following anti-corrosion treatments may be applied for steel piles or concrete-filled steel tubular composite piles according to the environmental conditions:

1 Covering with anti-corrosion coating or other covering layers in the outer walls.

2 Increasing the reserved thickness of the corrosion margin for the pipe wall. In marine environment, the average annual corrosion rate for one surface of a steel pile may be taken from Table 6.2.5, and it may also be determined by in-situ measurement if the measurement is available. Under other environment conditions, the average annual corrosion rate may be taken as 0.06 mm/year for pile bodies above the average low water level, and 0.03 mm/year for pile bodies below the average low water level.

Table 6.2.5 Average Annual Corrosion Rate in One Surface of a Steel Pile in the Marine Environment

Location	(mm/Year)	Location	(mm/Year)
Atmospheric zone	0.05 ~ 0.10	Water level changing zone, underwater zone	0.12 ~ 0.20
Splash zone	0.20 ~ 0.50	Under-mud zone	0.05

Note: 1. The average annual corrosion rate listed in the table is applicable to environmental conditions with pH value of 4 ~ 10. The rate shall be increased accordingly in a severely polluted environment.

2. The average annual corrosion rate in the corresponding parts shall be increased appropriately in case the structure is located in an estuary where salinity in the water is quite different, or in an environment of high average annual temperature, large waves, or high current velocity.

3 Using cathodic protection for structure parts underwater.

4 Choosing corrosion-resistant steels.

5 If the inner wall of the pipe pile is isolated from the outside environment, its anti-corrosion treatment may not be considered.

6.2.6 Pile arrangement and centre-to-centre spacing shall comply with the following requirements:

1 Pile groups may be arranged in symmetrical, quincuncial form, or circular shape.

2 The centre-to-centre spacing of friction piles shall be arranged in accordance with the following requirements:

For the piles driven by hammering or static pressure, the centre-to-centre spacing of the pile tip shall not be less than 3 times the pile diameter or the side length. The spacing shall be appropriately increased for the soft soil foundation. For piles driven into sand by vibration, the centre-to-centre spacing of the pile tip shall not be less than 4 times the pile diameter or the side length. The centre-to-centre spacing at the bottom of the pile cap shall not be less than 1.5 times the pile diameter or the side length.

1) The centre-to-centre spacing of drilled piles shall not be less than 2.5 times the pile diameter.

2) The centre-to-centre spacing of the bored piles may be taken as that of the drilled pile.

3 The centre-to-centre spacing of the drilled (bored) end-bearing piles supported or socketed in bedrock shall not be less than 2.0 times the pile diameter.

4 The centre-to-centre spacing of the cast-in-place drilled (bored) piles with belled bottom shall not be less than the larger value of 1.5 times the belled pile diameter and the belled pile diameter plus 1.0 m.

5 For piles with diameter less than or equal to 1.0 m, the distance from the outside of the border pile or corner pile to the edge of the pile cap shall not be less than 0.5 times the pile diameter or the side length, and shall not be less than 250 mm; for piles with diameter greater than 1.0 m, the distance shall not be less than 0.3 times of the pile diameter or the side length, and shall not be less than 500 mm.

6.2.7 Details of pile caps and lateral bracings shall comply with the following requirements:

1 The depth of a pile cap should not be less than 1.5 times the pile diameter and should not be less than 1.5 m. The concrete strength class of the pile cap shall not be less than C25. If reinforcing steel with a nominal strength value of 400 MPa or larger is used, the concrete strength class shall not be less than C30.

2 If the pile head is directly embedded into the pile cap, 1 to 2 layers of reinforcement meshes shall be set on the top of each pile. If the primary reinforcement of the pile extend into the cap, one layer of reinforcement mesh should be set at the bottom of the pile cap. The amount of reinforcing steel in the bottom should be 1200 ~ 1500 mm^2/m in each direction and the diameter of reinforcing steel should be 12 mm ~ 16 mm.

3 When lateral bracings are used to enhance the integrity of the piles, the sectional depth and width of the bracings may be 0.8 ~ 1.0 times and 0.6 ~ 1.0 times the diameter of the piles, respectively. The concrete strength class shall not be less than C25. If reinforcing steel with a nominal strength value of 400 MPa or larger is used, the concrete strength class shall not be lower than C30. The steel ratio of the primary reinforcement of lateral bracing shall not be less than 0.15%. The stirrup diameter shall not be less than 8 mm and the spacing shall not be greater than 400 mm.

6.2.8 The connection between pile and pile cap, pile and lateral bracing shall comply with the following requirements:

1 For the connection that a pile head is embedded directly into the pile cap, the embedded length shall not be less than two times the pile diameter (or side length) when the pile diameter (or side length) is less than 0.6m; the embedded length shall not be less than 1.2 m when the pile diameter (or side length) is 0.6 m ~ 1.2 m; the embedded length shall not be less than the pile diameter (or side length) when the pile diameter (or side

length) is greater than 1.2 m.

2 For the connection that the primary reinforcement of a concrete pile are extended into the pile cap, the depth of the pile to be embedded into the cap may be taken as 100 mm; the primary reinforcement of the pile extended into the cap may be shaped as a trumpet (tilt about 15° with respect to the vertical line); the length of the primary reinforcement extended into the pile cap shall not be less than $40d$ for the plain round steel bar of HPB300 (with hook), and no less than $35d$ for the ribbed bar (without hook) (d is the diameter of the primary reinforcement).

3 Cast-in-place piles with large diameters may be connected directly to the columns with or without lateral bracing.

4 For the connection between concrete pipe pile and pile cap, when the longitudinal reinforcing steels embedded into pile cap are used as steel dowels, their number shall not be less than 4 and their diameters shall not be less than 16 mm. The length anchored into the cap shall not be less than 35 times the bar diameter, and the length inserted into the filled concrete in the pipe pile head shall not be less than 1.0 m.

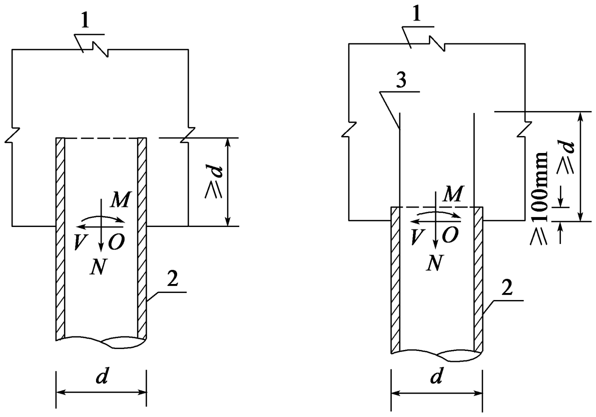
5 Steel pipe pile and pile cap shall be fixed together, and the connection part shall meet stress requirements. One or several combinations of the following methods may be used for fixed connection:

1) If the pile head is directly extended into the pile cap (Fig. 6.2.8a), shear keys shall be set at the part of the steel pipe pile that extends into the cap.

2) Anchors or anchorage reinforcement may be set at the pile head (Fig. 6.2.8 b), the length of the anchorage reinforcement extending into the cap shall comply with the provisions of Item 2 in this Clause.

3) The reinforced concrete core may be set at the pile head. The connection between the reinforced concrete core and the pile cap shall comply with the requirements of Item 2 in this Clause; the length and reinforcement of the concrete core in the steel pipe shall meet the stress requirements.

6 The primary reinforcement of lateral bracing shall be inserted into pile with a length no less than 35 times the diameter of the primary reinforcement.



a)Pile Head Extending Directly into the Pile Cap

b)The Pile Head Anchored by Anchors or Anchorage Reinforcement

Fig. 6.2.8 Connection between Steel Pipe Pile and Pile Cap

1-pile cap; 2-steel pile; 3-anchors or anchorage reinforcement

6.3 Analysis

6.3.1 Piles may be analyzed according to the following provisions:

1 It is assumed that all the loads above the pile cap bottom are supported by the piles;

2 The lateral earth pressure on the abutment is computed according to the earth height measured from the natural ground before filling.

6.3.2 In the ground with a thick layer of soft soil (or soft ground) and good bearing stratum, negative skin friction caused by the loads of subgrade fills, or by lowering groundwater level, etc., shall be taken into account in the calculation of pile foundation.

6.3.3 For the drilled (bored) cast-in-place pile supported by the soil layer, the characteristic value of axial compressive resistance of a single pile, R_a, may be calculated by the following equations:

$$R_a = \frac{1}{2}u\sum_{i=1}^{n} q_{ik} l_i + A_p q_r \tag{6.3.3-1}$$

$$q_r = m_0 \lambda [f_{a0} + k_2 \gamma_2 (h - 3)] \tag{6.3.3-2}$$

where:

R_a—characteristic value of axial compressive resistance of a single pile. The difference between pile self-weight and the correspondingly replaced soil weight shall be considered in the action effect (if the buoyancy of the pile is considered, that of the replaced soil shall be also considered);

u—pile perimeter (m);

A_p—sectional area of the pile tip (m^2). For the belled pile, it may be taken as the sectional area in bell bottom.

n—number of soil layers;

l_i—depth of the soil layers below pile cap bottom or local scour line (m), the length of $2d$ above the bell-bottomed part or variation section is not included;

q_{ik}—nominal value of skin friction of the pile side in each soil layer with a depth of l_i (kPa), which should be determined by the skin friction test of a single pile; when test conditions are unavailable, it can be selected from Table 6.3.3-1, and the skin friction in the bell-bottomed part and in the pile with a range of $2d$ length above the variable section is not considered;

q_r—adjusted characteristic value of bearing resistance of soil at the pile tip (kPa); if the bearing stratum is sand or gravel and the calculated value exceeds the following values, the following values should be adopted: 1000 kPa for silty sand; 1150 kPa for fine sand; 1450 kPa for medium sand, coarse sand, and gravelly sand; and 2750 kPa for gravelly soil;

f_{a0}—characteristic value of bearing resistance of soil at the pile tip (kPa). It is determined according to Clause 4.3.3;

h—bearing depth of the pile tip (m). It is taken from the general scour line for the pile foundation with scouring; it is taken from the natural ground level or from the actual ground level after excavation for the pile foundation without scouring; the calculated value of h shall not be greater than 40 m; and shall be taken as 40 m if the calculated value is larger than 40 m;

k_2—adjusting factor of depth for characteristic value of bearing resistance, which may be selected from Table 4.3.4 of the *Specifications* according to the soil types of bearing stratum at the pile tip;

γ_2—weighted average unit weight for the soil layers above the pile tip (kN/m^3). It shall be taken as the saturated unit weight if the bearing stratum is underwater and impermeable; and it shall be taken as the buoyant unit weight for the soil immersing in the water if the bearing stratum is permeable;

λ—adjusting factor for the characteristic value of bearing capacity, which is selected from Table 6.3.3-2;

M_0—bottom cleaning coefficient, which is selected from Table 6.3.3-3.

Table 6.3.3-1 Nominal Value for Skin Friction of Drilled Pile, q_{ik}

Soil types	Conditions	q_{ik} (kPa)
Medium dense slags, fly ash		40 ~ 60
Cohesive soil	Liquid	20 ~ 30
	Semi liquid	30 ~ 50
	Plastic, stiff plastic	50 ~ 80
	Stiff	80 ~ 120

continued

Soil types	Conditions	q_{ik} (kPa)
Silty soil	Medium dense	30 ~ 55
	Dense	55 ~ 80
Silty sand, fine sand	Medium dense	35 ~ 55
	Dense	55 ~ 70
Medium sand	Medium dense	45 ~ 60
	Dense	60 ~ 80
Coarse sand, gravelly sand	Medium dense	60 ~ 90
	Dense	90 ~ 140
Round gravel, Breccia	Medium dense	120 ~ 150
	Dense	150 ~ 180
Gravel andcobble	Medium dense	160 ~ 220
	Dense	220 ~ 400
Boulder, block stone	—	400 ~ 600

Note: The characteristic value for the skin friction of the bored pile may refer to this table.

Table 6.3.3-2 Adjusting factor, λ

Soil condition at the pile tip	l/d		
	4 ~ 20	20 ~ 25	> 25
Permeable soil	0.70	0.70 ~ 0.85	0.85
Impermeable soil	0.65	0.65 ~ 0.72	0.72

Table 6.3.3-3 Bottom Cleaning Coefficient, m_0

t_0/d	0.3 ~ 0.1
m_0	0.7 ~ 1.0

Note: 1. The t_0, d is the sediment thickness and pile diameter at the pile tip, respectively;

2. When $d \leqslant 1.5$ m, $t_0 \leqslant 300$ mm is taken; when $d > 1.5$ m, $t_0 \leqslant 500$ mm is taken; and the condition shall meet: $0.1 < t_0/d < 0.3$.

6.3.4 For the single cast-in-place pile that is post-grouted and complies with the requirements in Appendix K of the *Specifications*, the characteristic value of its axial compressive resistance, R_a, may be calculated according to the following equation:

$$R_a = \frac{1}{2}u\sum_{i=1}^{n}\beta_{si}q_{ik}l_i + \beta_p A_p q_r \tag{6.3.3-3}$$

where:

R_a—characteristic value of the axial compressive resistance of a single pile for the post-grouted cast-in-place piles (kN); the difference between pile self-weight and its correspondingly replaced soil weight shall be considered in the action effect (if the buoyancy of the pile is considered, that of the replaced soil shall be also considered);

β_{si}— enhancement coefficient of skin friction of the i^{th} soil layer, which may be taken according to Table 6.3.4. When the pile tip in saturated soil is grouted, only the skin friction of the pile within the range of 10.0 ~ 12.0 m above the pile tip is revised for its enhancement; when the pile tip in the unsaturated soil layer is grouted, only 5.0 – 6.0 m above the pile tip is revised for its enhancement; when the pile side in a saturated soil layer is grouted, only the pile skin friction within the range of 10.0 ~ 12.0 m above the grouting section is revised for its enhancement; when the side of the pile in an unsaturated soil layer is grunted, only the skin friction of the pile within the range of 5.0 ~ 6.0 m above and below the grouting section is revised for its enhancement; for pile body out of the enhanced influence range, $\beta_{si} = 1$.

β_p—enhancement coefficient of tip resistance, which may be taken according to Table 6.3.4.

Other symbols are the same as those in Eq. 6.3.3-1 of the *Specifications*.

Table 6.3.4 Enhancement Coefficients of Skin Friction and Tip Resistance after grouting, β_s and β_p, respectively

Name of soil layer	Muddy soil	Clay, silty clay	Silty soil	Silty sand	Fine sand	Medium sand	Coarse sand gravelly sand	Breccia, round gravel	Gravel, cobble	Completely weathered rock; highly weathered rock
β_s	1.2 ~ 1.3	1.3 ~ 1.4	1.4 ~ 1.5	1.5 ~ 1.6	1.6 ~ 1.7	1.7 ~ 1.9	1.8 ~ 2.0	1.6 ~ 1.8	1.8 ~ 2.0	1.2 ~ 1.4
β_p	—	1.6 ~ 1.8	1.8 ~ 2.1	1.9 ~ 2.2	2.0 ~ 2.3	2.0 ~ 2.3	2.2 ~ 2.4	2.2 ~ 2.5	2.3 ~ 2.5	1.3 ~ 1.6

Note: A greater value may be taken for moderately dense, loose sand and gravelly soil; and a smaller value for dense sand and gravelly soil.

6.3.5 The characteristic value of the axial compressive resistance of a single driven pile supporting on the soil layers, R_a, may be calculated as follows:

$$R_a = \frac{1}{2}\left(u\sum_{i=1}^{n}\alpha_i l_i q_{ik} + \alpha_r \lambda_p q_{rk}\right) \tag{6.3.5}$$

where:

R_a—characteristic value of the axial compressive resistance of a single pile (kN); the difference between pile self-weight and its correspondingly replaced soil weight shall be considered in the action effect (if the buoyancy of the pile weight is measured into buoyancy, the replaced soil weight shall also be measured into buoyancy);

u—perimeter of pile body (m);

n—number of soil layers;

l_i—depth of each soil layer below the bottom of the pile cap or the local scour line (m);

q_{ik}—nominal value for the skin friction of piles in each soil layer with a depth of l_i (kPa), which should be determined by the test of skin friction of a single pile or static cone penetration test; if the test condition is unavailable, it may be selected from Table 6.3.5-1;

q_{rk}—nominal value of bearing resistance of soil at the pile tip (kPa), which should be determined by a single pile test of tip resistance or static cone penetration test; if the test condition is unavailable, it may be selected from Table 6.3.5-2;

α_i, α_r—coefficient to consider the influence of vibration-driven pile on the skin friction of piles in each soil layer and the resistance of the pile tip, respectively; which is taken from Table 6.3.5-3; it is taken as 1.0 for driven piles by hammering or by static pressure.

λ_p—soil plugging coefficient at pile tip. For pipe piles with closed end, $\lambda_p = 1.0$; for pipe piles with open end, λ_p is 0.3 ~ 0.4 when $1.2\ \text{m} \leqslant d \leqslant 1.5\ \text{m}$, and 0.2 ~ 0.3 when $d > 1.5$ m.

Table 6.3.5-1 Nominal Value of Skin Friction for Driven Piles

Soil type	Status	Nominal value of skin friction, q_{ik} (kPa)
Cohesive soil	$1.5 \geqslant I_L \geqslant 1$	15 ~ 30
	$1 > I_L \geqslant 0.75$	30 ~ 45
	$0.75 > I_L \geqslant 0.5$	45 ~ 60
	$0.5 > I_L \geqslant 0.25$	60 ~ 75
	$0.25 > I_L \geqslant 0$	75 ~ 85
	$0 > I_L$	85 ~ 95
Silty soil	Slightly dense	20 ~ 35
	Medium dense	35 ~ 65
	Dense	65 ~ 80
Silty sand, fine sand	Slightly dense	20 ~ 35
	Medium dense	35 ~ 65
	Dense	65 ~ 80
Medium sand	Medium dense	55 ~ 75
	Dense	75 ~ 90
Coarse sand	Medium dense	70 ~ 90
	Dense	90 ~ 105

Note: 1. The liquidity indexes, I_L, of the soil listed in the table are the values measured by 76g stratocone.

2. Small value should be used for steel pipe piles.

Table 6.3.5-2 Nominal Value of Bearing Resistance of Soil at the Driven Pile Tip, q_{rk}

Soil type	Status	Nominal value of bearing resistance of soil at the pile tip, q_{rk} (kPa)		
Cohesive soil	$I_L \geq 1$	1000		
	$1 > I_L \geq 0.65$	1600		
	$0.65 > I_L \geq 0.35$	2200		
	$0.35 > I_L$	3000		
—		Relative depth for the pile tipembedded into bearing stratum		
		$> \frac{h_c}{d}$	$4 > \frac{h_c}{d} \geq 1$	$\frac{h_c}{d} > 4$
Silty soil	Medium dense	1700	2000	2300
	Dense	2500	3000	3500
Silty sand	Medium dense	2500	3000	3500
	Dense	5000	6000	7000
Fine sand	Medium dense	3000	3500	4000
	Dense	5500	6500	7500
Medium and coarse sand	Medium dense	3500	4000	4500
	Dense	6000	7000	8000
Round gravel	Medium dense	4000	4500	5000
	Dense	7000	8000	9000

Note: The h_c in the table is the depth of pile tip embedded into the bearing stratum (excluding pile shoes); d is the pile diameter or side length.

Table 6.3.5-3 Influence Coefficient, α_i, α_r

Pile diameter or side length, d (m)	Coefficient α_i, α_r			
	Clay	Silty clay	Silty soil	Sand
$0.8 \geq d$	0.6	0.7	0.9	1.1
$2.0 \geq d > 0.8$	0.6	0.7	0.9	1.0
$d > 2.0$	0.5	0.6	0.7	0.9

6.3.6 When the skin friction of the pile and bearing resistance of soil at the pile tip are measured by the static cone penetration test, q_{ik} and q_{rk} in the calculation of the characteristic values of compression resistance of the driven piles should be calculated as follows:

$$q_{ik} = \beta_i \bar{q}_i \tag{6.3.6-1}$$

$$q_{ir} = \beta_r \bar{q}_r \tag{6.3.6-2}$$

When $\bar{q}_r$ of the soil layer is greater than 2000 kPa and $\bar{q}_i / \bar{q}_r$ is less than or equal to 0.014, β_i and β_r should be calculated as follows:

$$\beta_i = 5.067(\bar{q}_i)^{-0.45} \tag{6.3.6-3}$$

$$\beta_r = 3.975(\bar{q}_r)^{-0.25} \tag{6.3.6-4}$$

Otherwise:

$$\beta_i = 10.045(\bar{q}_i)^{-0.55} \tag{6.3.6-5}$$

$$\beta_r = 12.064(\bar{q}_r)^{-0.35} \tag{6.3.6-6}$$

where:

$\bar{q}_i$—average value of the local skin friction of the pile side in the i^{th} layer of the soil measured by the static cone penetration test (kPa). When q_i is less than 5 kPa, it is taken as 5 kPa;

$\bar{q}_r$—average value of resistance at the pile tip within the range of elevations (excluding the pile shoes) from -4d to +4d (d is the diameter or side length of the pile) measured by the static cone penetration test (kPa). When the average value of resistance within 4d above pile tip elevation is greater than that within 4d below pile tip elevation, the average value of resistance is taken as the value within 4d below pile tip elevation;

$\beta_i \beta_r$—comprehensive adjusting factors for skin friction and tip resistance, respectively. Eqs. (6.3.6-3) ~ (6.3.6-6) are not appropriate for short piles in urban miscellaneous fill; when the piles are used for loess or other special soil regions, it is needed to carry out trail piling to verify the comprehensive adjusting factors.

6.3.7 The characteristic value of axial compressive resistance of drilled (bored) and driven piles supported by bedrock or socketed in the bedrock, R_a, may be calculated by the following equation:

$$R_a = c_1 A_p f_{rk} + u \sum_{i=1}^{m} c_{2i} h_i f_{rki} + \frac{1}{2} \xi_s u \sum_{i=1}^{n} l_i q_{ik} \tag{6.3.7}$$

where:

c_1—performing coefficient of the tip resistance determined by rock strength, rock fracture degree of rock, and other factors, as shown in Table 6.3.7-1;

A_p—sectional area of the pile tip (m^2). For the belled pile, it may be taken as the sectional area in bell bottom;

f_{rk}—nominal value of the uniaxial compressive strength of saturated rock at pile tip (kPa). It may be taken as the nominal value of uniaxial compressive strength under natural humidity for argillite; when f_{rk} is less than 2 MPa, R_a is calculated as the pile is supported on the soil layer;

f_{rki}—the value of f_{rk} in the i^{th} layer;

c_{2i}—performing coefficient of skin friction determined by rock strength, rock fracture degree and other factors, as shown in Table 6.3.7-1;

u—perimeter of pile body in each soil layer or rock stratum (m);

h_i—depth of the pile socketed in each rock stratum (m), excluding highly weathered and completely weathered strata as well as the rock stratum above the local scour line;

m—number of rock strata, excluding highly and completely weathered strata;

ξ_s—performing coefficient of skin friction for the overlying layer, which shall be determined according to f_{rk} at the pile tip, see Table 6.3.7-2;

l_i—depth of each soil layer below the pile cap bottom or local scour line (m);

q_{ik}—nominal value of skin friction in pile side of No. i^{th} soil layer (kPa), which shall be taken as the tested skin friction of the single pile; if there is no condition to conduct a test, it may be taken from Table 6.3.3-1 for drilled (bored) piles, from Table 6.3.5-1 for driven piles without consideration of the skin friction for the bell-bottom part;

n—number of soil layers. Highly or completely weathered rock strata are considered as soil layers.

Table 6.3.7-1 Performing Coefficient, c_1, c_2

Rock stratum conditions	c_1	c_2
Intact ormoderately intact	0.6	0.05
Moderatelyfractured	0.5	0.04
Veryfractured or extremely fractured	0.4	0.03

Note: 1. When the socketed depth of the pile into the rock is less than or equal to 0.5 m, c_1 is taken as 0.75 times the value listed in the table, and $c_2 = 0$ is taken.

2. For drilled piles, the values of C_1 and C_2 shall be reduced by 20%; for the sediment thickness t at pile tip, $t \leqslant 50$ mm is taken when $d \leqslant 1.5$ m, and $t \leqslant 100$ mm is taken when $d > 1.5$ m.

3. When the moderately weathered stratum is used as bearing stratum, C_1 and C_2 are multiplied by a reduction coefficient of 0.75.

Table 6.3.7-2 Performing Coefficient of Skin Friction of Overlying Layer, ξ_s

f_{rk} (MPa)	2	15	30	60
Performing Coefficient of skin friction, ξ_s	1.0	0.8	0.5	0.2

Note: values of ξ_s may be calculated by interpolation. When $f_{rk} > 60$ MPa, ξ_s may be taken according to $f_{rk} = 60$ MPa.

6.3.8 When a pile is designed as rock-socketed pile, the effective socketed depth of the pile in the bedrock may be calculated by the following equations:

1 Circular pile:

$$h_r = \frac{1.27H + \sqrt{3.81\beta f_{rk} dM_H + 4.84H}}{0.5\beta f_{rk} d} \tag{6.3.8-1}$$

2 Rectangular pile:

$$h_r = \frac{H + \sqrt{3\beta f_{rk} bM_H + 3H^2}}{0.5\beta f_{rk} b} \tag{6.3.8-2}$$

where:

h_r—effective socketed depth of the pile in the bedrock (excluding highly and completely weathered strata as well as the rock stratum above the local scour line) (m). It shall not be less than 0.5 m;

H—horizontal force at the top surface of the bedrock (kN);

M_H—bending moment at the top surface of the bedrock (kN · m);

b—side length of the pile perpendicular to the bending moment plane (m);

β—reduction coefficient for converting the vertical compressive strength of rock to the horizontal compressive strength, it is taken as 0.5 ~ 1.0. It is determined by the side structure of the rock stratum. The smaller value is taken for the reduction coefficient if the rock has developed joints, while the larger value is taken for the reduction coefficient if the rock has under-developed joints;

f_{rk}—nominal value of the uniaxial compressive strength of saturated rock (kPa).

6.3.9 For friction piles subjected to tension, the following requirements shall be met:

1 Pile shall not be in tension if the axial forces on the pile are caused by the combination of dead load, prestressing, soil weight, lateral earth pressure, vehicle load, and pedestrian load with their frequent values;

2 Pile is permitted to be in tension if the axial forces are caused by the combination of the above-mentioned loads and other variable actions or the accidental actions with frequent values, or caused by the accidental combination. The characteristic value of the axial tensile resistance of a single pile is calculated by the following equation:

$$R_t = 0.3u\sum_{i=1}^{n} \alpha_i l_i q_{ik} \tag{6.3.9}$$

where:

R_t—characteristic value of the axial tensile resistance of the single pile (kN);

u—perimeter of pile body (m). For a pile with constant diameter, $u = \pi d$; for a belled pile, in the length range $\sum l_i \leq 5d$ (counted from pile tip), $u = \pi D$ is taken; for other length ranges, $u = \pi d$ is taken (where D is the extensional pedestal diameter of the pile, and d is the diameter of the pile body);

α_i—coefficient to consider the influence of vibration-driven pile on the skin friction of pile in each soil layer, which is taken according to Table 6.3.5-3; for hammer-driven, static pressure-driven, and drilled pile, $=1$.

3 In calculating the axial force acting on the bottom of the pile cap caused by external loads, the self-weight of the pile shall be excluded.

6.3.10 In calculating the internal forces of a pile, the m method (see Appendix L and Appendix M) or other reliable methods may be used. The sectional stiffness of the concrete-filled steel tubular composite pile may be calculated according to the following equations:

$$EA = E_c A_c + E_s A_s \tag{6.3.10-1}$$

$$EI = E_c I_c + E_s I_s \quad (6.3.10\text{-}1)$$

$$GA = G_c A_c + G_s A_s \quad (6.3.10\text{-}1)$$

where:

EA—sectional compression stiffness of concrete-filled steel tubular composite pile;

EI—sectional flexural stiffness of concrete-filled steel tubular composite pile;

GA—sectional shear stiffness of concrete-filled steel tubular composite pile.

The subscripts *c* and *s* represent the corresponding parameters of concrete and steel tube, respectively.

6.3.11 For a multi-row pile group consisting of 9 or more friction piles, if the center-to-center spacing of the piles in the pile tip plane is less than 6 times the pile diameter, the pile group may be considered as a block foundation, where the end bearing resistance of the total area at the pile tip plane is checked in accordance with the provisions in the Appendix N of the *Specifications*. If a soft soil layer or weak ground is underlied the pile tip plane, the bearing resistance of the underlying soft or weak soil layer shall be checked according to Clause 5.2.6 of the *Specifications*.

6.3.12 For pile foundations with end-bearing piles or with piles whose centre-to-centre spacings are greater than 6 times the pile diameter (or side length), the settlement of a single pile may be taken as the total settlement of the pile foundation. In other cases, the settlement of the pile groups shall be calculated as a pier or abutment foundation according to the provisions of Clause 5.3.4 of the *Specifications*.

6.3.13 When the pile foundation is located in a seasonal frost heaving soil, the frozen resistant uplift stability of the pile shall be checked. The calculation may be performed according to the method in Appendix H of the *Specifications*.

7 Caisson Foundation

7.1 General

7.1.1 A caisson foundation may be adopted for pier or abutment foundation where the riverbed geology, hydrology, and construction conditions are suitable. However, the caisson foundation should not be applied where some obstacles are difficult to remove in the riverbed, such as boulders, logs of trees, and abandoned old bridge foundations, or where the rock stratum surface inclines greatly as well as quicksand may appear during the construction process.

7.1.2 According to the project scale, geological conditions, and construction methods, caissons may be made of concrete, reinforced concrete, concrete with permanent steel formwork, or other materials. Concrete caisson may be used in loose strata, and the caisson for floating may be made of thin-walled reinforced concrete structures or thin-walled concrete structures with permanent steel formwork.

7.1.3 The bearing depth of a caisson shall be in accordance with the requirements in Section 5.1 of the *Specifications*.

7.1.4 The bearing capacity, settlement, and stability of the caisson as an block foundation in service as well as in the sinking and floating process during the construction shall be checked in accordance with the provisions of the *Specifications*. Moreover, the ultimate limit state and the serviceability limit state of the structure shall be checked based on the materials used, according to the provisions of the current *Specification for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culvert*(JTG 3362) and other relevant industry specifications.

7.1.5 Necessary structural measures shall be set to ensure the smooth sinking of the caisson and to avoid the occurrence of adverse situations such as sinking difficulty, suddenly sunk, or tilting severely.

7.2 Detailing Requirements

7.2.1 The plane shape and size of a caisson shall be determined according to the size of the bottom surface of the pier or abutment body, bearing capacity of ground soil, and construction requirements. Additionally, they shall also meet the following requirements:

1 The offset width at the top surface of a caisson shall meet the construction requirements, including the allowable deviation in caisson construction, setups for the water-retaining structure, and space for the construction of the pier or abutment body. The offset width shall not be less than 0.4 m for floating caisson, and 0.2 m for other caissons; at the same time, the offset width shall not be less than 1/50 of the total height of the caisson.

2 The size and arrangement of the dredge holes shall meet the needs of the operation of mechanical dredgers. For a caisson with a cofferdam on its top, the cofferdam should be considered in the design of the size and arrangement of the dredge holes.

3 The corners of the caisson should be rounded or obtuse.

7.2.2 The height of each segment of a caisson may be determined according to its plane size, total height, ground conditions, and construction conditions, and shall meet the requirements for its sinking capacity and stability during the sinking process. The outer surface of the caisson may be made as a vertical plane, an inclined plane (the slope of vertical/horizontal is 20/1 ~ 50/1) or a stepped shape corresponding to the slope.

7.2.3 The thicknesses of the caisson wall and its diaphragm shall be determined according to factors such as the structural strength, gravity weight required for sinking, convenience for soil dredging and base cleaning of foundation, etc. They may be taken as 0.8 ~ 2.2 m. The wall thickness of floating caissons of reinforced concrete and concrete with permanent steel formwork shall also be determined by the calculation according to the floating requirements.

7.2.4 The cutting edge of a caisson may adopt sharp cutting edge or cutting edge with a lower plane according to the geological conditions, and shall also meet the following requirements:

1 Concrete structure should not be adopted. If the soil is stiff, the cutting edge surface shall be reinforced with shaped steel, or the bottom segment shell shall be a steel structure.

2 The width of the bottom surface of the cutting edge may be 0.1 ~ 0.2 m, and may be

appropriately widened when the caisson is supported on soft ground.

3 The intersection angle between the bevel of the cutting edge and the horizontal plane shall not be less than 45°.

4 The bottom surface of the internal diaphragms of a caisson shall be at least 0.5 m higher than the bottom surface of the cutting edge.

5 When the caisson is inevitable to seat on a slightly inclined rock surface, the cutting edge of the caisson should be made with a same inclined surface (higher and lower cutting edges).

7.2.5 The reinforcement arrangement of a reinforced concrete caisson shall be determined by calculation, and the reinforcement ratio shall not be less than 0.1%. The vertical primary reinforcement of the cutting edge shall be extended into the root of the cutting edge with a length no less than 0.5 times of the maximum effective span of the caisson calculated as a plane frame structure. The stirrups shall be set in the total height range of the cutting edge and shall be designed according to shear force or detailing requirements. The connected reinforcement steels shall be set in the vertical connection of the concrete caisson wall.

7.2.6 The concrete strength class for each part of the caisson shall comply with the following requirements:

1 The concrete strength class shall not be less than C30 for cutting edge, shall not be less than C25 for caisson wall.

2 When it is a thin-walled floating caisson, the concrete strength class shall not be less than C30 for the caisson wall and diaphragm, and not be less than C15 for the filling in the abdominal cavity.

3 The concrete strength class for bottom seal shall not be less than C25 for non-rock foundations, and not be less than C20 for rock foundations.

7.2.7 The concrete depth of the sealing base for the caisson shall be determined by calculation. The top surface of the sealing base shall not be less than 0.5 m higher than the root of the cutting edge (that is, the vertex of the slope of the cutting edge).

7.2.8 Whether the space inside the caisson is filled or not shall be determined according to the stress and stability requirements of the caisson. Filled or not, the caisson shall be designed to meet the following requirements:

1 The filling materials may be concrete, rubble concrete, or rubble grouting concrete; the filling materials may be coarse sand or sandy gravel filling in non-freezing areas.

2 Reinforced concrete cover plates shall be installed on the top surfaces for hollow caissons or caissons filled with coarse sand or sandy gravel. The thickness of cover plates shall be determined by calculation.

7.3 Analysis

7.3.1 The calculation of bearing capacity, settlement, and stability of a caisson as an block foundation shall comply with the following requirements:

1 If lateral earth pressure is not considered, the calculation may be carried out according to the relevant provisions in Chapter 5 of the *Specifications*. If the lateral elastic resistance of soil is considered, the calculation may be carried out according to Appendix M of the *Specifications*.

2 The elastic resistance of soil may be considered if grouting sleeves are used in construction and measures for restoring the restraint capacity of lateral soil are taken.

3 For caissons with higher and lower cutting edges, adverse factors such as rock surface inclination shall be considered in checking the stability against overturning and sliding, and necessary measures shall be taken to improve the stability.

7.3.2 The sinking capacity in construction of a caisson may be checked according to the following provisions:

1 When the soil in the caisson is dredged below the cutting edge and the supporting reaction force on the bottom surface of the cutting edge is zero, the sinking factor may be calculated according to the following formulas:

$$k_{st} = (G_k - F_{fw,k})/R_f \tag{7.3.2-1}$$

$$R_f = u(h - 2.5)q \tag{7.3.3-2}$$

where:

k_{st}—sinking factor, which is generally controlled within the range of 1.15 ~ 1.25;

G_k—nominal value of self-weight of the caisson (plus the nominal value of sinking-assistant weight) (kN);

$F'_{\mathrm{fw,k}}$—nominal value of the buoyancy of water during sinking (kN);

R_{f}—nominal value of the total skin friction of the caisson wall (kN). The skin friction per unit area is distributed along the caisson depth in a gradient, it follows a triangular distribution within 5 m from the ground level and is constant below 5 m;

u—perimeter of the base plan of the caisson (m). For caisson with stepped wall, the perimeter of the bottom plan of this step segment is taken as the u value for this step segment;

h—bearing depth of the caisson (m);

q—weighted average value of the nominal value of the skin friction between the caisson wall and the soil, which is weighted according to the soil layer depth (kPa). The nominal value of the skin friction between the caisson wall and the soil shall be determined according to actual measured data or practical experience. In case of lack of data, it may be selected from Table 7.3.2 according to the characteristics of the soils and construction measures.

Table 7.3.2 Nominal Value of Skin Friction Between Caisson Wall and Soil

Soil name	Nominal value of skin friction (kPa)
Cohesive soil	25 ~ 50
Sand	12 ~ 25
Cobble	15 ~ 30
Gravelly stone	15 ~ 20
Soft soil	10 ~ 12
Grouting sleeve	3 ~ 5

Note: The grouting sleeve is the thixotropic grouting jetted on the outer periphery of the caisson wall, which is a material assistant to sinking.

2 If the sinking factor is large or soft soil is encountered in the sinking process, the following equations may be used to check the stability of the caisson during sinking:

$$k_{st,s} = (G_k - F'_{fw,k})/(R'_f + R_b) \tag{7.3.2-3}$$

$$R_b = R_1 + R_2 \tag{7.3.2-4}$$

$$R_1 = U\left(a + \frac{b}{2}\right) \cdot 2 \cdot f_a \tag{7.3.2-5}$$

$$R_2 = 2 \cdot A_1 \cdot f_a \tag{7.3.2-6}$$

where:

$k_{\mathrm{st,s}}$—stability factor during caisson sinking, which is generally controlled in the range of 0.8 ~ 0.9;

$F'_{\mathrm{fw,k}}$—nominal value of buoyancy of water in the state under investigation (kN);

R'_{f}—nominal value of total skin friction of the caisson wall in the investigation state (kN), it may be calculated according to Formula (7.3.2-2);

R_b—sum of the nominal values of bearing capacity of ground soil under the cutting edge, the diaphragms, and base beams of the caisson (kN);

R_1—supporting force of the soil under the lower plane and the inclined surface of the cutting edge (kN);

U—outer perimeter of the wall (m);

a—the width of lower plane in the cutting edge (m);

b—the level projection of inclined plane of the cutting edge embedded into soil (m);

f_a—characteristic value of bearing capacity of ground soil (kPa), which may be determined in accordance with Section 4.3 in the absence of data;

R_2—supporting reaction force of the soil under the diaphragms and the bottom beams (kN);

A_1—total supporting area of the diaphragms and base beams (m^2).

7.3.3 The most unfavorable conditions that may occur in the actual construction shall be considered in checking the bearing capacity of the caisson wall during construction. The check may follow the provisions in Appendix P of the *Specifications*.

7.3.4 The most unfavorable conditions that may occur in the actual construction process shall be considered in checking the flexural resistance of the cutting edge of a caisson. The check may follow the provisions in Appendix Q of the *Specifications*.

7.3.5 The bearing capacity of the caisson wall and the inner diaphragm of the caisson segments above the bottom segment shall be checked accordingly under the most unfavorable conditions. The actions that shall be considered in the checking include the actions of hydrostatic water pressure, stream pressure, wave force, wind load, reaction force of guiding structure, tension force of anchor cable, lateral pressure of the filled concrete in the caisson, etc.

7.3.6 The depth of the bottom seal concrete shall be calculated and determined according to the water pressure at the foundation base, the upward reaction force of the ground soil, the filling cases inside the caisson, and the requirements of the bottom seal concrete at various stress stages. The following factors shall be considered in the calculation:

1 If dewatering and dry construction are required after bottom sealing, the bottom seal concrete is subjected to the upward reaction force of the water and soil under the seal plate, and the actual strength class of the sealing concrete in dewatering shall be taken as the concrete strength in the calculation.

2 If the space inside the caisson is not filled with concrete, the bottom seal concrete bears all the reaction forces generated by all loads of the caisson foundation. If the space inside the caisson is filled with sand, the gravity effect of the sand shall be deducted from the

reaction forces.

3 If the space inside the caisson is filled with concrete (or rubble concrete), the bottom seal concrete bears the reaction forces induced by all loads of the caisson foundation plus the hydrostatic pressure at the bottom of the caisson before concrete filling and minus the gravity weight of the fillings.

7.3.7 Lateral stability of a thin-walled caisson during floating (before sinking into the riverbed) shall be ensured. The stable inclination angle of the floating body of the caisson, φ, may be calculated according to the following formulas:

$$\varphi = \tan^{-1}\frac{M}{\gamma_w V(\rho - a)} \tag{7.3.7-1}$$

$$\rho = \frac{I}{V} \tag{7.3.7-2}$$

where:

φ—inclination angle of the caisson during floating, which shall not be greater than 6°, and shall satisfy $(\rho - a) > 0$;

M—external moment (kN · m);

V—volume of discharged water (m^3);

a—distance from the centroid of the caisson to the center of buoyancy (m). It is positive if the centroid is above the center of buoyancy, otherwise it is negative;

ρ—metacenter radius, that is, the distance from the metacenter to the buoyancy center (m);

I—moment of inertia of cross-sectional area of discharged water for the thin-walled floating caisson (m^4);

γ_w—unit weight of water, $\gamma_w = 10kN/m^3$.

8 Underground Diaphragm Walls

8.1 General

8.1.1 This chapter is applicable to the design of the cast-in-place concrete diaphragm walls as retaining structures for foundation pits and as bridge foundations in highway bridges.

8.1.2 The design safety level and importance factor for the diaphragm wall as retaining structures for foundation pits shall be determined according to Table 8.1.2, based on the severity of the influence on the adjacent facilities and the construction of underground structures induced by the damage of the retaining structure, instability of soil body or excessive deformation.

Table 8.1.2 DesignSafety Level and Importance Factor of Retaining Structures

Safety Level	Consequences of destruction	γ_0
Level 1	Very serious	1.1
Level 2	Serious	1.0
Level 3	Not serious	0.9

8.1.3 Diaphragm walls as retaining structures for foundation pits shall be designed in accordance with the following requirements:

1 Factors of engineering geology and hydrogeology, foundation type, superstructure conditions, excavation depth of foundation pit, dewatering, and drainage conditions, adjacent facility requirements and service life, etc., shall be comprehensively considered.

2 Safety shall be ensured during the excavation and construction of the underground structures.

8.1.4 Diaphragm walls as foundations shall be designed in accordance with the following requirements:

1 Factors of engineering geology and hydrogeology shall be comprehensively considered, and the design shall be reasonable and in accordance with the local conditions.

2 It shall be ensured that the settlement, horizontal movement, or tilt affecting the function of the superstructure would not occur.

8.1.5 Relevant requirements for quality inspection, environmental monitoring, and field tests shall be presented in the design of the diaphragm walls.

8.1.6 The diaphragm walls in areas with special geological conditions shall be carefully applied according to local engineering judgement.

8.2 Retaining Structures

8.2.1 For a diaphragm wall as a retaining structure system for a foundation pit, its scheme shall be analyzed and selected through technical and economic comparisons of various potential schemes. The strength, stability, and deformation of the retaining structure shall be calculated and checked.

8.2.2 The design of the supporting system of retaining structure shall meet the following requirements:

1 The structural system and connection structure of internal bracings shall be stable, and their stiffness shall meet the requirements of deformation. Structural layout, calculations on structural internal forces and deformations, checking calculations on strength and stability of components, connection detailing as well as the installation and dismantling process of components shall be included in the design.

2 The design of the soil anchor rods (anchor cables) shall include the structural layout arrangement, checking calculations on axial resistance, and soil stability.

3 The design of the internal ring beams and inner linings shall include structural layout arrangement, and verification of stresses, strength, and stability.

8.2.3 The design of the retaining structure shall consider the influence of horizontal deformation of the structure and the changes of groundwater on the horizontal and vertical deformation of the surrounding soil body and adjacent facilities. For foundation pit projects with safety level 1 or safety level 2 that have the requirements for limiting deformation of the surrounding environment, the limit value of horizontal deformation of the retaining structure shall be determined according to the factors of the importance of the surrounding environment, the adaptive ability for deformation, and the properties of soil.

8.2.4 The bearing depth of a diaphragm wall below the excavation surface of a foundation pit shall meet the requirements for the stability and deformation of the retaining structure of the foundation pit. It may be preliminarily selected according to the static equilibrium conditions; after the check of the stability and wall deformation, it can be comprehensively determined by referring to local engineering experiences. If the bearing depth of the diaphragm wall is close to the rock stratum, the wall should be socketed into the rock in case this will not increase the project cost too much.

8.2.5 Retaining structures shall be designed in accordance with the following requirements for the ultimate limit state and serviceability limit state based on different design conditions:

1 For the ultimate limit state, the following calculation contents shall be included:

1) Calculations on the stability of the soil body;

2) Calculations on strength and stability of the wall structure;

3) Calculations on resistance and stability of the supporting system.

2 For the serviceability limit state, the checking calculations of structural deformation, crack resistance, and crack width shall be included.

8.2.6 Combination of actions for a retaining structure shall be determined according to different design states and different cases in its construction process.

8.2.7 Detailing of the retaining structures shall comply with the following requirements:

1 The section form and segment length of the wall shall be determined according to the overall plan layout, stress conditions, trench wall stability, environmental conditions, construction conditions, etc. A unit length of wall segment may be 4 m ~ 8 m; the wall thickness shall be determined by calculation considering the excavation capacity of the trenching machine and should not be less than 600 mm; the trench verticality shall not be greater than 1/200.

2 The diaphragm wall shall meet the requirements for seepage prevention; the concrete strength class of the wall, internal bracings, internal ring beams (including vertical ribs), and inner linings shall not be lower than C25; if the groundwater is corrosive, applicable anti-erosion concrete shall be selected. The concrete strength class, raw materials and main mix ratio indexes shall also meet the relevant provisions in the *Specification for Deterioration Prevention of Highway Concrete Structures* (JTG/T B07—01).

3 The net thickness of concrete cover for the primary reinforcement of the walls shall be determined according to the service requirements, geological conditions, construction conditions, and environmental conditions, and should not be less than 70 mm. The diameter of the primary reinforcement of the walls should not be smaller than 20 mm and shall not be larger than 40 mm, and the diameter of the auxiliary reinforcement due to detailing requirements should not be less than 16 mm.

4 The pipe connectors may be used between the trench wall units. If the requirements of integrity and impermeability are very high, the milled connectors, steel partition connectors, or box-type connectors should be adopted.

5 The reinforcement arrangement of reinforcement cages in diaphragm walls shall meet the following requirements:

1) The vertical primary reinforcement shall be placed on the inner side, their clear spacing shall not be less than 75 mm; the spacing of auxiliary reinforcement due to detailing requirements shall not be greater than 300 mm. When double-layer reinforcement must be implemented, the spacing between the inner and outer rows of the reinforcements shall not be less than 100 mm.

2) The positions with small stresses shall be selected as the location of the vertical connectors of reinforcement cages. The division length of the reinforcement cages shall be determined according to the length of the unit trench segment, connector type, and lifting capacity of the lift equipment.

3) The bottom of the reinforcement cage should be appropriately narrowed in the wall thickness direction, and have a gap of 100 ~ 500 mm to the wall bottom. The primary reinforcement shall be extended into the top cap beam of the wall, and the extension length shall not be less than the required development length.

4) When the pipe connectors are used, a gap of 150 mm ~ 200 mm should be left between the

side end of the reinforcement cage and the connector pipe; when milled connectors are used, a gap no less than 250 mm should be left between the side end of the reinforcement cage and the concrete end surface.

5) Mechanical connection should be used for reinforcement connection. If a binding or lapping connection is adopted, it shall comply with the provisions in the *Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTG 3362).

6 Concrete cap beams shall be set at the top of the wall, and each side of the cap beam shall be wider than the wall no less than 150 mm.

7 The internal bracings of the straight diaphragm walls may be of steel or concrete structures, and shall meet the following requirements:

1) The vertical height of the section of the cast-in-place concrete internal bracings shall not be less than l/20 of the effective span in the vertical plane;

2) The horizontal dimension of the intermediate beam section shall not be less than 1/8 of its horizontal effective span, and the vertical dimension of the section shall not be less than the section depth of the internal bracing;

3) The vertical spacing of anchorage stems of anchor rods (anchor cables) should not be less than 2.5 m, and their horizontal spacing should not be less than 1.5 m;

4) The thickness of the overlying soil layer on the anchor stem should not be less than 4.0 m;

5) The inclination angle of the inclined anchorage rods should be 15°~30°;

6) The development length of the anchor rods shall be determined by calculation and shall not be less than 4.0 m. The free length of the anchor rods should not be less than 5.0 m and shall surpass the potential fracture surface no less than 1.5 m.

8 The section depth and thickness of the internal ring beams (including vertical ribs) or inner linings of the circular diaphragm wall as retaining structure shall be determined according to calculations, and the vertical ribs may be reinforced according to the detailing requirements.

8.2.8 The lateral actions of diaphragm walls shall include earth pressure, water pressure, temperature action, frost heave effect, wave action as well as lateral pressure caused by adjacent buildings (structures) around foundation pit, ground overload, construction load, etc.

8.2.9 Active and passive lateral earth pressure may be calculated using Rankine or Coulomb earth pressure theory. When the horizontal displacement of the retaining structure is strictly restricted, the at-rest earth pressure shall be used in the calculation.

8.2.10 In principle, earth pressure and water pressure acting on the retaining structure should be calculated independently for sand ground; while they should be calculated together for cohesive soil ground. They may also be calculated according to local experience.

8.2.11 If deformation control is used as the principle in the design of a retaining structure, the earth pressure acting on the diaphragm walls may be determined according to the soil-wall interaction theorem, and may be calculated according to the method in Appendix R of the *Specifications*.

8.2.12 The calculation of a straight diaphragm wall as a retaining structure shall comply with the following requirements:

1 Stability against overturning (stability for socket), overall stability against sliding, safety in resisting basal heave, anti-seepage, and anti-surge stability for groundwater shall be checked.

2 Internal forces and deformations of diaphragm walls may be calculated as vertically elastic beam foundation specified in Appendix S of the *Specifications*.

8.2.13 The components of a straight diaphragm wall as a retaining structure shall be analyzed according to the following requirements:

1 Walls, bracings, and columns shall be analyzed as eccentric compression members.

2 Intermediate beams may be analyzed as horizontal flexural members. If an intermediate beam intersects obliquely with horizontal internal bracing, or an intermediate beam is used as the chord of the side truss, the waist beam shall be analyzed as an eccentric compression member.

3 The stem of the soil anchor rod (or anchor cable) shall be analyzed as an axial tension member. The free length and development length of the anchor rods, the diameter and shape of the anchor stems, and the strength of the slurry shall be determined according to the designed axial tension forces of the anchor rod (or anchor cable), the soil layer's pull-

out resistance and the bonding force. The outer anchor head and intermediate beam shall be designed according to the anchor load value.

8.2.14 The calculation of circular diaphragm wall as retaining structure shall comply with the following requirements:

1 The stability shall be checked, and the checking contents shall comply with the provisions of Item 1 of Clause 8.2.13.

2 The stability of the structure under the actions of earth pressure and water pressure shall be checked. In calculating the critical load of structure buckling, the structure should be analyzed as a spatial structure, or the structure may also be simplified as a ring and be checked according to the following equations:

$$q_{pk} = 3EI/(R_0^3 h) \tag{8.2.14-1}$$

$$Kq_{tk} \leqslant q_{pk} \tag{8.2.14-2}$$

where:

q_{pk}—nominal value of critical load distributed along the ring (kN/m^2);

E—modulus of elasticity of concrete (kN/m^2);

I—moment of inertia of the section within the investigated height range (m^4);

R_0—radius of the centerline of the ring in the calculation (m);

h—height of the ring (m);

q_{tk}—combination of actions with characteristic values (kN/m^2);

K—stable safety factor, it is taken as 4.

3 The circular diaphragm wall as retaining structure should be analyzed as a spatial structure; it may also be analyzed as a vertical beam on elastic foundation by taking a unit width of the diaphragm wall according to the axisymmetric structure, or analyzed by the method specified in Appendix T of the *Specifications*, where the hooping effect of the wall, the internal ring beam, or the inner lining may be simplified to equivalent elastic supports by considering the diaphragm wall as an axisymmetric structure.

4 The internal forces and deformations of internal ring beams or inner linings may beanalyzed as plane rigid-frame ring beams. In the calculations, the non-uniformity distribution of the strata, groundwater, and ground load shall be considered, and the restraint effect of the soil (in the deformation area of the ring) to the internal ring beams or inner linings shall also be considered.

8.3 Foundations

8.3.1 According to the wall panel unit connection, plane layout and the functional use, diaphragm walls as foundations may be classified into the diaphragm wall as strip foundation, caisson foundation, and a part of other foundations.

8.3.2 The wall bottom shall be embedded into the favorable bearing stratum with large bearing capacity, and the bearing depth of the wall in the bearing stratum shall be greater than the wall thickness. If the bearing stratum is a non-rock bed, the increase of the bearing depth of the wall shall be firstly considered to improve the vertical compressive resistance.

8.3.3 In design of the cross-sectional shape and plane layout of the foundation, the centroid of the foundation should be coincided with the acting point of the resultant force induced by the permanent actions on the superstructure.

8.3.4 Foundation structures shall be designed in accordance with the following requirements for the ultimate limit state and serviceability limit state based on different design conditions:

1 For the ultimate limit state, the following calculation contents shall be included:

1) Calculation of the ground bearing capacity;

2) Calculation of the strength of diaphragm wall structures;

3) Calculation of the strength of the top plate.

2 For the serviceability limit state, the checking calculation of the structural deformation, crack resistance, and crack width of diaphragm walls and top plates shall be included.

8.3.5 When the soil around the foundation is subject to ground settlement due to self-weight consolidation or large-area ground load, the influence of negative skin friction of the wall-side on the vertical compressive resistance and settlement of the wall shall be considered.

8.3.6 The vertical and horizontal resistance of the foundation should be determined through field loading tests.

8.3.7 The detailing of the foundation shall comply with the following requirements:

1 In addition to the thickness of the wall, the structural design of the wall shall comply with the provisions of Items 1 to 5 of Clause 8.2.7 in the *Specifications*.

2 The wall thickness shall be determined by calculation considering the mechanical capacity of the trench and arrangements of the wall panels, and shall not be less than 800 mm. The width of a single cell of caisson diaphragm wall as foundation should not be less than 5 m, and should not be greater than 10 m; the outer peripheral wall and diaphragm should be of the same thickness.

3 Top plate shall be set on the wall top, its concrete strength class shall not be lower than C30. The wall shall be inserted into the top plate 100-200 mm; the vertical reinforcement shall be extended into the top plate with a length no less than the sum of $b/2$ and reinforcement development length l_a (Fig. 8.3.7). It may be not necessary to set a top plate at the wall top for a single-wall type diaphragm.

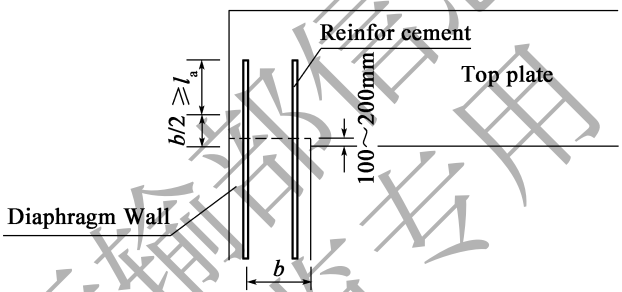


Fig. 8.3.7 Top Plate Structure of Diaphragm Wall

b-the distance from the outer vertical reinforcement to the inner side of the wall

4 The vertical tensile reinforcement shall not be less than 0.3% the effective cross-sectional area of the wall, the horizontal tensile reinforcement shall not be less than 0.2% the effective cross-section area of the wall, and the horizontal reinforcement ratio at the connector locations should not be less than 2 times of the horizontal reinforcement ratio at general parts.

5 Rigid connectors must be used between the outer peripheral wall panels of the diaphragm wall as caisson foundation; the rigid connectors should be used for the internal diaphragm, and the hinged connectors may be used if the condition is not allowed for the rigid connectors.

8.3.8 Reliable methods shall be used for mechanical analysis of a diaphragm wall structure by taking it as a spatial structure.

8.3.9 The calculation of components of diaphragm wall as caisson foundation shall comply with the following requirements:

1 The strength in the vertical box section is calculated according to the internal forces of the section at each depth, which are obtained by spatial structure.

2 The mechanical analysis for the foundation subjected to horizontal forces is carried out by considering it as a rigid frame in plane structure.

3 The top plate is calculated as a slab beam supported on the diaphragm wall, and the supporting effect of soil inside the diaphragm wall is not considered in the calculation. When the depth of the top plate exceeds 0.5 times (simply supported) or 0.4 times (continuous beam) the effective span, it may be calculated as a deep beam.

8.3.10 The wall of the foundation that also serves as the retaining structure of the foundation pit shall comply with the provisions of Section 8.2 of the *Specifications*.

9 Special Grounds and Foundations

9.1 Soft Grounds

9.1.1 In design of bridges and culverts on soft grounds, the requirements for settlement and bearing resistance shall be fully considered, and reasonable superstructure and foundation forms shall be selected; whenever necessary, soft grounds shall be reinforced.

9.1.2 Soft grounds may be reinforced by the methods of replacement layer of sandy gravel, sandy stone piles, and preloading method with sand drain wells, or reinforced according to actual conditions by the methods of cement deep mixing piles, lime piles, vibratory crushed stone piles, hammer compaction, dynamic compaction and grout pouring.

9.1.3 Replacement layer of sandy gravel (sand blanket) may be used to reinforce the shallow layers of mud, muddy soil, hydraulically placed fills, plain fills, and miscellaneous fills. Medium sand, coarse sand, gravelly sand, and crushed (pebble) stone may be used as replacement layer materials. The particle size of these gravel materials should not be greater than 50 mm; the material should not contain impurities such as plant residues, in which the clay content shall not be greater than 5% and the silt content shall not be greater than 25%.

9.1.4 The size of the top surface for a replacement layer of sandy gravel shall be wider than the size of the foundation base by no less than 0.3 m on each side. The depth of replacement layer should not be less than 0.5 m and not larger than 3 m. The size of replacement layer shall also comply with the following requirements:

1 The depth of the replacement layer, z, shall be determined according to the bearing resistance of the underlying soil layer, and shall comply with the requirements of the following formulas:

For strip foundations: $$p_{0i} = \frac{b(p'_{0ik} - \mathrm{p}'_{\mathrm{gi}})}{b + 2z\tan\theta} \tag{9.1.4-2}$$

For rectangular foundations: $$p_{0k} = \frac{b(p'_{0k} - \mathrm{p}'_{\mathrm{gk}})}{(b + 2z\tan\theta)(l + 2z\tan\theta)} \tag{9.1.4-3}$$

Note: The strip foundation is a rectangular foundation with a ratio of length-to-width greater than or equal to 10.

where:

P_{0k}—superimposed compressive stress at the bottom of the replacement layer (kPa);

p_{gk}—compressive stress of soil self – weight at the bottom of the replacement layer (kPa);

f_a—characteristic value of ground bearing capacity at the bottom of the replacement layer (kPa). It is taken according to the provisions of Clause 4.3.4 or 4.3.5;

b—width of bottom surface of rectangular or strip foundation (m);

l—length of the bottom surface of rectangular foundation (m);

p'_{0x}—compressive stress on the bottom surface of the foundation (kPa);

$p'_{\text{水}}$—compressive stress of soil self-weight at the foundation base (kPa);

z—depth of the replacement layer under the foundation base (m);

θ—pressure spreading angle of the replacement layer (°), which may be taken according to Table 9.1.4.

Table 9.1.4 Pressure Spreading Angle of the Replacement layer, θ

Material of replacement layer	Medium sand, coarse sand, gravelly sand, round gravel, breccia, cobbles, gravel	
z/b	0.25	≥0.5
θ(°)	20	30

Note: when $0.25 < z/b < 0.5$, the value of θ may be determined by interpolation; when $z/b < 0.25$, $\boldsymbol{\theta}$ is taken as 0°.

2 The width of the replacement layer shall meet the requirements for pressure spreading of the foundation base. It may be determined according to the following formula or local experience.

$$b_1 \geqslant b + 2z\tan\theta \tag{9.1.4-4}$$

where:

b_1—width of the bottom surface of the replacement layer (m);

θ—pressure spreading angle of the replacement layer, which may be taken according to Table 9.1.4. When $z/b < 0.25$, it may be taken from Table 9.1.4 according to $z/b = 0.25$.

9.1.5 The characteristic value of the bearing resistance of the replacement layer, f_a, should be determined by field tests. If the test data are unavailable, it may be taken according to Table 9.1.5.

Table 9.1.5 Characteristic Values of Bearing Resistance of Various Replacement Layers, f_a

Construction method	Material of replacement layer	Compaction coefficient, λ_c		Characteristic value of bearing resistance, f_a (kPa)
		Heavy compaction test	Light compaction test	
Rolling, vibrating or compact	Gravel, cobbles	≥0.94	≥0.97	200 ~ 300
	Sand androck (gravel and cobbles account for 30%-50% of the total mass)			200 ~ 250
	Soil and stone (gravel and cobbles account for 30%-50% of the total mass)			150 ~ 200
	Medium sand, coarse sand, gravelly sand			150 ~ 200

Note: 1. The compaction coefficient λ_c is the ratio of the controlled dry density of the soil, ρ_d, to the maximum dry density $\rho_{d,m}$.

2. The maximum dry density of the soil should be determined by compaction test; the maximum dry density may be taken as 2000 ~ 2200 kg/m^3.

9.1.6 The settlement of the ground with a replacement layer of sandy gravel may be calculated using the following formulas:

$$s = s_{cu} + s_s \tag{9.1.6-1}$$

$$s_{cu} = p_m \cdot \frac{z}{E_{cu}} \tag{9.1.6-2}$$

where:

s—settlement of ground with replacement layer of sandy gravel (mm);

s_{cu}—compressive deformation of the replacement layer itself (mm);

s_s—settlement value of the underlying layer (mm), which may be calculated according to the provisions of Clauses 5.3.4 ~ 5.3.7 in the *Specifications*;

P_m—average compressive stress in the replacement layer (MPa), that is, the average value of the compressive stresses at the top and at the bottom of the replacement layer of sandy gravel;

z—depth of the replacement layer of sandy gravel (mm);

E_{cu}—compressive modulus of the replacement layer of sandy gravel (MPa). If the test data are unavailable, it may be taken as 12 ~ 24 MPa.

9.1.7 The sandy stone piles may be used to reinforce the ground of loose sand, plain fill, or miscellaneous fill. They may also be used to treat the saturated clay ground for which settlement should not be controlled. Application of sandy stone piles for ground reinforcement shall comply with the following requirements:

1 The ground width compacted by sandy stone piles shall be larger than the foundation width. It should have 1-3 rows of sandy stone piles arranged in the widened width at each side. If sandy stone piles are used to prevent sand liquefaction, the widened width of each side should not be less than 1/2 of the treatment depth and also should not be less than 5 m; if the liquefiable layer is covered with a non-liquefiable layer with a depth of more than 3 m, the widened width of each side should not be less than 1/2 of the depth of the liquefiable layer, and also shall not be less than 3 m.

2 The diameter of the sandy stone pile should be determined according to the properties of the ground soil and the pile forming equipment. The diameter should be 0.3 m ~ 0.8 m, and a large diameter should be selected for saturated cohesive soil ground.

3 Gravelly sand, coarse sand, medium sand, round gravel, breccia, cobbles, gravels, etc. should be used as fillers in the sandy stone piles. The fillers shall not have mud content greater than 5%, and should not contain granules with a particle size greater than 50 mm.

9.1.8 The center-to-center distance of sandy stone piles shall be determined by field test, but should not be greater than 4 times the diameter of sandy stone piles. Sandy stone piles should be arranged as shown in Fig. 9.1.8, and the center-to-center distance may be estimated according to the following formulas:

Arrangement in equilateral triangle shape: $$l_s = 0.95d\frac{\sqrt{(1+e_0)}}{e_0 - e_1} \tag{9.1.8-1}$$

Arrangement in square shape: $$l_s = 0.90d\frac{\sqrt{(1+e_0)}}{e_0 - e_1} \tag{9.1.8-2}$$

$$e_1 = e_{\max} - D_{r1}(e_{\max} - e_{\min}) \tag{9.1.8-3}$$

where:

l_s—center-to-center distance of sandy stone piles (m);

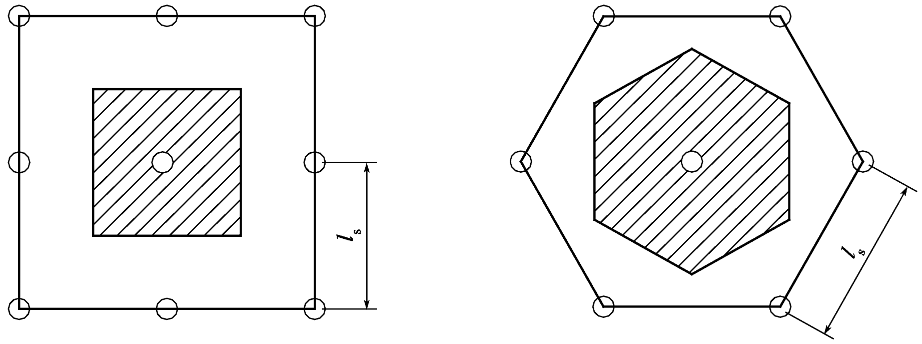
d—diameter of sandy stone pile (m);

e_0—void ratio of sand before ground treatment, which may be determined according to the undisturbed soil sample test, or according to comparative tests (like comparison of dynamic test and static cone penetration test);

e_1—required void ratio after the ground is compacted;

e_{max} and e_{min}—maximum and minimum void ratio of sand, respectively;

D_{r1}—required relative density after the ground is compacted, which may be taken as 0.70 ~ 0.85.



a)Square b)Equilateral triangle

Fig. 9.1.8 Layout and Center-to-center Distance of Sandy-Stone Piles

9.1.9 The preloading method with sand drain wells may be used for the treatment of saturated cohesive soil grounds with muddy soil, mud, or hydraulically placed fills. The diameter of the ordinary sand drain well, d_w, may be 300 mm ~ 500 mm; the diameter of the bagged sand drain well, d_w, may be 70 mm ~ 100 mm; the equivalent transformed diameter of prefabricated vertical drains may be calculated as follows:

$$D_p = \alpha \frac{2(b+\delta)}{\pi} \qquad (9.1.9)$$

where:

D_p—equivalent transformed diameter of the prefabricated vertical drain (mm);

α—transformed factor, α = 0.75 ~ 1.00 may be taken if the test data are unavailable;

b—width of a prefabricated vertical drain (mm);

δ—thickness of prefabricated vertical drain (mm).

9.1.10 The plane layout of sand drain wells may be arranged in equilateral triangle or square shape. The center-to-center distance of the sand drain wells, l_s, may be calculated according to the following formulas:

Arrangement in equilateral triangle shape: $l_s = \frac{d_e}{1.05}$ (9.1.10-1)

Arrangement in square shape: $l_s = \frac{d_e}{1.13}$ (9.1.10-2)

$$d_e = nd_w \qquad (9.1.10\text{-}3)$$

Ordinary sand drain well: $n = 6 \sim 8$ (9.1.10-4)

Bagged sand drain well or prefabricated vertical drain: $n = 15 \sim 20$ (9.1.10-5)

where:

d_e—effective diameter of the drainage cylinder by a sand drain well (mm);

d_w—diameter of sand drain well (mm), which is specified in Clause 9.1.9 of the *Specifications*;

n—ratio of the effective diameter of the drainage cylinder by a sand drain well to the diameter of the sand drain well, d_e / d_w.

9.1.11 The depth of the sand drain well shall be determined according to the following provisions based on the requirements for the stability and deformation of the ground required by the bridge and culvert structures:

1 For structures dominated mainly by the stability against sliding of the ground, the depth of the sand drain well shall exceed the most dangerous sliding surface by at least 2m.

2 For bridges and culverts controlled by settlement, if the depth of the compressed soil layer is not large, the sand drain well should thoroughly penetrate the compressible layer.

3 For bridges and culverts dominated by settlement, if the depth of the compressible soil layer is large, the depth of the sand drain well shall be determined according to the deformation value to be reached in the preset time for preloading.

9.1.12 The sand material for sand drain wells should be medium-coarse sand, and the mud content of the sand shall be less than 3%.

9.1.13 If the preloading method with sand drain wells is used to treat the ground, a drainage layer of sandy gravel shall be laid on the earth surface, and the method should be used in accordance with the following requirements:

1 The depth should be greater than 400 mm.

2 The sand in the drainage layer of sandy gravel should be medium-coarse sand with a mud content less than 5%, itis allowable for the sand to have a small amount of gravel with a particle size less than 50 mm. The dry density of the drainage replacement layer of sandy gravel should be greater than 1500 kg/m^3.

3 A blind ditch connected with the drainage replacement layer of sandy gravel should be set in the preloading zone, and the water discharged from the foundation should be drawn out of the preloading zone.

9.2 Collapsible Loess Grounds

9.2.1 The collapsibility of loess shall be determined by the collapsibility coefficient δ_s. The δ_s may be determined by the following provisions:

1 According to the indoor compression test, δ_s may be calculated using the equation as follows:

$$\delta_s = \frac{h_\mathrm{p} - h'_\mathrm{p}}{h_0} \qquad (9.2.1)$$

where:

δ_s—collapsibility coefficient;

h_p—height of the soil sample after its settlement is stable (mm), in which the sample has been pressurized to the specified pressure while its natural humidity and structure are maintained as original one;

h'_p—height of the soil sample after the additional settlement is stable (mm), in which the sample is first pressurized to a stable state and then immersed in the water (to be saturated);

h_0—original height of the soil sample (mm).

2 The pressure for determining the collapsibility coefficient, δ_s, may be taken according to the following regulations:

1) For bridge and culvert foundations with compressive stress at the foundation base no greater than 300 kPa, the pressure may be taken as 200 kPa for the soil layers from the foundation base to a depth of 10 m; the pressure may be taken as the compressive stress of the saturated soil self-weight overlying above the investigated layer for soil layers from 10m depth to the top surface of the non-collapsible soil layer; and it may be taken as 300 kPa if the compressive stress of the self-weight of the saturated soil overlying above the investigated layer is larger than 300kPa.

2) For bridge and culvert foundations with compressive stress at the foundation base larger than 300 kPa, the pressure may be taken as the actual compressive stress.

3) For the newly accumulated loess with high compressibility, the pressure should be taken as 100 ~ 150 kPa for the soil layer within 5 m below the foundation base; the pressure may be taken as 200 kPa for the soil layer 5 m ~ 10 m below the foundation base; for the soil layer from 10m depth to the top surface of the non-collapsible soil layer, the pressure may be taken as the compressive stress of the self-weight of the saturated soil overlying on it.

9.2.2 The self-weight collapsibility coefficient, δ_s, may be calculated using the equation as follows:

$$\delta_{zs} = \frac{h_z - h'_z}{h_0} \qquad (9.2.2)$$

where:

δ_z—self-weight collapsibility coefficient;

h_z—height of the soil sample after its settlement is stable (mm), in which the sample has been pressurized to a value same as the pressure of the self – weight of the saturated soil overlying on the investigated sample while its natural humidity and structure is maintained as original one;

h'_z—height of the soil sample after additional settlement is stable (mm), in which the sample is first pressurized to a stable state and then immersed in the water (to be saturated);

h_0—original height of the soil sample (mm).

9.2.3 The type of ground collapse for foundations of bridges and culverts located in loess areas shall be determined according to self – weight collapse settlement Δ_{zs}. If the self – weight collapse settlement $\Delta_{zs} \leqslant 70$ mm, the ground is not a self – weight collapsible loess; if $\Delta_{zs} > 70$ mm, the ground is a self – weight collapsible loess. The self – weight collapse settlement of the collapsible loess, Δ_{zs}, may be calculated using the equation as follows:

$$\Delta_{zs} = \beta_0 \sum_{i=1}^{n} \delta_{zsi} h_i \tag{9.2.3}$$

where:

Δ_{zs}—self-weight collapse settlement (mm);

δ_{zsi}—self-weight collapsibility coefficient of the i^{th} layer of soil;

h_i—depth of the i^{th} layer soil (mm). The layers from the natural ground level to the top of the non – collapsible loess layer are taken in the calculation, in which the soil layers with a self – weight collapsibility coefficient δ_{zs} less than 0.015 may be ignored;

β_0—modification coefficient varied with the soil quality in different regions, which shall be in accordance with the provisions in the *Code for Building Construction in Collapsible Loess Regions* (GB 50025—2004).

9.2.4 The collapse settlement of the ground below the foundation base, Δ_s, may be calculated using the equation as follows:

$$\Delta_s = \sum_{i=1}^{n} \beta \delta_{si} h_i \tag{9.2.4}$$

where:

Δ_s—collapse settlement of the ground below the foundation base (mm);

δ_{si}—collapsibility coefficient of the i-layer soil accounted from the foundation base, which shall be calculated in accordance with the provisions of Clause 9.2.1 of the *Specifications*;

β—modification coefficient considering the influence of the lateral extrusion or water immersion probability of ground soil. It may be taken as $\beta = 1.5$ for the ground soil within 5 m below the foundation base; $\beta = 1.0$ for ground soil 5 ~ 10 m below the foundation base; and $\beta = 0$ for the ground soil from a depth 10 m below the foundation base to the top of the non – self – weight collapsible loess layer, $\beta = 0$ is also taken for the non – collapsible loess layer. For self – weight collapsible loess layer, β may be taken as β_0 appeared in Formula (9.2.3) of the *Specifications*.

h_i—depth of the i-layer soil below the foundation base (mm), which is accumulated from the foundation base, and to a depth of 10 m below the foundation base (or to the compressed ground layer) for non- self-weight collapsible loess; to the top of non-collapsible loess layer for self-weight collapsible loess. The soil layer with a collapsibility coefficient δ_s (which is δ_{zs} for the soil layer 10 m under the foundation base) less than 0.015 may not be considered in the depth h_i.

9.2.5 The collapsibility level of the collapsible loess ground shall be determined according to Table 9.2.5 based on the self-weight collapse settlement Δ_{zs} and the collapse settlement of the ground below the foundation base, Δ_{zs}.

Table 9.2.5 Collapsibility Level of Collapsible Loess Ground

Collapse type		Non- Self-weight collapsible ground	Self-weight collapsible ground	
Self-weight collapse settlement, Δ_{zs} (mm)		$\Delta_{zs} \leqslant 70$	$70 < \Delta_{zs} \leqslant 350$	$\Delta_{zs} > 350$
Collapse settlement of the ground below the foundation base, Δ_s (mm)	$\Delta_{zs} \leqslant 300$	Ⅰ (slight)	Ⅱ (medium)	—
	$300 < \Delta_{zs} \leqslant 700$	Ⅱ (medium)	Ⅱ (medium) or Ⅲ* (serious)	Ⅲ (serious)
	$\Delta_s > 700$	Ⅱ (medium)	Ⅲ (serious)	Ⅳ (very serious)

Note: In the table, Level Ⅲ* is specified to the soil with a calculated collapse settlement $\Delta_s >$ 600 mm and the calculated self-weight collapse settlement $\Delta_{zs} >$ 300 mm, other cases may be judged as Level Ⅱ.

9.2.6 The bridge and culvert foundations in collapsible loess area should be set on the original ditch beds, structures that can endure large settlements should be adopted; separated foundations shall not be used for culverts.

9.2.7 In the design of a bridge and culvert foundation in a collapsible loess area, corresponding measures and treatment schemes shall be taken according to the collapsibility level of the loess, the structure category, and the water flow characteristics. The measures in Table 9.2.7 may be taken as a reference for foundation treatment, and other reliable measures may also be adopted according to local experiences. As for the structure category, it may be conducted according to the structural importance, structural characteristics, damage degree after being immersed in water (wetting) as well as the difficulty of rehabilitation, as shown in Table 9.2.7-2.

Table 9.2.7-1 Treatment Measures for Grounds in Collapsible Loess Regions

Types and measures		Flow characteristics and collapsible level							
		Constant flow (or large possibility of wetting)				Seasonal flow (or less possibility of wetting)			
		Ⅰ	Ⅱ	Ⅲ	Ⅳ	Ⅰ	Ⅱ	Ⅲ	Ⅳ
A	Measures	①				①			
B	Measures	②、③	②、③	①、②	①	③		②、③	②
	Treatment depth (m)	2.0 ~ 3.0	3.0 ~ 5.0	4.0 ~ 6.0	6.0	0.8 ~ 1.0	1.0 ~ 2.0	2.0 ~ 3.0	5.0
C	Measures	③			②	③			
	Treatment depth (m)	0.8 ~ 1.0	1.0 ~ 1.5	1.5 ~ 2.0	3.0	0.5 ~ 0.8	0.8 ~ 1.2	1.2 ~ 2.0	2.0
D	Measures	④				④			

Note: In the table, ①, ②, ③ and ④ are the serial numbers of the measures, which are explained as follows: ① Open excavation spread footings, caissons or pile foundations are adopted as the foundations of the piers or abutments, taking the non-collapsible soil layers as the foundation beds; ② Dynamic compaction method or pile compaction method are adopted for the ground, and waterproof measures as well as measures through structure design are also adopted; ③ Compaction by heavy hammer is adopted, and waterproof measures as well as measures through structure design are also adopted; ④ Ground surface is compacted.

Table 9.2.7-2 Category of Structures in Collapsible Loess Area

Category	Structure
Category A	High piers or abutments with a height equal to or larger than 20 m, or external statically indeterminate bridges
Category B	General foundation of bridge, arch culvert
Category C	General culvert and inverted siphon
Category D	Auxiliary of bridge and culvert

9.2.8 The range of the river bed pavements and the vertical skirts for bridges and culverts in collapsible loess regions shall be appropriately larger and deeper than those for bridges and culverts in non-collapsible regions. The settlement joints of culverts shall be treated with waterproof seal.

9.3 Steep Slope Grounds and Foundations

9.3.1 Steep slope ground may refer to the ground with a slope rate no less than 1:2.5 where the bridge and culvert foundation is located, or with a slope that is prone to slippage deformation though the slope rate is less than 1:2.5. In the design of the steep slope ground and foundation, not only the requirements stipulated in this section but also those in other sections of the *Specifications* shall be complied with.

9.3.2 The design of steep slope ground shall meet the following requirements:

1 If a foundation is setin a steep slope ground, the stability and deformation of the ground

that bears the loads transmitted from the foundation shall be analyzed. The stability safety factors of the ground under different working conditions shall not be lower than the values specified in Table 9.3.2.

Table 9.3.2 Requirements for the Stability Safety Factors of the Grounds Under Different Working Conditions

Working condition	Ground soil state	Safety factor
Normal working condition	Natural state	1.35
Abnormal working condition Ⅰ	The state subjected to heavy rain or continuous rain	1.2
Abnormal working condition Ⅱ	The state subjected to heavy rain or continuous rain, or earthquake	1.1

2 In analyzing the stability of the steep slope ground, the beneficial effect of the pile foundation on the stability of the steep slope ground should not be considered.

3 If the stability of the steep slope cannot meet the design requirements, the steep slope shall be reinforced in advance.

9.3.3 The design of a foundation on steep slope ground shall meet the following requirements:

1 In addition to meeting the requirements in Section 5.1 of the*Specifications*, the bearing depth of the foundation on steep slope soil ground shall also comply with the following formula (Fig. 9.3.3):

$$H \geqslant 3d\tan\beta \tag{9.3.3}$$

where:

H—bearing depth of the foundation (m);

d—width or diameter of the foundation (m);

β—slope degree (°).

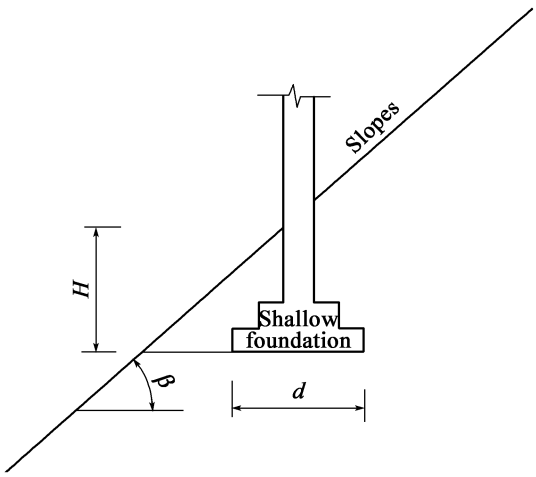


Fig. 9.3.3 Schematic Diagram of Steep Slope Ground and Foundation

2 If the deformation of the steep slope has an influence on the bridge foundation, the deformation and internal forces of the bridge foundation under the action of the slope deformation should be analyzed, and the deformation and bearing resistance of the foundation under the most unfavorable conditions shall be checked.

3 In determining thelateral pressure of the slope acting on the foundation, the stability and deformation characteristics of the slope shall be considered; in analyzing the deformation and internal force of foundation structure, the slope actions on it should be considered, coupling effects of vertical and horizontal forces carried by piles should also be considered.

4 In calculating the internal force and displacement of the pile foundation on the steep slope, the reduction of the horizontal earth resistance within a certain depth below the ground in the slope toe side where the pile foundation is located shall be considered. The specific values may be determined according to local experiences.

9.4 Karst Grounds and Foundations

9.4.1 Design scheme for grounds and foundations of bridges and culverts in karst regions shall be selected in accordance with the following principles:

1 The design scheme of the bridge and culvert foundation in a karst region shall be selected by comprehensively considering the requirements for structural resistance, requirements for structural deformation, and the spatial-temporal characteristics of underground cavity development.

2 According to the design scheme of the ground and foundation of bridges and culverts, the influence on the environment of the existing earth surface water and groundwater shall be evaluated. Significant changes in the hydrological environment in the area induced by the construction of the bridge or culvert foundation shall be avoided.

9.4.2 The grounds for bridges and culverts in karst regions shall be designed in accordance with the following principles:

1 The stability of grounds for bridges and culverts in karst regions shall be evaluated. The evaluation should follow the principles that the bearing resistance of the karst cave roof is dominant controlling factor while the deformation is the supplement controlling factor.

2 If the evaluation results of the karst cave or soil cave roofs cannot reach a 'stable' state, the grounds and foundations of bridges and culverts shall be treated. The treatment schemes of grounds and foundations shall be proposed according to the local environment, and the hydrology characteristics of the earth surface water and groundwater. It is more appropriate to guide the karst groundwater than to block it, and excessive changing the environment of the original groundwater shall be avoided.

9.4.3 The shallow foundations of bridges or culverts in karst regions shall be designed in accordance with the following requirements:

1 For small bridges and culverts located in karst regions where the karst is mainly developed in the vertical direction, shallow foundations should be selected. The depth of the rock or soil layer below the shallow foundation base shall be greater than 3 times the foundation width and shall not be less than the width of the underlying karst cave; otherwise, the size of the foundation base shall be larger than the plane size of the cave with sufficient supporting length, if this requirement can not be met, the underlying karst cave shall be treated.

2 Reinforced monolithic slab footing shall be selected as the shallow foundation in karst region to decrease the stresses on the foundation base as much as possible, to improve the capacity of the foundation for resisting the uneven deformation of the ground.

3 For ground mixed with soil and rock stone, a gravel mattress layer shall be set at the bottom of the shallow foundation, and the depth of the mattress layer shall not be less than 1.0 m.

4 For a shallow foundation spans the underlying karst cave, the calculation for its flexible resistance under adverse supporting conditions shall be added in the design.

5 If a shallow foundation is located on the bedrock but is close to lapies, karst ditch, karst fissure, karst funnel, etc., the overall stability of the foundation shall be analyzed and targeted treatment for the foundation shall be applied because the rock strata under the foundation is possible to slide toward the free face.

6 If the foundation is laid on the inclined rock layer, the foundation sliding shall be analyzed. If stability against sliding can not meet the design requirement, treatment measures shall be taken for the foundation.

9.4.4 The pile foundations of bridges and culverts in karst area shall be designed in accordance with the following requirements:

1 For the pile foundation of a bridge in karst area, the elevation of the pile tip shall be setin the karst cave roof with a certain depth. The depth of the karst cave roof should not be less than 3 times of pile diameter.

2 In determining the elevation of the pile tip, the rock-socketed depth of a pile shall be as small as possible, to ensure the integrity of the cave roofon the premise of meeting the requirements for bearing resistance and minimum rock-socketed depth. The minimum rock-socketed depth of a pile may be taken as 0.5 m.

3 If the pile top deformations of the piles in the same pier or abutment are quite different, the sum of compressive resistance of the piles shall meet the requirements of the superstructure. Moreover, the influence of the deformation differences of the piles on the superstructure shall be analyzed. If necessary, measures shall be taken to coordinate the vertical loads and deformations of the piles.

4 In calculating the vertical compressive resistance of the pile passing through the karst cave, whether to consider the side skin friction of the rock and soil above the karst cave roof that is penetrated by the pile shall be determined according to factors such as the pile-soil interface condition and the stability of the cave roof; in checking the horizontal resistance, the horizontal resistance of the rock and soil above the karst cave roof that is penetrated by the pile may be considered.

9.5 Foundation of Pile with Expanded Branches and Plates

9.5.1 In the design of a pile with expanded branches and plates, factors including the applicable soil layers for the branches and plates, the loading characteristics, the requirements for deformation, the construction equipment, the construction quality control, and social and economic benefits shall be comprehensively considered.

9.5.2 The arrangement and structure detailing of branches and plates in the piles with expanded branches and plates shall meet the following requirements (Fig. 9.5.2):

1 The branches and plates should be set in plastic, stiff plastic, stiff cohesive soil, medium-dense sand or gravelly soil, or compact sand or gravelly soil. They should not be set in soft soil, swelling soil, frozen soil, and liquefiable soil.

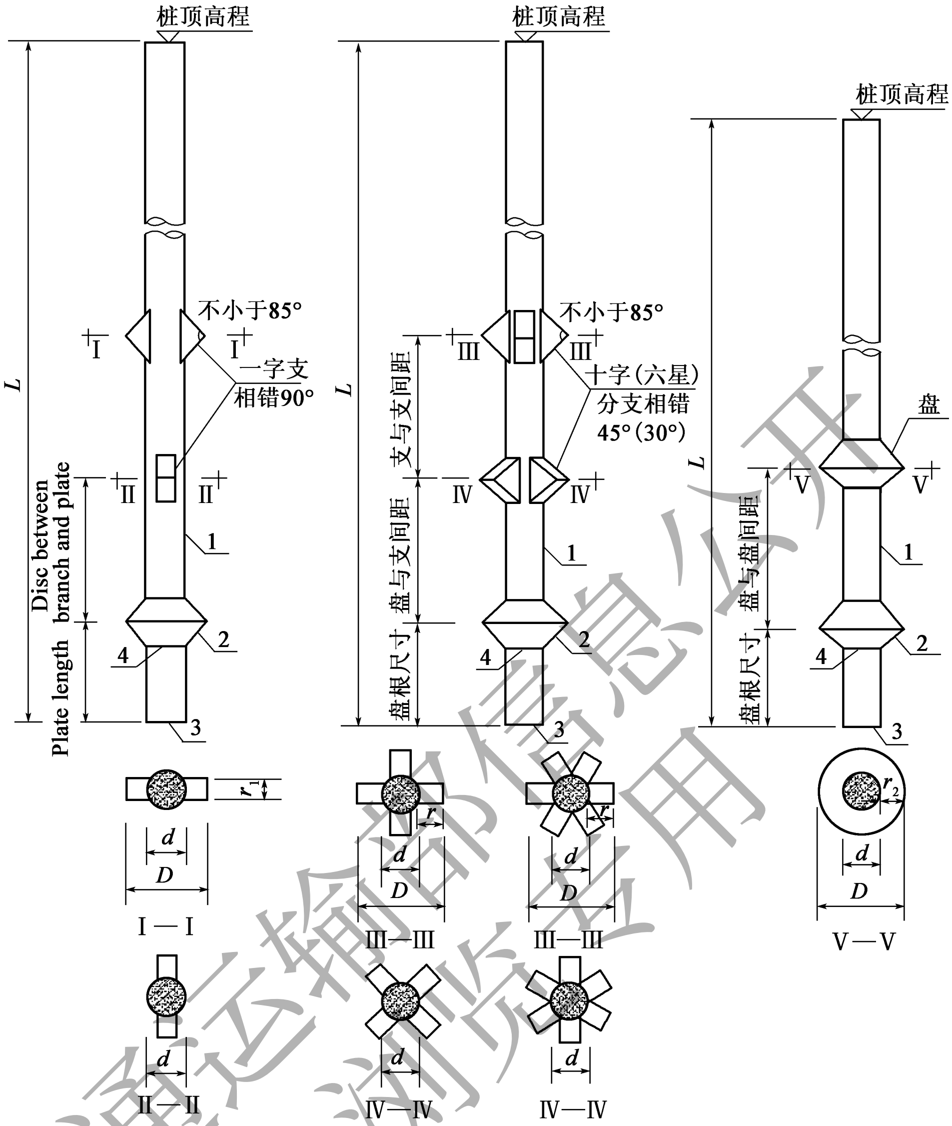


Fig. 9.5.2 Arrangement Scheme of Branches and Plates

1-main pile; 2-bearing plate; 3- pile tip; 4-plate bottom; d-pile diameter; D-branch diameter; L-pile length; r-branch length; r_1-branch width; r_2-plate ring width

2 The length of the branch or the width of the plate ring should be determined according to the design diameter of the pile, the construction technology, and equipment. It may be 300 ~ 1000 mm or half of the design diameter of the pile.

3 A certain number of standby positions for branches and plates shall be reserved according to geological conditions and structural requirements.

4 The thickness of the appropriate bearing stratum for the expanded plate should be greater than 6 times the width of the plate ring; the thickness of the appropriate bearing stratum for the expanded branch should be greater than 4 times the branch length.

5 The depth of the expanded branch or plate embedded into the bearing stratum should be greater than 1.0 time the depth of the branch or plate generally, and it should be greater than 0.5 times the depth of the branch or plate in stiff soils, such as gravelly soil, highly weathered rock, and softening rock. If there is a weak underlying layer, the distance between the lowest bottom of the branch or the plate and the top surface of the weak underlying layer should not be less than 9 times the length of the branch or the width of the plate ring.

6 The minimum vertical distance between plates or between plate and branch should not be less than 8 times the plate ring width or branch length. The minimum vertical distance between the in-line branches should not be less than 3 times the branch length when they are arranged in stagger with 90°. The minimum vertical distance between cross branches should not be less than 4 times the branch length when they are arranged in stagger with 45°. The minimum vertical distance between six-star branches should not be less than 5 times the branch length when they are arranged in stagger with 30°.

7 The first branch or plate from the ground should be set below the position where the bending moment and shear force of the pile is zero.

9.5.3 The center-to-center distance between the piles with expanded branches and plates shall meet the requirements for the center-to-center distance of friction drilled piles in Chapter 6 of the *Specifications*. The center-to-center distance shall also not be less than 1.5 times the diameter of the expanded branch, in which the largest diameter shall be taken as the pile diameter if the pile has a variable diameter.

9.5.4 The characteristic value of axial compressive resistance of a single pile with expanded branches and plates, R_a, may be calculated according to the following formulas:

$$R_{\mathrm{a}} = \frac{1}{2\mathrm{u}}\sum_{i=1}^{m} q_{ik} l_i + \sum_{j=1}^{n} A_{pj} q_{rj} + A_p q_r \tag{9.5.4-1}$$

$$q_{rj} = m_0 \lambda [f_{cj} + k_2 \gamma_2 (h_j - 3)] \tag{9.5.4-2}$$

where:

l_i—depth of the i^{th} layer of soil below the bottom of the pile cap or the local scour line (m). When there are branches and plates in the soil, 1.5 times the height of each branch and plate shall be subtracted;

A_{pj}—area of the j^{th} branch or plate (the area of the main pile is deducted) (m^2);

q_{rj}—characteristic value of bearing resistance of soil at the end of the j^{th} branch or plate (kPa);

n—total number of branches and plates;

f_{aj}—characteristic value of bearing resistance of soil at the branch and plate (kPa). It is determined in accordance with Clause 4.3.3;

Other symbols can be found in Clause 6.3.3, and their values shall comply with the provisions provided in the clause.

9.5.5 The resistance of the pile with expanded branches and plates shall be further verified by comparing the information collected in the construction process (to expand and compact the cavity of the branches and the plates) and the geological survey data. Whenever necessary, the number of branches and plates shall be increased by using the reserved positions for standby branches and plates.

9.5.6 The characteristic value of the axial tension resistance of the pile with expanded branches and plates may be calculated by the following formula, in which the arrangement and structure detailing of the bearing stratum above the branches and plates shall comply with the requirements in Clause 9.5.2.

$$R_t = 0.3u\sum q_{ik}l_i + 0.8\sum A_{pj}q_{rj} \tag{9.5.6}$$

where:

R_t—characteristic value of the axial tensile resistance of the single pile (kN).

9.5.7 In calculating the horizontal resistance of the pile with expanded branches and plates, the contribution of branches and plates may not be considered.

Appendix A
Geotechnical Classification of Grounds in Bridges and Culverts

A. 0. 1　Rock ground of bridge and culverts may be classified by the rock mass rating of strength, weathered degree and intactness degree, as shown in Table A. 0. 1-1 ~ Table A. 0. 1-3.

Table A. 0. 1-1　Qualitative Classification of Rock Mass Rating of Strength

Rock Mass Rating of Strength	Qualitative identification		Rock type
Strong rock	Very Strong rock	When hammered, it sounds very crispy, the hammer will spring back rapidly, and the hand feels a shock. The rock is very difficult to be crushed by hammer blow. The rock has no reaction of water absorption.	Unweathered to slightly weathered granite, diorite, diabase, basalt, andesite, gneiss, quartzite, quartzy sandstone, siliceous conglomerate, siliceous limestone, etc.
	Moderately strong rock	When hammering, it sounds crispy, the hammer will spring back slightly, and the hand feels a slight shock. The rock is very difficult to be crushed by hammer blow. The rock has a slight reaction of water absorption.	1. Slightly weathered strong rock; 2. Unweathered to slightly weathered marble, slate, limestone, dolomite, calcareous sandstone, etc.
Weak rock	Moderately weak rock	When hammering, it sounds non-crisp and the hammer will not spring back. The rock is easy to be crushed by a hammer blow. The rock can be imprinted with a nail after being immersed in water.	1. Moderately weathered to highly weathered strong rock or moderately weak rock; 2. Unweathered to slightly weathered tuff, phyllite, marlite, sandy mudstone, etc.
	Very weak rock	When hammering, it sounds dumb and the hammer will not spring back. Dents appear in the rock after hammering. The rock is easy to be crushed by a hammer blow. The rock can be broken by hands after being immersed in water.	1. Highly weathered strong rock or highly weathered moderately strong rock; 2. Moderately weathered to highly weathered weak rock; 3. Unweathered to slightly weathered shale, mudstone and argillaceous sandstone, etc.

continued

Rock Mass Rating of Strength	Qualitative identification	Rock type
Extremely weak rock	When hammering, it sounds dumb and the hammer will not spring back. Deep dents appear in the rock after hammering. The rock can be crushed between the finger and thumb and may be kneaded into a ball after being immersed in water.	1. Various completely weathered rocks; 2. Various hypabyssal rocks.

Table A.0.1-2 Classification of Rock Mass Based on Weathering

Weathering degree	Field characteristics	Coefficient index for the degree of weathering	
		Wave velocity ratio, kv	Weathering coefficient, kf
Unweathered	Rock is fresh, only a few weathered traces can be seen occasionally.	0.9 ~ 1.0	0.9 ~ 1.0
Slightly weathered	Rock structure is essentially unchanged, only joint planes are rendered or slightly discolored, with a few weathering fissures.	0.8 ~ 0.9	0.8 ~ 0.9
Moderately weathered	Rock structure has been partially damaged with secondary minerals distributed along joint planes; the weathering fissure is developed, and rock mass is cut into rock blocks. The rock is difficult to dig by picks except by core driller.	0.6 ~ 0.8	0.4 ~ 0.8
Highly weathered	Most of the rock structures have been damaged. Mineral components have been obviously changed. Weathering fissures are well-developed. The rock mass is broken and can be dug with a pick, but it is still difficult to drill by dry drilling.	0.4 ~ 0.6	< 0.4
Completely weathered	Almost all rock structures have been damaged but the rock can still be identified. It has residual structural strength, and the rock can be dug with a pick and drilled by dry drilling.	0.2 ~ 0.4	—
Residual soil	All the rock structures are destroyed and have been weathered into the soil state. It is easy to be dug with a spade or pick and to be drilled by dry drilling. It shows a certain plasticity.	<0.2	—

Note: 1. Wave velocity ratio k_v refers to the ratio of compressional wave velocity of weathered rock to that of fresh rock;
2. Weathering coefficient k_f refers to the ratio of uniaxial compressive strength of weathered rock to that of fresh rock;
3. Besides the field characteristics and quantitative index as listed in the table, the weathering degree of rock may also be classified by the local experiences;
4. The granite-type rocks may be divided into highly weathered, completely weathered, and residual soil by standard penetration test;
5. Mudstone and hypabyssal rocks may not need to be classified according to the weathering degree.

Table A.0.1-3 Qualitative Classification of Rock Mass Based on Intactness

<table>
<tr><th rowspan="2">Intactness</th><th colspan="2">Development degree of Structural plane</th><th rowspan="2">Combination degree of main structural plane</th><th rowspan="2">Type for main structural plane</th><th rowspan="2">Corresponding structure types</th></tr>
<tr><th>Structural plane group</th><th>Average interval (m)</th></tr>
<tr><td>Intact</td><td>1 ~ 2</td><td>> 1.0</td><td>Well combined or normally combined</td><td>Fissure, bedding</td><td>Structure in intact state or with very thick layer</td></tr>
<tr><td rowspan="2">Moderately intact</td><td>1 ~ 2</td><td>> 1.0</td><td>Poorly combined</td><td>Fissure, bedding</td><td>Structure in blockage or with thick layer</td></tr>
<tr><td>2 ~ 3</td><td>1.0 ~ 0.4</td><td>Well combined or normally combined</td><td>—</td><td>Structure in blockage</td></tr>
<tr><td rowspan="3">Moderately fractured</td><td>2 ~ 3</td><td>1.0 ~ 0.4</td><td>Poorly combined</td><td rowspan="3">Fissure, bedding, minor fault</td><td>Structure with fissure block or with medium thick layer</td></tr>
<tr><td rowspan="2">≥3</td><td rowspan="2">0.4 ~ 0.2</td><td>Well combined</td><td>Structure with embedded rupture</td></tr>
<tr><td>Ordinarily combined</td><td>Structure with mediumand thin layers</td></tr>
<tr><td rowspan="2">Very fractured</td><td rowspan="2">≥3</td><td>0.4 ~ 0.2</td><td>Poorly combined</td><td rowspan="2">Various types of structural plane</td><td>Fissuring blocky structure</td></tr>
<tr><td>≤ 2</td><td>Ordinarily or poorly combined</td><td>Fractured Structure</td></tr>
<tr><td>Extremely fractured</td><td>Disordered</td><td>—</td><td>Very poorly combined</td><td>—</td><td>Loose structure</td></tr>
</table>

Note: Average interval refers to the average interval value of the primary structural planes (1-2 groups).

A.0.2 Field identification for the density of gravelly soil shall be comprehensively carried out according to the characteristics specified in Table A.0.2.

Table A.0.2 Field identification for density of gravelly soil

Density	Skeleton particle content and arrangement	Digging ability	Drillability
Loose	The mass of the skeleton particle is less than 60% the total mass. The particles are arranged in disorder. Most of them do not contact with each other.	It can be dug with a spade. The wall of the dug well is liable to collapse. After large particles are taken out, the well wall collapses immediately.	It is easy to drill. During drilling, the drill rod may jump slightly. The wall of the drilled hole is liable to collapse.

continued

Density	Skeleton particle content and arrangement	Digging ability	Drillability
Medium dense	The mass of the skeleton particle equals to 60% ~ 70% of the total mass. The particles are staggered in arrangement. Most of them contact with each other.	It can be dug with a spade and pick, and the blocks may fall from the well wall. The concave can remain on the wall where large particles are removed.	It is moderately difficulty to drill. The drill rod and the sash weigh may jump, but not violently. There is a collapse phenomenon on the wall of the drilled hole.
Dense	The mass of the skeleton particle is larger than 70% of the total mass. The particles are staggered in arrangement. The particles contact with each other continuously.	It is difficult to dig with a spade or a pick and only can be loosened by a crowbar. The wall of the dug well is moderately stable.	It is difficult to drill with violent jumping of drill rod and sash weight in drilling. The wall of the drilled hole is moderately stable.

Appendix B
Essentials for Shallow Plate Loading Test

B.0.1　浅层平板载荷试验可用于确定浅部地基承压板下压力主要影响范围内土层的承载力。

承压板面积不应小于0.25 m^2,特殊情况下应符合下列规定：

1　对软土地基不应小于0.5 m^2。

2　对复合地基不应小于一根桩加固的面积。

3　对强夯处理后的地基,不应小于2.0 m^2。

B.0.1　Shallow plate loading test may be used to determine the bearing resistance of the soil layers within the range mainly affected by the pressure of the bearing plate in shallow grounds. The bearing plate area shall not be less than 0.25 m^2, and shall comply with the following provisions for special circumstances:

1　The bearing plate area shall not be less than 0.5 m^2 for soft soil ground.

2　The bearing plate area shall not be less than the area reinforced by one pile for composite ground.

3　The bearing plate area shall not be less than 2.0 m^2 for the ground after being treated by dynamic compaction.

B.0.2　The width of the testing foundation pit shall not be less than three times the width *b* or diameter *d* of the bearing plate; it shall maintain the original structure and natural humidity of the soil layer to be tested. The loading surface should be leveled up with coarse sand or medium sand with a thickness no thicker than 20 mm.

B.0.3 The loading stages shall not be less than 8. The maximum load shall not be less than the twice required design load.

B.0.4 At each loading stage, the settlement should be measured at intervals of 10 min, 10 min, 10 min, 15 min, and 15 min within the first hour. After that, the settlement should be measured every half hour. When the settlement is less than 0.1 mm per hour within two consecutive hours, the settlement is considered to be stable and the next loading stage can be carried out.

B.0.5 The loading process may be terminated under one of the following conditions:

1 Soil is obviously extruded out at the lateral direction around the bearing plate;

2 Settlement s increases sharply, and a steep drop segment is found in the load-settlement ($p-s$) curve;

3 Under one load stage, the settlement rate can not be stable within 24 hours;

4 The ratio of the settlement to the width or diameter of the bearing plate is greater than or equal to 0.06.

B.0.6 The characteristic value of the ground bearing capacity, f_{a0}, shall be determined in accordance with the following provisions:

1 When there is a proportional limit in the $p-s$ curve, the loading value corresponding to the proportional limit shall be taken as f_{a0};

2 When one of the conditions for termination of loading in the first three items of Clause B.0.5 of the *Specifications* is met, the corresponding load at the previous stage shall be defined as the ultimate load. If the ultimate load is less than 2 times of the corresponding proportional limit load value, half of the ultimate load value is taken as f_{a0}.

3 If it can not be determined by the above two items and the bearing plate area is 0.25 ~ 0.50 m^2, the load value corresponding to s/b (or s/d) = 0.01 ~ 0.015 may be taken as f_{a0}, but its value shall not be larger than half of the maximum applied load.

B.0.7 The number of testing points involved in the statistical analysis in one soil layer shall not be less than three. If the range of the measured values in the test does not exceed 30% of its average value, the average value shall be taken as the characteristic value of the ground bearing capacity of the soil layer, f_{a0}. If the range does not meet the requirements, the cause shall be found, and the statistical units of the ground shall be redivided for evaluation if necessary.

Appendix C
Essentials for Deep Plate Loading Test

C.0.1 Deep plate loading test may be used to determine the bearing resistance of the soil layers within the range mainly affected by the pressure of the bearing plate in the deep ground and at the pile tip for large-diameter pile.

C.0.2 The bearing plate for the deep plate loading test shall adopt a rigid plate with a diameter of 0.8 m, and the depth of the soil layer around the bearing plate shall not be less than 0.8 m.

C.0.3 The load for each loading stage may be applied stepwise by 1/10 ~ 1/15 of the estimated bearing resistance.

C.0.4 At each loading stage, the settlement should be measured at intervals of 10 min, 10 min, 10 min, 15 min, and 15 min in the first hour. After that, the settlement should be measured every half hour. When the settlement is less than 0.1 mm per hour within two consecutive hours, the settlement is considered to be stable and the next loading stage can be carried out.

C.0.5 Loading process may be terminated under one of the following conditions:

1 Settlement s increases sharply. A steep drop that can be used to determine the bearing resistance is found in the load-settlement ($p-s$) curve, and the settlement exceeds 0.04 times the diameter of the bearing plate;

2 Under one load stage, the settlement rate cannot be stable within 24 hours.

3 The settlement at the current stage is 5 times greater than the one at the previous stage;

4 If the soil layer in the bearing stratum is stiff and the settlement is small, the maximum

applied load shall not be less than the twice required design load.

C.0.6 The characteristic value of the ground bearing capacity, f_{a0}, shall be determined in accordance with the following provisions:

1 When there is a proportionate limit in the $p-s$ curve, the load value corresponding to the proportional limit shall be taken as f_{a0};

2 When one of the conditions for termination of loading in the first three items of Clause C.0.5 of the *Specifications* is met, the corresponding load at the previous stage shall be defined as the ultimate load. If the ultimate load is less than 2 times the corresponding proportional limit load value, half of the ultimate load value is taken as f_{a0}.

3 If it can not be determined by the above two items, the load value corresponding to s/b (or s/d) $=0.01\sim0.015$ may be taken as f_{a0}, but its value shall not be more than half of the maximum load applied.

C.0.7 The number of testing points involved in the statistical analysis in one soil layer shall not be less than three. If the range of the measured values in the test does not exceed 30% of its average value, the average value shall be taken as the characteristic value of the ground bearing capacity of the soil layer, f_{a0}. If the range does not meet the requirements, the cause shall be found, and the statistical units of the ground shall be redivided for evaluation if necessary.

Appendix D
Essentials for Loading Test of Rock Ground

D. 0. 1 Loading test of rock ground may be applicable to determine the bearing resistance when an intact, moderately intact, moderately fractured rock grounds are used as a natural ground or bearing stratum of pile foundation.

D. 0. 2 A circular rigid bearing plate with a diameter of 300 mm shall be used. If the rock is deeply buried, a reinforced concrete pile may be used, but measures shall be taken on the pile surface to eliminate the skin frictional force between the pile body and the soil.

D. 0. 3 Before loading, the measurement system shall be read every 10 min. If the values in the consecutive three readings remain unchanged, the measured data is considered to be stable and can be taken as the initial values. Then, the test can be started.

D. 0. 4 Single cyclic loading shall be applied. The load shall be increased stage by stage till fracture of the rock, and then unloaded stage by stage.

D. 0. 5 The load in the first stage is 1/5 the estimated design load, and the incremental load in each of the following stages is 1/10 the estimated design load.

D. 0. 6 Measurement shall be recorded immediately after the loading, and every 10 min thereafter.

D. 0. 7 The value shall be considered stable if the difference of three consecutive measurement is no more than 0. 01 mm.

D. 0. 8 The loading may be terminated under one of the following conditions:

1 The settlement keeps changing and the settlement rate tends to increase within 24 hours;

2 Load cannot be applied or cannot be stable after being applied reluctantly.

Note: Even if the loading capacity is limited for the test, the maximum applied load shall also not be less than twice the load required in the design.

D.0.9 Unloading shall be investigated according to the following requirements:

1 The unloading value for each step can be twice of the loading value. If the loading stage number is odd, the unloading value in the first stage can be the sum of the loading value at the last three stages.

2 After each unloading stage, the spring back value is measured once every 10 min. After three measurements, the next unloading stage can be started.

3 When the spring back value is less than 0.01 mm in half an hour after all load has been removed, it may be considered stable.

D.0.10 The characteristic value of the bearing resistance of rock ground shall be determined in accordance with the following provisions:

1 The endpoint at the first straight-line segment on the $p-s$ curve is the proportionate limit. When the condition for stopping loading is met, the corresponding load at the previous stage is the ultimate load. The small one of the ultimate load divided by safety factor 3 and the load corresponding to the proportionate limit is adapted as the characteristic value of bearing resistance of the rock ground.

2 The number ofloading tests on each site shall not be less than 3, and the minimum value shall be taken as the characteristic value of the bearing capacity of the rock ground.

3 The bearing resistance of rock ground is not modified bybearing depth.

Appendix E
Nominal Frost Depth Diagram and Behavior Classifications of Frozen Soil

E.0.1 Nominal frost depth diagram of seasonally frozen soil may be determined according to Appendix F of *Code for Design of Building Foundation* (GB 50007-2011)

E.0.2 Based on the classification of frost heave, the ground soil of highway bridges and culverts may be classified into: no frost heave, slightly frost heave, moderately frost heave, strong frost heave, and extremely strong frost heave according to Table E.0.2.

Table E. 0.2 Behavior Classification of Frost Heave of Ground Soil in Highway Bridges and Culverts

Soil name	Natural water content before frozen, w (%)	Minimum distance from groundwater level to the design frost depth before frozen, z (m)	Average frost heave factor η (%)	Frost heave grade	Frost heave type
Gravel, gravelly sand, coarse sand, medium sand (the content of grain with grain size less than 0.075 mm are not greater than 15%), fine sand (the content of grain with grain size less than 0.075 mm are not greater than 10%)	Not consider	Not consider	$\eta \leqq 1$	Ⅰ	No frost heave
gravelly soil, gravelly sand, coarse sand, medium sand (the content of grain with grain size less than 0.075 mm are greater than 15%), fine sand (the content of grain with grain size less than 0.075 mm are greater than 10%)	$w \leqq 12$	$z > 1.0$	$\eta \leqq 1$	Ⅰ	No frost heave
		$z \leqq 1.0$	$1 < \eta \leqq 3.5$	Ⅱ	Slightly frost heave
	$12 < w \leqq 18$	$z > 1.0$			
		$z \leqq 1.0$	$3.5 < \eta \leqq 6$	Ⅲ	Moderately frost heave
	$w > 18$	$z > 0.5$			
		$z \leqq 0.5$	$6 < \eta \leqq 12$	Ⅳ	Strong frost heave

continued

Soil name	Natural water content before frozen, w (%)	Minimum distance from groundwater level to the design frost depth before frozen, z (m)	Average frost heave factor η (%)	Frost heave grade	Frost heave type
Fine sand and silty sand	$w \leqq 14$	$z > 1.0$	$\eta \leqq 1$	Ⅰ	No frost heave
		$z \leqq 1.0$	$1 < \eta \leqq 3.5$	Ⅱ	Slightly frost heave
	$14 < w \leqq 19$	$z > 1.0$			
		$z \leqq 1.0$	$3.5 < \eta \leqq 6$	Ⅲ	Moderately frost heave
	$19 < w \leqq 23$	$z > 1.0$			
		$z \leqq 1.0$	$6 < \eta \leqq 12$	Ⅳ	Strong frost heave
	$w > 23$	Not consider	$\eta > 12$	Ⅴ	Extremely strong frost heave
Silty soil	$w \leqq 19$	$z > 1.5$	$\eta \leqq 1$	Ⅰ	No frost heave
		$z \leqq 1.5$	$1 < \eta \leqq 3.5$	Ⅱ	Slightly frost heave
	$19 < w \leqq 22$	$z > 1.5$			
		$z \leqq 1.5$	$3.5 < \eta \leqq 6$	Ⅲ	Moderately frost heave
	$22 < w \leqq 26$	$z > 1.5$			
		$z \leqq 1.5$	$6 < \eta \leqq 12$	Ⅳ	Strong frost heave
	$26 < w \leqq 30$	$z > 1.5$			
		$z \leqq 1.5$	$\eta > 12$	Ⅴ	Extremely strong frost heave
	$w > 30$	Not consider			
Cohesive soil	$w \leqq w_p + 2$	$z > 2.0$	$\eta \leqq 1$	Ⅰ	No frost heave
	$w_p + 2 < w \leqq w_p + 5$	$z \leqq 2.0$	$1 < \eta \leqq 3.5$	Ⅱ	Slightly frost heave
		$z > 2.0$			
	$w_p + 5 < w \leqq w_p + 9$	$z \leqq 2.0$	$3.5 < \eta \leqq 6$	Ⅲ	Moderately frost heave
		$z > 2.0$			
	$w_p + 9 < w \leqq w_p + 15$	$z \leqq 2.0$	$6 < \eta \leqq 12$	Ⅳ	Strong frost heave
		$z > 2.0$			
		$z \leqq 2.0$	$\eta > 12$	Ⅴ	Extremely strong frost heave

Note: 1. w_p—Water content at the plastic limit (%), w—average value of natural water content in the frozen soil layer before frozen.

2. Saline frozen soil is not involved in this classification.

3. If the plasticity index of the soil is greater than 22, its frost heave grade shall be increased by one grade.

4. If the content of particles with a grain size less than 0.005 mm in the soil is greater than 60%, it is non-frost heave soil.

5. If gravelly soil is used as filling material and its mass is more than 40% of the total mass, the frost heave behavior of the ground soil may be evaluated according to filling soil.

E.0.3 The permafrost of ground soil of highway bridges and culverts may be classified into no thaw-subsidence, slightly thaw-subsidence, moderately thaw-subsidence, strong thaw-subsidence, and extremely strong thaw-subsidence (thaw collapse) according to Table E.0.3.

Table E.0.3 Classification Table of Permafrost

Soil name	Water content, w (%)	Average coefficient of thaw-subsidence, δ_0	Thaw-subsidence grade	Thaw-subsidence type	Frozen soil type
Gravel (cobble), gravelly sand, coarse and medium sand (the content of grains with a grain size less than 0.075 mm are not greater than 15%)	$w<10$	$\delta_0 \leqslant 1$	Ⅰ	No thaw-subsidence	Ice-short frozen soil
	$w \geqslant 10$	$1<\delta_0 \leqslant 3$	Ⅱ	Slightly thaw-subsidence	Ice frozen soil
Gravel (cobble), gravelly sand, coarse and medium sand (the content with grain size less than 0.075 mm are greater than 15%)	$w<12$	$\delta_0 \leqslant 1$	Ⅰ	No thaw-subsidence	Ice-short frozen soil
	$12 \leqslant w<15$	$1<\delta_0 \leqslant 3$	Ⅱ	Slightly thaw-subsidence	Ice-short frozen soil
	$15 \leqslant w<25$	$3<\delta_0 \leqslant 10$	Ⅲ	Moderately thaw-subsidence	Ice-rich frozen soil
	$w \geqslant 25$	$10<\delta_0 \leqslant 25$	Ⅳ	Strong thaw-subsidence	Ice-saturated frozen soil
Silty sand, fine sand	$w<14$	$\delta_0 \leqslant 1$	Ⅰ	No thaw-subsidence	Ice-short frozen soil
	$14 \leqslant w<18$	$1<\delta_0 \leqslant 3$	Ⅱ	Slightly thaw-subsidence	Ice frozen soil
	$18 \leqslant w<28$	$3<\delta_0 \leqslant 10$	Ⅲ	Moderately thaw-subsidence	Ice-rich frozen soil
	$w \geqslant 28$	$10<\delta_0 \leqslant 25$	Ⅳ	Strong thaw-subsidence	Ice-saturated frozen soil
Silty soil	$w<17$	$\delta_0 \leqslant 1$	Ⅰ	No thaw-subsidence	Ice-short frozen soil
	$17 \leqslant w<21$	$1<\delta_0 \leqslant 3$	Ⅱ	Slightly thaw-subsidence	Ice frozen soil
	$21 \leqslant w<32$	$3<\delta_0 \leqslant 10$	Ⅲ	Moderately thaw-subsidence	Ice-rich frozen soil
	$w \geqslant 32$	$10<\delta_0 \leqslant 25$	Ⅳ	Strong thaw-Lsubsidence	Ice-saturated frozen soil

continued

Soil name	Water content, w (%)	Average coefficient of thaw-subsidence, δ_0	Thaw-subsidence grade	Thaw-subsidence type	Frozen soil type
Cohesive soil	$w < w_p$	$\delta_0 \leqslant 1$	Ⅰ	No thaw-subsidence	Ice-short frozen soil
	$w_p \leqslant w < w_p + 4$	$1 < \delta_0 \leqslant 3$	Ⅱ	Slightly thaw-subsidence	Ice frozen soil
	$w_p + 4 \leqslant w < w_p + 15$	$3 < \delta_0 \leqslant 10$	Ⅲ	Moderately thaw-subsidence	Ice-rich frozen soil
	$w_p + 15 \leqslant w < w_p + 35$	$10 < \delta_0 \leqslant 25$	Ⅳ	Strong thaw-subsidence	Ice-saturated frozen soil
Soil-containing ice layer	$w \geqslant w_p + 35$	$\delta_0 > 25$	Ⅴ	Extremely Strong thaw-subsidence (Thaw collapse)	Ice layer with soil

Note: 1. The total water content w involves ice and unfrozen water.

2. Thesaline frozen soil, frozen peat soil, humus, and highly plastic cohesive soil are not listed in this table.

Appendix F
Calculations ofSuperimposed Vertical Compressive Stress on Abutment Foundation Base or Pile Tip Plane Caused by the Fills behind the Abutment

F. 0. 1 The superimposed vertical compressive stress (see Fig. F. 0. 1) on the abutment foundation base or pile tip plane caused by backfill behind the abutment shall be calculated in accordance with the following provisions:

1 Thesuperimposed compressive stress on the ground of the abutment foundation base or pile tip plane caused by backfill behind the abutment, p_1, may be calculated by the following formula:

$$p_1 = \alpha_1 \cdot \gamma_1 \cdot H_1 \tag{F. 0. 1-1}$$

2 For spill-through abutments, thesuperimposed compressive stress on the front edge of the abutment foundation base or pile tip plane caused by the front cone, p_2, may be calculated by the following formula:

$$p_2 = \alpha_2 \cdot \gamma_2 \cdot H_2 \tag{F. 0. 1-2}$$

3 The total stress of the ground edge on the abutment foundation base or pile tip plane is the sum ofp_1 and p_2 as well as stresses caused by other loads.

Symbols in formula F. 0. 1-1, F. 0. 1-2 andFig. F. 0. 1:

α_1, α_2—coefficient of superimposed vertical compressive stress, see Table F. 0. 1-1 and Table F. 0. 1-2;

γ_1—unit weight of backfill (kN/m^3);

γ_2—unit weight of cone fill (kN/m^3);

H_1—height of backfill behind the abutment (m);

H_2—height of the cone at the front edge of the foundation base or the pile tip plane (m), which is taken as the distance from the original ground to the intersection point of the cone slope line and the vertical line of the front edge at the foundation base;

P_1—soil pressure in the original ground caused by backfill behind the abutment (kPa);

P_2—soil pressure on the front edge of the abutment foundation base or pile tip plane caused by the front cone (kPa);

b_a—foundation length between the front and back edges at foundation base or pile tip plane (m);

h—depth from the original ground to the foundation base or pile tip plane (m), which is also the bearing depth of the foundation.

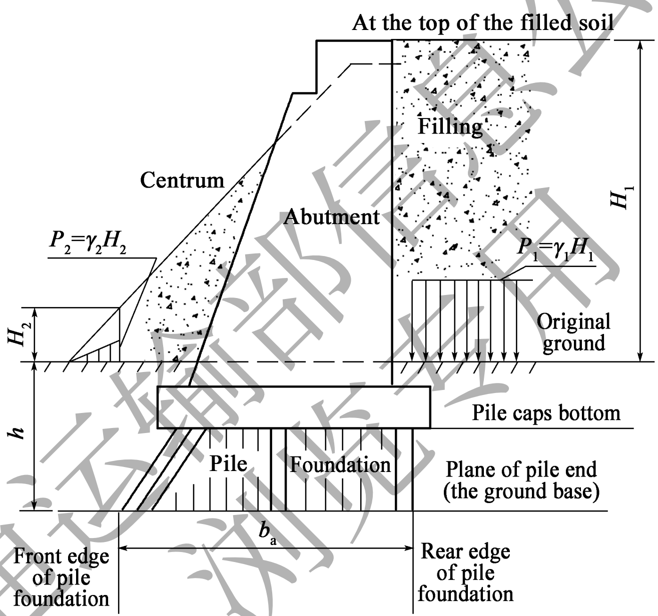


Fig. F.0.1 Superimposed Compressive Stress of Abutment Foundation Base Caused by Backfill behind the Abutment

Table F.0.1-1 Table of Coefficient, α_1

Bearing depth of foundation, h (m)	Height of backfill, H_1 (m)	Abutment edge			
		Back edge	Front edge, according to the foundation length at the foundation base plane, b_a (m)		
			5	10	15
5	5	0.44	0.07	0.01	0
	10	0.47	0.09	0.02	0
	20	0.48	0.11	0.04	0.01

continued

Bearing depth of foundation, h (m)	Height of backfill, H_1 (m)	Abutment edge			
		Back edge	Front edge, according to the foundation length at the foundation base plane, b_a (m)		
			5	10	15
10	5	0.33	0.13	0.05	0.02
	10	0.40	0.17	0.06	0.02
	20	0.45	0.19	0.08	0.03
15	5	0.26	0.15	0.08	0.04
	10	0.33	0.19	0.10	0.05
	20	0.41	0.24	0.14	0.07
20	5	0.20	0.13	0.08	0.04
	10	0.28	0.18	0.10	0.06
	20	0.37	0.24	0.16	0.09
25	5	0.17	0.12	0.08	0.05
	10	0.24	0.17	0.12	0.08
	20	0.33	0.24	0.17	0.10
30	5	0.15	0.11	0.08	0.06
	10	0.21	0.16	0.12	0.08
	20	0.31	0.24	0.18	0.12

Note: The embankment considered in the Table is cohesive soil.

Table F.0.1-2 Table of Coefficient, α_2

Bearing depth of foundation, h (m)	Height of backfill behind abutment, H_1 (m)	
	10	20
5	0.4	0.5
10	0.3	0.4
15	0.2	0.3
20	0.1	0.2
25	0	0.1
30	0	0

Appendix G
Calculation of Stress Redistribution on Rock Ground with Rectangular Section Subjected to Biaxial Eccentric Compression or with Circular Section Subjected to Eccentric Compression

G. 0. 1 The stress of rock ground with rectangular section subjected to biaxial eccentric compression after consideration of the stress redistribution may be calculated by the following formula if data is unavailable:

$$p_{\max} = \lambda \frac{N}{A} \tag{G.0.1}$$

where:

λ—obtained from Fig. G. 0. 1 according to e_y/d and e_x/b;

N—axial force on the section;

A—foundation base area;

e_x, e_y—eccentricity of N in x and y direction, respectively;

b, d—width and height of the section in the x and y direction, respectively.

G. 0. 2 If the eccentricity $n > 0.125$, the maximum stress of the circular section subjected to eccentric compression after consideration of the stress redistribution may be calculated by the following formulas:

$$p_{\max} = \lambda \frac{N}{A} \tag{G.0.2-1}$$

$$n = \frac{e}{d} \tag{G.0.2-2}$$

where:

N—axial force on the section (N);

A—foundation base area (mm^2);

e—eccentricity (mm);

d—circular section diameter (mm);

n—ratio of eccentricity to circular section diameter;

λ—revised coefficient for characteristic value of bearing capacity, which can be taken from Table G.0.2 according to n.

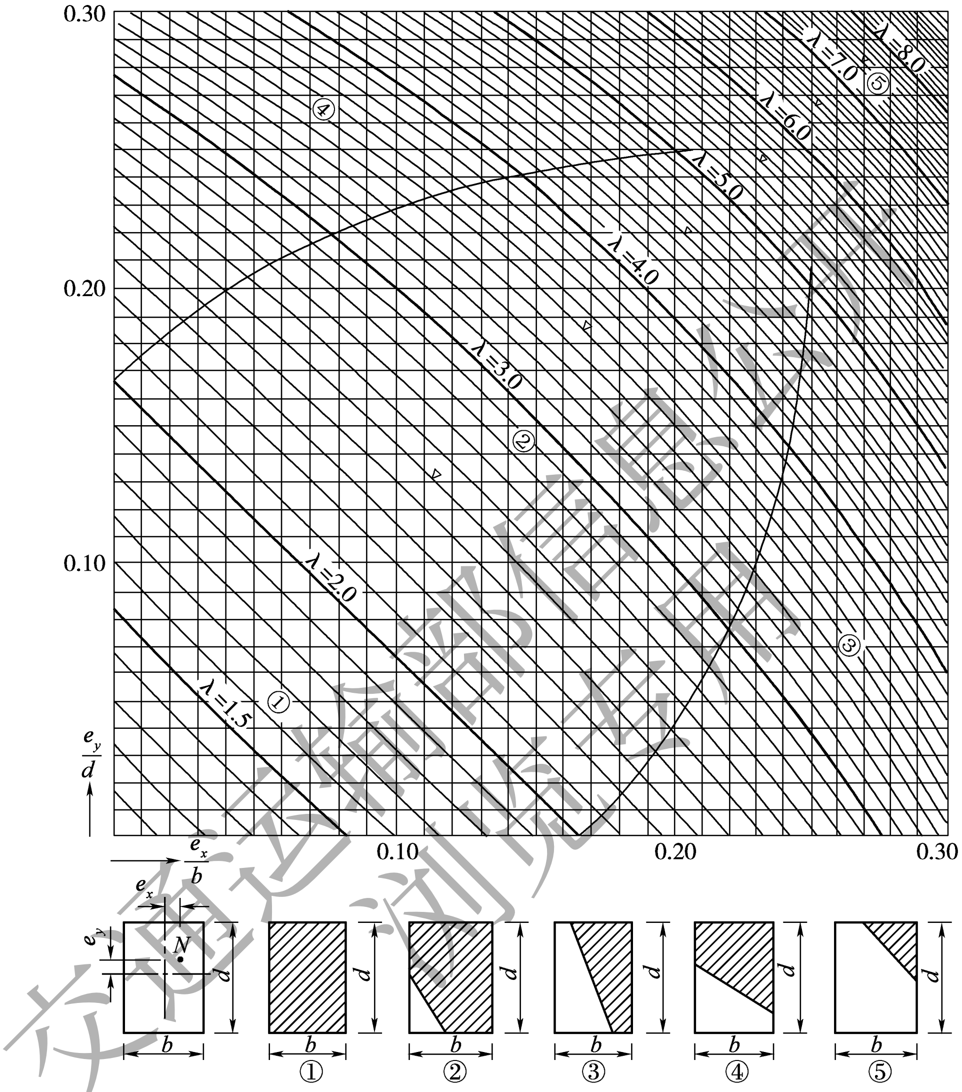


Fig. G.0.1 Diagram of Stress Redistribution on Rectangular Section Subjected to Biaxial Eccentric Compression

Table G.0.2 Table of Revised Coefficient, λ

$n=\frac{e}{d}$	λ	$n=\frac{e}{d}$	λ	$n=\frac{e}{d}$	λ	$n=\frac{e}{d}$	λ
0.1250	2.000	0.1752	2.457	0.2310	3.208	0.2945	4.729
0.1260	2.012	0.1780	2.487	0.2347	3.271	0.2980	4.828
0.1270	2.015	0.1787	2.499	0.2380	3.321	0.3020	4.949
0.1290	2.034	0.1815	2.524	0.2415	3.382	0.3050	5.074

continued

$n=\frac{e}{d}$	λ	$n=\frac{e}{d}$	λ	$n=\frac{e}{d}$	λ	$n=\frac{e}{d}$	λ
0.1330	2.064	0.1848	2.571	0.2452	3.465	0.3080	5.230
0.1370	2.102	0.1886	2.608	0.2470	3.497	0.3115	5.334
0.1384	2.109	0.1890	2.620	0.2490	3.540	0.3150	5.484
0.1414	2.134	0.1916	2.645	0.2529	3.610	0.3190	5.634
0.1430	2.151	0.1951	2.690	0.2565	3.692	0.3220	5.793
0.1441	2.160	0.1989	2.736	0.2597	3.768	0.3260	5.957
0.1468	2.181	0.2020	2.777	0.2620	3.803	0.3310	6.130
0.1500	2.213	0.2022	2.773	0.2640	3.859	0.3330	6.311
0.1532	2.242	0.2055	2.823	0.2678	3.949	0.3380	6.512
0.1562	2.268	0.2070	2.851	0.2718	4.046	0.3390	6.700
0.1580	2.288	0.2122	2.920	0.2741	4.161	0.3430	6.911
0.1593	2.296	0.2160	2.967	0.2770	4.193	0.3470	7.141
0.1625	2.327	0.2174	2.996	0.2789	4.245	0.3500	7.368
0.1654	2.358	0.2200	3.036	0.2826	4.356	0.3540	7.620
0.1680	2.378	0.2232	3.080	0.2868	4.471	0.3570	7.881
0.1686	2.391	0.2271	3.143	0.2907	4.593	0.3600	8.157
0.1716	2.421	0.2300	3.193	0.2940	4.715	0.3690	8.467

Appendix H
Check for Stability against Frozen Uplift for Frozen Soil Ground

H.0.1 The stability against frozen uplift of the piers and abutments as well as their foundations (including strip foundations) on the ground of seasonally frozen soil may be calculated by the following formulas:

$$F_k + G_k + Q_{sk} \geqslant kT_k \tag{H.0.1-1}$$

$$T_k = z_d \tau_{sk} \mu \tag{H.0.1-2}$$

where:

F_k—self-weight of structure acting on the foundation (kN);

G_k—self-weight of foundation and soil over the offset of the foundation (kN);

Q_{sk}—nominal value of the skin friction of the thawing layer around the foundation (kN), which is calculated by Formula (H.0.2-2);

k— adjusting factor for frost heave force. Before construction or erection of the superstructures, k is taken as 1.1. After construction or erection of superstructures, k is taken as 1.2 for the externally static determinate structure and as 1.3 for the externally static indeterminate structure;

T_k—nominal value of tangential frost heave to the foundations (kN);

z_d—design frost depth (m), which can be referred to Clause 5.1.2 in the *Specifications*. If the bearing depth of the foundation, h, is less than Z_d, h is taken as Z_d;

τ_{sk}—nominal value of tangential frost heave for seasonally frozen soil (kPa), which is taken from Table H.0.1;

u—average perimeter of the foundation and pier bodies inner the seasonally frozen soil layer (m).

Table H. 0. 1 Nominal Value of Tangential frost heave for Seasonally Frozen Soil, τ_{sk} (kPa)

Foundation type	Frost heave type				
	Non-frost heave	Slightly frost heave	Moderately frost heave	Strong frost heave	Extremely strong frost heave
Pier, abutment, column, pile foundations	0 ~ 15	15 ~ 80	80 ~ 120	120 ~ 160	160 ~ 200
Strip foundation	0 ~ 10	10 ~ 40	40 ~ 60	60 ~ 80	80 ~ 100

Note: 1. Strip foundation refers to the foundation whose length-to-width ratio is equal to or greater than 10.

2. For the precast pile with smooth surface, the τ_{sk} is multiplied by 0.8.

H. 0. 2 The stability against frozen uplift for the piers and abutments as well as their foundations (including strip foundation) on permafrost ground may be checked by the following formulas (see Fig. H. 0. 2):

$$F_k + G_k + Q_{sk} + Q_{sp} \geqslant kT_k \quad \text{(H. 0. 2-1)}$$

$$Q_{sk} = q_{sk} A_s \quad \text{(H. 0. 2-2)}$$

$$Q_{pk} = q_{pk} A_p \quad \text{(H. 0. 2-3)}$$

where:

Q_{sk}—nominal value of skin friction of the thawing layer around the foundation (kN); when the seasonally frozen soil layer is connected with the permafrost layer, $Q_{sk} = 0$; when the seasonally frozen soil layer is not connected with the permafrost layer, it is calculated by Eq. (H. 0. 2-2);

A_s—side face area of the foundation in a thawing layer (m^2);

q_{sk}—nominal value of skin friction between the foundation side surface and the thawing layer (kPa). In the absence of measured data, it may be taken as 20 ~ 30 kPa for the cohesive soil and 30 ~ 40 kPa for sand and gravelly soil;

Q_{pk}—nominal value of freezing force between the foundation side surface and the permafrost (kN), which is calculated according to Formula (H. 0. 2-3);

A_p—foundation side surface area in the permafrost layer (m^2);

q_{pk}—nominal value of freezing stress between the permafrost and the foundation side face (kPa), which may be obtained from Table H. 0. 2;

Other symbols are the same as those in ClauseH. 0. 1.

Note: As shown in Fig. H. 0. 2, the ground layers betweenseasonally frozen soil and permafrost may be divided into ground with and without connection of seasonally frozen soil and permafrost layer. When a permafrost layer top surface is located below the seasonally frozen soil layer, it is a ground with connection ($Q_{sk} = 0$). When there is a thawing layer or a thawing layer plus permafrost in an alternate manner below the seasonally frozen soil layer, it is a ground without connection (Q_{sk} is calculated by Formula H.0.2-2).

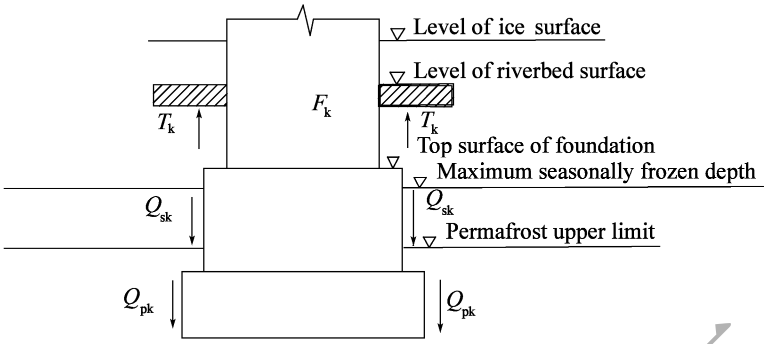


Fig. H.0.2 Diagram for the Frost Heave Force of the Permafrost Foundation

T_k—tangential frost heave to the foundation;

Q_{sk}—skin friction of the foundation in the thawing layer;

Q_{pk}—freezing force between the foundation and the permafrost.

Table H.0.2 Nominal value of Frozen Force between the Permafrost and the Foundation, q_{pk} (kPa)

Soil type and thaw-subsidence grade		Temperature (℃)						
		−0.2	−0.5	−1.0	−1.5	−2.0	−2.5	−3.0
Silty soil, cohesive soil	Ⅲ	35	50	85	115	145	170	200
	Ⅱ	30	40	60	80	100	120	140
	Ⅰ, Ⅳ	20	30	40	60	70	85	100
	Ⅴ	15	20	30	40	50	55	65
Sand	Ⅲ	40	60	100	130	165	200	230
	Ⅱ	30	50	80	100	130	155	180
	Ⅰ, Ⅳ	25	35	50	70	85	100	115
	Ⅴ	10	20	30	35	40	50	60
Gravelly soil (content of grain with grain size less than 0.075 mm is less than or equal to 10%)	Ⅲ	40	55	80	100	130	155	180
	Ⅱ	30	40	60	0	100	120	135
	Ⅰ, Ⅳ	25	35	50	60	70	85	95
	Ⅴ	15	20	30	40	45	55	65
Gravelly soil (content of grain with grain size less than 0.075 mm is greater than 10%)	Ⅲ	35	55	85	115	50	170	200
	Ⅱ	30	40	70	90	115	140	160
	Ⅰ, Ⅳ	25	35	50	70	5	95	115
	Ⅴ	15	20	30	35	45	55	60

Note: 1. The thaw-subsidence grade of permafrost can be found in Table E.0.3 of the *Specifications*;
2. The nominal value of frozen forces for a foundation made of precast concrete, wood, and metal, the value listed in Table H.0.2 shall be multiplied by a coefficient of 1.0, 0.9, and 0.66, respectively.
3. The nominal value of frozen force between permafrost and driven piles are taken as the value for the thaw-subsidence Grade Ⅳ category.

H.0.3 The stability against frozen uplift for pile (column) foundation may be checked by the following formulas:

$$F_k + G_k + Q_{fk} \geqslant kT_k \tag{H.0.3-1}$$

$$Q_{fk} = 0.4\mu \sum q_{ik} I_i \tag{H.0.3-2}$$

where:

F_k—self-weight of structures acting on top of the pile (column) (kN);

G_k—self-weight of the pile (column) (kN), which is taken as buoyant unit weight if the pile (column) is below water level and the soil layer at its bottom is permeable;

Q_{fk}—sum of nominal value of skin friction of piles (columns) in all soil layers below the depth of frost potential, which is calculated by Formula (H.0.3-2);

u—pile perimeter (m);

q_{ik}—nominal value of skin friction of all soil layers below the depth of frost potential (kPa), see Table 6.3.3-1 or Table 6.3.5-1 of the *Specifications*;

l_i—depth of i^{th} soil layers below the depth of frost potential (m);

T_k—tangential frost heave of each pile (column) (kN), which is calculated by Formula (H.0.1-2).

H.0.4 If the tangential frost heave is large, the tensile strength at the weak sections of the pier, abutment, foundation, and pile (column) shall be checked.

Appendix J
Superimposed Compressive Stress Coefficient α and Average Superimposed Compressive Stress Coefficient $\bar{\alpha}$ of Foundation Bases in Bridge and Culvert

J.0.1 Superimposed compressive stress coefficient α at the centroid of underlying stratum in the bridge and culvert foundation bases under uniform load may be taken from Table J.0.1.

Table J.0.1 Superimposed Compressive Stress Coefficient α at the Centroid of Underlying Stratum in the Foundation Bases

z/b	l/b											
	1.0	1.2	1.4	1.6	1.8	2.0	2.4	2.8	3.2	3.6	4.0	5.0	≥10strip
0.0	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.1	0.980	0.984	0.986	0.987	0.987	0.988	0.988	0.989	0.989	0.989	0.989	0.989	0.989
0.2	0.960	0.968	0.972	0.974	0.975	0.976	0.976	0.977	0.977	0.977	0.977	0.977	0.977
0.3	0.880	0.899	0.910	0.917	0.920	0.923	0.925	0.928	0.928	0.929	0.929	0.929	0.929
0.4	0.800	0.830	0.848	0.859	0.866	0.870	0.875	0.878	0.879	0.880	0.880	0.881	0.881
0.5	0.703	0.741	0.765	0.781	0.791	0.799	0.810	0.812	0.814	0.816	0.817	0.818	0.818
0.6	0.606	0.651	0.682	0.703	0.717	0.727	0.737	0.746	0.749	0.751	0.753	0.754	0.755
0.7	0.527	0.574	0.607	0.630	0.648	0.660	0.674	0.685	0.690	0.692	0.694	0.697	0.698
0.8	0.449	0.496	0.532	0.558	0.578	0.593	0.612	0.623	0.630	0.633	0.636	0.639	0.642
0.9	0.392	0.437	0.473	0.499	0.520	0.536	0.559	0.572	0.579	0.584	0.588	0.592	0.596
1.0	0.334	0.378	0.414	0.441	0.463	0.482	0.505	0.520	0.529	0.536	0.540	0.545	0.550
1.1	0.295	0.336	0.369	0.396	0.418	0.436	0.462	0.479	0.489	0.496	0.501	0.508	0.513
1.2	0.257	0.294	0.325	0.352	0.374	0.392	0.419	0.437	0.449	0.457	0.462	0.470	0.477

continued

z/b	l/b												
	1.0	1.2	1.4	1.6	1.8	2.0	2.4	2.8	3.2	3.6	4.0	5.0	≥10strip
1.3	0.229	0.263	0.292	0.318	0.339	0.357	0.384	0.403	0.416	0.424	0.431	0.440	0.448
1.4	0.201	0.232	0.260	0.284	0.304	0.321	0.350	0.369	0.383	0.393	0.400	0.410	0.420
1.5	0.180	0.209	0.235	0.258	0.277	0.294	0.322	0.341	0.356	0.366	0.374	0.385	0.397
1.6	0.160	0.187	0.210	0.232	0.251	0.267	0.294	0.314	0.329	0.340	0.348	0.360	0.374
1.7	0.145	0.170	0.191	0.212	0.230	0.245	0.272	0.292	0.307	0.317	0.326	0.340	0.355
1.8	0.130	0.153	0.173	0.192	0.209	0.224	0.250	0.270	0.285	0.296	0.305	0.320	0.337
1.9	0.119	0.140	0.159	0.177	0.192	0.207	0.233	0.251	0.263	0.278	0.288	0.303	0.320
2.0	0.108	0.127	0.145	0.161	0.176	0.189	0.214	0.233	0.241	0.260	0.270	0.285	0.304
2.1	0.099	0.116	0.133	0.148	0.163	0.176	0.199	0.220	0.230	0.244	0.255	0.270	0.292
2.2	0.090	0.107	0.122	0.137	0.150	0.163	0.185	0.208	0.218	0.230	0.239	0.256	0.280
2.3	0.083	0.099	0.113	0.127	0.139	0.151	0.173	0.193	0.205	0.216	0.226	0.243	0.269
2.4	0.077	0.092	0.105	0.118	0.130	0.141	0.161	0.178	0.192	0.204	0.213	0.230	0.258
2.5	0.072	0.085	0.097	0.109	0.121	0.131	0.151	0.167	0.181	0.192	0.202	0.219	0.249
2.6	0.066	0.079	0.091	0.102	0.112	0.123	0.141	0.157	0.170	0.184	0.191	0.208	0.239
2.7	0.062	0.073	0.084	0.095	0.105	0.115	0.132	0.148	0.161	0.174	0.182	0.199	0.234
2.8	0.058	0.069	0.079	0.089	0.099	0.108	0.124	0.139	0.152	0.163	0.172	0.189	0.228
2.9	0.054	0.064	0.074	0.083	0.093	0.101	0.177	0.132	0.144	0.155	0.163	0.180	0.218
3.0	0.051	0.060	0.070	0.078	0.087	0.095	0.110	0.124	0.136	0.146	0.155	0.172	0.208
3.2	0.045	0.053	0.062	0.070	0.077	0.085	0.098	0.111	0.122	0.133	0.141	0.158	0.190
3.4	0.040	0.048	0.055	0.062	0.069	0.076	0.088	0.100	0.110	0.120	0.128	0.144	0.184
3.6	0.036	0.042	0.049	0.056	0.062	0.068	0.080	0.090	0.100	0.109	0.117	0.133	0.175
3.8	0.032	0.038	0.044	0.050	0.056	0.062	0.072	0.082	0.091	0.100	0.107	0.123	0.166
4.0	0.029	0.035	0.040	0.046	0.051	0.056	0.066	0.075	0.084	0.090	0.095	0.113	0.158
4.2	0.026	0.031	0.037	0.042	0.048	0.051	0.060	0.069	0.077	0.084	0.091	0.105	0.150
4.4	0.024	0.029	0.034	0.038	0.042	0.047	0.055	0.063	0.070	0.077	0.084	0.098	0.144
4.6	0.022	0.026	0.031	0.035	0.039	0.043	0.051	0.058	0.065	0.072	0.078	0.091	0.137
4.8	0.020	0.024	0.028	0.032	0.036	0.040	0.047	0.054	0.060	0.067	0.072	0.085	0.132
5.0	0.019	0.022	0.026	0.030	0.033	0.037	0.044	0.050	0.056	0.062	0.067	0.079	0.126

Note: l, b—Length of the long and short sides of the rectangular foundation, respectively (m); z—Distance from the foundation base to the top surface of the underlying stratum (m).

J.0.2 Average superimposed compressive stress coefficient $\overline{\alpha}$ at the centroid of the underlying stratum in the rectangular foundation under uniform load may be taken from Table J.0.2.

Table J.0.2 AverageSuperimposed Compressive Stress Coefficient $\overline{\alpha}$ at the Centroid of Underlying Stratum in the Rectangular Foundation under Uniform Load

z/b	l/b												
	1.0	1.2	1.4	1.6	1.8	2.0	2.4	2.8	3.2	3.6	4.0	5.0	≥10strip
0.0	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
0.1	0.997	0.998	0.998	0.008	0.998	0.998	0.998	0.998	0.998	0.998	0.998	0.998	0.998
0.2	0.987	0.990	0.991	0.992	0.992	0.992	0.993	0.993	0.993	0.993	0.993	0.993	0.993
0.3	0.967	0.973	0.976	0.978	0.979	0.979	0.980	0.980	0.981	0.981	0.981	0.981	0.981
0.4	0.936	0.947	0.953	0.956	0.958	0.965	0.961	0.962	0.962	0.963	0.963	0.963	0.963
0.5	0.900	0.915	0.924	0.929	0.933	0.935	0.937	0.939	0.939	0.940	0.940	0.940	0.940
0.6	0.858	0.878	0.890	0.898	0.903	0.906	0.910	0.912	0.913	0.914	0.914	0.915	0.915
0.7	0.816	0.840	0.855	0.865	0.871	0.876	0.881	0.884	0.885	0.886	0.887	0.887	0.888
0.8	0.775	0.801	0.819	0.831	0.839	0.844	0.851	0.855	0.857	0.858	0.859	0.860	0.860
0.9	0.735	0.764	0.784	0.797	0.806	0.813	0.821	0.826	0.829	0.830	0.831	0.830	0.836
1.0	0.698	0.728	0.749	0.764	0.775	0.783	0.792	0.798	0.801	0.803	0.804	0.806	0.807
1.1	0.663	0.694	0.717	0.733	0.774	0.753	0.764	0.771	0.775	0.777	0.773	0.780	0.782
1.2	0.631	0.663	0.686	0.703	0.715	0.725	0.737	0.744	0.749	0.752	0.754	0.756	0.758
1.3	0.601	0.633	0.657	0.674	0.688	0.698	0.711	0.719	0.725	0.728	0.730	0.733	0.735
1.4	0.573	0.605	0.629	0.648	0.661	0.672	0.687	0.696	0.701	0.705	0.708	0.711	0.714
1.5	0.548	0.580	0.604	0.622	0.637	0.648	0.667	0.673	0.679	0.683	0.686	0.690	0.693
1.6	0.524	0.556	0.580	0.599	0.613	0.625	0.641	0.651	0.658	0.663	0.666	0.670	0.675
1.7	0.502	0.533	0.558	0.577	0.591	0.603	0.620	0.631	0.638	0.643	0.646	0.651	0.656
1.8	0.482	0.513	0.537	0.556	0.571	0.588	0.600	0.611	0.619	0.624	0.629	0.633	0.638
1.9	0.463	0.493	0.517	0.536	0.551	0.563	0.581	0.593	0.601	0.606	0.610	0.616	0.622
2.0	0.446	0.475	0.499	0.518	0.533	0.545	0.563	0.575	0.584	0.590	0.594	0.600	0.606
2.1	0.429	0.459	0.482	0.500	0.515	0.528	0.546	0.559	0.567	0.574	0.578	0.585	0.591
2.2	0.414	0.443	0.466	0.484	0.499	0.511	0.530	0.543	0.552	0.558	0.563	0.570	0.577
2.3	0.400	0.428	0.451	0.469	0.484	0.496	0.515	0.528	0.537	0.544	0.548	0.554	0.564
2.4	0.387	0.414	0.436	0.454	0.469	0.481	0.500	0.513	0.523	0.530	0.535	0.543	0.551
2.5	0.374	0.401	0.423	0.441	0.455	0.468	0.486	0.500	0.509	0.516	0.522	0.530	0.539
2.6	0.362	0.389	0.410	0.428	0.442	0.473	0.473	0.487	0.496	0.504	0.509	0.518	0.528
2.7	0.351	0.377	0.398	0.416	0.430	0.461	0.461	0.474	0.484	0.492	0.497	0.506	0.517
2.8	0.341	0.366	0.387	0.404	0.418	0.449	0.449	0.463	0.472	0.480	0.486	0.495	0.506
2.9	0.331	0.356	0.337	0.393	0.407	0.438	0.438	0.451	0.461	0.469	0.475	0.485	0.496

continued

z/b	l/b												
	1.0	1.2	1.4	1.6	1.8	2.0	2.4	2.8	3.2	3.6	4.0	5.0	≥10strip
3.0	0.322	0.346	0.366	0.383	0.397	0.409	0.429	0.441	0.451	0.459	0.465	0.474	0.487
3.1	0.313	0.337	0.357	0.373	0.387	0.398	0.417	0.430	0.440	0.448	0.454	0.464	0.477
3.2	0.305	0.328	0.348	0.364	0.377	0.389	0.407	0.420	0.431	0.439	0.445	0.455	0.468
3.3	0.297	0.320	0.339	0.355	0.368	0.379	0.397	0.411	0.421	0.429	0.436	0.446	0.460
3.4	0.289	0.312	0.331	0.346	0.359	0.371	0.388	0.402	0.412	0.420	0.427	0.437	0.452
3.5	0.282	0.304	0.323	0.338	0.351	0.362	0.380	0.393	0.403	0.412	0.418	0.429	0.444
3.6	0.276	0.297	0.315	0.330	0.343	0.354	0.372	0.385	0.395	0.403	0.410	0.421	0.436
3.7	0.269	0.290	0.308	0.323	0.335	0.346	0.364	0.377	0.387	0.395	0.402	0.413	0.429
3.8	0.263	0.284	0.301	0.316	0.328	0.339	0.356	0.369	0.379	0.338	0.394	0.405	0.442
3.9	0.257	0.277	0.294	0.309	0.321	0.332	0.349	0.362	0.372	0.380	0.387	0.398	0.415
4.0	0.251	0.271	0.288	0.302	0.311	0.325	0.342	0.355	0.365	0.373	0.379	0.391	0.408
4.1	0.246	0.265	0.282	0.296	0.308	0.328	0.335	0.348	0.358	0.366	0.372	0.384	0.402
4.2	0.241	0.260	0.276	0.290	0.302	0.312	0.328	0.341	0.352	0.359	0.366	0.377	0.396
4.3	0.236	0.255	0.270	0.284	0.296	0.306	0.322	0.335	0.345	0.353	0.359	0.371	0.390
4.4	0.231	0.250	0.265	0.278	0.290	0.300	0.316	0.329	0.339	0.347	0.353	0.365	0.384
4.5	0.336	0.245	0.260	0.273	0.285	0.294	0.310	0.323	0.333	0.341	0.347	0.359	0.378
4.6	0.222	0.240	0.255	0.268	0.279	0.289	0.305	0.317	0.327	0.335	0.341	0.353	0.373
4.7	0.218	0.235	0.250	0.263	0.274	0.284	0.299	0.312	0.321	0.329	0.336	0.347	0.367
4.8	0.214	0.231	0.245	0.258	0.269	0.279	0.294	0.306	0.316	0.324	0.330	0.342	0.362
4.9	0.210	0.227	0.241	0.253	0.265	0.274	0.289	0.301	0.311	0.319	0.325	0.337	0.357
5.0	0.206	0.223	0.237	0.249	0.260	0.269	0.284	0.296	0.306	0.313	0.320	0.332	0.352

Note: l, b- Length of long and short sides of rectangular foundation, respectively (m); z- Soil layer depth measured from the bottom of the foundation (m).

Appendix K
Post-Grouting Technical Parameters of Piles

K.0.1 Water-to-cement ratio of grout shall be determined by the soil saturation and permeability. It should be taken as 0.5 ~ 0.7 for the saturated soil, and 0.7 ~ 0.9 for non-saturated soil (0.5 ~ 0.6 for the loose gravelly soil and sandy gravel). Grout with a low water-to-cement ratio should be mixed with a superplasticizer; and when there is a flowing groundwater, it shall be mixed with cement accelerator.

K.0.2 The grouting pressure at pile tip when grouting is finished shall be determined by the properties of the soil and the depth of the grouting point. For weathered rock, non-saturated cohesive soil, and silty soil, it should be taken as 3.0 ~ 10.0 MPa. For saturated soil, it should be taken as 1.2 ~ 4.0 MPa, in which a low value should be taken for soft soil, while a high value should be taken for dense soil. The grouting pressure at the pile sides when grouting is finished should be 1/3 ~ 1/2 of that at the pile tip.

K.0.3 The holding time for grouting is 5 min.

K.0.4 Grouting flow should not be larger than 75 L/min.

K.0.5 Grout quantity for a single pile shall be determined by pile diameter, pile length, properties of soil at the pile tip and pile side, compressive resistance increment for a single pile after grouting, etc. It may be calculated by the following equation:

$$G_c = \sum_{i=1}^{m} \alpha_{si} d + \alpha_p d \tag{K.0.5}$$

where:

G_c—grout quantity for a single pile (t);

α_{si}, α_{p}—empirical coefficients of grout quantity for the i^{th} grouting section at the pile sides and pile tip (t/m), respectively. The range of coefficients α_{si} and α_{p} are shown in Table K.0.5;

m—cross section number for grouting at the pile sides;

d—pile diameter (m).

Table K.0.5 Empirical Coefficient of Grouting at Pile Side, α_{si}, and Empirical Coefficient of Grouting at Pile Tip, α_p

Name of soil layer	Cohesive soil, silty cohesive soil	Silty soil	Silty sand	Fine sand	Medium sand	Coarse sand, gravelly sand	Breccia, gravel	Gravelly stone, cobble	Completely and highly weathered rocks
α_{si}	0.7 ~ 0.8	0.8 ~ 0.9	0.8 ~ 0.9	0.8 ~ 0.9	0.9 ~ 1.1	0.9 ~ 1.1	0.8 ~ 0.9	0.8 ~ 0.9	0.8 ~ 0.9
α_p	2.0 ~ 2.4	2.1 ~ 2.5	2.4 ~ 2.7	2.4 ~ 2.7	2.3 ~ 2.7	2.7 ~ 3.0	2.9 ~ 3.2	2.3 ~ 2.8	2.3 ~ 2.5

Note: For slightly dense and loose sand as well as gravelly soil, the high value may be taken; for the dense sand and gravelly soil, the low value may be taken.

Appendix L
Calculations of Horizontal Displacement and Action Effect for Elastic Piles by Using the *m* Method

L.0.1 The effective width of a pile may be calculated by the following formulas:

When $d \geqslant 1.0$ m:

$$b_1 = kk_f \tag{L.0.1-1}$$

When $d < 1.0$ m:

$$b_1 = kk_f \tag{L.0.1-2}$$

For multi-row piles with $L_1 \geqslant 0.6h_1$ or single-row piles:

$$k = 0 \tag{L.0.1-3}$$

For multi-row piles with $L_1 < 0.6h_1$:

$$k = b_2 + \frac{1 - b_2}{0.6} \cdot \frac{L_1}{h_1} \tag{L.0.1-4}$$

where:

b_1——effective width of the pile (m), $b_1 \leqslant 2d$;

d——diameter or width of the pile perpendicular to the horizontal force (m);

k_f——transformed factor for pile shape, which is determined by the horizontal force acting plane (perpendicular to the horizontal force acting direction); for circular section, k_f is taken as 0.9; for rectangular section, k_f is taken as 1.0; for composite section of round end and rectangular, k_f is taken as $k_f = \left(1 - 0.1\frac{a}{b}\right)$ (Fig. L.0.1-1);

k——coefficient for mutual influence between piles in the direction parallel to the horizontal force;

L_1——clear distance between piles in the direction parallel to the horizontal force (Fig. L.0.1-2) (m). It may be calculated by the projection distance between piles in the direction of the horizontal force (Fig. L.0.1-3) for piles arranged in quincuncial shape if the center-to-center distance between two adjacent piles, c, is less than $(d+1)$;

h_1——effective bearing depth of a pile below the ground or local scour line (m). It may be taken as $h_1 = 3(d+1)$, but shall not be greater than the pile bearing depth below the ground or local scour line, h (Fig. L.0.1-2).

b_2——coefficient for pile number (n) in one row in the direction parallel to the horizontal force. When $n = 1$, $b_2 = 1.0$; when $n = 2$, $b_2 = 0.6$; when $n = 3$, $b_2 = 0.5$; and when $n \geqslant 4$, $b_2 = 0.45$.

In the plane layout of the piles, if the pile number of each row in the direction parallel to the horizontal force is not the same, and the center-to-center distance between adjacent piles (any direction) is equal to or greater than $(d+1)$, then the same coefficient for mutual influence between piles, k, may be taken in the calculation for all the piles, in which its value is adopted according to the row with the largest pile number. In addition, if there are n piles in the direction perpendicular to the horizontal force, the effective width shall be taken as nb_1, but it also shall satisfy $nb_1 \leqslant B + 1$, where B is the distance of outer edges of n piles in the direction perpendicular to the horizontal force, taking meter as unit (see Fig. L.0.1-4).

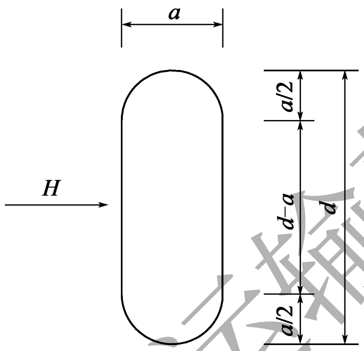


Fig. L.0.1-1 Schematic Diagram for Calculation of k_f for Composite Section of Round-end and Rectangular

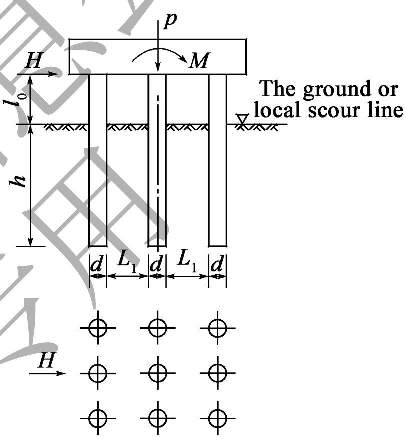


Fig. L.0.1-2 Schematic Diagram for Calculation of k for Pile Foundation

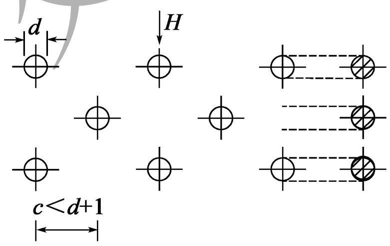


Fig. L.0.1-3 Schematic Diagram for Width Calculation of Quincuncial Piles

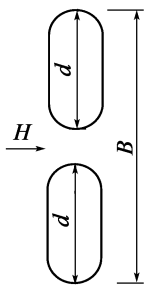


Fig. L.0.1-4 Diagram for Width Calculation of Single Pile

L.0.2 The deformation coefficient of a pile in a pile foundation may be calculated by the following formulas:

$$\alpha = \sqrt[5]{\frac{mb_1}{EI}} \tag{L.0.2-1}$$

$$EI = 0.8E_cI \quad (L.0.2\text{-}2)$$

where:

α——deformation coefficient of a pile (1/m);

EI——flexural stiffness of a pile. For a reinforced concrete pile mainly subjected to bending moment, it is adopted according to the provisions in the current *Specifications for Designw of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTG 3362);

m_0——proportionality factor for partial factor in non-rock ground. The partial factor in non-rock ground increases proportionally with the pile bearing depth. The factor of horizontal resistance at depth z *is* $C_z = m \times z$, and the factor of vertical resistance at the pile tip is $C_0 = m_0 \times h$ (when $h < 10$, $C_0 = 10 \times m_0$ is taken), where m is the proportionality factor for partial factor in non-rock ground, and m_0 is the proportionality factor for partial factor in non-rock ground at the pile tip ground. Both m and m_0 shall be determined by tests; in case of lack of test data, it shall be adopted from Table L.0.2-1 according to ground soil type and state. When there are two soil layers below the ground surface aside from the foundation or below the local scour line with a depth of $h_m = 2(d+1)$ (m) (in case of $ah \leqslant 2.5$, $h_m = h$ is taken) as shown in Fig. L.0.2, the proportionality factor for such two soil layers shall be transformed into one m value by Formula L.0.2-3, and this m is used for the overall depth. The partial factor for rock ground does not change with the rock buried depth, thus $C_z = C_0$ is taken in the calculation, where C_0 may be taken from Table L.0.2-2 or determined by tests.

$$m = \gamma m_1 + (1-\gamma)m_2 \quad (L.0.2\text{-}3)$$

$$\begin{cases} 5(h_1/h_m)^2 & h_1/h_m \leqslant 0.2 \\ 1-1.25(1-h_1/h_m)^2 & h_1/h_m > 0.2 \end{cases} \quad (L.0.2\text{-}4)$$

Table L.0.2-1 Values of m and m_0 for Non-rock Soils

Soil name	m and m_0 (kN/m^4)	Soil name	m and m_0 (kN/m^4)
Liquid cohesive soil with $I_L > 1.0$, semi-liquid cohesive soil with $1.0 \geqslant I_L > 0.75$, mud	3000 ~ 5000	Stiff, semi-stiff cohesive soil with $I_L \leqslant 0$, coarse sand, dense silty soil	20000 ~ 30000
Plastic cohesive soil with $0.75 \geqslant I_L > 0.25$, silty sand, slightly dense silty soil	5000 ~ 10000	Gravelly sand, breccia, round gravel, gravel, cobble	30000 ~ 80000
Stiff plastic cohesive soil with $0.25 \geqslant I_L > 0$, fine sand, medium sand, medium dense silty soil	10000 ~ 20000	Dense cobble plus coarse sand, dense boulder, cobble	80000 ~ 120000

Note: 1. This table is applicable where the maximum horizontal displacement of the foundation on the ground does not exceed 6 mm. In case of larger displacement, the values in the table shall be reduced accordingly.

2. If there is a slope or step at the foundation side and the slope ratio or the ratio between the total width and depth of the steps is greater than 1:20, the m value in the table shall be reduced by 50% for application.

Table L.0.2-2 Partial Factor for Rock Ground, C_0

No.	f_{rk} (kPa)	C_0 (kN/m^4)
1	1000	300000
2	≥25000	15000000

Note: f_{rk}—the nominal value of the uniaxial compressive strength of saturated rock. For the sample that can not be saturated, the nominal value of uniaxial compressive strength of a sample with natural content of water may be used. When $1000 < f_{rk} < 25000$, C_0 may be determined by the straight-line interpolation method.

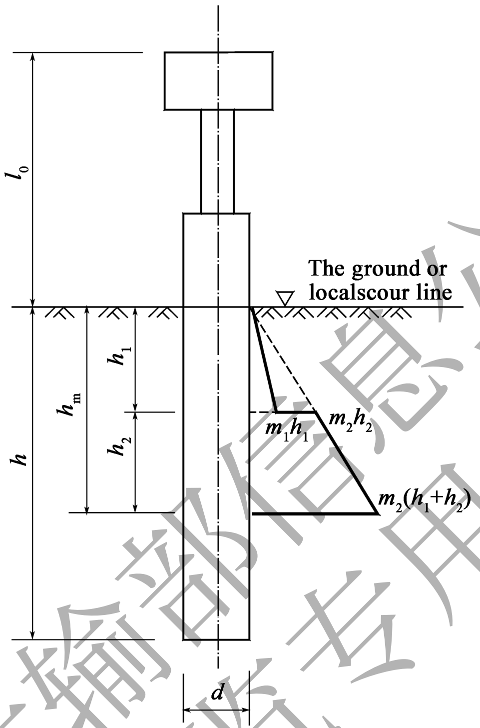
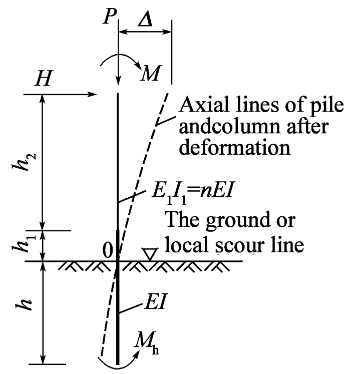
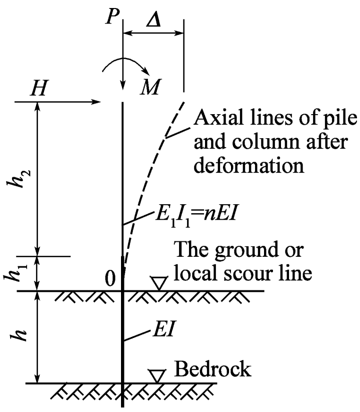


Fig. L.0.2 Schematic Diagram for Calculation of Transformed Value *m* for Two Soil Layers

L.0.3 When $ah > 2.5$, the action effect and displacement of the pile bent pier with single-row piles under the load at pier top may be calculated according to Table L.0.3.

Table L.0.3 Calculation Table for Pile Bent Pier with Single-row Piles under Load at Pier Top

<table>
<tr>
<td>Calculated scheme</td>
<td>(1) Pile bent pier with single-row piles whose pile top is free and whose pile tip is supported by non-rock soil or bedrock surface.

</td>
<td>Pile bent pier with single-row piles whose pile top is free but whose pile tip is socketed into bedrock.

</td>
</tr>
</table>

continued

<table>
<tr><td colspan="2" rowspan="2">Action effect of the pile on the ground or local scour line</td><td>Bending moment</td><td colspan="2">$M_0 = M + H(h_2 + h_1)$</td></tr>
<tr><td>Shear force</td><td colspan="2">$H_0 = H$</td></tr>
<tr><td rowspan="4">Deformation produced on the force acting section when unit ‘force’ is applied on the ground or local scour line</td><td rowspan="2">Action of $H_0 = 1$</td><td>Horizontal displacement</td><td>$\delta^{(0)}_{HH} = \frac{1}{\alpha^3 EI} \times \frac{(B_3D_4 - B_4D_3) + k_h(B_2D_4 - B_4D_2)}{(A_3B_4 - A_4B_3) + k_h(A_2B_4 - A_2B_2)}$</td><td>$\delta^{(0)}_{HH} = \frac{1}{\alpha^3 EI} \times \frac{B_2D_1 - B_1D_2}{A_2B_1 - A_1B_2}$</td></tr>
<tr><td>Rotation angle(rad)</td><td>$\delta^{(0)}_{HH} = \frac{1}{\alpha^3 EI} \times \frac{B_3D_4 - B_4D_3 + k_h(B_2D_4 - B_4D_2)}{(A_3B_4 - A_4B_3) + k_h(A_2B_4 - A_4B_2)}$</td><td>$\delta^{(0)}_{MH} = \frac{1}{\alpha^2 EI} \times \frac{A_2D_1 - A_1D_2}{A_2B_1 - A_1B_2}$</td></tr>
<tr><td rowspan="2">Action of $M_0 = 1$</td><td>Horizontal displacement</td><td>$\delta^{(0)}_{HM} = \delta^{(0)}_{MH} = \frac{1}{\alpha^2 EI} \times \frac{B_3C_4 - B_4C_3 + k_h(B_2C_4 - B_4C_2)}{A_3B_4 - A_4B_3 + k_h(A_2B_4 - A_4B_2)}$</td><td>$\delta^{(0)}_{HM} = \delta^{(0)}_{MH} = \frac{1}{\alpha^2 EI} \times \frac{B_2C_1}{A_2B_1 - A_1B_2}$</td></tr>
<tr><td>Rotation angle(rad)</td><td>$\delta^{(0)}_{MM} = \frac{1}{\alpha EI} \times \frac{A_3C_4 - A_4C_3 + k_h(A_2C_4 - A_4C_2)}{(A_3B_4 - A_4B_3) + k_h(A_2B_4 - A_4B_2 3)}$</td><td>$\delta^{(0)}_{MM} = \frac{1}{\alpha EI} \times \frac{A_2C_1 - A_1C_2}{A_2B_1 - A_1B_2}$</td></tr>
<tr><td rowspan="2">Displacement of the pile on the ground or local scour line</td><td colspan="2">Horizontal displacement</td><td colspan="2">$X_0 = H_0\delta^{(0)}_{HH} + M_0\delta^{(0)}_{HM}$</td></tr>
<tr><td colspan="2">Rotation angle(rad)</td><td colspan="2">$\varphi_0 = -\left(H_0\delta^{(0)}_{MH} + M_0\delta^{(0)}_{MM}\right)$</td></tr>
<tr><td rowspan="2">Internal force of the pile section at depth z below the ground or local scour line</td><td colspan="2">Bending moment</td><td colspan="2">$M_z = \alpha^2 EI\left(x_0A_3 + \frac{\varphi_0}{\alpha}B_3 + \frac{M_0}{\alpha^2 EI}C_3 + \frac{H_0}{\alpha^3 EI}D_3\right)$</td></tr>
<tr><td colspan="2">Shear force</td><td colspan="2">$Q_z = \alpha^3 EI\left(x_0A_4 + \frac{\varphi_0}{\alpha} + \frac{M_0}{\alpha^2 EI}C_4 + \frac{H_0}{\alpha^3 EI}D_4\right)$</td></tr>
<tr><td>Horizontal displacement at the head of the pile column</td><td colspan="4">$\Delta = x_0 - \varphi_0(h_2 + h_1)\Delta_0$
where
$\Delta \frac{H}{E_1I_1}\left[\frac{1}{3}(nh_1^3 + h_2^3) + nh_1h_2(h_1 + h_2)\right] + \frac{M}{2E_1I_1}[h_2^2 + nh_1(2h_2 + h_1)]$</td></tr>
</table>

Note: Physical meanings of δ^0_{HH}, $\delta^{(0)}_{MH}$, $\delta^{(0)}_{HM}$ and $\delta^{(0)}_{MM}$ in the table refer to Fig. L.0.3.

The h_1 is the length of the pile that is over the ground, h_2 is the height of the pier.

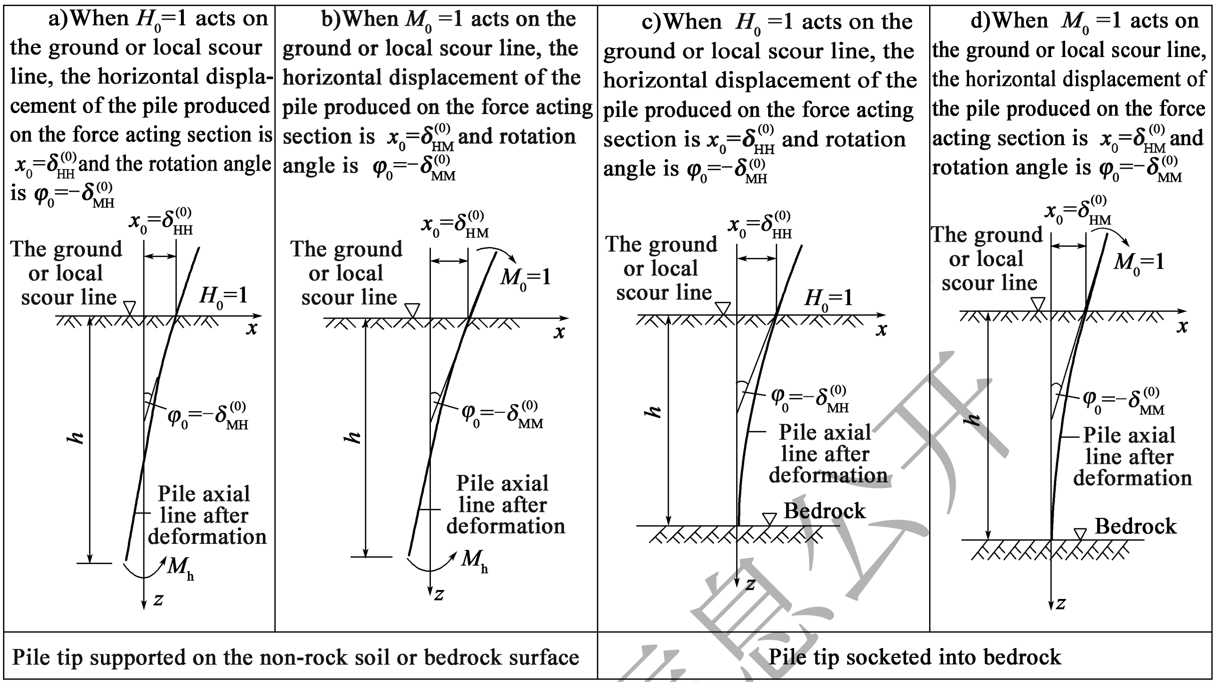


Fig. L. 0. 3 Deformation of Pile Under Load Action

L. 0. 4 If $\alpha h > 2.5$, the action effect and the displacement of the pile bent abutment with single-row piles under the earth pressure on the pile column side surface may be calculated according to Table L. 0. 4, and shall comply with the following requirements:

1 Table L. 0. 4 is applicable for piles with $\alpha h > 2.5$. For piles with $\alpha h \leqslant 2.5$, refer to Appendix M of the *Specifications*.

2 In calculating $\delta_{HH}^{(0)}$, $\delta_{MH}^{(0)}$, $\delta_{HM}^{(0)}$ and $\delta_{MM}^{(0)}$, the coefficients A_i, B_i, C_i, D_i ($i = 1, 2, 3, 4$) are taken from Table L. 0. 8 according to $\bar{h} = \alpha z$. In calculating M_z and Q_z, they are taken from Table L. 0. 8 according to $\bar{h} = \alpha z$. For $h = \alpha h$ and $\bar{h} = \alpha z$, h = 4 is considered when $h > 4$.

3 The $k_h = \dfrac{C_0}{\alpha E} \times \dfrac{I_0}{I}$ is the influence coefficient of the soil resistance at the pile tip due to rotation of the pile tip to $\delta_{HH}^{(0)}$, δ_{MH}^{0}, $\delta_{MM}^{(0)}$ and $\delta_{MM}^{(0)}$, where C_0 is determined according to Clause L. 0. 2, I and I_0 are the moment of inertia of cross-section of the pile body below ground or local scour line and the pile-tip, respectively. When the pile tip is located on a non-rock soil and $\alpha h \geqslant 2.5$, or when the pile tip is located on a rock bed and $\alpha h \geqslant 3.5$, $k_h = 0$.

4 The n is the ratio of $E_1 I_1$ to EI, where $E_1 I_1$ and EI are the flexural stiffness of the upper and

lower parts of the pile bent pier, respectively. EI is calculated according to Clause L. 0. 2, while $E_1I_1 = 0.8E_CI_c$, where E_c is the compressive elastic modulus of concrete of the pile body and I_1 is the moment of inertia for the cross-section of the upper part of the pile.

5 The q_1, q_2, q_3, and q_4 are the earth pressures acting on the piles (kN/m). The earth pressure and the effective width of the pile may be determined according to Clause 4. 2. 3 of *General Specifications for Design of Highway Bridges and Culverts* (JTG D60). If the pile cross-section above the ground or local scour line is constant, h_2 is taken as the full height, and $h_1 = 0$.

6 When the pile has anbearing depth $h \geqslant 4/a$, the action effect of the pile below depth $z = 4/a$ may be ignored (that is, the embedment length of pile can be taken as $z = 4/a$).

7 When there are two soil layers within $h_m = 2(d+1)$ (if $\alpha h \leqslant 2.5$, $h_m = h$ is taken) below the ground surface at the foundation side or the local scour line, the actual maximum bending moment of the pile body may be modified by the following formula;

$$M_{\max} = \xi M_{z\max} \tag{L. 0. 4-1}$$

where:

$M_{z\max}$——maximum bending moment of the pile body calculated by Table L. 0. 3 or Table L. 0. 4;

$M_{\max}$——actual maximum bending moment of the pile body;

ξ——modification coefficient for maximum bending moment, which may be calculated by the following formulas:

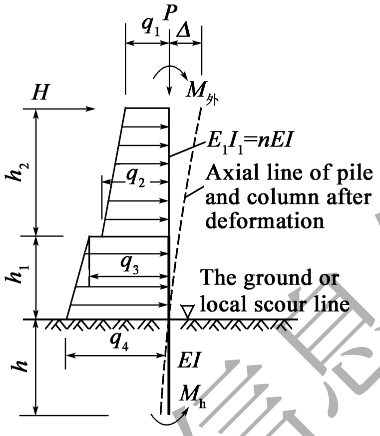
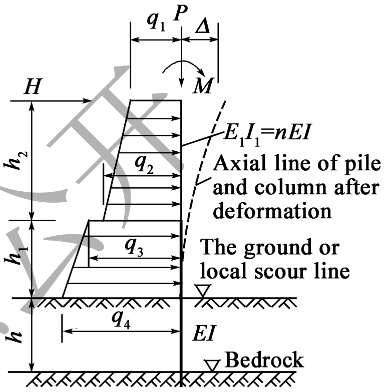
$$\begin{cases} \xi = \dfrac{2\delta}{\delta+2}\dfrac{h_1}{h_m}+1 & \dfrac{h_1}{h_m} \leqslant \dfrac{1}{6}(\delta+2) \\ \xi = \dfrac{2\delta}{\delta-4}\dfrac{h_1}{h_m}+\dfrac{4+\delta}{4-\delta} & \dfrac{h_1}{h_m} > \dfrac{1}{6}(\delta+2) \end{cases} \tag{L. 0. 4-2}$$

$$\delta = \frac{H_0}{H_0 + 0.1M_0}\lg\frac{m_2}{m_1} \tag{L. 0. 4-3}$$

where, the unit of H_0 is kN and the unit of M_0 is kN · m.

注：表中 $\delta_{HH}^{(0)}$, $\delta_{MH}^{(0)}$, $\delta_{HM}^{(0)}$, $\delta_{MM}^{(0)}$ 的物理意义见图 L. 0. 3.

Table L. 0. 4　Table for Calculations of Pile Bent Abutment with Single-Row Piles Under Earth Pressure on the Pile Column Side Surface

<table>
<tr>
<td colspan="3">Computational scheme</td>
<td>For pile bent abutment with a single-row piles, the pile column is subjected to trapezoidal loads, the pile column top is free and the pile tip is supported by the non-rock soil or bedrock surface.
</td>
<td>For pile bent abutment with a single-row piles, the pile column top is subjected to trapezoidal loads, the pile column top is free and the pile tip is socketed into the bedrock.
</td>
</tr>
<tr>
<td colspan="2" rowspan="2">Action effect on the pile section at the ground or local scour line</td>
<td>Bending moment</td>
<td colspan="2">$M_0 = M + H(h_1 + h_2) + \frac{1}{6}h_2[(2q_1 + q_2)h_2 + 3(q_1 + q_2)h_1] + \frac{1}{6}(2q_3 + q_4)h_1^2$</td>
</tr>
<tr>
<td>Shear force</td>
<td colspan="2">$H_0 = H + \frac{1}{2}(q_1 + q_2)h_2 + \frac{1}{2}(q_3 + q_4)h_1$</td>
</tr>
<tr>
<td rowspan="4">Deformation produced on the force acting section when a unit ‘force’ is applied on the ground or local scour line</td>
<td rowspan="2">Action of $H_0 = 1$</td>
<td>Horizontal displacement</td>
<td>$\delta_{HH}^{(0)} = \frac{1}{\alpha^3 EI} \times \frac{(B_3D_4 - B_4D_3) + k_h(B_2D_4 - B_4D_2)}{(A_3B_4 - A_4B_3) + k_h(A_2B_4 - A_4B_2)}$</td>
<td>$\delta_{HH}^{(0)} = \frac{1}{\alpha^3 EI} \times \frac{B_2D_12 - B_1D_2}{A_2B_1 - A_1B_2}$</td>
</tr>
<tr>
<td>Rotation angle(rad)</td>
<td>$\delta_{MH}^{(0)} = \frac{1}{\alpha^2 EI} \times \frac{(A_3D_4 - A_4D_3) + k_h(B_2D_4 - B_4D_2)}{(A_3B_4 - A_4B_3) + k_h(B_2D_4 - B_4D_2)}$</td>
<td>$\delta_{MH}^{(0)} = \frac{1}{\alpha^2 EI} \times \frac{A_2D_1 - A_1D_2}{A_2B_1 - A_1B_21}$</td>
</tr>
<tr>
<td rowspan="2">Action of DG 56.109mm $M_0 = 1$</td>
<td>Horizontal displacement</td>
<td>$\delta_{HM}^{0} = \delta_{MM}^{(0)} = \frac{1}{\alpha^2 EI} \times \frac{(B_3C_4 - B_4C_3) + k_h(B_2C_4 - B_4C_2)}{(B_3C_4 - A_4B_3) + k_h(A_2B_4 - A_4B_2)}$</td>
<td>$\delta_{MH}^{(0} = \delta_{HM}^{(0)} = \frac{1}{\alpha^2 EI} \times \frac{B_2C_1 - B_1C_2}{A_2B_1 - A_1B_2}$</td>
</tr>
<tr>
<td>Rotation angle(rad)</td>
<td>$\delta_{MM}^{(0)} = \frac{1}{\alpha EI} \times \frac{(A_3C_4 - A_4C_3) + k_h(A_2C_4 - A_4C_2)}{(A_3B_4 - A_4B_3) + k_h(A_2B_4 - A_4B_2)}$</td>
<td>$\delta_{MM}^{(0)} = \frac{1}{\alpha EI} \times \frac{A_2C_1 - A_1C_2}{A_2B_1 - A_2B_2}$</td>
</tr>
</table>

continued

Displacement of the pile section at the ground or local scour line	Horizontal displacement	$x_0 = H_0 \delta_{\mathrm{HH}}^{(0)} + M_0 \delta_{\mathrm{HM}}^{(0)}$
	Rotation angle (rad)	$\varphi_0 = -\left(H_0 \delta_{\mathrm{MH}}^{(0)} + M_0 \delta_{\mathrm{MM}}^{(0)} \right)$
Internal force of the pile section at depth z below the ground or local scour line	Bending moment	$M_z = \alpha^2 EI \left(x_0 A_3 + \frac{\varphi_0}{\alpha} B_3 + \frac{M_0}{\alpha^2 EI} C_3 + \frac{H_0}{\alpha^3 EI} D_3 \right)$
	Shear force	$Q_z = \alpha^3 EI \left(x_0 A_4 + \frac{\varphi_0}{\alpha} B_4 + \frac{M_0}{\alpha^2 EI} C_4 + \frac{H_0}{\alpha^3 EI} D_4 \right)$
Horizontal displacement at the head of the pile column		$\Delta = x_0 - \varphi_0 (h_2 + h_1) \Delta_0$ $\Delta_0 = \frac{M}{2E_1 I_1}(nh_1^2 + 2nh_1 h_2 + h_2^2) +$ Where: $\frac{H}{3E_1 I_1}(nh_1^3 + 3nh_1^2 h_2 + 3nh_1 h_2^2 + h_2^3) +$ $\frac{1}{120E_1 I_1}[(11h_2^4 + 40nh_2^3 h_1 + 20nh_1 h_1^3 + 50nh_2^2 h_1^2]q_1 +$ $4(h_2^4 + 10nh_2^2 h_1^2 + 5nh_2^3 h_1 + 5nh_2^3 h_1 + 5nh_2 h_1^3) q_4 +$ $(11nh_1^4 + 15nh_3 h_1^3) q_3 + (4nh_1^4 + 5nh_2 h)_1^3 q_4$

Note: The physical meanings of $\delta_{\mathrm{HH}}^{(0)}$, $\delta_{\mathrm{MH}}^{(0)}$, $\delta_{\mathrm{HM}}^{(0)}$ and δ_{MM}^{0} in the table are referred to Fig. L. 0. 3.

L. 0. 5 The maximum and minimum compressive stress of the soil mass at the pile tip shall meet the requirements of the following formulas:

$$P_{\substack{\max \\ \min}} = \frac{N_{hk}}{A_0} \pm \frac{M_{hk}}{W_0} \leqslant qr(\text{bored pile}) \text{ or } \alpha_r\, q_{rk}(\text{driven pile}) \quad (\text{L. 0. 5})$$

where:

$p_{\max}$, $p_{\min}$——maximum and minimum stresses of soil mass at the pile tip, respectively;

$\mathrm{N_{hk}}$——nominal value for the axial force at the pile tip. For non-rock ground, $N_{hk} = P_k + G_k - T_k$. For rock ground, $N_{hk} = P_k + G_k$;

P_k——nominal value of axial force on pile column top;

G_k——self-weight of all the pile columns. For drilled (bored) piles in non-rock ground, the self-weight of the pile below the local scour line should be deducted from the replaced soil weight (if the buoyancy of the pile is considered, that of the replaced soil shall also be considered);

T_k——sum of nominal value for the skin friction of the pile below the local scour line;

M_{hk}——bending moment at the pile tip, it is calculated using the formulas in Table L. 0. 4 for calculating M_z by setting $z = h$. When $\alpha h \geqslant 4$, $M_{hk} = 0$;

A_0, W_0——area and section modulus of pile tip, respectively;

q_r——characteristic value of bearing resistance of soil at pile tip (kPa), which is calculated according to provisions in Clause 6. 3. 3 of the *Specifications*;

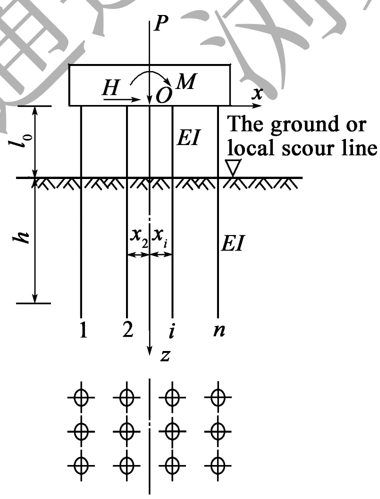
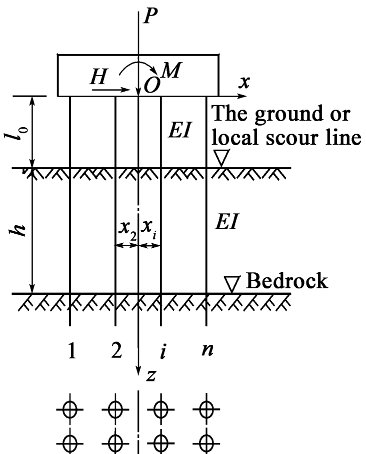
q_{rk}——nominal value of bearing resistance of soil at pile tip (kPa), which is taken from Table 6.3.5-2 of the *Specifications*.

α_p——influence coefficient for end bearing resistance of the driven pile, see Table 6.3.5-3 of the *Specifications*.

In addition, for piles supported on non-rock soil, rock surface with $\alpha h > 3.5$, or socketed into rock with $\alpha h > 4$, the pressure at the pile tip is considered to be evenly distributed, the compressive stress of the soil at the pile tip may not need to be checked. For pile supported by the bedrock surface, when $e > \rho$ (e is the load eccentricity, ρ is the core radius of the pile tip section), the redistribution of pressure on the pile tip shall be considered (refer to Appendix K of the *Specifications*); for the pile socketed into bedrock, the strength at the socketed section shall be checked.

L.0.6 If $\alpha h > 2.5$, the action effect and displacement of pile bent pier with multi-row (vertical) piles under load at the pile top may be calculated according to Table L.0.6.

Table L.0.6 Table for Calculations of the Action Effect and Displacement in Pile Bent Pier with Multi-Row Vertical Piles under Load at the Pile Top

<table>
<tr><td rowspan="2">Computational scheme</td><td colspan="2">Foundation of pile bent pier with multi-row symmetrically-arranged (vertical) piles
(pile foundation of high-rise cap)</td></tr>
<tr><td>(1) Pile tip set on non-rock soil or bedrock surface

</td><td>(2) Pile tip socketed into the bedrock

</td></tr>
</table>

continued

Displacement produced on the pile top under a unit force at the pile top	Action of $H=1$	Horizontal displacement	$\delta_{HH}=\frac{l_0^3}{3EI}+\delta^{(0)}_{MM}l_0^2+2\delta^0_{MH}l_0+\delta^{(0)}_{HH}$	δ^0_{HH}、δ^0_{MH}、δ^0_{HM} and δ^0_{MM} are calculated by the formula listed in Table L.0.3 or L.0.4 according to the restrained conditions of the pile tips
		Rotation angle	$\delta_{MH}=\frac{l_0^2}{2EI}+\delta^{(0)}_{MM}l_0+\delta$	
	Action of $M=1$	Horizontal displacement	$\delta_{HM}=\delta_{MH}=\frac{l_0^2}{2EI}+\delta^{(0)}_{MM}+\delta^{(0)}_{HM}$	
		Rotation angle	$\delta_{MM}=\frac{l_0}{EI}+\delta^{(0)}_{MM}$	
Action effect at the pile top section produced by a unit displacement at the pile top	Axial force produced at the pile top section by a unit displacement along the pile axis		$\rho_{pp}=\cfrac{1}{\cfrac{l_0+\xi h}{EA}+\cfrac{1}{C_0A_0}}$	ξ- coefficient; for end-bearing pile, $\xi=1$; for friction pile (or friction bearing pipe pile), when it is driven by hammering or vibration, $\xi=2/3$; when it is drilled by bore or dig, $\xi=1/2$ A- average sectional area of the buried part of the pile; A_0- is calculated by the following formula: Friction pile: $A_0=\begin{cases}\pi\left(\frac{d}{2}+h\tan\frac{\bar{\varphi}}{4}\right)^2\\ \frac{\pi}{4}S^2\end{cases}$ End-bearing pile: $A=\pi d^2/4$ $\bar{\varphi}$——average internal friction angle of the soil layers the pile penetrating through; s——center-to-center distance of the pile tips; d——diameter of pile tip.
	Horizontal force produced at the pile top section by a unit displacement perpendicular to the pile vertical axis		$\rho_{HH}=\frac{\delta_{MM}}{\delta_{HH}\delta_{MM}-(\delta_{MH})}$	
	Bending moment produced at the pile top section by a unit displacement perpendicular to the pile vertical axis		$\rho_{MH}=\frac{\delta_{MH}}{\delta_{HH}\delta_{MM}-(\delta_{MH})}$	
	Horizontal force produced at the pile top section by a unit rotation angle at the pile top		$\rho_{HM}=\rho_{MH}$	
	Bending moment produced at the pile top section by a unit rotation angle at the pile top		$\rho_{MM}=\frac{\delta_{HH}}{\delta_{HH}\delta_{MM}-(\delta_{MH}}$	

continued

<table>
<tr><td rowspan="4">Sum of reaction forces of all pile top to the pile cap, under a unit displacement at the pile cap</td><td>Sum of vertical reaction forces at the pile top, under a unit vertical displacement at the pile cap</td><td>$\gamma_{cc} = n\rho_{pp}$</td><td rowspan="4">n——total number of piles;
x_i——distance from the origin of coordinate O to each pile axis;
k_i——number of piles in i^{th} row.</td></tr>
<tr><td>Sum of horizontal reaction forces at the pile top, under a unit horizontal displacement at the pile cap</td><td>$\gamma_{aa} = n\rho_{HH}$</td></tr>
<tr><td>Sum of horizontal reaction forces at the pile top, under a unit rotation angle around the center O of the pile cap; or, sum of reaction bending moments at the pile top, under a unit horizontal displacement at the pile cap</td><td>$\gamma_{\alpha\beta} = \gamma_{\beta\alpha} = -n\rho_{HM} = -$</td></tr>
<tr><td>Sum of reaction bending moments at the pile top, under a unit rotation angle at the pile cap</td><td>$\gamma_{\beta\beta} = n\rho_{MM} + \rho_{PP}\sum K_i$</td></tr>
<tr><td rowspan="3">Cap displacement</td><td>Vertical displacement</td><td>$c = \dfrac{p}{\gamma_{cc}}$</td><td rowspan="3">$P$, H, and M are the vertical force, horizontal force, and bending moment at the origin of coordinate O of the bottom of the pile cap, respectively</td></tr>
<tr><td>Horizontal displacement</td><td>$a = \dfrac{\gamma_{\beta\beta}H - \gamma_{\alpha\beta}M}{\gamma_{aa}\gamma_{\beta\beta} - (\gamma_{\alpha\beta})^2}$</td></tr>
<tr><td>Rotation angle (rad)</td><td>$\beta = \dfrac{\gamma_{aa}M - \gamma_{\alpha\beta}H}{\gamma_{aa}\gamma_{\beta\beta} - (\gamma_{\alpha\beta})^2}$</td></tr>
<tr><td rowspan="3">Action effect at the pile top</td><td>Axial force at any pile top</td><td>$N_i = (c + \beta x_i)\rho_{pp}$</td><td>$x_i$ value is positive when it is in the right side of the origin of coordinate O, while negative in the left side.</td></tr>
<tr><td>Shear force at any pile top</td><td colspan="2">$Q_i = a\rho_{HH} - \beta\rho_{HM} = \dfrac{H}{n}$</td></tr>
<tr><td>Bending moment at any pile top</td><td colspan="2">$M_i = \beta\rho_{MM} - a\rho_{MH}$</td></tr>
<tr><td rowspan="2">'Force' acting on the section of top pile at the ground or local scour line</td><td colspan="2">Horizontal force</td><td>$H_0 = Q_i$</td></tr>
<tr><td colspan="2">Bending moment</td><td>$M_0 = M_i + Q_i l_0$</td></tr>
</table>

Note: The physical meanings of δ_{HH}, δ_{MH}, δ_{HM} and δ_{MM} in the table are referred to Fig. L. 0. 6.

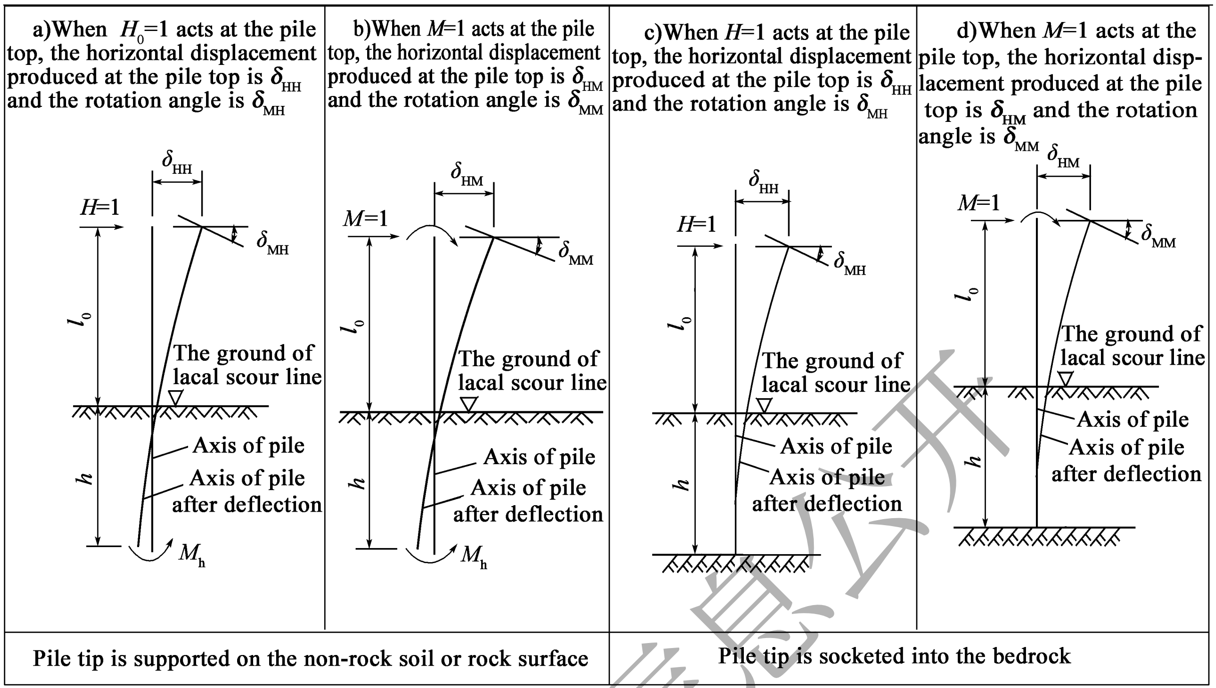


Fig L. 0. 6 Deformation of Pile Under Load Action

L. 0. 7 If $\alpha h > 2.5$, the action effect and displacement of an abutment with multi-row vertical piles under earth pressure on pile column side surfaces may be calculated according to Table L. 0. 7. The calculation shall comply with the following provisions.

1 The q_1 and q_2 are the earth pressure acting on the pile. The earth pressure and the effective width of the pile may be determined according to the current *General specifications for Design of Highway Bridges and Culverts* (JTG D60).

2 If the piles are arranged asymmetrically, the origin of coordinate O of the pile cap bottom may be selected freely; if the piles are arranged symmetrically, it should be selected on the symmetric axis as shown in Table L. 0. 7.

3 If the vertical piles are arranged asymmetrically, calculation formulas are as follows:

1) If no lateral earth pressure acts on the side surface of the pile, the cap vertical displacement c, the horizontal displacement a, and the rotation angle β shall be obtained by solving simultaneously the equation group as follows:

$$\left.\begin{aligned} c\gamma_{cc} + \beta\gamma_{c\beta} - P = 0 \\ a\gamma_{aa} + \beta\gamma_{\alpha\beta} - H = 0 \\ a\gamma_{\beta a} + c\gamma_{\beta c} + \beta\gamma_{\beta\beta} - M = 0 \end{aligned}\right\} \tag{L.0.7-1}$$

2) If lateral earth pressure acts on the side surface of the pile, the cap vertical displacement c, the horizontal displacement a, and the rotation angle β shall be obtained by solving simultaneously the equation group as follows:

$$\left.\begin{aligned} c\gamma_{cc} + \beta\gamma_{c\beta} - P = 0 \\ a\gamma_{aa} + \beta\gamma_{\alpha\beta} - (H - \sum Q_q) = 0 \\ a\gamma_{\beta a} + c\gamma_{\beta\beta} - (M - \sum M_q) = 0 \end{aligned}\right\} \tag{L.0.7-2}$$

where:

$\gamma_{c\beta} = \gamma_{\beta c} = \rho_{pp} \sum K_i x_i$——sum of vertical reaction forces of all pile tops acting on the pile cap when there is a unit rotation angle around the origin of coordinate O of the cap, or sum of reaction bending moments of all pile tops acting on the pile cap when there is a unit vertical displacement on the pile cap;

x_i——distance from the origin of coordinate O to each pile axis. It is positive when it is located on the right side of the origin O, while negative on the left side;

$\sum Q_q, \sum M_q$——sum of Q_q and M_q for all piles under the direct actions of earth pressures, respectively.

3) If the ground or local scour line are located above the pile cap bottom, the soil around the pile cap may be regarded as an elastic medium, and the shape coefficients $\gamma_{cc}, \gamma_{aa}, \gamma_{a\beta}, \gamma_{c\beta}$ as well as $\gamma_{\beta\beta}$ may be calculated by the following formulas:

$$\left.\begin{aligned} \gamma_{cc} &= n\rho_{\mathrm{pp}} \\ \gamma_{aa} &= n\rho_{\mathrm{HH}} + b_1 F^c \\ \gamma_{ap} &= \gamma_{\beta a} = -n\rho_{\mathrm{HM}} + b_1 S^c = -n\rho_{\mathrm{MH}} + b_1 S^c \\ \gamma_{c\beta} &= \gamma_{\beta c} = \rho_{\mathrm{pp}} \sum k_i x_i \\ \gamma_{\beta\beta} &= n\rho_{\mathrm{MM}} + \rho_{\mathrm{pp}} \sum k_i x_i^2 + b_1 I^c \end{aligned}\right\} \tag{L.0.7-3}$$

where:

b_1——effective width of the bottom of the pile cap perpendicular to the horizontal force, which is determined by Clause L.0.1;

F^c, S^c, I^c——graph area of the horizontal coefficient of ground reaction above the bottom of the pile cap, and its section modulus and moment of inertia with respect to the axis of the bottom surface of the pile cap.

$$\left.\begin{aligned} F^c &= \frac{C_c h_c}{2} \\ S^c &= \frac{C_c h^2}{6} \\ I^c &= \frac{C_c h^3}{12} \end{aligned}\right\} \tag{L.0.7-4}$$

where:

C_c—— horizontal coefficient of ground reaction at the pile cap bottom, $c_c = mh_c$;

h_c——depth of the pile cap bottom buried below the ground or local scour line.

In the calculation of ρ_{PP}, ρ_{HH}, ρ_{MH} and ρ_{MM}, let l_0 in the related formulas. In taking coefficients $A_1, B_1 \ldots C_4, D_4$ from the table, z in the transformed depth $\bar{h} = \alpha z$ is measured from the pile cap bottom, and the earth pressure on the side surface of the pile may be ignored.

4 After the pile bending moment M_0 (it is M_0' for pile subjected to trapezoidal load directly) and horizontal force H_0 (it is H_0' for pile subjected to direct trapezoidal load directly) at the ground or local scour line are obtained according to Tables L. 0. 6 and L. 0. 7, the horizontal displacement x_0 and rotation angle φ_0 at the ground or local scour line, the bending moment M_z and shear force Q_z for each cross-section at depth z below the ground or local scour line, as well as the maximum and minimum compressive stress $P_{\max}$ and $P_{\min}$ at the pile tip can be calculated according to Tables L. 0. 3 and Table L. 0. 4.

5 The meanings of other symbols in this table are the same as those in Tables L. 0. 3 and L. 0. 4.

6 The horizontal displacement on the top of the pier or abutment with multi-row piles, Δ, is calculated by the following formula:

$$\Delta = a + \beta l + \Delta_0 \tag{L.0.7-5}$$

where:

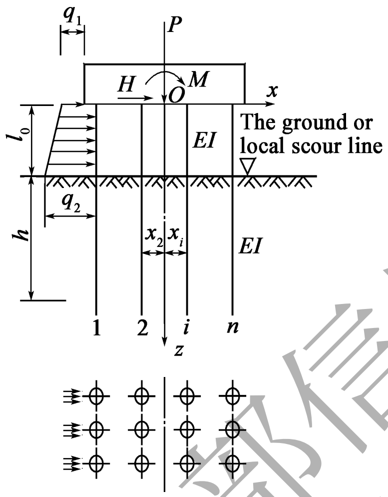
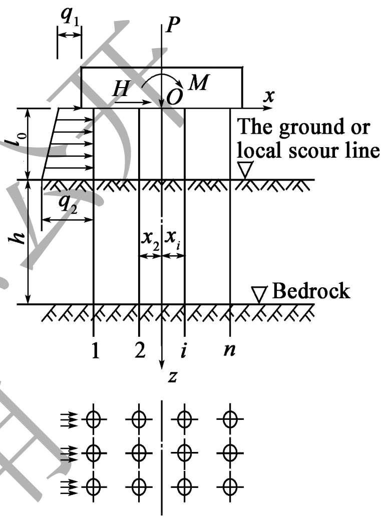
a——horizontal displacement at the cap bottom;

β——rotation angle at the cap bottom;

l——distance from pier/abutment top to its cap bottom;

Δ_0——horizontal displacement of pier/abutment top caused by the elastic deflection between the pile cap bottom and the pier/abutment top surface.

Table. L. 0. 7 Calculation Table for the Abutment with Multi-Row Vertical Piles under Earth Pressure on the Pile Sides

<table>
<tr><td rowspan="2">Computational scheme</td><td colspan="4">Abutment foundation with multi-row, symmetrically-arranged vertical piles (pile foundation of high-rise cap, the piles are subjected to trapezoidal load)</td></tr>
<tr><td colspan="3">(1) Pile tip supported on non-rock soil or bedrock surface

</td><td>(2) Pile tip socketed into the bedrock

</td></tr>
<tr><td rowspan="4">Displacement produced on the pile top under a unit force at the pile top</td><td rowspan="2">Action of H = 1</td><td>Horizontal displacement</td><td>$\delta_{HH} = \frac{l_0^3}{3EI} + \delta_{MM}^{(0)} l_0^2 + 2\delta_{MH}^0 l_0 + \delta_{HH}^{(0)}$</td><td rowspan="4">$\delta_{HH}^{(0)}$, δ_{MH}^0, δ_{HM}^0, and δ^{MM} are calculated by the formulas in Table L. 0. 3 or L. 0. 4 based on the embedment conditions of the pile tip</td></tr>
<tr><td>Rotation angle (rad)</td><td>$\delta_{MH} = \frac{l_0^2}{2EI} + \delta_{MM}^{(0)} l_0 + \delta_{MH}^0$</td></tr>
<tr><td rowspan="2">Action of M = 1</td><td>Horizontal displacement</td><td></td></tr>
<tr><td>Rotation angle (rad)</td><td>$\delta_{MM} = \frac{l_0}{EI} + \delta_{MM}^{(0)}$</td></tr>
</table>

continued

<table>
<tr>
<td rowspan="5">Action effect at the pile top section produced by a unit displacement at the pile top</td>
<td>Axial force produced at the pile top section by a unit displacement along the pile axis</td>
<td>$\rho_{PP} = \dfrac{1}{\dfrac{l_0 + \xi h}{EA} + \dfrac{1}{C_0 A_0}}$</td>
<td rowspan="5">ξ——coefficient; for end-bearing pile, $\xi = 1$; for friction pile (or friction-bearing pipe pile), when it is driven by hammering or vibration, $\xi = 2/3$; when it is bored or excavated, $\xi = 1/2$
A——average sectional area of the buried part of the pile;
A_0——it is calculated by the following formulas:
For friction pile:
$A_0 = \begin{cases} \pi\left(\dfrac{d}{2} + h\tan\dfrac{\overline{\varphi}}{4}\right)^2 \\ \dfrac{\pi}{4}S^2 \end{cases}$
For end-bearing pile:
$A_0 = \pi d^2/4$
$\overline{\varphi}$——average internal friction angle of the soil layer the pile penetrating through;
S——center-to-center distance of the pile tips;
d——diameter of pile tip.</td>
</tr>
<tr>
<td>Horizontal force produced at the pile top section by a unit displacement perpendicular to the pile vertical axis</td>
<td>$\rho_{HH} = \dfrac{\delta_{MM}}{\delta_{HH}\delta_{MM} - (\delta_{MH})^2}$</td>
</tr>
<tr>
<td>Bending moment produced at the pile top section by a unit displacement perpendicular to the pile vertical axis</td>
<td>$\rho_{MH} = \dfrac{\delta_{MH}}{\delta_{HH}\delta_{MM} - (\delta_{MH})^2}$</td>
</tr>
<tr>
<td>Horizontal force produced at the pile top section by a unit rotation angle at the pile top</td>
<td>$\rho_{HM} = \rho_{MH}$</td>
</tr>
<tr>
<td>Bending moment produced at the pile top section by a unit rotation angle at the pile top</td>
<td>$\rho_{MM} = \dfrac{\delta_{HH}}{\delta_{HH}\delta_{MM} - (\delta_{MH})^2}$</td>
</tr>
<tr>
<td rowspan="4">Sum of reaction forces of all pile top to the pile cap, under a unit displacement at the pile cap</td>
<td>Sum of vertical reaction forces at the pile top, under a unit vertical displacement at the pile cap</td>
<td>$\gamma_{cc} = n\rho_{pp}$</td>
<td rowspan="4">n——total number of piles;
x_i——distance from the origin of coordinate O to each pile axis;
K_i——number of piles in i^{th} row.</td>
</tr>
<tr>
<td>Sum of horizontal reaction forces at the pile top, under a unit horizontal displacement at the pile cap</td>
<td>$\gamma_{aa} = n\rho_{HH}$</td>
</tr>
<tr>
<td>Sum of horizontal reaction forces at the pile top, under a unit rotation angle around the center O of the pile cap; or, sum of reaction bending moments at the pile top, under a unit horizontal displacement at the pile cap</td>
<td>$\gamma_{a\beta} = \gamma_{\beta a} = -n\rho_{HM} = -n\rho_{MH}$</td>
</tr>
<tr>
<td>Sum of reaction bending moments at the pile top, under a unit rotation angle at the pile cap</td>
<td>$\gamma_{\beta\beta} = n\rho_{MM} + \rho_{pp}\sum K_i x_i^2$</td>
</tr>
</table>

continued

Cap displacement	Vertical displacement	$c = \dfrac{p}{\gamma_{cc}}$	P, H, and M are the vertical force, horizontal force, and bending moment at the origin of coordinate O of the bottom of the pile cap, respectively. $\sum M_q$ and $\sum Q_q$ are the sum of reaction bending moment and shear force from the pile top which are caused by the earth pressure on the pile, respectively. See the last term in this table.
	Horizontal displacement	$a = \dfrac{\gamma_{\beta\beta}(H - \sum Q_q) - \gamma_{a\beta}(M - \sum M)}{\gamma_{aa}\gamma_{\beta\beta} - (\gamma_{a\beta})^2}$	
	Rotation angle (rad)	$b = \dfrac{\gamma_{aa}(M - \sum M_q) - \gamma_{a\beta}(H - \sum Q_q)}{\gamma_{aa}\gamma_{\beta\beta} - (\gamma_{a\beta})^2}$	
Action effect at the pile top	Axial force at any pile top	$N_i = (c + \beta x_i)\rho_{pp}$	x_i value is positive or negative when it is in the right side or left side of the origin of coordinate O, respectively.
	Shear force at any pile top	$Q_i = \alpha\rho_{HH} - \beta\rho_{HM}$ For pile subjected to earth pressure directly: $Q_i' = Q_i + Q_q$	
	Bending moment at any pile top	$M_i = \beta\rho_{MM} - a\rho_{MH}$ For pile subjected to earth pressure directly: $M_i' = M_i + M_q$	
'Force' acting on the pile section at the ground or local scour line	Horizontal force	$H_0 = Q_i$ For pile directly subjected to earth pressure: $H_0' = Q_i + Q_q + \left(\dfrac{q_1 + q_2}{2}\right)l_0$	q_1, q_2——earth pressure intensity acting on the pile top and pile section at the ground level, respectively; M_q, Q_q—— bending moment and shear force acting on the pile cap from the forces at the pile tops (connected with the pile cap), the piles are subjected to earth pressure directly, as illustrated in Fig. L.0.7, where M_q and Q_q are all in positive direction.
	Bending moment	$M_0 = M_i + Q_i l_0$ For pile directly subjected to earth pressure: $M_0 = M_i + M_q + (Q_i + Q_q)$ $l_0 + \left(\dfrac{2q_1 + q_2}{6}\right)l_0^2$	

continued

M_q and Q_q are solved by simultaneous equations	$$M_{l_0} = M_q + Q_q l_0 + \left(\frac{q_1}{2! + \frac{q_2 - q_1}{3!}}\right) l_0^2$$ $$Q_{l_0} = Q_q + \left(q_1 + \frac{q_2 - q_1}{2!}\right) l_0$$ $$\frac{1}{EI}\left[\frac{M_q l_0^2}{2!} + \frac{Q_q l_0^3}{3!} + \frac{q_1 l_0^4}{4!} + \frac{(q_2 - q_1) l_0^4}{5!}\right]$$ $$M_{l_0} \delta_{HM}^{(0)} + Q_{l_0} \delta_{HH}^{(0)}$$ $$\frac{1}{EI}\left[M_q l_0 + \frac{Q_q l_0^2}{2!} + \frac{q_1 l_0^3}{3!} + \frac{(q_2 - q_1) l_0^3}{4!}\right]$$ $$= -[M_{l_0} \delta_{MM}^{(0)} + Q_{l0} \delta_{MH}^{(0)}]$$	M_{l_0}, Q_{l_0}——bending moment and shear force of the pile at the ground, in which the pile is subjected to earth pressure directly, the upper end of the pile is considered to be socketed rigidly in the pile cap, while the pile tip is considered to be socketed elastically in the ground, as shown in Fig. L. 0. 7.

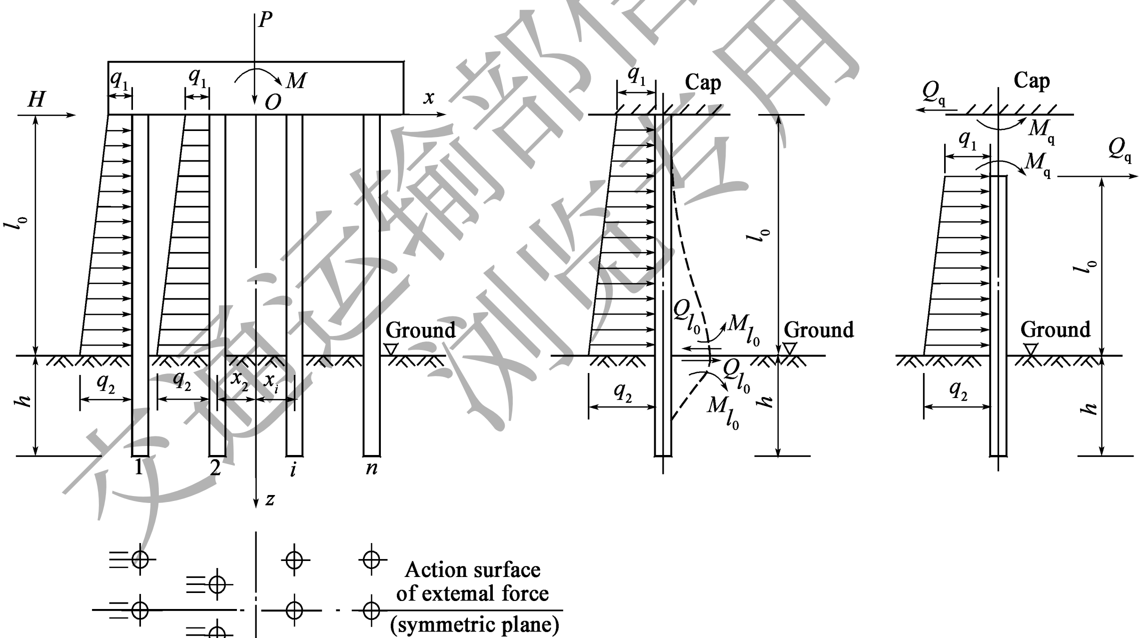


Fig. L. 0. 7 Schematic Diagram for Calculation of M_q and Q_q

L. 0. 8 The dimensionless coefficients for calculation of pile action effects shall be taken according to Table L. 0. 8.

Table. L. 0. 8 Table of Dimensionless Coefficients for Calculations of Action Effects of Pile

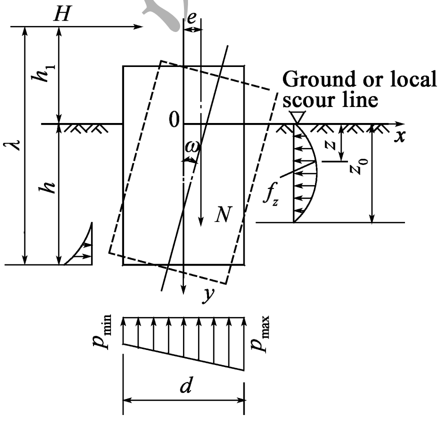
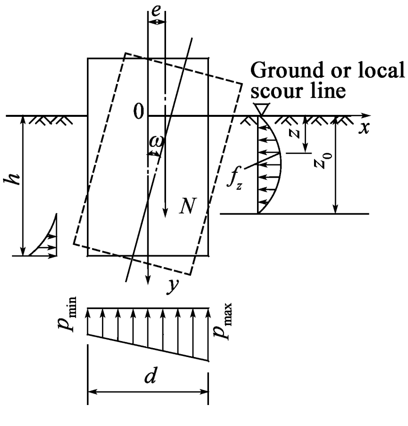
$\overline{h}=\alpha z$	A_1	B_1	C_1	D_1	A_2	B_2	C_2	D_2	A_3	B_3	C_3	D_3	A_4	B_4	C_4	D_4
0	1.00000	0.00000	0.00000	0.00000	0.00000	1.00000	0.00000	0.00000	0.00000	0.00000	1.00000	0.00000	0.00000	0.00000	0.00000	1.00000
0.1	1.00000	0.10000	0.00500	0.00017	0.00000	1.00000	0.10000	0.00500	−0.00017	−0.00001	1.00000	0.10000	−0.00500	−0.00033	−0.00001	1.00000
0.2	1.00000	0.20000	0.02000	0.00133	−0.00007	1.00000	0.20000	0.02000	−0.00133	−0.00013	0.99999	0.20000	−0.02000	−0.00267	−0.00020	0.99999
0.3	0.99998	0.30000	0.04500	0.00450	−0.00034	0.99996	0.30000	0.04500	−0.00450	−0.00067	0.99994	0.30000	−0.04500	−0.00900	−0.00101	0.99992
0.4	0.99991	0.39999	0.08000	0.01067	−0.00107	0.99983	0.39998	0.08000	−0.01067	−0.00213	0.99974	0.39998	−0.08000	−0.02133	−0.00320	0.99966
0.5	0.99974	0.49996	0.12500	0.02083	−0.00260	0.99948	0.49994	0.12499	−0.02083	−0.00521	0.99922	0.49991	−0.12499	−0.04167	−0.00781	0.99896
0.6	0.99935	0.59987	0.17998	0.03600	−0.00540	0.99870	0.59981	0.17998	−0.03600	−0.01080	0.99806	0.59974	−0.17997	−0.07199	−0.01620	0.99741
0.7	0.99860	0.69967	0.24495	0.05716	−0.01000	0.99720	0.69951	0.24494	−0.05716	−0.02001	0.99580	0.69935	−0.24490	−0.11433	−0.03001	0.9944
0.8	0.99727	0.79927	0.31988	0.08532	−0.01707	0.99454	0.79891	0.31983	−0.08532	−0.03412	0.99181	0.79854	−0.31975	−0.17060	−0.05120	0.98908
0.9	0.99508	0.89852	0.40472	0.12146	−0.02733	0.99016	0.89779	0.40462	−0.12144	−0.05466	0.98524	0.89705	−0.40443	−0.24284	−0.08198	0.98032
1.0	0.99167	0.99722	0.49941	0.16657	−0.04167	0.98333	0.99583	0.49921	−0.16652	−0.08329	0.97501	0.99445	−0.49881	−0.33298	−0.12493	0.96667
1.1	0.98658	1.09508	0.60384	0.22163	−0.06096	0.97317	1.09262	0.60346	−0.22152	−0.12192	0.95975	1.09016	−0.60268	−0.44292	−0.18285	0.94634
1.2	0.97927	1.19171	0.71787	0.28758	−0.08632	0.95855	1.18756	0.71716	−0.28737	−0.17260	0.93783	1.18342	−0.71573	−0.57450	−0.25886	0.91712
1.3	0.96908	1.28660	0.84127	0.36536	−0.11883	0.93817	1.27990	0.84002	−0.36496	−0.23760	0.90727	1.27320	−0.83753	−0.72950	−0.35631	0.87638
1.4	0.95523	1.37910	0.97373	0.45588	−0.15973	0.91047	1.36865	0.97163	−0.45515	−0.31933	0.86573	1.35821	−0.96746	−0.90754	−0.47883	0.82102
1.5	0.93681	1.46839	1.11484	0.55997	−0.21030	0.87365	1.45259	1.11145	−0.55870	−0.42039	0.81054	1.43680	−1.10468	−1.11609	−0.63027	0.74745
1.6	0.91280	1.55346	1.26403	0.67842	−0.27194	0.82565	1.53020	1.25872	−0.67629	−0.54348	0.73859	1.50695	−1.24808	−1.35042	−0.81466	0.65156
1.7	0.88201	1.63307	1.42061	0.81193	−0.34604	0.76413	1.59963	1.41247	−0.80848	−0.69144	0.64637	1.56621	−1.39623	−1.61340	−1.03616	0.52871
1.8	0.84313	1.70575	1.58362	0.96109	−0.43412	0.68645	1.65867	1.57150	−0.95564	−0.86715	0.52997	1.61162	−1.54728	−1.90577	−1.29909	0.37368
1.9	0.79467	1.76972	1.75090	1.12637	−0.53768	0.58967	1.70468	1.73422	−1.11796	−1.07357	0.38503	1.63969	−1.69889	−2.22745	−1.60770	0.18071
2.0	0.73502	1.82294	1.92402	1.30801	−0.65822	0.47061	1.73457	1.89872	−1.29535	−1.31361	0.20676	1.64628	−1.84818	−2.57798	−1.96620	−0.05652
2.2	0.57491	1.88709	2.27217	1.72042	−0.95616	0.15127	1.73110	2.22299	−1.69334	−1.90567	−0.27087	1.57538	−2.12481	−3.35952	−2.84858	−0.69158
2.4	0.34691	1.87450	2.60882	2.19535	−1.33889	−0.30273	1.61286	2.51874	−2.14117	−2.66329	−0.94885	1.35201	−2.33901	−4.22811	−3.97323	−1.59151
2.6	0.033146	1.75473	2.90670	2.72365	−1.81479	−0.92602	1.33485	2.74972	−2.62126	−3.59987	−1.87734	0.91679	−2.43695	−5.14023	−5.35541	−2.82106
2.8	−0.38548	1.49037	3.12843	3.28769	−2.38756	−1.17548	0.84177	2.86653	−3.10341	−4.71748	−3.10791	0.19729	−2.34558	−6.02299	−6.99007	−4.44491
3.0	−0.92809	1.03679	3.22471	3.85838	−3.05319	−2.82410	0.06837	2.80406	−3.54058	−5.99979	−4.68788	−0.89126	−1.96928	−6.76460	−8.84029	−6.51972
3.5	−2.92799	−1.27172	2.46304	4.97982	−4.98062	−6.70806	−3.58647	1.27018	−3.91921	−9.54367	−10.34040	−5.85402	1.07408	−6.78895	−13.69240	−13.82610
4.0	−5.85333	−5.94097	−0.92677	4.54780	−6.53316	−12.15810	−10.60840	−3.76647	−1.61428	−11.73066	−17.91860	−15.07550	9.24368	−0.35762	−15.61050	−23.14040

Note: z—Depth below ground or below the maximum scour line.

Appendix M
Calculation Methods of Displacement and Action Effect for Rigid Pile

M.0.1 This appendix is applicable to the calculations of horizontal displacement and action effect for pile foundation with $\alpha h \leqslant 2.5$ and caisson foundation. For foundations supported on non-bedrock and deep foundations supported on bedrock, the methods shown in Table M.0.1-1 and Table M.0.1-2 may be used, respectively.

Table M.0.1-1 Calculation Method of Horizontal Displacement and Action effect of Rigid Pile Supported on Non-bedrock

Calculated scheme	(1) When horizontal force H acts together with eccentric vertical force N 	(2) When only eccentric vertical force N acts 
Rotation angle of the foundation	$\omega = \dfrac{6H}{Amh}$	$\omega = \dfrac{2\beta(Ne)}{mhB} = \dfrac{2\beta M}{mhB}$

continued

Distance from the foundation rotating center to the ground or to the local scour line	$z_0 = \dfrac{\beta b_1 h^2(4\lambda - h) + 6dW_0)}{2\beta b_1 h(3\lambda - h)}$	$z_0 = \dfrac{2h}{3}$
Bending moment on the foundation section at depth z below the ground or local scour line	$M_z = H(\lambda - h + z) - \dfrac{Hb_1 z^3}{2hA}(2z_0 - z)$	$M_z = M_1 - \dfrac{\beta M b_1 z^3}{6Bh}(2z_0 - z)$
Horizontal pressure on the foundation side face at depth z below ground or local scour line	$p_z = \dfrac{6H}{Ah}z(z_0 - z)$	$p_z = \dfrac{2\beta M}{Bh}z(z_0 - z)$
Vertical pressure at the bottom of the foundation	$p_{\substack{\max \\ \min}} = \dfrac{N}{A_0} \pm \dfrac{3dH}{A\beta}$	$p_{\substack{\max \\ \min}} = \dfrac{N}{A_0} \pm \dfrac{dM}{B}$
Coefficients in the table	$A = \dfrac{\beta b_1 h^3 + 18dW_0}{2\beta(3\lambda - h)}$; $B = \dfrac{1}{18}\beta b_1 h^3 + d \cdot W_0$; $\beta = \dfrac{mh}{c_0} = \dfrac{mh}{m_0 h} = \dfrac{m}{m_0}$; $\lambda = \dfrac{\sum M}{H}$	

Note: β——ratio of the coefficient of ground resistance at the side surface of foundation to the coefficient of ground resistance above the foundation base at the depth h; when the foundation base is supported on non-bedrock ground, m and m_o is taken from Table L. 0. 2-1; when it is supported on bedrock, C_0 is taken from Table L. 0. 2-2;

$\lambda = (\sum M)/H$—ratio of total bending moments at the centroid of the foundation base to the horizontal force (m), in which the bending moments are induced by the horizontal and vertical forces above the ground or local scour line;

d——foundation diameter or width on the horizontal force acting plane (perpendicular to the horizontal force acting direction) (m);

W_0——elastic section modulus of the edge in the foundation base;

b_1——effective width of the foundation (m), see Clause L. 0. 1;

A_0——area of foundation base (m^2);

N——nominal value of vertical force at the foundation base (including the foundation self-weight) (kN);

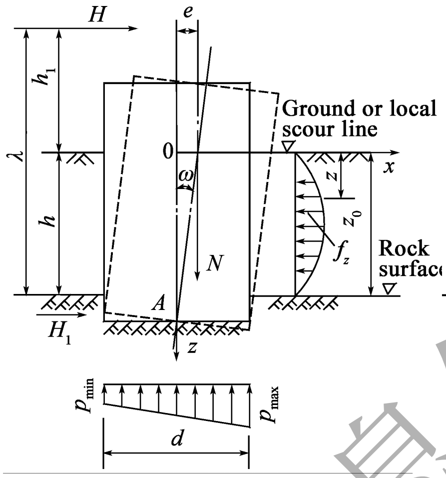
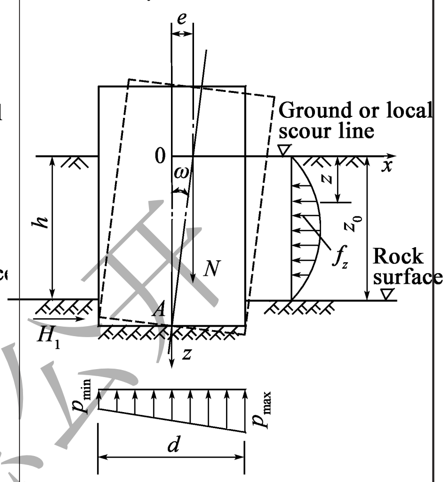
e——eccentricity of vertical force at the foundation base (m);

M——nominal value of bending moment at the foundation base induced by eccentric vertical force (kN · M);

N_1——vertical force on the foundation section at z depth (including the foundation self-weight above z) (kN);

M_1——bending moment on the foundation section at z depth induced by the eccentric vertical force N_1 (including the foundation self-weight above z) (kN · M), $M_1 = N_1 e_1$, where e_1 is the eccentricity of N_1 at depth z. If the foundation shape is symmetric, then $M_1 = N_1 e$.

Table M.0.1-2 Calculation Method of Horizontal Displacement and Action effect of Rigid Pile Supported on Bedrock

Calculated scheme	(1) When horizontal force H acts together with eccentric vertical force N: 	(2) When only eccentric vertical force N acts: 
Rotation angle of the foundation	$\omega = \frac{H}{mhD_0}$	$\omega = \frac{Ne}{D_1 mh} = \frac{M}{D_1 mh}$
Distance from the foundation rotating center to the ground or local scour line	$z_0 = h$	$z_0 = h$
Bending moment on foundation section at depth z below the ground or local scour line	$M_z = H(\lambda - h + z) - \frac{z^3 b_1 M}{12D_0 h}(2h - z)$	$M_z = M_1 - \frac{z^3 b_1 M}{12D_1 h}(2h - z)$
Horizontal pressure on foundation section at depth z below the ground or local scour line	$p_z = (h - z) z \frac{H}{D_0 h}$	$p_z = (h - z) z \frac{M}{D_1 h}$
Vertical pressure at the bottom of the foundation	$p_{\substack{\max \\ \min}} = \frac{N}{A_0} \pm \frac{dH}{2\beta D_0}$	$p_{\substack{\max \\ \min}} = \frac{N}{A_0} \pm \frac{dM}{2\beta D_1}$
Horizontal force at the foundation socketed point	$H_1 = H\left(\frac{b_1 h^2}{6D_0} - 1\right)$	$H_1 = b_1 \frac{h_2 M}{6D_1}$
Coefficient in the table		$D_0 = \frac{b_1\beta h^3 + 6dW_0}{12\lambda\beta}$; $D_1 = \frac{b_1\beta h^3 + 6dW_0}{12\beta}$; $\beta = \frac{mh}{c_0} = \frac{mh}{m_0 h} = \frac{m}{m_0}$; $\lambda = \frac{\sum M}{H}$

Note: the meanings of the symbols are the same as those in Table.0.1-1.

M. 0. 2 In order to ensure that the foundation is socketed reliably in soil, the horizontal pressure p_z on the foundation side face shall meet the following conditions:

$$\left.\begin{aligned} p_{h/3} &\leqslant \frac{4}{\cos\varphi}\left(\frac{\gamma}{3}h\tan\varphi + c\right)\eta_1\eta_2 \\ p_h &\leqslant \frac{4}{\cos\varphi}\gamma h\tan\varphi + c\eta_1\eta_2 \end{aligned}\right\} \tag{M.0.2}$$

where:

$p_{h/3}$, p_h —— horizontal pressure at depth of $z = h/3$ and $z = h$, respectively;

φ, γ, c —— internal friction angle, unit weight and cohesion of soil. For permeability soil, γ shall be taken as the buoyant unit weight; if there are several soil layers within the range of investigated depth, it shall be taken as the weighted average value of all layers of the soil;

η_1 —— coefficient; for piers/abutments in external statically indeterminate arch bridges, $\eta_1 = 0.7$; for pier/abutment in other bridges, $\eta_1 = 1.0$;

η_2 —— coefficient for considering the percentage of the structure self-weight in the total load, $\eta_2 = 1 - 0.8\frac{M_g}{M}$;

M_g —— bending moment produced by structure self-weight on the centroid of the foundation base;

M —— bending moment produced by all loads on the centroid of the foundation base.

M. 0. 3 The calculation of horizontal displacement at the pier/abutment top may be applied by the following formula:

$$\Delta = k_1\omega z_0 + k_2\omega l_0 + \delta_0 \tag{M.0.3}$$

where:

l_0 —— height from the ground or local scour line to the pier/abutment top;

δ_0 —— horizontal displacement at the pier/abutment top caused by the deformation of the pier/abutment stem within l_0 and the foundation;

k_1, k_2 —— coefficients considering the influence of foundation rigidity, which is adopted from Table M. 0. 3.

Table M. 0. 3 Coefficients k_1, k_2

Transformed depth $\bar{h} = \alpha h$	Coefficient	λ/h				
		1	2	3	5	∞
1.6	k_1	1	1	1	1	1
	k_2	1	1.1	1.1	1.1	1.1
1.8	k_1	1	1.1	1.1	1.1	1.1
	k_2	1.1	1.2	1.2	1.2	1.3

continued

Transformed depth $\bar{h} = \alpha h$	Coefficient	λ/h				
		1	2	3	5	∞
2	k_1	1.1	1.1	1.1	1.1	1.2
	k_2	1.2	1.3	1.4	1.4	1.4
2.2	k_1	1.1	1.2	1.2	1.2	1.2
	k_2	1.2	1.5	1.6	1.6	1.7
2.4	k_1	1.1	1.2	1.3	1.3	1.3
	k_2	1.3	1.8	1.9	1.9	2
2.5	k_1	1.2	1.3	1.4	1.4	1.4
	k_2	1.4	1.9	2.1	2.2	2.3

Note: 1. For $\alpha h < 1.6$, $k_1 = k_2 = 1.0$.

2. If there is only theeccentric vertical force, $\lambda/h \to \infty$.

Appendix N
Calculations for Pile Group as a Block Foundation

N. 0. 1 When a pile group (friction piles) is used as an block foundation, the pile foundation may be regarded as a block foundation within the range of '*acde*' as shown in Fig. N. 0. 1.

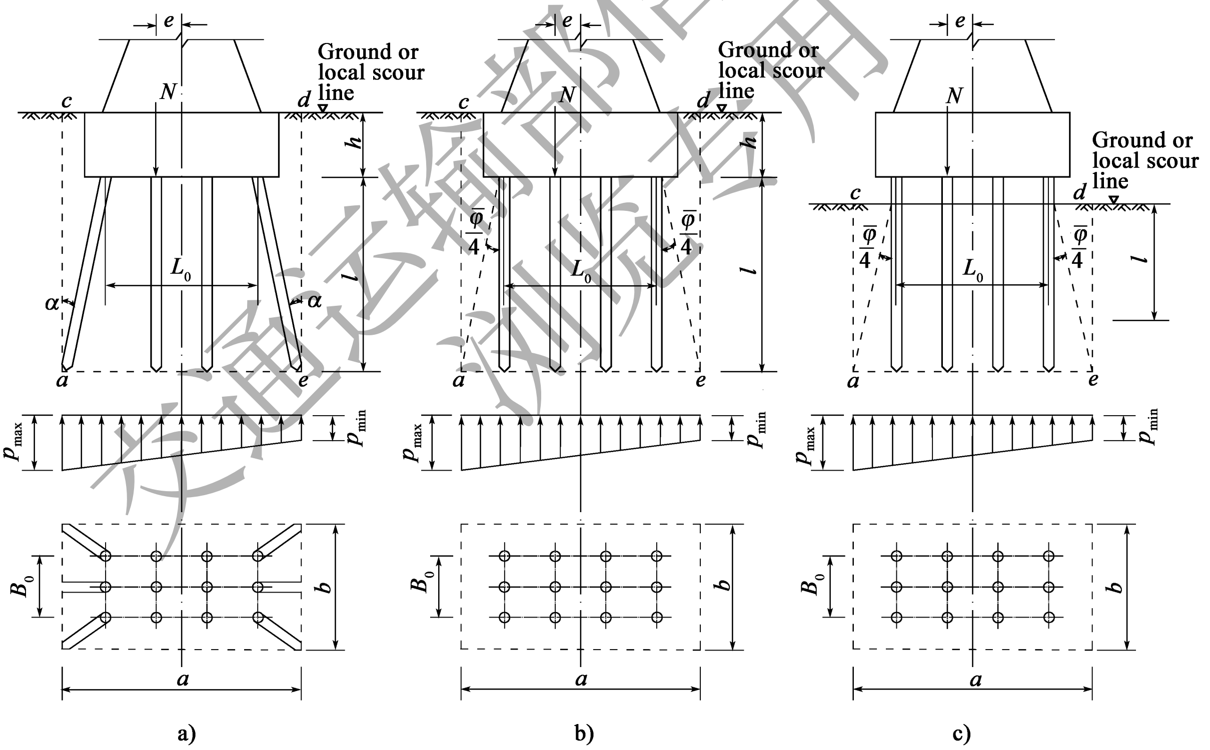


Fig. N. 0. 1 Calculated Scheme of Pile Group Acting as a Block Foundation

N. 0. 2 The calculation of the pile group as an block foundation shall comply with the following requirements:

1 Under axial compression:

$$p = \bar{\gamma} l + \gamma h - \frac{BL\gamma h}{A} + \frac{N}{A} \leqslant f_a \tag{N.0.2-1}$$

2 Under eccentric compression, it shall satisfy the following conditions beside Item 1:

$$p_{\max} = \overline{\gamma} l + \gamma h - \frac{BL\gamma h}{A} + \frac{N}{A}\left(1 + \frac{eA}{W}\right)A \leqslant \gamma_R f_a \quad (\text{N.0.2-2})$$

$$A = ab \quad (\text{N.0.2-2})$$

If the inclination ofa batter pile $\alpha \leqslant \frac{\overline{\varphi}}{4}$:

$$a = L_0 + d + 2l\tan\frac{\overline{\varphi}}{4} \quad (\text{N.0.2-4})$$

$$b = B_0 + d + 2l\tan\frac{\overline{\varphi}}{4} \quad (\text{N.0.2-5})$$

If the inclination of a batter pile $\alpha > \frac{\overline{\varphi}}{4}$:

$$a = l_0 + d + 2l\tan\alpha \quad (\text{N.0.2-6})$$

$$b = B_0 + d + 2l\tan\alpha \quad (\text{N.0.2-7})$$

$$\overline{\varphi} = \frac{\varphi_1 l_1 + \varphi_2 l_2 + \cdots + \varphi_n l_n}{l} \quad (\text{N.0.2-8})$$

where:

p ——average pressure at pile tip plane (kPa);

$p_{\max}$——maximum pressure at pile tip plane (kPa);

$p_{\min}$——minimum pressure at pile tip plane (kPa);

γ——average unit weight of soil (including the pile gravity) from the pile cap bottom to the plane of the pile tip (kN/m^3);

l —— pile depth (m);

γ——unit weight of soil above the pile cap bottom (kN/m^3);

L——cap length (m);

B ——cap width (m);

N——vertical component force of the resultant force acting on the pile cap bottom (kN);

A——effective area of the pile tip plane in the assumed block foundation (m^2);

a, b——effective width and length of the pile tip plane in the assumed block foundation (m), respectively;

L_0——length of the rectangular contour formed by the centers of the surrounding piles (m);

B_0——width of the rectangular contour formed by the centers of the surrounding piles (m);

d——pile diameter (m);

W——section modulus of the assumed block foundation with respect to the pile tip plane (m^3);

e——eccentricity of the vertical force component of the resultant force acting on the pile cap bottom with respect to the gravity axis of the effective area at the pile tip plane (m);

$\overline{\varphi}$——average inner friction angle of the soil layers the pile penetrates through (°);

—— product of the internal friction angle of each soil layer and the corresponding soil layer depth;

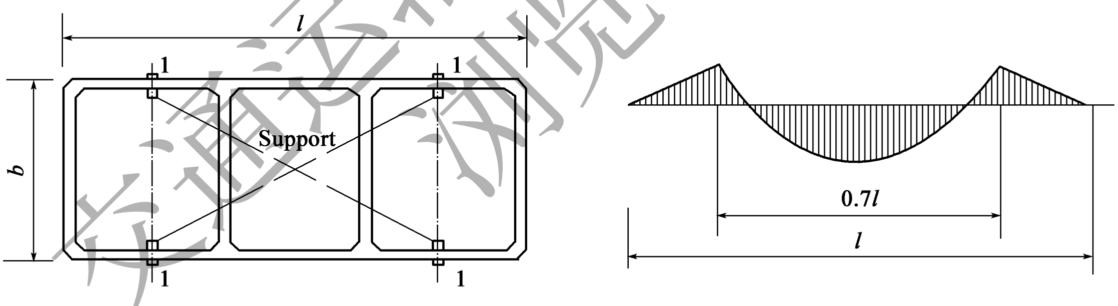
f_a——adjusted characteristic value of bearing resistance of soil at the pile tip plane (kPa), which is taken according to provisions in Clauses 4.3.4 and 4.3.5 of the *Specifications*, and shall be increased appropriately according to provisions in Clause 3.0.7 of the *Specifications*;

γ_R——partial factor, see Clause 3.0.7 of the *Specifications*.

Appendix P
Calculations for Caisson Walls during Caisson Sinking Process

P. 0. 1 The bearing capacity and deformation of caisson wall at the bottom segment during the sinking process of caisson shall be checked. The action effect may be calculated based on the frame model as follows:

1 When a caisson is sunk through an excavation in drain soil, the bottom segment of the caisson can be assumed to be vertically supported at the four points marked as '1' in Fig. P. 0. 1-1.

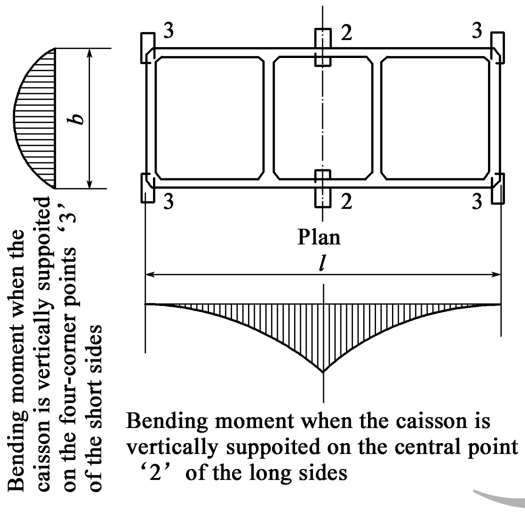


a)Plane b)Bending moment diagram

Fig. P. 0. 1-1 Caisson sunk by dewatering

2 When a caisson is sunk without dewatering, the bottom segment of the caisson can be assumed to be vertically supported on the central point of the long side marked as '2' in Fig. P. 0. 1-2, or on the four-corner points at two ends of the short sides marked as '3' in Fig. P. 0. 1-2.

P. 0. 2 The vertical tensile strength of the wall during the caisson sinking process shall be checked. The checking may be carried out according to the following provisions:



P. 0. 1-2 Caisson sunk by excavation in water

1 It is assumed that the caisson is socketed by the skin frictions from the surrounding soil, and the soil under the edge has been excavated.

2 It is assumed that the tensile stress at the connector is not beard by concrete but all by steel bars there.

3 If a caisson has constant wall section andits top surface is at the ground level, it is assumed that the skin friction of its wall is distributed in triangle along the full height, i. e. , the skin friction is zero at the bottom of the cutting edge and the maximum on the ground section. In this case, the most unfavorable section is located at 1/2 of the buried depth of the caisson (Fig. P. 0. 2a) and the maximum vertical tension force $P_{\max}$ is 1/4 of the total self-weight of the caisson, G_k.

$$P_{\max} = \frac{G_k}{4} \tag{P. 0. 2-1}$$

4 The tensile force in the changing section of the caisson wall in a stepped caisson shall be checked. The tensile force P_x (Fig. P. 0. 2b) of the changing section of the caisson wall may be calculated according to the following equations:

$$P_x = G_{xk} = \frac{1}{2} u q_x x \tag{P. 0. 2-2}$$

$$q_x = \frac{x}{h} q_d \tag{P. 0. 2-3}$$

where:

P_x——tensile force of the caisson wall in changing section with a distance of x away from the cutting edge bottom (kN);

G_{xk}——self-weight of the caisson within the height of x(kN);

u——perimeter of the caisson wall(m);

q_x——skin friction in changing section with a distance of x away from the cutting edge bottom (kPa);

q_d——skin friction on the caisson top(kPa);

h——total height of the caisson(m);

x——height from the cutting edge bottom to the changing section or the checked section (m).

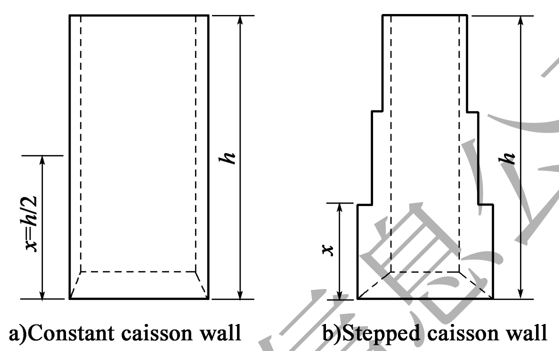


Fig. P. 0. 2 Vertical Tension of Caisson Wall

P. 0. 3 The horizontal resistance of the caisson wall during the sinking process shall be checked. In the checking, the caisson may be modeled as a horizontal frame. The horizontal load on the frame may be determined according to the following provisions:

1 The water pressure and earth pressure acting on the caisson wall are determined according to the excavation condition: in dry or in water.

2 For a caisson sunk by slurry jacket method, when the slurry pressure is larger than the horizontal forces such as water pressure and earth pressure, the slurry pressure is taken as the pressure on the caisson wall. For a caisson sunk by air curtain method, the pressure on the caisson is same as the pressure in calculation of ordinary caisson.

3 In addition to the horizontal load within the investigated range of the caisson wall, the horizontal shear force transmitted from the cantilever of the cutting edge shall also be considered.

4 A caisson wall segment with a height equal to the wall thickness measured from the root of the cutting edge is taken for calculation of the root of the cutting edge. The average load q acting on this caisson wall segment is calculated according to Formulas(P. 0. 3-1) ~ (P. 0. 3-5):

$$q = W + E + Q \tag{P.0.3-1}$$

$$W = \frac{W_1 + W_2}{2} \cdot t \tag{P.0.3-2}$$

$$W_1 = \lambda h_1 \gamma_w \tag{P.0.3-3}$$

$$W_2 = \lambda h_2 \gamma_w \tag{P.0.3-4}$$

$$E = \frac{E_1 + E_2}{2} \cdot t \tag{P.0.3-5}$$

where:

q——average load acting on the caisson wall segment with a height of t (kN/m);

W——water pressure acting on the caisson wall segment with a height of t (kN/m). The distance from the acting point of the resultant force of the water pressure to the root of the cutting edge is $\frac{W_2 + 2W_1}{W_2 + W_1} \cdot \frac{t}{3}$;

W_1——unit water pressure acting on section A with a height t above the root of cutting edge (kPa);

W_2——unit water pressure acting on section B of the root of cutting edge(kPa);

t——thickness of the caisson wall(m);

h_1, h_2——depth of the investigated sections A and B from the water surface(m), respectively;

γ_w——unit weight of water(10 kN/m3);

λ——reduction factor. When a caisson is excavated in dry, there is no water pressure inner the caisson, water pressure outside the caisson depends on the soil property, $\lambda = = 1.0$ is taken for sand and $\lambda = 0.7$ is taken for cohesive soil. When a caisson is excavated in water, the water pressure outside the caisson is calculated as 100%, that is $\lambda = 1.0$, and the water pressure inner the caisson is calculated as 50%, that is $\lambda = 0.5$;

E——lateral earth pressure acting on the caisson wall of a segment with a height of t(kN/m). The distance from the action point of the resultant force of the earth pressure to the root of the cutting edge is $\frac{E_2 + 2E_1}{E_2 + E_1} \cdot \frac{t}{3}$;

E_1——unit earth pressure acting on section A with a height t above the root of cutting edge (kPa), which may be calculated according to the earth pressure formula in the current *General Specifications for Design of Highway Bridges and Culverts*(JTG D60);

E_2——unit earth pressure acting on section B of the root of the cutting edge(kPa);

Q——horizontal force transmitted by the cutting edge(kN/m), which is equal to the horizontal force acting on the cantilever beam of the cutting edge multiplied by the distribution coefficient α, see formula(Q.0.5-1).

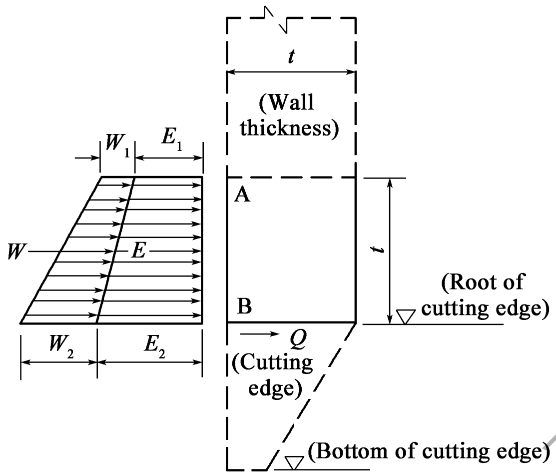


Fig. P. 0. 3 Load Distribution of a Caisson Wall Segment above the Cutting Edge Root with a Height Equals to the Wall Thickness

5 For other segments of the caisson wall, the segments of the caisson wall with a unit height above the changed sections are taken for analysis. The average load q acting on the frame is calculated according to Formula (P. 0. 3-1), but the horizontal shear force transmitted by the cantilever of the cutting edge is not considered.

Appendix Q
Calculations for Cutting Edge during the Caisson Sinking Process

Q.0.1 In checking the flexural resistance of the cutting edge of a caisson, the cantilever beam and frame structure model may be adopted for the checking in the vertical direction and on the horizontal plane, respectively.

Q.0.2 If the cutting edge behaves as a cantilever beam deflecting outward when its flexural resistance is checked, the action forces may be calculated according to the following provisions (Fig. Q.0.2-1, Fig. Q.0.2-2):

1 It is assumed that the inside of the cutting edge is cut into the soil by 1 m, and the caisson is exposed above the ground or above the water surface with a certain height, or the caisson wall has a certain exposed height after all the wall is cast.

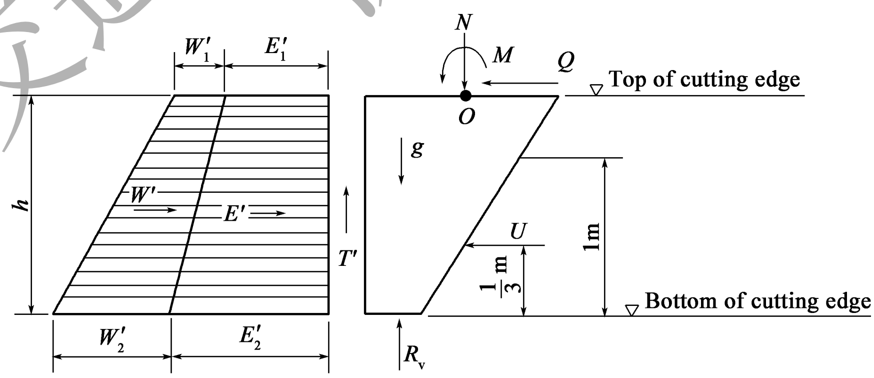


Fig. Q.0.2-1 Analytical Model of a Typical Cutting Edge

2 The lateral earth pressure E_1', E_2', E' and water pressure W_1', W_2', W' acting on the cutting edge are calculated according to the provisions in Appendix P of the Specifications by taking a segment with unit width along the periphery of the cutting edge.

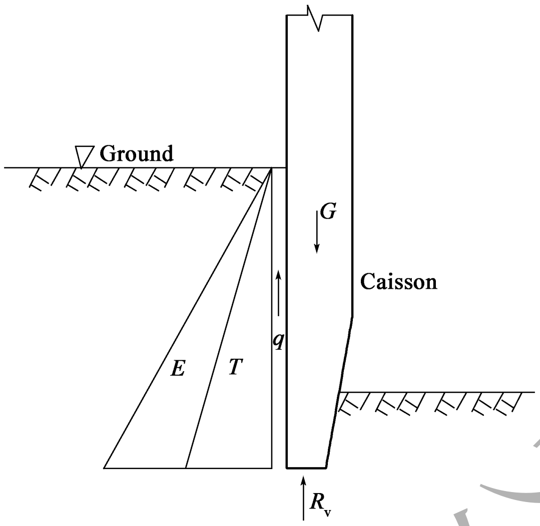


Fig. Q. 0. 2-2　Skin Friction*T* of Caisson Wall and Soil Reaction Force *R* under the Cutting Edge

3　The 70% hydrostatic pressure is taken as the sum of lateral earth pressure and water pressure acting on the outside of the cutting edge if the sum is greater than 70% of the hydrostatic pressure.

4　The total skin friction of the caisson side surface with a unit height along the wall perimeter is calculated according to the following formulas, and the lesser value is taken.

$$T = \mu \cdot E \tag{Q.0.2-1}$$

$$T = q \cdot A \tag{Q.0.2-2}$$

where:

T——total skin friction of the caisson side surface with a unit height along the wall perimeter (kN/m);

μ——friction coefficient, $\mu = \tan\varphi$;

φ——internal friction angle of the soil, generally it is taken as $\tan\varphi = 0.5$;

q——unit skin friction between the soil and the caisson wall, is taken from Table 7.3.2 of the Specifications;

A——total area per unit height of the caisson side surface in contact with the soil (m^2), $A = 1 \times h = h$, (h is the height of the caisson, in meters);

E——total earth pressure per meter width acting on the caisson wall (kN/m).

5　The vertical reaction force R_V of the soil on the unit perimeter below the cutting edge is calculated according to the following equation:

$$R_v = G - T \tag{Q.0.2-3}$$

where:

G——self-weight of the caisson wall with unit height along the perimeter of the outer surface of the caisson(kN/m). Its value is equal to the total weight of the caisson within the height divided by the perimeter of the caisson. When the caisson is sunken by excavation in water, the buoyancy of the submerged portion of the caisson shall be deducted from the total self-weight of the caisson.

6 The action point of Rv is calculated according to the following provisions (see Fig. Q.0.2.3):

1) It is assumed that the soil reaction force acting on the slope of the cutting edge is distributed in a triangle, with an angle of β between its direction and the normal of the slope. The β is taken as the external friction angle between the soil reaction force and the slope of the cutting edge (generally $\beta = 30°$ is adopted),.

2) Thevertical force component of the soil reaction force acting on the slope of the cutting edge, V_2, is calculated according to Formula (Q.0.2-4), in which the distance between the force acting point to the outer wall of the cutting edge is a + $\frac{b}{3}$.

$$V_2 = \frac{b}{2a+b} \cdot R_v \qquad (Q.0.2\text{-}4)$$

where:

a——bottom width of cutting edge tread(m);

b——horizontal projection(m) of the slope where the cutting edge enters the soil, $b = \cot\alpha$, where α is the angle formed by the slope of the cutting edge to the horizontal plane.

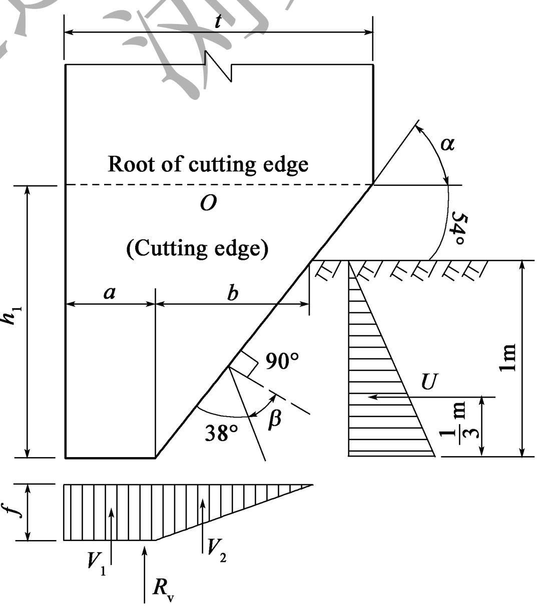


Fig. Q.0.2-3 Action Point of R_V below the Cutting Edge

3) The vertical reaction force acting on the bottom surface of the cutting edge, V_1, is calculated according to Formula (Q.0.2-5), and the distance between the acting point and the exterior surface of the cutting edge is $\frac{a}{2}$.

$$V_1 = R_v - V_2 \tag{Q.0.2-5}$$

7 The horizontal force component acting on the slope of the cutting edge, U, is calculated by the following formula. Its action point is at 1/3 of the height (m) from the bottom surface of the cutting edge.

$$U = V_2 \tan(\alpha - \beta) \tag{Q.0.2-6}$$

8 The unit weight of the cutting edge, g, is calculated as follows:

$$g = \gamma_h \cdot h_1 \frac{t + a}{3} \tag{Q.0.2-7}$$

where:

γ_h——unit weight of concrete (kN/m^3). If the caisson is sunk without dewatering, the buoyancy shall be deducted;

h_1——slope height of the cutting edge (m).

9 The skin friction acting on the outside of the cutting edge, T', may be taken as the larger value calculated by the following formulas.

$$T' = u \cdot E' \tag{Q.0.2-8}$$

$$T' = q \cdot A' \tag{Q.0.2-9}$$

where:

A'——total area per unit height of the outer surface of the cutting edge contacting with the soil (m^2), $A' = 1 \times h_1 = h_1$;

E'——total earth pressure per meter width within the height of the cutting edge (kN/m).

10 The horizontal force acting on the side of the cantilever cutting edge is the maximum horizontal force on the cutting edge multiplied by the distribution coefficient α. Value α is calculated according to Clause Q.0.5 of the Specifications.

Q.0.3 If the cutting edge behaves as a cantilever beam deflecting inward when its flexural resistance is checked, the action forces may be calculated according to the following provisions:

1 A segment with unit width along the periphery of the cutting edge in the horizontal direction is taken for calculation under the assumption that the caisson has reached its design elevation andthe soil below the bottom of the cutting edge has been hollowed out, as shown in Fig. Q.0.3.

2 The earth pressure and water pressure acting on the outside of the cutting edge may be calculated in accordance with the provisions in Appendix P of the Specifications.

3 If the caisson is sunk without dewatering, outside water pressure shall be calculated by 100%, while the inside water pressure shall be calculated by 50% or by the water head difference possible appeared in the construction. If the caisson is sunk by dewatering, the outside water pressure may be calculated as 70% of hydrostatic pressure in permeable soil.

4 The skin friction acting on the outside of the caisson wall, T', is taken as the lesser value from the calculated results by Formula (Q.0.2-8) and Formula (Q.0.2-9).

5 The unit weight of the cutting edge, g, is calculated according to Formula (Q.0.2-7).

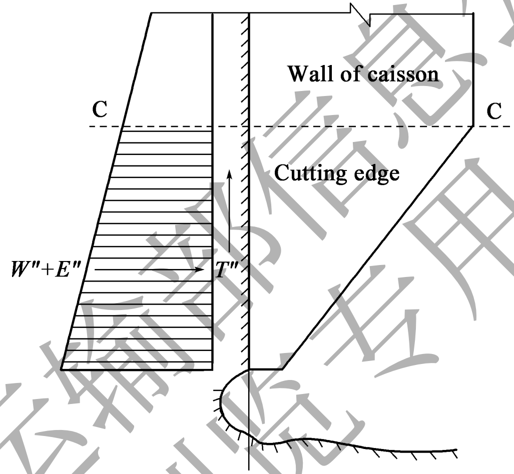


Fig. Q.0.3 Cutting Edge Deflecting inward

Q.0.4 When the horizontal resistance of a cutting edge is checked by modeling the caisson as a horizontal frame, the calculation may be carried out according to the following provisions (Fig. Q.0.4):

1 Assume the caisson has already been lowered to the design elevation, the soil under the cutting edge has been removed totally, and the unit height is intercepted along the cutting edge in the vertical direction to form a horizontal frame structure.

2 The forces on the plane frame are calculated following the provisions in Clause Q.0.3 of the Specifications; if necessary, the horizontal forces subjected from inside and outside of the frame are considered according to the construction situation.

3 The finaluniformly distributed load subjected on the entire perimeter of the plane frame is the product of maximum horizontal force on the cutting edge multiplied by the distribution

coefficient β, and its value is calculated according to the provisions in Clause Q. 0. 5 of the *Specifications*.

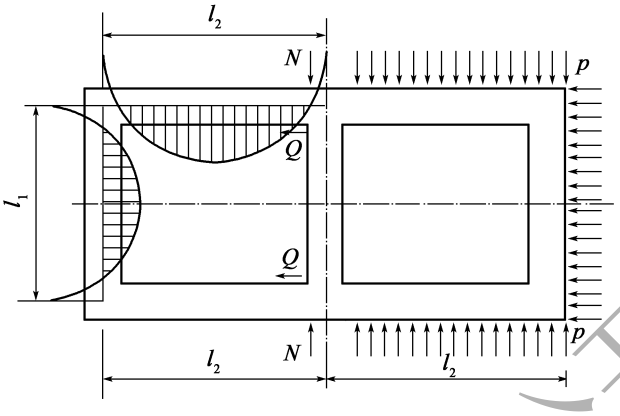


Fig. Q. 0. 4 Plane Frame of the Cutting Edge of a Rectangular Caisson

Q. 0. 5 For a rectangular caisson, the distribution coefficient of the horizontal force on the cutting edge may be calculated according to the following approximation method:

1 The cutting edge shall be regarded as a cantilever in the vertical direction and its length is equal to the height of the inclined plane portion. If the distance from the bottom surface of the internal diaphragm to the bottom surface of the cutting edge is 0. 5 m, or it is larger than 0. 5 m but has been strengthened by vertical supports, the horizontal force on the cantilever portion may be multiplied by the distribution coefficient α:

$$\alpha = \frac{0.1 l_1^4}{h^4 + 0.05 l_1^4} \leqslant 1.0 \qquad \text{(Q. 0. 5-1)}$$

where:

l_1——maximum effective span of the outer wall supported by the internal diaphragms (m);

h——height of the inclined plane portion of the cutting edge (m).

2 The cutting edge may be regarded as a closed frame in plan. If the horizontal force for the cantilever of the cutting edge is multiplied by the distribution coefficient α, the horizontal force acting on the plane frame may be multiplied by the distribution coefficient β:

$$\beta = \frac{h_1^4}{h_1^4 + 0.05 l_2^4} \qquad \text{(Q. 0. 5-2)}$$

where:

l_2——minimum effective span of the outer wall supported on the internal diaphragms (m);

h_2——height of the inclined plane portion of the cutting edge (m).

Appendix R
Calculations for Horizontal Earth Pressure Based on the Principle of Soil-Retaining Structure Interaction

R. 0. 1 When the retaining structure is designed by the principle of deformation control, the earth pressure on the diaphragm wall may be determined by the principle of soil-structure interaction. Considering the influence of wall horizontal deformation to the horizontal earth pressure on the wall, the horizontal earth pressure may be calculated by the following formulas:

$$E_{jk} = E_{0k} - K\delta \tag{R.0.1-1}$$

$$E_{0k} = K_0 (q_k + \sum \gamma_i h_i) \tag{R.0.1-2}$$

$$K = mz \tag{R.0.1-3}$$

where:

E_{jk}——horizontal earth pressure on the wall (kPa); when $E_{jk} < E_a$, $E_{jk} = E_a$; when $E_{jk} > E_p$, $E_{jk} = E_p$, where E_a and E_p are the active and passive horizontal earth pressure on the wall, respectively, which involve the effects of the soil self-weight and the surcharge load on the ground aside the wall, and they may be calculated by Coulomb or Rankine earth pressure theory;

E_{0K}——at-rest horizontal earth pressure on the wall (kPa);

K——coefficient of horizontal reaction of the ground soil beside the wall (kN/m^3), which should be determined by in-situ test, or based on reliable methods or experience. In absence of reliable methods or experience, it may be calculated by Formula (R. 0. 1-3)

m——proportionality coefficient for the factor of horizontal reaction of ground (kN/m4), which is increased proportionally with the depth, and may be determined by the horizontal loading test or based on the engineering judgment;

δ——horizontal deformation of the wall (m). The deformation towards the direction of the earth pressure is positive, while the deformation away from the direction of the earth pressure is negative;

K_0——coefficient of at-rest earth pressure. For normally consolidated soil, $K_0 = 1 - \sin\phi'_K$; for over consolidated soil, $K_0 = \sqrt{1 - \sin\phi'_k}$, where ϕ'_k is the effective internal friction angle (°) of the soil layer at the investigated point;

q_k——uniformly distributed vertical load on the ground (kPa);

——unit weight of soil in i^{th} layer above the investigated plane (kN/m^3);

h_i——thickness of soil in the i^{th} layer above the investigated plane (m);

Z——depth of the investigated point to the ground surface beside the wall (m).

Appendix S
Calculations for Straight Diaphragm Wall as Retaining Structures

S.0.1 If the method of vertically elastic foundation beam is applied to calculate a straight diaphragm wall as retaining structure, the internal force and deformation of the wall panel may be calculated by the finite element method with bar elements. The calculation scheme is shown in Fig. S.0.1.

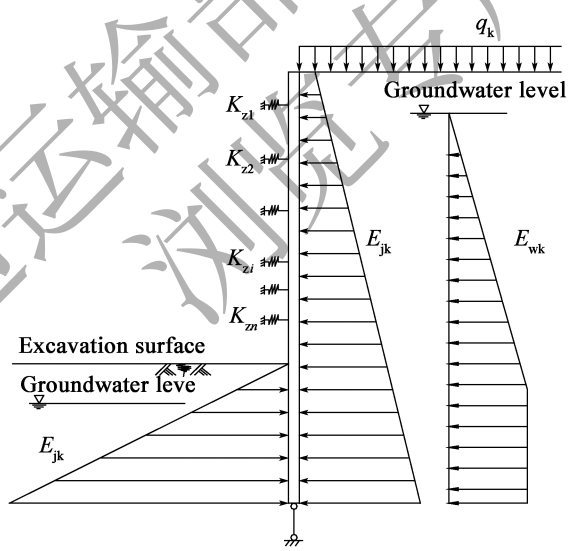


Fig. S.0.1 Calculation Schema of a Typical Straight Diaphragm Wall as Retaining Structure

where:

$K_{z1}, K_{z2} \cdots K_{zi} \cdots K_{zn}$——elastic coefficients of strut, horizontal bracket, earth anchor (or cable) and other supports;

q_k——uniformly distributed vertical load on the ground (kPa);

E_{jk}——horizontal earth pressure on the wall (kPa), which is calculated by Formula (R.0.1-1);

E_{wk}——water pressure on the wall (kPa). Water pressure may be calculated as hydrostatic pressure when water pressure and soil pressure are calculated separately; if specific experiences are available, the influence of seepage on water pressure may also be considered.

Appendix T Calculations for Circular Diaphragm Wall as Retaining Structures

T.0.1 If the method of vertical elastic foundation beam is applied to calculate the circular diaphragm wall as a retaining structure, the internal force and deformation of the wall panel may be calculated by the finite element method with bar elements. The calculation scheme is shown in Fig. T.0.1.

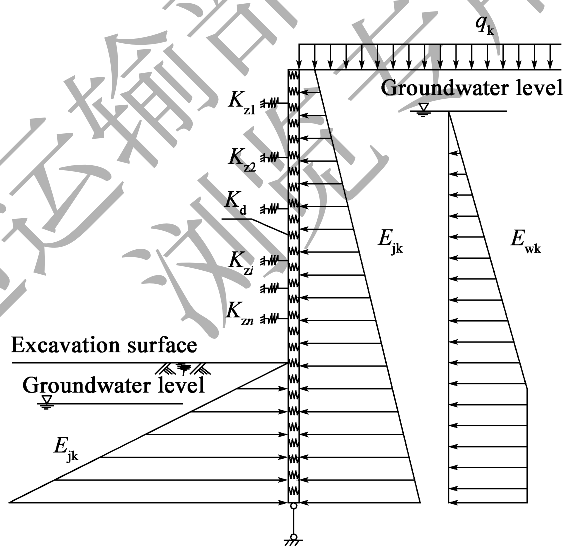


Fig. T.0.1 Calculation Schema of the Circular Diaphragm Wall as Retaining Structure

where:

$K_{z1}, K_{z2} \cdots k_{zi} \cdots K_{zn}$——elastic coefficients of internal ring beam or inner lining, which are calculated by Formula T.0.2;

K_d——elastic coefficient of equivalent distribution along the depth direction of the wall panel, which is calculated by Formula T.0.3;

q_k——uniformly distributed vertical load on the ground (kPa);

E_{jk}——horizontal earth pressure on the wall (kPa), which is calculated by

Formula(R.0.1-1);

E_{wk}——water pressure on the wall(kPa). Water pressure may be calculated as hydrostatic pressure when water pressure and soil pressure are calculated separately; if specific experiences are available, the influence of seepage on water pressure may also be considered.

T.0.2 When the circular diaphragm wall as retaining structure is supported by the internal ring beams or inner linings, the action due to the internal ring beams or inner linings may be simulated as the action of equivalent springs, as shown in Figs T.0.2-1 and T.0.2-2. The equivalent elastic coefficient of an internal ring beam or inner lining per unit width wall panel may be calculated by the following equation:

$$K_z = \frac{E_z A_z}{R_z^2} \tag{T.0.2}$$

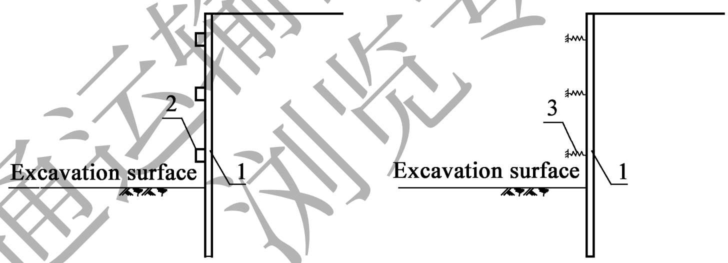
where:

K_z——elastic coefficient of equivalent spring for internal ring beams or inner linings per unit width of the wall panel(kN/m);

E_z——elastic modulus of the materials of the internal ring beam or inner lining(kN/m);

A_z——effective sectional area for one internal ring beam or one inner lining (m^2), the influence of deviation in the construction shall be considered in the calculation;

R_z——radius of the cross-section centerline of the internal ring beam or inner lining(m).



a)Internal ring beam b)Equivalent spring support

1- Diaphragm wall panel; 2-Internal ring beam; 3-Equivalent spring

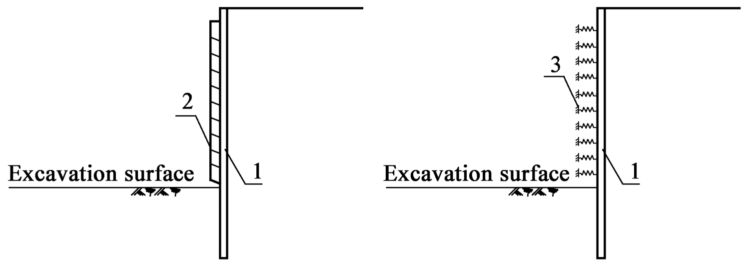
Fig. T.0.2-1 Internal Ring Beam and Its Equivalent Springs in Typical Circular Diaphragm Wall as Retaining Structure

T.0.3 The circumferential effect of the panel of the circular diaphragm wall may be stimulated by spring supports distributing along the depth, as shown in Fig. T.0.1. The equivalent distributed elastic coefficient per unit width of the panel of the diaphragm wall may be calculated by the following formula:

$$K_d = \alpha \frac{Ed}{R_0^2} \tag{T.0.3}$$

where:

K_d——equivalent distributed elastic coefficient per unit width of the panel of the diaphragm wall(kN/m^2);



1-Diaphragm wall panel; 2-Inner lining; 3-Equivalent spring

Fig. T. 0. 2-2 Inner Lining and Its Equivalent Springs in Typical Circular Diaphragm Wall as Retaining Structure

E——elastic modulus of the panel material of the diaphragm wall((kN/m^2);

d——effective thickness of the panel of the diaphragm wall(m), influence of deviation in the construction shall be considered in the calculation;

R_0——radius of the centerline of the panel of the diaphragm wall(m);

α——adjusting factor, which shall be adopted according to the actual engineering conditions. If practical experience is unavailable, it may be taken as $\alpha = 0.4 \sim 0.7$; and the lesser value is taken when R_0 is large or the number of the trench segments is great.

Wording Explanation for the *Specifications*

1 Words for strictness in implementation of the *Specifications*:

1) "Must" or "must not" is used for a mandatory requirement in any circumstances.
2) "Shall" or "shall not" is used for a mandatory requirement in normal circumstances.
3) "Should" or "should not" is used for an advisory requirement.
4) "May" or "may not" is used for a permissive condition that no requirement is intended.

2 The following expressions are used in citation of other specifications:

1) To state the relation between the Specifications and other specifications in Chapter "General Provisions", it is expressed as "In addition to the Specifications, … shall also comply with the provisions in the current relevant national and industrial standards".

2) When referring to national and industrial standards in clauses of the Specifications, it is expressed as "shall comply with the relevant provisions in ( )".

3) When citing other clauses in the Specifications, it is expressed as "shall comply with the relevant provisions in Chapter of the Specifications", "shall comply with the relevant provisions in Section . of the Specifications", "shall comply with the relevant provisions in Clause . . of the Specifications", or "shall be implemented in accordance with the relevant provisions in Clause . . of the Specifications".

Background to Provisions

1 General Provisions

1.0.1 The *Specifications* is a revision of the *Specifications for Design of Ground Base and Foundation of Highway Bridges and Culverts* (JTG D63-2007) (hereinafter referred to as '2007 Version *Specifications*'). During the revision, many amendments and supplements were carried out based on 2007 Version *Specifications* by summing up the practical experiences and referencing the research results from China and other countries.

1.0.3 ~ 1.0.4 The ground soils are extremely complex for their properties, their mechanical indexes vary greatly even in the same foundation and ground. Moreover, many hidden or exposed adverse geological conditions exist here and there, together with difference properties of soils. All these aspects emphasize that the foundation design shall follow the principle of the adaptation to local conditions. Bridge foundations are important and huge projects in rivers, lakes, and seas, which need a large amount of soil, rock, and concrete materials. Most of the foundations underwater are constructed directly in water or in a dewatered site, resulting difficult and workload works. Therefore, the geological conditions of the specific projects must be practically gathered and reasonable schemes must be selected by the designers, and the accidents caused by any unknown situations or by wrong design must be avoided. In addition, foundation engineering is closely related to water issues, so that the scheme shall be designed according to the actual conditions, the large volumes in excavation and filling shall be avoided, and pollution of water resources shall be prevented.

The type of bridge foundation generally needs to be selected reasonably according to the construction conditions proposed in this Clause. In case of complicated situations, it may be necessary to retrofit the natural conditions partially, or to propose several schemes and then select an optimal solution by comparison of them in aspect of technology and economics.

1.0.5 The engineering geology has great influence in the scheme selection and structural design for not only the foundation but also the bridge type. Engineering geological exploration shall be emphasized, especially when the faults or karsts exist at the bridge site, partially weak soil layer is buried in the uneven stratum, or foundations are built on undulating or sloping rock strata.

1.0.6 The structural durability has been attracting people's attentions. The durability of the foundation structure is affected not only by the harmful substances contained in the materials (such as concrete and reinforced concrete), but also by the natural environment such as air, water, and soil where the foundations are located. Therefore, the durability for foundation structures needs to be designed according to different environment conditions.

2 Terms and symbols

This chapter only lists the terms that are present and need to be clearly defined in the *Specifications*. General terms related to the bridge profession and the terms that have been clarified in the Clauses are not listed in this chapter.

In this revision, some terms in the 2007 Version *Specifications* are deleted, including safety level, combination of short-term effects, combination of long-term effects, and allowable value of resistance; some terms are supplemented according to the revision of the provisions in the *Specifications*, including the characteristic values of ground bearing capacity, shallow foundation, pile with expanded branches and plates.

Most of the explanations of the terms are only of general meanings. The terms are not standardized names and are cited for reference only.

Symbols in this chapter are listed by four parts: resistance and material properties of ground, actions and action effects, geometric parameters, calculation coefficients and others. These main symbols are adopted in accordance with the provisions in the current national standards of China. If no provision is specified in current standards to follow, they are adopted from the 2007 version *Specifications* or as customary. Not all symbols used in the *Specifications* are listed, only some primary ones are listed in this chapter.

3 GeneralRules

3.0.7 The partial factor of ground bearing capacity, γ_R, specified in this Clause is determined according to load conditions and loading stages.

Due to the importance of the ground bearing capacity, the partial factor is generally taken as 1.0, and it can be appropriately increased if there are sufficient evidence. The 2007 Version *Specifications* uses 'shall' for situations that can be improved, however it is revised to 'may' in the *Specifications*, which means the designer can determine to increase or not the factor according to the specific situation.

The action combinations carried by the ground in the service stage can be taken according to the provisions in Clause 3.0.6 of the *Specifications*. The frequent combination in the *Specifications* is equivalent to the combination of short-term effects of the 2007 Version *Specifications*, which refers to the combination with all permanent and variable actions that may occur simultaneously and will produce adverse effects to the ground bearing capacity. Such a combination has the maximum value due to all the frequent factor of the actions in it are taken as 1.0, whereas the probability of simultaneous occurrence of them is not considered. Therefore, the action effects on the ground in the computation with such an action combination must be larger than the actual ones. Therefore, the characteristic value of ground bearing capacity, f_a, needs to be multiplied by a partial factor greater than 1.0.

However, if the action combination only includes the self-weight of the structure, the preload, the soil weight, the lateral earth pressure, the vehicle, and the pedestrian, *i. e.*, the actions directly acted on the structures, the distribution of the compressive stress of foundation base is close to a rectangular, which is close to the actual ground bearing capacity. Therefore, f_a does not need to be multiplied by γR, that is, let $\gamma R = 1.0$.

The accidental combination of actions in the *Specifications* is equivalent to that in the 2007 Version *Specifications*. The accidental action occurs instantaneously with very small probability. Though sometimes it may cause large compressive stress in the base with a distribution of trapezoidal shape, but the maximum value in one side of the base is of local and temporary. According to the previous various versions of the *Specifications*, the characteristic value of bearing capacity under this condition is also considered to be multiplied by the partial factor $\gamma_R = 1.25$.

The actions on the ground during construction are of short-term, compared to those in the service stage, thus a larger partial factor is generally adopted. The provisions of the *Specifications* follow the values in the 2007 Version *Specifications*.

3.0.8 The deformation characteristics of the ground shall be considered in calculating the foundation settlement. Since the ground soil is a type of material with large deformation and long-term effect, the settlement of foundation needs to be calculated according to the quasi-permanent combination of actions at the serviceability limit state. This combination consists of the characteristic values of permanent actions and the quasi-permanent value of variable actions, in which the former only includes the structural self-weight, soil weight, soil lateral pressure and normal buoyancy, and the latter only includes the vehicle and pedestrian load. This combination is consistence with that in the 2007 Version *Specifications*. According to the investigation statistics, such a combination has a large probability to act on the bridge, stays there some long time, has large influence on the foundation settlement, therefore it is relatively reasonable to be adopted in settlement calculation.

3.0.9 Stability check for foundation structures is one of the contents for the design of ultimate limit state. The fundamental idea is to regard the foundation as a rigid body, to keep it in static balance with a certain safety factor for stability. In the check, characteristic values are adopted in the combination of actions that will produce balance effect or unbalanced effect, and all factors in the expression for effects of combination of actions are taken as 1.0. Moreover, the most adverse effects of the combination of actions need to be considered. All the variable actions that cause structure instability in the same direction and may appear simultaneously need to be considered, while for the variable actions in favor for structure stability in the same direction but may not appear simultaneously, the domestic actions are considered but other variable actions are not taken in the combination in order to minimize the effect of stabilization.

4 Geotechnical Classification, Engineering Characteristic and Bearing Capacity of Ground

4.1 Geotechnical Classification of Ground

4.1.1 Classification of rock and soil includes geological classification or engineering classification. Geological classification is mainly based on its geological origin, mineral composition, structure, and weathered degree. It may be expressed by geological name (i.e., petrological name) plus weathering degree, such as highly weathered granite, slightly weathered sandstone, etc. This is necessary for the survey and design of engineering project. Engineering classification is mainly based on the engineering properties of rock masses, enabling engineers to have a clear concept of engineering properties of the rock and soil. Geological classification is a basic classification. Engineering classification shall be carried out based on geological classification to better generalize the engineering properties of the rock and soil, and to facilitate engineering evaluation.

4.1.2 Rock mass rating of strength aims to determine the bearing capacity of the ground. For rocks with bearing resistance larger than 30 MPa, their bearing resistance do not depend on the strength of the rocks themselves, however, in order to consistence with the other geotechnical classifications, the rock stiffness is still divided into two categories in the *Specifications*, *i.e.*, very strong rock and moderately strong rock. For rocks with bearing resistance no larger than 30 MPa, their bearing resistance are impacted significantly by the strength of the rocks, so that a more detailed classification for their stiffness is necessary. They are classified as moderately weak rock, very weak rock and extremely weak rock in the *Specifications*. It is very important to distinguish extremely weak rocks, because such rocks are not only extremely soft, but also often have special engineering properties. For example, some mudstones have a very high swelling property; argillaceous sandstones and completely weathered granites have strong softening properties (the uniaxial compressive strength of saturation sample may be equal to zero); some tertiary sandstones disintegrate in contact with water and have the nature of quicksand. For rocks that disintegrate in water or those their compressive strength cannot be tested for saturated samples, qualitative methods are generally used to determine their classifications.

4.1.4 Attention shall be paid for the softening rocks, because their bearing resistance will significantly decrease after they are immersed in water. The softening coefficient is the ratio of the compressive strength of the saturated sample to that of the dry sample. Using softening coefficient 0.75 as the threshold is based on relevant specifications in China and other countries as well as engineering judgement accumulated decades.

4.1.5 According to design experience, the bearing resistance of rock with $f_{rk} > 30$MPa is not controlled by its strength, but by the intactness of the rock mass. For rock grounds, special attention shall be paid to weak rock, extremely weak rock, very fractured and extremely fractured rock, and rock with a basic quality grade of V. For those types of rocks whose undisturbed sample can be taken, the geotechnical test methods are used to test their behaviors, their physical and mechanical properties. When the intactness of the rock mass is extremely fractured, the rock mass rating of strength is not performed.

4.1.6 In 85 Version of the *Specifications for Design of Ground and Foundation of Highway Bridges and Culverts* (hereinafter referred to as '85 Version *Specifications*'), fracture degree and rock mass rating of strength are used to determine the bearing capacity of rock grounds. In revision of the *Specifications*, the method in the 85 Version *Specifications* is still reserved for reference by engineers. However, the fracture degree in the 85 Version *Specifications* has been modified to the development degree of joint in the 2007 Version *Specifications*, and this modification is followed in the *Specifications*.

4.1.7 Gypsum, rock salt and other soluble rocks, swelling mudstone, collapsible sandstone, etc., are of special natures and have great hazards to the engineering projects. Special studies are necessary in the design, therefore they are specifically listed in the *Specifications*.

4.1.9 The application of heavy dynamic cone penetration test is popular. In this Clause, heavy dynamic cone penetration test method is used to classify the density of gravelly soil into four levels, *i. e.*, loose, medium dense, slightly dense and dense. In Appendix A.0.2, the field identification method is used to classify the gravelly soil according to its density because it still has practical value, but the identification results often vary from person to person and are difficult to maintain objectivity, therefore the gravelly soil can only be roughly divided into three levels, i.e., loose, medium dense, and dense. The 'slightly dense' and 'loose' identified by dynamic cone penetration test corresponds to the 'loose' identified in the field. Since the results obtained by these two identification methods may be inconsistent, it is necessary to state whether the identification is based on the 'field identification' or 'heavy dynamic cone penetration test' in the investigation report.

4.1.15 The engineering properties of cohesive soil are closely related to the age of deposition. In the *Specifications*, the classification for the age of deposition of cohesive soil is retained from the *previous Specifications*. Cohesive soils deposited in the Late Pleistocene (Q_3) of the Quaternary Period and its predecessors generally have higher strength and lower compressibility than others. The cohesive soil deposited in the Quaternary Holocene (Q_4) is generally considered as normally deposited cohesive soil. The cohesive soil deposited since the cultural period is generally under-consolidated and has low strength.

4.1.18 In the *Specifications*, the indexes of soft soil refer to the relative provisions in *Specifications for Design of Highway Subgrades*(JTG D30-2015). Considering that 'cohesive soil' and 'silty soil' are not appeared in the classifications in the *Specifications for Design of Highway Subgrades*(JTG D30-2015), only part of identification indexes in the *Specifications for Design of Highway Subgrades*(JTG D30-2015) are used in the *Specifications*.

4.1.21 The collapsible soil in this Clause refers to the natural soil(except loess) that can produce additional settlement over a certain amount under load when immersed in water. Only the characteristics of collapsible soil is given in this Clause, while for the identification and classification methods, they can refer to the *Specification for investigation of geotechnical engineering* (GB 50021-2001).

4.2 Engineering Characteristic Index

4.2.1 Mature experiences on in-situ tests, including static cone penetration test, dynamic cone penetration test and standard penetration test, have been obtained in China and can be used to determine the bearing capacity of foundations in practice, therefore, they are included in this Clause. However, it should be noted that they should not be used alone but should be used through a comprehensive analysis with indoor test results.

4.2.2 How to select the representative values of engineering characteristic properties is very important for foundation calculation. The selection principle is clearly stipulated in this Clause. The nominal value is the 0.05 quantile of its probability distribution. The characteristic value is taken for the bearing capacity of the foundation.

4.2.3 Loading test is one of the main methods to determine the bearing capacity of rock and soil. In the loading tests, the test results vary with the variation of the loading plate size, the loading time interval and the standards for settlement stability. Results from different tests are comparable only when the tests are carried out by unified operating procedures. Therefore, the key points of the tests are included in the appendix in the *Specifications* similar as in previous versions.

4.2.4 The compressibility properties of soil is the basic data for calculating the settlement of the structure. Compressive modulus and compressive factor are determined by compression test, which is applicable for fine-grained soil but inapplicable for coarse-grained soil. In calculation of the settlement of coarse-grained soil, the deformation modulus obtained from the field loading test should be used. In order to be consistent with the force conditions in the settlement calculation, the maximum compressive stress applied in the indoor or field foundation compression test shall exceed the sum of the effective self-weight stress of the soil and the expected superimposed stress, and the compressed segment same as that in the actual project should be taken to calculate the deformation parameters. Generally, the compressive factor $a_{1\text{-}2}$ is only used for the comparison of the compressibility of different soil bodies, but not for the calculation of foundation settlement.

4.3 Bearing Capacity of Grounds

4.3.1 Grounds are designed according to the requirements for the serviceability limit state. The designated ground bearing capacity in design is its characteristic value, this is because the soil is a material with large deformation. The deformation of ground will increase correspondingly with the increase of the load, and the bearing capacity of the ground will also increase gradually, so that it is difficult to define a true 'limit value' for the bearing capacity of ground. In addition, bridge and culvert structures in service should meet the requirements for their functions. It is a common case that the ground bearing capacity has potential to be utilized, but the deformation of the ground has reached or exceeded the serviceability limit state.

It is required to determine the characteristic value of the ground bearing capacity byloading tests or other in-situ tests in the *Specifications*, but sometimes these tests cannot be conducted in the grounds of bridges and culverts. To provide tables for characteristic values of ground bearing capacity in the *Specifications* is helpful for surveyors and designers of highway engineering. Therefore, tables for ground bearing capacity are still remained in the *Specifications*.

4.3.3 The items of this Clause are the tables for the characteristic value of various types of ground bearing capacity, f_{a0}.

1 Bearing capacity of rock ground

The bearing capacity of rock ground is related to the genesis, structure, mineral components, age of formation, degree of joint development, and the influence of water immersing, etc. The influence degrees by various factors are varied according to the specific situations, usually depending mainly on the strength of the rock block and the degree of rock fracture. For fresh and integral rock mass, the bearing capacity is mainly influenced by the strength of the rock mass; while for the rock mass affected by tectonic and weathering, its strength is low and its fragmentation is high, so that the bearing capacity is not only influenced by the strength of the rock mass, but also by its fracture degree. Therefore, the rock foundation is classified according to the strength of the rock, and is subclassified further according to the degree of joint development. Such a classification approach is clear in concept and can reflect the objective reality. The following points shall be paid attention in using the table of the allowable value of the ground bearing capacity in the *Specifications*:

(1) Due to insufficient test data, the accurate influence value of water on the bearing capacity of rock cannot be given. When such a situation is encountered on site, it shall be determined according to specific research. For example, when the easily weathered rock is used as the ground, special attention shall be paid to the possible changes in the hydrogeological conditions after the construction, and the f_{a0} value shall be carefully selected. If necessary, it should be determined by loading test.

(2) When the rock mass has been weathered into soil, sand or gravel, the characteristic value of the ground bearing capacity may be determined referring to residual soil or sand. However, for the recently weathered residual sand and gravel, the characteristic value of their bearing capacity may be taken some higher than that of the corresponding soil type, because they still maintain a certain relationship with the parent rock mass and the particles have cohesive force (or cementing force).

(3) The values in the table shall be appropriately selected according to the strength, thickness, fracture development degree, and other factors of the rock block. It would be better to use a smaller value for softening rocks and extremely softening rocks immersed in water.

(4) For softening rock with f_{rk} higher than 30 MPa, its ground bearing capacity needs to be determined comprehensively according to the actual situation. It cannot be taken directly from the ground bearing capacity of very strong rock or moderately strong rock in the table.

2 Bearing capacity of gravelly soil ground

The determination of allowable bearing capacity of gravelly soil is mainly based onloading test. Because the compressibility of most of the gravelly soil is low, the foundation settlement is small and the settlement process is fast, so the deformation is not the main control factor in the foundation design. The allowable bearing capacity $[\sigma_0]$ in the 85 Version *Specifications* was determined by proportional limits or by 1/3 of the ultimate load.

4 Bearing capacity of silty soil ground

The recommended values of ground bearing capacity of silty soil refer to the recommended values in the *Code for Design of Building Foundation* (GBJ 7-89) and *Code for Geology Investigation of Railway Engineering* (TB 10012-2001). Data from soil samples with saturation greater than 90% were accounted for 36%.

A variety of methods were adopted in the statistical calculations, including stepwise regression, binary regression with selected independent variable combination, and single index e statistical analysis. The selected independent variables are as follows: characteristics of soil (plasticity index I_p and liquid limit w_L), soil density index (void ratio e), and soil state indicators (liquidity index I_L and water content ratio a_w). In addition, the water content w is also included in the independent variables because it is an index that can reflect not only the soil state but also the density of saturated soil to a certain extent.

Natural void ratio e and moisture content w were selected as independent variables through analysis. The equation is written as follows:

$$f_{a0} = 148.6e^{-1.692} \times w^{-0.1912} \tag{4-5}$$

where:

f_{a0} - Characteristic value of ground bearing capacity (kPa).

The complex correlation coefficient $R = 0.785$ and the residual variance $\sigma = 0.0944$. When the actual table is built, the value in the table is adjusted by taking into account the error of σ in Eq. (4-5) and the guaranteed probability of 85%.

5 Bearing capacity of old cohesive soil ground

Eq. (4-6) is obtained from 53 data for the bearing capacity of old cohesive soil ground with a correlation coefficient of $R = 0.52$, which is used to get the Table 4.3.3-5.

$$f_{a0} = 308.9 + 79E_s \tag{4-6}$$

where:

E_s - compression modulus (MPa).

For the old cohesive soil $E_s < 10$MPa, the above formula is not applicable due to lack of data, so its bearing capacity may be taken by considering it as the general cohesive soil.

6 Bearing capacity of general cohesive soil ground

After comparisons of varies groups of factors, the liquid index and void ratio were finally selected as the basic factors for the table making. After considering the modification value for depth, $k_2\gamma_2 h$ (among them, k_2 is selected according to Table 4.3.4 in the provisions of the *Specifications*, $\gamma_2 = 15$ kN/m^3, $h = 1.5$ m), and the adjustment of individual values according to the past experiences, the calculated results from the regression equation were taken for Table 4.3.3-6 in the provisions of the *Specifications*.

Considering application range of Table 4.3.3-6 in the provisions is limited, the value cannot be found from the table for those general cohesive soil with physical property indicators beyond the range, the formula with the variable E_s was established and listed as a supplement in Note 2 of Table 4.3.3-6 in the provisions.

7 Bearing capacity of newly deposited cohesive soil ground

Due to the lack of information, the characteristic value of ground bearing capacity of the newly deposited cohesive soil, f_{a0}, in Table 4.3.3-7 are directly cited from Table 5 of Appendix III in the *Code for Design of Building Foundation* (TJ7-74).

4.3.4 Eq. (4.3.4) is established based on the ground theory of shallow foundation. It is clear in the mechanical concept to separate f_{a0} from the width and depth adjusting factors. The units of f_a and f_{a0} are the same, both are in kPa, and k_1 and k_2 are dimensionless coefficients.

1 Adjusting factors of width and depth of foundation base on cohesive soil ground

No modification is considered in the *Specifications* for the foundation width to the characteristic value of the bearing capacity of ground with various cohesive soils, f_{a0}, that is, $k_1 = 0$ is adopted. This is because large settlement is detrimental to the normal operation of bridges and culverts, while the wider the ground, the larger the later settlements of the clay and loess after the ground is compressed. When f_{a0} is determined by the load-settlement curve, it is generally determined based on the relative sinking of the load plate by 2%. When the width increases, taking $k_1 = 0$ for cohesive soil and loess can ensure that the foundation does not cause excessive settlement.

(2) Adjusting factors of width and depth of foundation base for silty soil

Silty soil has a certain degree of plasticity and presents some characteristics of sand, it is conservative in design to regard it as cohesive soil and to take its adjusting factor of width of foundation base as $k_1 = 0$. The particle size of silty soil is smaller than that of silty sand, and its adjusting factor of depth of foundation base is smaller than that of silty sand, $k_2 = 1.5$ is taken for it by reference of cohesive soil.

(3) Adjusting factors of width and depth of foundation base for sand or gravelly soil

The settlements of ground of sand and gravelly soil are almost completed during the construction period, the settlements in the later period are very small. Therefore, the bearing capacity of the ground is not controlled by the settlement, and the foundation strength can be improved when the width of foundation is increased.

(4) In principle, the ground bearing capacity of the rock may be revised by the adjusting factors of width and depth of the foundation base, but it is rather complex on how to modify it. At present, experimental data on it is still lack. It is recommended that no modification for the width and depth of the foundation base is made for rocks with under-developed joints or developed joints; modification is made by k_1 and k_2 for rocks with well-developed joints or very fracture, the factors may be determined by referring to those of gravelly soil; for rocks that have been weathered into soil or sand, the adjusting factors may be selected by referring to those of cohesive soil or sand.

(5) When the soil is immersed in water, the unit weights γ_1 and γ_2 in the formula are specified as follows: γ_1 is the weight of the base bearing stratum, when the bearing stratum is permeable soil, γ_1 is the effective weight; on the contrary, when the bearing stratum is impermeable, γ_1 is the saturated weight γ_b; if it is difficult to determine whether the bearing stratum is permeable or impermeable, it is safe to take effective weight for γ_1. The γ_2 is generally considered as the overload acting on the base. When the bearing stratum is permeable, the force acting on it includes not only the gravity of the soil particles but also the gravity of the water in the pores no matter if the soil over the bearing stratum is permeable or not, that is, γ_2 is the saturated weight. Saturated weight γ_s and effective weight γ_b are calculated as follows:

$$\gamma_s = \frac{d_s + e}{1 + e}\gamma_w \tag{4-7}$$

$$\gamma_b = \gamma_s - \gamma_w \tag{4-8}$$

or

$$\gamma_b = \frac{d_s - 1}{1 + e}\gamma_w \tag{4-9}$$

where:

d_s——specific weight of soil particles;

γ_w——unit weight of water, $\gamma_w = 10\ \mathrm{kN/m^3}$ in general;

e——natural void ratio of soil.

(6) When the bearing stratum at the foundation base is impermeable soil, the foundation base is not affected by the buoyancy of water, and the water gravity and saturated soil gravity on the sides of the foundation are regarded as the overload of the foundation. If the bearing stratum is permeable soil, it is generally affected by the buoyancy of water, and the water gravity is not considered or only the buoyancy of soil is considered. However, for deep-water foundations or complex soil layers, it is difficult to determine the water permeability of the bearing stratum. The most unfavorable combination of loads is used to determine whether to consider the buoyancy of the foundation or not.

4.3.5 In order to ensure the safety and normal use of bridge and culvert structures, both the requirements for stability and deformation must be considered in determining the allowable bearing capacity of soft ground. The method for determining the bearing capacity of the untreated soft soil ground is the same as that of the 85 Version Specifications, which can used in the design for foundations of small and medium bridges and culverts where loading test or in-situ tests cannot be carried out or no other more reliable methods are available. The basic characteristic value of bearing capacity of soft soil foundation treated by drainage consolidation method shall be determined by loading test or other in-situ tests.

The natural moisture content of saturated soft clay has a unique relationship with its strength. The soil particle density is about 2.7kN/m^3. Therefore, the void ratio is close to 1.0 when the water content is 36%, and is about 2.0 when the water content is 75%. Table 4.3.5 of the bearing capacity of the soft soil in the *Specifications* refers to the provisions of *Code for Design of Building Foundation* (GBJ 7-89), which is simple and applicable for the determination of the bearing capacity of the ground for small bridges and culverts.

5 Shallow foundations

5.1 Bearing depth

5.1.1 The safety value of bearing depth of foundation base for the pier and abutment on non-rock riverbed is taken from the Table 8.6.3 of the *Hydrological Specifications for Survey and Design of Highway Engineering* (JTG C30-2015).

5.1.2 For the foundation bases of piers and abutments of bridges and culverts which are directly set in the natural grounds, their bearing depths need to be comprehensively considered according to the soil properties and frost heave, scouring situations by flowing water, and the properties of the bridge and culvert structures.

If a foundation of pier of abutment is set at aseasonally frozen soil, the minimum bearing depth of the foundation base is the design frost depth minus the allowable maximum depth of frozen layer below the foundation base. The average value of the local measured maximum frost depth minus the average surface frost heave is adopted for the design frost depth as far as possible. If the above information is lack, the design frost depth is adopted as the nominal frost depth z_0 multiplied by various influence coefficients; the nominal frost depth z_0 in the *Specifications* is adopted from the Appendix F of the *Code for Design of Building Foundation* (GB 50007-2011) (referred to as GB 50007-2011 Version), in which three coefficients ψ_{zs}, ψ_{zw} and ψ_{ze} are considered, namely: the influence coefficient of frost depth by soil type, by the frost heave of soil, and by the environment. In addition to above three coefficients, two other coefficients (ψ_{zg}, ψ_{zf}) should also be taken into account, thus there are five coefficients in total.

The values of the above five coefficients specified in the Specifications follow the regulations of 2007 Version Specifications, including:

(1) The influence coefficient of frost depth by frost heave of soil, ψ_{zw}, is calculated based on both the multi-year tests conducted in the science testing site of frozen soil and the expression of the influence of frost depth by the frost heave of soil, $\psi_{zw} = 0.94_{\exp}^{-0.0175k_d}$, proposed by the regression analysis of test data, where k_d (%) is the frost heave rate of ground.

(2) The influence coefficient of frost depth by the foundation, ψ_{zf}, is calculated based on both

the tests of different bearing depths(h = 1.4,1.6,1.8,2.0 m) of the concrete foundation in the science testing site of frozen soil, and the expression of influence coefficient of frost depth by foundation proposed by the analysis of the test data, $\varphi_{zf} = 0.09 + 0.19\ln(100h)$, where h is the depth of foundation(m).

(3) In calculating the allowable maximum depth of frozen layer below the foundation base, h_{max}, the residual thicknesses of frozen layer of the soils with various frost heave rate are first computed based on the 20 mm as the allowable frost heave deformation of the bridge and culvert structure; its relationship with the frost heave rate d_k is expressed as $h_{\max} = 154.3 - 47\ln k_d$ by regression analysis. The computed results are shown in Table 5-1. The recommended values of $h_{\max}$ in the Table 5-1 are the values specified in Table 5.1.2-5 of the *Specifications*.

Table 5-1 The Allowable Maximum Thickness of Frozen Layer under the Foundation Base for Different Types of Soil Frost Heave

Type of soil frost heave		Weak frost heave	Frost heave	Strong frost heave	Extra strong frost heave
$h_{\max}$	Calculated value	$0.45z_0$	$0.33z_0$	$0.18z_0$	$0.096z_0$
	Recommended value	$0.38z_0$	$0.28z_0$	$0.15z_0$	$0.08z_0$

Note: 1. The z0 is the nominal frost depth, which can be found in Clause E.0.1 of Appendix E of the Specifications.
2. The recommended value is the calculated value divided by 1.2.

(4) In addition, the influence coefficient of frost depth by the topographic aspect, ψ_{zg}, also refers to the relevant regulations of the *Code for Engineering Geological Investigation of Frozen Ground*(GB50324-2014) and *Design Code for Anti-frost-heave of Canal and Its Structure*(SL 23).

5.2 Check for Ground Bearing Capacity and Eccentricity of Foundation Base

5.2.1 The foundations of piers and abutments are important components of bridges. The foundations and the bearing stratum below the foundation base must have enough strength and stability to ensure the safety of the bridges. Therefore, in design of pier and abutment, the stability of the foundation and the bearing capacity of the ground shall be checked by considering the combination of those loads, which produce the most adverse effects and may occur simultaneously during construction and in service. Whenever necessary, the settlement of the foundation shall also be checked.

When the abutment backfill is high and the ground is poor, the ground will suffer thesuperimposed compression stress induced by the high backfill, which generally results the total stress of the ground exceeding its allowable bearing capacity, causes the instability of the abutment. Therefore, the adverse effects by high backfill shall be taken into account in checking the stresses of the abutment ground.

5.2.2 The stress value and distribution shape in the foundation base is a complex problem, because the ground and the foundation are not the same material, their stiffnesses differ greatly and their deformations cannot be coordinated. Shallow foundation is a type of rigid foundations, has very high flexural stiffness and almost has no elastic deformation under the action of loads. Therefore, the original plane base of the foundation can remain a plane after settlement. If the resultant load on the base passes through its centroid, the settlement of the base is uniform. However, according to the field measurement by buried earth pressure boxes and theoretical analysis, the stress distribution shape of the base may be saddle-, parabolic- and bell-shape according to the magnitude of the load on the base center (Fig. 5-1). It can be seen that the foundation with great flexural stiffness has the 'spanning effect', that is, at the same time to adjust the base settlement to make the stress distribution tending uniformly, it causes the base stresses transferring from the middle to the edge. Under axial loading, the base stress distribution is saddle-shape if the load value is normal; with the increase of the load, the base stress distribution will change from saddle-shape to parabolic-shape, because plastic deformation zone will occur in the soil at the base edge if the load is large, thus the stresses at the base edges will not increase while the stresses in the middle part of the base will continue to increase. When the load is close to the failure load of the foundation base, the stress pattern changes from a parabolic-shape to a bell shape protruding in the middle.

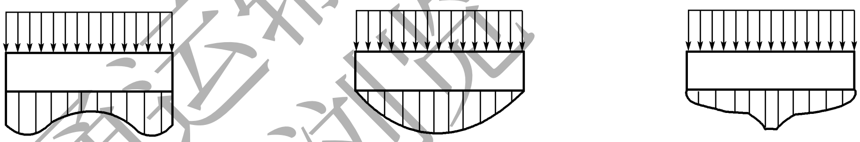


Fig. 5-1 Stress Distribution Diagram at the Base of Rigid Foundation

The shallow foundations of piers and abutments generally can be regarded as rigid foundation with a saddle-shape stress distribution. The loads on these foundations are limited by the ground bearing capacity and are not very large, and the foundations have certainbearing depths, their stress distribution may be considered as uniform under axial loading. In addition, according to the Saint-Venant principle, the soil stress under a certain depth below the foundation base (about 1.5 to 2.0 times the foundation width) induced by foundation load has almost nothing to do with the distribution shape of the load at the foundation base, but only related to the resultant force and the location of the action point. Therefore, in engineering practice, the pressure below the foundation base is assumed to be a linear distribution, which may be simplified according to formulas of elastic material mechanics, as shown in Eq. (5.2.2-1), (5.2.2-2), and (5.2.2-3) in the *Specifications*.

5.2.3, 5.2.4 The eccentricity of the resultant force on the foundation base of the abutment or pier on the bedrock is allowed to exceed the core radius(i. e. , $e_0 > \rho$) , but its value shall not exceed the value specified in Clause 5. 2. 5 of the *Specifications*. When the eccentricity of the resultant force on the foundation base exceeds the core radius, the tensile stress will be occurred, but the bedrock cannot withstand the tensile stress, so that the foundation will debond from the bedrock with gap and the stresses will be redistributed. The stresses after redistribution for foundation subjected to uniaxial eccentric compression with an eccentricity larger than the core radius is calculated by Eq. (5. 2. 3) in the *Specifications*. The calculation of the stresses after redistribution for foundation subjected to biaxial eccentric compression with the eccentricity larger than the core radius is problematical in mathematical solution, you can refer to the relevant information when necessary, the Appendix G of the *Specifications* is available for use, which is referred to the calculation norm diagram of *Railway Design Manual* in 1962. The norm diagram is originally used in calculation of stresses as well as their redistribution for a section subjected to biaxial eccentric compression in elastic material mechanics. It is also applicable to check of foundation base pressure because the same theory is applied in the check.

5. 2. 5 To limit eccentricity aims mainly to have the uniformity of the pressure on the ground. For the soil ground, the difference between the maximum and the minimum pressure shall not be too large; for the rock ground, it is allowed to consider the pressure redistribution after tension stress appeared in the calculation. For a rectangular cross-section, if uniaxial eccentricity $e_0 \leqslant 0.1\rho$($\rho$is the core radius), the ratio of the maximum to the minimum pressure is $p_{max}/p_{min} \leqslant 1.22$; if $e_0 \leqslant 0.75\rho$, $p_{max}/p_{min} \leqslant 7$. An abutment bears the earth pressure of the backfill, its eccentricity is much larger than that of a pier; but the bottom area of an abutment foundation is also larger than that of a pier, resulting a smaller maximum pressure compared with that of a pier. Therefore, some large value is specified for allowable eccentricity for the bottom of the abutment foundation, which is based on the actual stress conditions, considering its pressure is not large in general and some large allowable value will not affect the bearing capacity and settlement of the ground too much. These regulations have been using since 1950s.

For piers and abutments under combination of actions, the calculated eccentricity is larger than that under only permanent action, and the eccentricity direction is variable. Therefore, comparing to that under permanent action, the allowable value of the eccentricity for non-rock foundation is relaxed to $[e_0] \leqslant \rho$; for rock ground, tensile stress is permitted to appear in the calculation, but after that, stress redistribution shall be considered and stability against overturning shall be ensured. According to the Eq. (5. 4. 1-1) in the *Specifications*, for rectangular section under uniaxial eccentric compression, the corresponding factor of stability against overturning is $k_0 \geqslant 3.0$, $k_0 \geqslant 2.5$, and $k_0 \geqslant 2.0$ for $e_0 \leqslant \rho$, $e_0 \leqslant 1.2\rho$, and $e_0 \leqslant 1.5\rho$. The calculated factor of stability against overturning according to the eccentricity in the above is higher than the factor from the stability check. It seems that such a comparison is made under the same loading conditions, but in fact, if there is no sufficient assurance, the buoyancy force is not considered in the bearing capacity calculation, while it is considered in the calculation of stability against overturning, therefore, the loading conditions are not the same in some cases.

The calculation formulas of ρ (core radius) or e_0/ρ for biaxial eccentric compression (uniaxial eccentric compression is one of its special case) are derived as follows:

Assuming a rectangular section (Fig. 5-2), the axial force acts on the first quadrant, the bending momentM_x about x-axis, the bending moment M_y about y-axis, and the oblique bending moment M are:

$$M_x = Ne_y = Ne_0 \sin\alpha \tag{5-1}$$

$$M_y = Ne_x = Ne_0 \cos\alpha \tag{5-2}$$

$$M = Ne_0 \tag{5-3}$$

All the symbols are illustrated in Fig. 5-2.

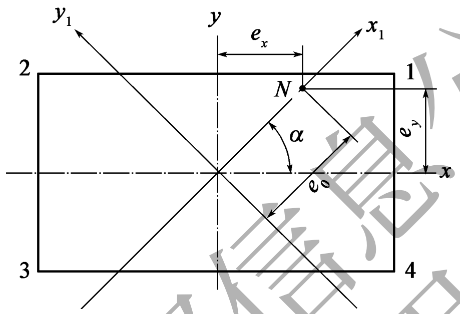


Fig. 5-2 Biaxial Eccentric Compression on the Bottom of Rectangular Foundation

$$M_x = Ne_y = Ne_0 \sin\alpha \tag{5-1}$$

$$M_y = Ne_x = Ne_0 \cos\alpha \tag{5-2}$$

$$M = Ne_0 \tag{5-3}$$

Rotate the$x$- and y-axis by an angle α, and take x_1 as the abscissa and y_1 as the ordinate, the minimum compressive stress on the foundation base may be obtained as:

$$p_{\min} = \frac{N}{A} - \frac{M}{W} \tag{5-4}$$

According to the definition of the core radius of the section in the material mechanics, $\rho = W/A$, the Eq. (5-4) may be written as:

$$p_{\min} = \frac{N}{A} - \frac{Ne_0}{\rho A} = \frac{N}{A}\left(1 - \frac{e_0}{\rho}\right)$$

that is

$$\rho = \frac{e_0}{1 - \dfrac{p_{\sin} A}{N}} \tag{5-5}$$

In calculating ρ, $p_{\min}$ may be obtained by the following equation:

$$p_{\min} = \frac{N}{A} = -\frac{Ne_0 \sin\alpha}{W_x} - \frac{Ne_0 \cos\alpha}{W_y} \tag{5-6}$$

where:

W_x——section modulus of the bottom area of the foundation about X-axis at the edge in the non-eccentric direction;

W_y——section modulus of the bottom area of the foundation about Y-axis at the edge in the non-eccentric direction.

When p_{min} is a negative value, it is tensile stress. In the calculation, stress redistribution after cracking is not considered, the negative value (tensile stress) can be substituted.

For any non-rectangular cross-section, the above calculation formulas are also applicable.

For arch bridges, the horizontal thrust is relatively large. Based on the actual stress conditions, when piers and abutments only subjected to permanent actions, it is required in the Specifications that the resultant forces shall be acted as close as possible to thecentroid of the foundation base for non-rock grounds, which follows the requirements in the 2007 Version Specifications; when piers and abutments subjected to frequency combination of actions, the resultant forces on the non-rock ground shall not exceed the core radius. For robust piers of arch bridges supported on non-rock or rock ground, the eccentricity may be not necessary to be controlled due to the one-direction thrust is a temporary case in abnormal usage case, however, ground bearing capacity, stability against overturning and sliding shall still meet the requirements.

5.2.6 The bearing capacity shall be checked when there is a soft ground or soft soil layer under the foundation base. As the value of the superimposed pressure stress of the underlying layer decreases with the increase of its depth, it is generally not necessary to check the underlying layer of all piers and abutments, it is necessary only when there is the collapsible soil or the cohesive soil with a liquid index greater than 0.6 under the foundation base. The pressure stress at the bottom of the foundation is a trapezoidal distribution, and is simplified approximately as a rectangle shape in this Clause. The pressure value, p, at the bottom of the foundation is adopted in accordance with this Clause. This is also used in settlement calculations. This method is also adopted in Clause 3.2.1 and Clause 5.2.1 of the *Specification for Design on Subsoil and Foundation of Railway Bridges and Culvert* (TB10093-2017). As the foundation deepens, the superimposed compressive stress will gradually decrease, and the influence of compressive stress of foundation base will also decrease, so the pressure stress shape can be simplified.

5.3 Check for Settlement

5.3.1 The settlement of foundation of pier and abutment will inevitably cause the settlement of the superstructure, which will affect the clear height beneath the bridge and the usage of deck expansion joints, bearings, and link slab in multi-span deck-only continuous girders. Generally, the settlements of the foundation base of pier and abutment shall be checked under the following circumstances:

1 The span lengths of the two adjacent spans are obviously different.

2 In determining the clear height beneath an overpass bridge or overpass aqueduct, the settlement values of their piers and abutments need to be calculated in advance.

3 When the piers are built on a ground with complicated geology, uneven stratum, and poor bearing capacity.

4 Bridges are reconstructed or widened.

5.3.3 In order to make the bridge facilities, such as deck expansion joints, link slabs and bearings, can accommodate the settlement differences of adjacent piers/abutments, the relative requirements are specified in this item.

The settlement differences of foundations will cause additional internal forces in the external statically indeterminate structures (continuous girders, true arches, rigid frames, etc.). Therefore, the relative requirements are specified in this item.

5.3.4 ~ 5.3.7 The calculation method for final settlement of pier and abutment foundations in the *Specifications* follows the method specified in the 2007 Version *Specifications*. This method is a simplified layer-wise summation method. In the layer-wise summation method, the foundation is regarded as a straight-line deformation body, and the deformation under external load is assumed to occur only in the range of the limited depth z (*i. e.* compression layer). The compression layers are divided into several layers, and the stress of each layer is calculated. Then, the deformation of each layer is calculated by the 'stress-strain' relationship of the soil, and the deformation of each layer is summed up to obtain the final settlement of the foundation. The layer-wise summation method is simplified or improved from the following aspects to form the method in the *Specifications*:

(1) In the layer-wise summation method, $h_i \leq 0.4b$ is required for layering (where h_i is the depth of the layer and b is the width of the foundation), which has a large analysis workload. In the method specified in the *Specifications*, each natural soil layer is basically treated as a layer to calculate the settlement.

(2) The averagesuperimposed stress coefficient, $\bar{\alpha}$, is used in the *Specifications* instead of the superimposed stress coefficient α in the original layer-wise summation method, where both $\bar{\alpha}$and α can be found in the Appendix J of the *Specifications*.

The α is derived as follows: the formula of deformation of each layer in the layer-wise summation method may be changed to (see Sections 4.3 and 4.4 in *Soil Mechanics Foundation and Foundation Difficult Interpretation* edited by Zhao Minghua et al., published by *China Architecture & Building Press*):

$$\Delta_{si} = \frac{e_{1i} - e_{2i}}{1 + e_{1i}} h_i = \frac{\alpha_i (p_{2i} - p_{1i})}{1 + e_{1i}} h_i = \frac{p_{2i} - p_{1i}}{E_{si}} \mathrm{h_i} = \frac{\mathrm{p_i}}{E_{si}} \mathrm{h_i} \tag{5-7}$$

where:

$e1$i and $e2$i ——the void ratio of the ground under the self-weight compressive stress, $p1$, and under the sum stresses of the self-weight compressive stress plus the superimposed compressive stress, $p2$, respectively;

hi ——layer depth(h i $\leqslant 0.4b$ is required in the layer-wise summation method);

α_i——superimposed stress coefficient;

p_{1i}——average value of self-weight compressive stress;

p_{2i}——average value of superimposed compressive stress;

E_{si}——compressive modulus of the i^{th} layer below the foundation base. It is calculated in the range of the compressive stress from the compressive stress of the soil taken from the self-weight compressive stress to the sum of the self-weight compressive stress of the soil and the superimposed compressive stress;

pzi ——superimposed compressive stress at z_i below the foundation base induced by the foundation self-weight, $p_{zi} = \alpha_i\ p_0$, where p_0 is the superimposed compressive stress of the foundation base. In the above formula, $p_{zi}\ h_i$ may be regarded as the superimposed compressive stress area A_{3456} enclosed by the shadow lines as shown in Fig. 5-3(a), and the compressive stress area is $A3456 = A1234 - A1256$.

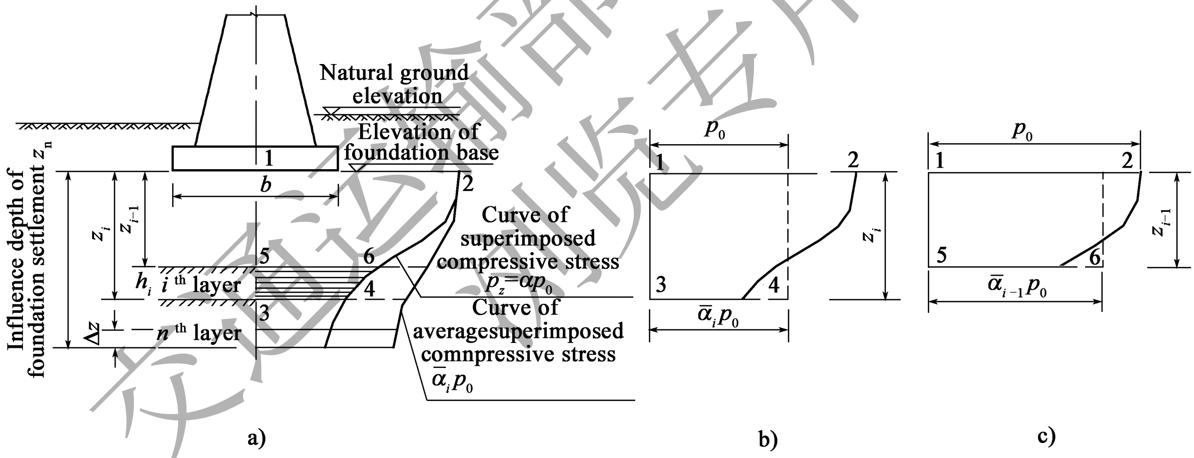


Fig. 5-3 Layering Diagram for Calculation of Foundation Settlement

The A_{1234} represents the compressive stress area of the vertical superimposed compressive stress pz in the range of zi [see Fig. 5-3(b)]. In order to simplify the calculation, an average superimposed stress coefficient is introduced as $\bar{\alpha}_i = \dfrac{A_{1234}}{p_0 z_i}$. The $\bar{\alpha}_i\ p_0 z_i$ in the above formula has the equivalent value of the pressure area $A1234$, while $A1234$ is the area of the vertical superimposed compressive stress p_z within the depth zi. Similarly, $\bar{\alpha}_{i-1} p_0 z_{i-1}$ is the equivalent value of the pressure area $A1256$ of the vertical superimposed compressive stress within the depth z_{i-1} [see Fig. 5-3(c)]. Thus, the equation of the layer-wise summation method is:

$$\Delta_{si} = \frac{p_{zi} h_i}{E_{si}} = \frac{A_{3456}}{E_{si}} = \frac{A_{1234} - A_{1256}}{\mathrm{E}_{si}} = \frac{p_0}{E_{si}}(z_i\ \bar{\alpha}_i - z_{i-1}\alpha_{i-1}) \tag{5-8}$$

The averagesuperimposed stress coefficient, $\bar{\alpha}_i$, is expressed as follows:

$$\bar{\alpha} = \frac{A}{pz} = \frac{\int_0^z p_z dz}{p_0 z} = \frac{p_0 \int_0^z \alpha dz}{p_0 z} = \frac{\int_0^z \alpha dz}{z} \qquad (5\text{-}9)$$

where:

$\int_0^z \alpha \mathrm{d}z$——the area of superimposed compressive stress at depth z, which may be found in a table using numerical integration.

Therefore, the method proposed in the *Specifications* is also called 'pressure area method'.

(3) The investigated depth for the foundation deformation, z_n, has been re-specified in the *Specifications*. In the layer-wise summation method, 0.2 or 0.1 is taken as the control value for the ratio of the superimposed compressive stress of the foundation to the self-weight compressive stress. This method is referred to as the pressure ratio method in short. However, the structure and properties of the soil layers are not considered and the influence of the load on the compression layer is emphasized too much, while attention on the more important factor of foundation size is inadequate. The relative deformation is taken as the control value in the *Specifications* (refer to as deformation method), that is, it is required to meet:

$$\Delta S_n \leqslant 0.025 \sum_{i=1}^{n} \Delta S_i \qquad (5\text{-}10)$$

where:

$\Delta S_i'$——calculated deformation value of the i^{th} layer of soil within the investigated depth Z_n;

$\Delta S_n'$——calculated settlement value of soil layer with depth Δz taken upward at the investigated depth, Z_n [see Fig. 5-3]. The Δz can be seen from Table 5.3.6 of the *Specifications*.

(4) Introduction of empirical coefficient of settlement, ψ_s

The above simplified approach will inevitably cause some deviations. This deviation coupled with the theoretical deviation of the layer-wise summation method itself will cause difference between the calculation results and the actual situation. A large amount data of settlement investigation shows that when the foundation soil layer is dense, the calculated settlement value is rather large; when the soil layer is weak, the calculated settlement value is rather small. As a result, an empirical coefficient ψ_s is introduced in the *Specifications* to modify the calculation result. The ψ_s is obtained by mathematical statistical analysis from a large number of actual settlement investigation data of real engineering projects, which can comprehensively reflect the influence of many factors, such as the assumption of lateral confinement conditions; the assumption of the homogeneity of the ground soil in calculating the superimposed pressure, the influence of the inconsistency of the actual layering of the ground soil on the superimposed pressure; the various difference between the calculated settlement value and the actual measured value for ground soil with different compressibility, etc. Therefore, the method proposed by the *Specifications* can give a result close to the real value.

The values of theψ_s are listed in Table 5.3.5 of the Specifications. The ψ_s is given according to the foundation superimposed pressure $P_0$and the equivalent value $\overline{Es}$ of compressive modulus within the range of Z_n. In the 2007 Version *Specification*s, weighted average value of depth was adopted. Though it was simple, it ignored the characteristics of the superimposed pressure along the depth. Therefore, for the compressed soil layer composed of multiple layers, the obtained $\overline{Es}$ values are not equivalent to the $\overline{\mathrm{E}}\mathrm{s}$ values for the compressed soil layer composed only a single layer. Thus, a weighted average method based on the layered deformation $\overline{ES}$ was proposed in the *Specifications*, namely:

$$\overline{E}_s = \frac{\sum A_i}{\sum \frac{A}{E_{si}}} = \frac{p_0 \sum (z_i \overline{\alpha}_i - z_{i-1}\overline{\alpha}_{i-1})}{E_0 \sum \frac{(z_i \overline{\alpha}_i - z_{i-1}\overline{\alpha}_{i-1})}{E_{si}}} = \frac{\sum (z_i \overline{\alpha}_i - z_{i-1}\overline{\alpha}_{i-1})}{\sum_{si}} \qquad (5\text{-}11)$$

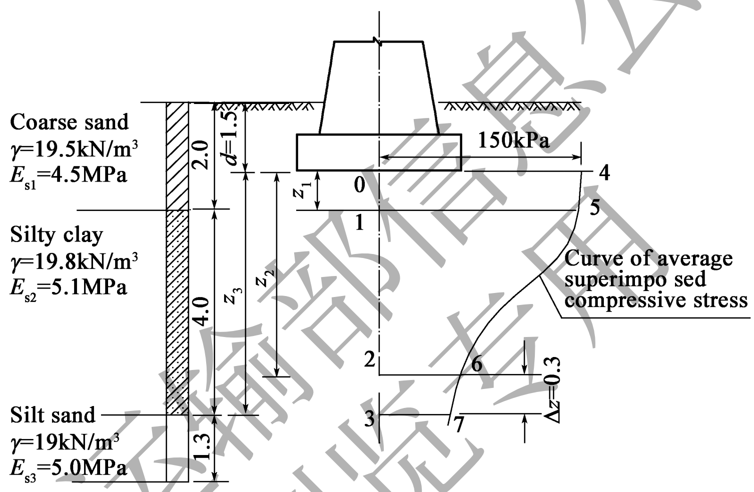


Fig. 5-4 Calculation diagram of $\overline{Es}$ (dimension unit: m)

(5) Determination of the investigated depth for the foundation settlement, Z_n.

For the investigated depth for the foundation settlement (including the influence of adjacent loads), the relative deformation is used as the control standard (referred to as the deformation method), namely: $\Delta s'_n \leqslant 0.025 \sum_{i=1}^{n} \Delta s_i$ [See Eq. (5.3.6) of the Specifications].

(6) The pressure graph in the bottom of the foundation is approximately simplified as a rectangle in this Clause, the pressure p of the foundation base is adopted in accordance with the provisions of this Clause. The reason for such a treatment please refers to the commentary of Clause 5.2.2 of the *Specifications*. For some piers or abutments with complex foundation conditions, the corner point method may be used to calculate the settlement at the center point of the foundation according to the actual trapezoidal pressure diagram, and the relevant calculation tables in the appendix of *Code for Design of Building Foundation* (GB 50007-2011) may be used in the calculation.

5.4 Check for Stability

5.4.2 There are two possibilities for foundation sliding. One is that the horizontal thrust overcomes the frictional resistance at the interface between the foundation bases and their soil to make the structure slide along the interface; the other is that the horizontal thrust overcomes the frictional resistance inside the soil to make the foundation together with part of the bearing stratum slide. The latter rarely occurs in piers and abutments, because their foundations are generally buried deeply, and a certain safety factor has been taken into account in the allowable pressure of the ground soil, which ensures that local limit equilibrium condition will not produce in the soil under the foundation bases, and plastic flow in the soil will not occur. Therefore, only the stability against the former sliding is stipulated to be checked in the *Specifications*. The safety factor of stability against sliding is the ratio of sliding resistance forces to the sliding forces.

5.4.3 The safety factors of stability against overturning or sliding of pier and abutment foundation in the *Specifications* are the same as in 2007 Version *Specifications*. The safety factor of stability against overturning is slightly greater than that against sliding under the same conditions of combination of actions. This is because the soil surrounds the foundation can play a larger role in resisting sliding than in resisting overturning.

The safety factors of stability against overturning of pier or abutment foundations, k_0, have a certain relationship with the eccentricity of resultant force, e_0, which is specified in Clause 5.2.5 of the *Specifications*. Now take the rectangular section with uniaxial eccentricity as an example, the explanation is as follows:

From Formula (5.4.1-1) of the *Specifications*, $k_0 = s/e_0$., we will have $e_0 = s/k_0 = (b/2)/k_0$ (where b is the foundation width). For a uniaxial eccentric loaded rectangular section, taking the foundation in unit length for analysis, $\rho = 0.167b$ (b is the foundation width), we can obtain e_0 under various k_0, as shown in Table 5-2.

Table 5-2 Comparison Table of Safety Factor of Stability against Overturning, K_0, Eccentricity and P_{max}/P_{min} in Uniaxial Eccentric Loaded Rectangular Section

k_0	Eccentricity e_0		P_{max}/P_{min}
	Expressed by b	Expressed by ρ	
1.2	$0.417b$	2.497ρ	$(3.502/-1.502) \times N/A$
1.3	$0.385b$	2.305ρ	$(3.31/-1.31) \times N/A$
1.5	$0.333b$	1.994ρ	$(2.998/-0.998) \times N/A$

Note: 1. The P_{max} and P_{min} are the maximum and minimum stresses, respectively, $p_{max} = N(1+6e_0/b)/A$, $p_{min} = N(1-6e_0/b)/A$, negative value represents tensile stress.

2. N is the vertical force; A is the area of the foundation base.

It can be seen from Table 5-2 that when the safety factor of stability against overturning for a uniaxial eccentric loaded rectangular section is equal to the specified limit value, the corresponding eccentricity is larger than that specified in Table 5.2.5 of the Specifications. This indicates that limitation of eccentricity will control the stability design in resisting overturning of pier and abutment foundation. For the relationship between the eccentricity limitation and the safety factor of stability against overturning, see commentary on Clause 5.2.5.

6 Pile Foundations

6.1 General

6.1.1 The pile foundations are designed according to the following two limit states:

(1) Ultimate limit state: the pile foundations achieve the ultimate bearing capacity or the overall instability, or the large deformation that unfits to keep on loading;

(2) Serviceability limit state: the pile foundations achieve the deformation limit specified in normal use or achieve some sort of limit values required for durability.

The calculation of the bearing capacity and check of the stability for pile foundations are the specific contents in ultimate limit state design, and shall be carried out in a targeted manner in accordance with the specific conditions of the project. Pile foundation deformation is the specific content of the serviceability limit state design, covering the settlement and horizontal displacement. The latter includes displacements caused by long-term horizontal loads, by horizontal earthquake actions in high seismic intensity areas as well as by wind loads. Pile settlement is the basic benchmark for calculation of the absolute settlement and the differential settlement.

6.1.2 Reasonable selection of pile types is important in the design of pile foundations. The classification of piles is explained as follows:

1. Classified by load-bearing properties:

Under the vertical load, the loads on the pile top are carried together by the pile skin friction and pile tip resistance. The values of the pile skin friction, pile tip resistance, as well as the load sharing ratio between them are mainly determined by the physical and mechanical properties of the ground soil at the pile side and the pile tip, the size of the pile, and the construction technology used. The piles are divided into friction pile and end-bearing pile in traditional classification method. Friction pile can be further divided into only friction pile and friction pile with end-bearing. End-bearing pile can be further divided into only end-bearing pile and end-bearing pile with skin friction.

2. Classified by pile forming methods:

A lot of engineering practices show that the soil squeezing effect in pile formation has a great influence on the compressive resistance of piles, pile quality control, and the environment. Therefore, according to the pile forming methods and the compaction effect on soil in the pile forming process, piles are classified into three types: non-soil-compaction piles, partial soil-compaction piles, and soil-compaction piles.

For soil-compaction piles in saturated soft soil, a significant compaction effect will be produced in the pile forming process if the piles are not designed and constructed properly, which will cause pipe pile fracture, pile upwelling and shift, ground uplift, etc., thereby reducing the compressive resistance of the pile. Sometimes it can damage the neighboring buildings. After the pile foundations are constructed, reconsolidation settlement of the soil layer may occur after dissipation of pore water pressure in the saturated soft soil, which will result negative skin friction in the pile, reduce the resistance and increase the settlement of the pile foundation. Soil-compaction piles can receive good technical and economical results only if they are properly designed and constructed.

In the unsaturated loose soil, the bearing resistance of soil-compaction pile is significantly higher than that of the non-soil-compaction pile. Therefore, the correct selection of pile forming methods and technology are important in the design of pile foundations.

For non-compaction piles, since they do not have the negative effects of soil compaction, while have the capacity to pass through various stiff interlayers, to be rock-socketed, and to enter various stiff bearing stratums, therefore, the selection range of the pile geometric size and the resistance of a single pile for non-compaction piles is large.

6.1.5 The settlement of friction piles is generally greater than that of end-bearing piles. To prevent uneven settlement of the pile foundations, it is not recommended to use friction piles and end-bearing piles in a same pile foundation. It is also not recommended to use piles with different diameters, different materials, and excessively different pile tip depths in a same pile foundation, not only because it is complicated in design, but also it is prone to have errors in construction.

6.1.6 Commonly used piles are those with a slenderness ratio less than 40. Static loading test methods include heap load method, anchor pile method, self-balance method, etc. Static loading test methods are often used to test the resistance of piles, the test results are intuitive and reliable. In the cases the piles are located in narrow sites, slopes, foundation pit bottoms, water (sea) or the piles with super large bearing capacity, the traditional static loading test methods (heaping method or anchor pile method) are often restricted by factors such as site conditions or loading capacity of the tests. The self-balancing method is a new method of static loading test for piles. It has the advantages of time-saving, labor-saving, safety, no pollution, low comprehensive cost, and no restrictions of site conditions and low loading tonnage. At present, the self-balancing method has been used in cast-in-place drilled piles, manual bored piles, caisson piles, pipe piles, and deep foundations (caissons, diaphragm walls), including friction piles, end-bearing friction piles, friction end-bearing piles, end-bearing piles, and uplift piles according to classification of piles by load-bearing properties. The self-balancing method tests currently has a corresponding industry standard

Static Loading Test of Foundation Pile—Self-balancing Method (JT/T 738-2009).

6.2 Detailing Requirements

6.2.1 The diameter of a pile is determined by the comprehensive factors, such as the values of forces subjected, the forms of pile foundation, and the construction conditions. The diameters of reinforced concrete pipe piles generally are 0.4 ~ 1.2 m, which can facilitate the applications of the current driven equipment in the construction. The minimum thickness of the pipe wall is 80 mm, which refers to the wall thickness when the pipe is manufactured by the centrifugal rotating machine.

6.2.2 Reinforced concrete square piles and pipe piles driven by hammering or vibration are all prefabricated piles. They shall be reinforced to meet not only the strength requirements of the foundation structure, but also the stress requirements during transportation, lifting and driving process of the piles, therefore the reinforcing bars shall be arranged along the full length of the pile. In the driving process by hammering or vibration, the two ends of the pile are stressed greatly, especially in the stiff soil layer. Therefore, the spacing of stirrups or spirals at both ends of the pile shall be appropriately increased.

The drilled (bored) piles are made by drilling (boring) a hole first, and then poured concrete in-site, without process of pile hoisting, driving, and so on. Therefore, the drilled (bored) piles are only reinforced segment by segment according to the requirements for structural stresses. If no reinforcing bar is needed from the analysis of the internal force, the piles are reinforced according to the detailing requirements.

6.2.3 In order to ensure the welding quality, welding are performed in factory as far as possible and double-sided welding shall be used. If double-sided welding is not available, it needs inside lining plate for single-sided welding or other reliable welding processes.

In order to prevent the buckling failure of steel pipe piles, the required minimum wall thickness is stipulated.

6.2.4 The basic principles of concrete-filled steel tube composite piles are: the stability and stiffness of steel tube is improved by the inner filled concrete; the compressive strength and deformation resistance of core concrete is enhanced due to the hooping effect from the outside steel tube, which make the core concrete under three-direction compression. Because of the advantages of high ductility, great resistance, and convenient construction, the concrete-filled steel tube composite piles have a great development prospect in offshore pile foundation engineering. The section reinforcement ratio of concrete-filled steel tube composite pile is generally controlled by the requirement of horizontal resistance. Considering the durability of the piles, the adverse effects of accidental collision of ships and construction factors, and referring to the minimum reinforcement ratio in specifications of China and other countries, the ratio given in the Specifications is 0.6%. The end-bearing piles, uplift piles, and piles with negative skin friction are reinforced along the whole pile lengths because they have large axial pressures in their whole pile lengths. For piles on a slope or shoreside where soft soil exist, they are generally subjected to the shear forces induced by the additional horizontal thrusts and are also reinforced along the whole pile length.

6.2.5 The corrosion of steel belongs to electrochemical corrosion. Due to difference in environmental conditions, water quality and climate in the sites of the projects, the corrosion characteristics of steel are different. Various factors shall be considered comprehensively in anti-corrosion design, including the importance of the project, the function requirements, the parts of the structure, the possibility of the anti-corrosion treatments in construction, the methods of maintenance, and the source of materials.

6.2.6 Piles are arranged according to the stress and construction conditions. Pile groups are generally arranged symmetrically. If the area of the cap is limit while the number of piles is large, pile groups can be arranged in a plum or a ring.

From the point of view of mechanical behaviors, the center-to-center spacing of friction pile group had better to be large enough to make the pressure distribution range at each pile tip not overlap, thus can fully play the carrying capacity of each pile. According to this requirement, the center distance is $6d$ (d is diameter or side length) according to test results. However, if pile spacing is $6d$, a large cap is required. Therefore, the spacing in a pile group generally is less than $6d$. To prevent the pressure of the soil at the plane of pile tip induced by adjacent piles to be overlapped too much, and to prevent too compactness of the soil to let the pile to be driven down, it is stipulated according to the experiences that the center distance of piles at the plane of pile tip shall no less than $3d$ for the piles driven by hammering or static pressure. For piles driven by vibration, it is stipulated that the center distance at the pile tip plane shall not be less than $4d$ because the compactness effect of the soil is more significant. The center distance of piles at the bottom of pile cap shall not less than 1.5 times of pile diameter (or side length).

For drilled piles during construction, there is no interaction effect among piles and no phenomenon that the pile cannot be driven to the required depth, therefore the spacing of the piles may be appropriately reduced to save the area of the pile cap. However, if the center-to-center spacing is too small, the skin friction between the soil and the pile will be reduced, thus it stipulated that the spacing shall not be less than $2.5d$.

For end-bearing piles, as long as the construction permits, the spacing can be appropriately reduced compared to the friction piles because there is no phenomena of pressure overlap at the pile tips.

The distance from the outside of the border pile (or corner pile) to the edge of pile cap shall be large enough to ensure the protection cover after the primary reinforcement of the pile top is bent into a flared shape, and to ensure the masonry of the cap edge will not be broken under the action of the bending moment of the pile top and the lateral force.

6.2.7 The depth, the reinforcement and the concrete strength class of the pile cap are generally determined according to the requirements for stresses of the pile cap. However, the stresses of the pile cap are generally complex, and there is no mature calculation method. According to the current *Specification for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTG 3362), the depth of a pile cap should not be less than 1.5 times the pile diameter and 1.5 m. The concrete strength class shall not be less than C25. If reinforcing steel with a nominal strength of 400 MPa or larger are used, the concrete strength class shall not be less than C30 and one layer of reinforcement mat shall be set at the top of the pile around the bottom of the pile cap. When the primary reinforcement of the pile is extended into the pile cap for connection, the reinforcement mesh shall pass through the top of the pile by full length, and shall be tied with the primary reinforcement of the pile to prevent cracks in the tension area of the pile cap, as shown in Fig. 6-1. When the pile head is embedded directly into the cap without trumpet connection, one or two layers of local reinforcement mesh shall be set on the top of the pile, in which the diameter of the steel bar should not less than 12 mm, the length of each side of the mesh should not less than 2.5 times the pile diameter, and the mesh size should be 100 ×100 mm ~ 150 × 150 mm.

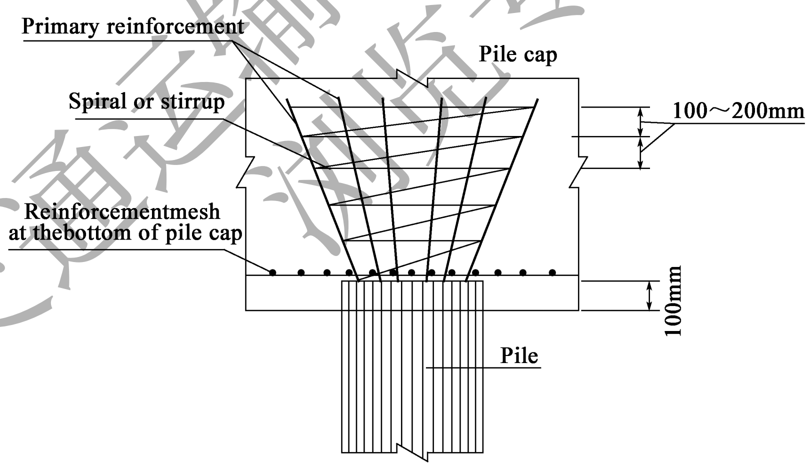


Fig. 6-1 Connection Between Pile head and Pile Cap

The calculation of the pile cap is performed in accordance with the relevant chapters of the current *Specification for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTG 3362).

The auxiliary reinforcement of lateral bracing is arranged no less than 0.15% the cross-sectional area of the bracing.

6.2.8 In order to strengthen the connection between piles and the pile cap, it is stipulated in the

Specifications that the concrete pile heads shall be embedded into the pile cap 100 mm.

Hinged structure is complicated in details, and unfavorable to the corrosion resistance of the pile head, therefore fixed connection is generally used in real engineering design. The stress state of the fixed pile head is complicated, and is generally computed by the stress superposition method. The fixed form of the pile head shall meet the following requirements:

When the pile head is fixed, the connection needs to bear the bending moment, shear force, and axial force of the pile head. The checking contents are listed in Table 6-1.

Table 6-1 Checking Items for Fixed Connection of Pile Head

Fixed connection form / Load condition	Pile head is extended directly into the pile cap	Pile head is extended into the pile cap through anchors
Axial compression	Squeezing and punching of concrete in pile head	
Axial tension	Anchor depth of the pile head	Cross-sectional area, development length, and welding length of the anchor
Horizontal shear and bending moment	Squeeze stress of the concrete on the side of the pile	Extrusion stress of concrete in the pile side and stress of the steel members

6.3 Analysis

6.3.1 From the excavation inspection of some old bridges, it was found that some bottom surfaces of the pile caps were separated from the ground, thus the ground below the pile caps is not considered to share the vertical load above the bottoms of the caps.

The earth pressure of abutment is generally calculated from the original ground before filling, and when there is excavation, it is calculated from the bottom of the foundation pit. For old fills or alluvial fills, the so-called ‘original ground’ still refers to the ground before filling with new soil. When the steep ridge in front of the abutment is close to the abutment, the earth pressure is calculated from the ground under the steep ridge; when the abutment is constructed after filling and the filling quality is fully guaranteed, the earth pressure is calculated according to the ground after the filling.

6.3.2 The negative skin friction (down drag) of the compressed soil layer to the pile shall be considered in the following two cases: (1) when a pile passes through soft soil and/or weak foundation soil layer and reaches the solid soil layer, and there are vertical loads (such as approach fill) acting on the soft soil layer around the pile, which will cause compressive settlement deformation larger than the displacement of the pile (including pile compression and pile tip settlement); (2) the groundwater in the soil layer is substantially lowered and caused the ground to sink in a large area, which will make the compression settlement speed of the soil layer greater than the displacement speed of the pile body.

At present, the calculation methods for negative skin friction are not mature, though there are many methods but the estimation results are quite different among them. The negative skin friction could be obtained from field tests, but it requires a large investment and a long period. Therefore, the negative skin friction is usually estimated by empirical formulas based on relevant data. It is proposed by the *Specifications* to calculate the negative skin friction of a single pile as follows:

$$N_u = u\sum_{i=1}^{n} qnili \tag{6-1}$$

$$qni = \beta\sigma' vi \tag{6-2}$$

where:

Nn——negative skin friction of a single pile (kN);

u——perimeter of a pile (m);

li——depth of each soil layer above the neutral point (m); the neutral point depth l_n shall be determined according to the condition that the settlement of the soil around the pile is equal to the settlement of the pile. If it is available to be calculated, it can also be determined according to Table 6-2;

qni——calculated value of negative skin friction (kPa) of each soil layer with respect to l_i, when the calculated value is greater than the positive skin friction, take the positive skin friction value;

β——negative skin friction coefficient, it may be taken according to Table 6-3;

σ'_{vi}——average vertical effective stress of the i^{th} layer of the pile side soil (kPa), which is calculated by Eq. (6-3);

$$\sigma'_{vi} = p + \gamma'_i \cdot zi \tag{6-3}$$

$\gamma' i$——weighted average buoyant unit weight calculated by the depth of the soil around the pile above the bottom of the i^{th} soil layer;

zi——midpoint depth of the i^{th} layer of soil from the ground;

p——uniform load acting on the ground.

Table 6-2 Neutral Point Depth, l_n

Bearing stratum properties	Cohesive soil, silty soil	Medium dense sand or more dener sand	Gravel, cobbles	Bedrock
Neutral depth ratio l_n/l_0	0.5 ~ 0.6	0.7 ~ 0.8	0.9	1.0

Note: 1. The l_n and l_0 are the neutral point depth and the lower limit depth of the soil around the pile and with settlement deformation, respectively;

2. When the pile passes through the self-weight collapsible loess layer, the depth is increased by 10% of the value listed in the table (except the bearing stratum is bedrock).

Table 6-3 Coefficient of Negative Skin Friction, β

Soil	β	Soil	β
Saturated soft soil	0.15 ~ 0.25	sand	0.35 ~ 0.50
Cohesive soil, silty soil	0.25 ~ 0.40	Self-weight collapsible loess	0.25 ~ 0.30

Note: 1. In the same type of soil, the larger value in the table is used for driven piles or cast-in-place piles of driven pipe; the smaller value in the table is used for cast-in-place bored (punching) piles;

2. For coefficient of negative skin friction of a fill with various components, the largest value among all the coefficients of the components in the Table is taken.

It is worth noting that the value of negative skin friction of a single pile calculated according to Eq. (6-1) shall not be greater than the sinking down self-weight of the soil surrounding the pile which is shared by the single pile, i. e., the self-weight of the soil cylinder which has the center axis same as the pile, with a radius of 1/2 pile spacing and a high of l_n. As for the negative skin friction of pile group, it is recommended to calculate the downdrag load of each single pile in the pile group according to the calculation method for negative skin friction of single pile.

In design of the pile foundation, necessary measures (such as asphalt coating on the surface of the precast pile, etc.) can be applied to reduce or eliminate the negative skin friction.

6.3.3 In the calculation formula of the characteristic value of the axial compressive resistance of a single drilled (bored) pile supported in the soil: $Ra = \frac{1}{2} u \sum_{i=1}^{n} qikli + A_p q_r$, the first item is the characteristic value of the total skin friction of the pile side, and the second item is the characteristic value of the total bearing resistance of the pile tip.

(1) On the nominal value of skin friction of pile side soil, *qik* (kPa)

The classification of soil shall be taken according to the provisions of Section 4.1 of the Specifications, *qik* is taken as the data in the 2007 Version Specifications.

(2) Theadjusted characteristic value of the bearing resistance of the soil at pile tip, q_r (kPa), see Eq. (6-4).

$$q_r = m_0 \lambda [f_{ao} + k_2 \gamma_2 (h - 3)] \tag{6-4}$$

The calculation formula for q_r is still adopted the provisions in the 2007 Version *Specifications*.

The upper limit of q_r is not the maximum value calculated by the formula, but is obtained based on a large amount of measured data.

When the bearing stratum at pile tip is cohesive soil, the upper limit of q_r is not limited, because the measured data by some test piles are greater than the possible maximum value calculated by the formula.

When the bearing stratum at pile tip is sand, the upper limit of q_r is specified according to the three categories: 1000 kPa for silty sand, 1150 kPa for fine sand, and 1450 kPa for medium sand, coarse sand, and gravelly sand.

When the bearing stratum at pile tip is gravelly soil, 2750 kPa is taken as the upper limit of q_r.

When reliable test results show that the q_r value exceeds the above-mentioned specified values, the actual measurement results can be taken as the q_r value.

(3) Regarding the self-weight of the pile below the ground or local scouring line, the q_{ik} and q_r in the recommended formulas in the Specifications adopt the numerical value and calculation formula of the 2007 Version Specifications. Most of these values are determined based on static loading tests for medium and short lengths of pile with medium and small diameters. The self-weight of the pile body has been balanced in the soil before the static loading test, and it is not necessary to consider the self-weight of the pile body in the design. However, considering that the difference between the self-weight of the pile body and the weight of the replacement soil will cause settlement, to ensure safety, the difference between the self-weight of the pile body and the replacement soil is considered as overload.

6.3.4 According to the different grouting locations, the type of post-grouting for a pile can be divided into post grouting at pile tip (pile tip post-grouting), post grouting at pile side (pile side post-grouting), and combined post grouting at pile tip and pile side (combined post-grouting).

The pile side post-grouting employs the embedded pipe at the pile body for grouting. It may be divided into two forms according to the grouting devices: ① the grouting pattern pipe is set longitudinally along the reinforcement cage; ② the grouting pattern pipe are set along the ring of the steel cage according to the pile diameter; ③ the pressure grouting devices in the pile side are set along the longitudinal direction of the reinforcement cage.

For the post-grouting at the pile tip, the grout diffuses in the sediment and the soil in a certain range of the pile tip through the types of infiltration (coarse-grained soil) and splitting (fine-grained soil), thereby playing a reinforcement role. Test results showed that the grout spread upward through the mud skin and weak disturbance layer on the side of the pile to a height of 10-12 m (low value for coarse-grained soil and high value for fine-grained soil), which enhances the pile skin friction. This indicated that the pile tip grouting not only enhanced the tip resistance but also increased the lateral skin friction in a certain range above the pile tip. This phenomenon has been confirmed by investigations through excavation and by pile force tests.

During the revision of the Specifications, test data of 716 post-grouting piles in China were obtained for statistics. The enhancement coefficient in Table 6.3.4 was obtained through statistical induction. According to the test data of 178 post-grouting piles with good enhancement results, the R_{ac} was calculated according to the formula in Clause 6.3.4. Among them, q_{ik} and q_r were taken from the experience values provided in the survey reports or the experience values listed in the Specifications. The skin friction enhancement coefficient, β_{si}, and the tip resistance enhancement coefficient, β_p, were taken as the upper limit values listed in Table 6.3.4. The scatter plot of the measured value R_{ac} and the calculated value R_{ac} is shown in Fig. 6-2. It can be seen that the actual measured values are basically close to 450 line or above the 450 line, that is, they are all higher or close to the calculated values, indicating that the characteristic value of the axial compressive resistance of the post-grouting cast-in-place pile is properly reliable when calculated in accordance with Clause 6.3.4.

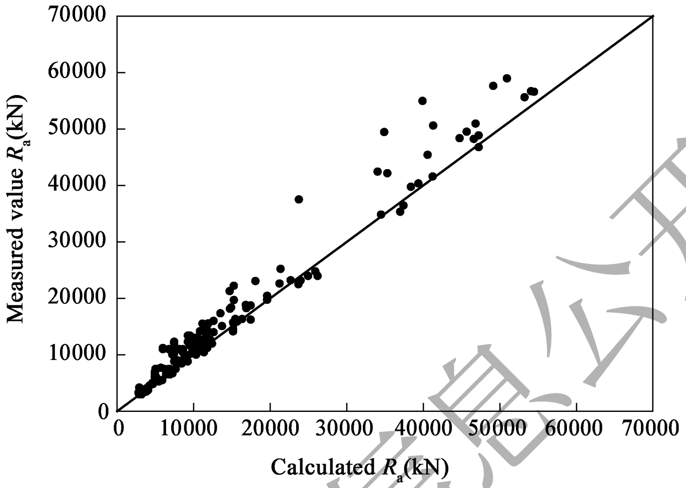


Fig. 6-2　Relationship Between Allowable Measured Value and Calculated Value of Axial Compressive Resistance of Single Pile of Post-grouting Pile

In order to ensure that the post-grouting can improve the resistance of the pile, attention shall be paid for post-grouting piles to the following technical indicators: ① water-cement ratio of the grout; ② grouting pressure at pile tip when grouting is finished; ③ loading duration; ④grouting flow; and ⑤grouting quantity. See Appendix K of the *Specifications* for details.

6.3.5　According to the usage and test results in recent years, the calculated pile resistance based on the *Code for the Design of Ground Base and Foundation of Highway Bridges and Culverts* (JTG D63-2007) does not differ greatly from the resistance in actual situation. For pipe piles with open ends, in order to simplify the calculation, the pile tip soil plugging coefficient is introduced.

The bearing capacity mechanism and the relationship between the resistance and related factors of pipe piles with open ends are more complicated than those of pipe piles with closed ends, this is due to the 'soil plugging' formed by part of the soil at the end of the pile pressed into the pipe with open end during the pile driving process. The height and occlusion effect of the soil plug vary with many factors, such as soil properties, pipe diameter, wall thickness, and the depth of the pile entering into the bearing stratum. The degree of occlusion of the pile tip soil directly affects the bearing behavior of the pile, which is called the soil plug effect. The difference in the degree of occlusion causes two types of failure modes for the tip resistance. In one failure mode, the soil plug is squeezed upwards along the pipe, or a large amount of soil at the pile tip is pressed because the compression deformation of the soil plug is large. This state is called incomplete occlusion, which will cause a reduction in the tip resistance. In the other failure mode, the pile is destroyed like a pipe pile with closed end, this phenomenon is called complete occlusion. The degree of occlusion of the soil plug is mainly related to the relative depth of the pile tip embedded into the bearing stratum and the pile diameter.

6.3.7 The compressive resistance of a single rock-socketed pile (excluding the pile supported on highly weathered and completely weathered rock) given in the *Specifications* generally consists of three parts: the total skin friction provided by the soil around the pile, the total skin friction in the rock-socketed part, and the total tip resistance.

Regarding the skin friction of the overlying soil layer, a former concept is like that: all rock-socketed piles must be end-bearing piles, and the skin friction of the soil layer shall not be considered in all end-bearing piles. Research indicates that the performance characteristics of the pile skin friction and tip resistance vary with the property and depth of the overlying soil layers, the property and depth of the embedded bedrock, and the sediment depth at the pile tip. A lots of field test results show that under normal circumstances, even if the pile tip is placed in fresh or slightly weathered bedrock, the overlying soil layer may still exert its skin friction to the pile.

The rock-socketed pile mentioned in this Clause refers to the pile that its end is embedded into medium weathered rock, slightly weathered rock or fresh rock, in which the rock mass at the pile tip can be sampled for uniaxial compressive strength test. For rock-socketed pile supported on highly weathered rock, the strength of the rock cannot be determined by the uniaxial compression test due to available to be sampled. The nominal value of the ultimate bearing capacity of the segment of the rock-socketed pile in such highly weathered rock could be calculated as the pile is supported in sand or gravelly soil according to the degree of weathering of the rock mass.

6.3.8 According to the special research report of the editing team of the *Specifications*, the formula for calculating the socketed depth of rock-socketed pile is applicable for pile socked in a rock with $f_{rk} \geqslant 2$ MPa. The formula is obtained according to the following assumptions:

1) The stresses on the sidewall of the socket rock change linearly. Among them, the top surface of the bedrock (the top surface of the socket rock) and the sidewall stresses of the soil layer at the pile tips are consistent and equal to the allowable stress of the sidewall. The sidewall stresses in a certain depth range below the top surface of the bedrock are assumed to be the same, and it is assumed that the sum of the stresses in this segment with constant stress is equal to the load on the loaded segment. In other words, the stress distribution of the pile within the socket depth, h, is in such a pattern that it is a trapezoid distribution at the upper part and a triangle distribution at the lower par, and the triangle in the trapezoid has the same shape of the lower triangle shape (Fig. 6-3);

2) For the distribution of pilelateral pressure, it is assumed that the maximum pressure $\sigma_{\max}$ is equal to the average compressive stress σ for rectangular piles. While for circular piles, it is assumed that the maximum pressure $\sigma_{\max}$ is equal to 1.27 times of the average compressive

stress σ;

3) The rock-socketed pile is assumed to be rigid when compared with pile in the soil layer;

4) The frictional forces and adhesion between the pile and surrounding rock and/or soil is ignored.

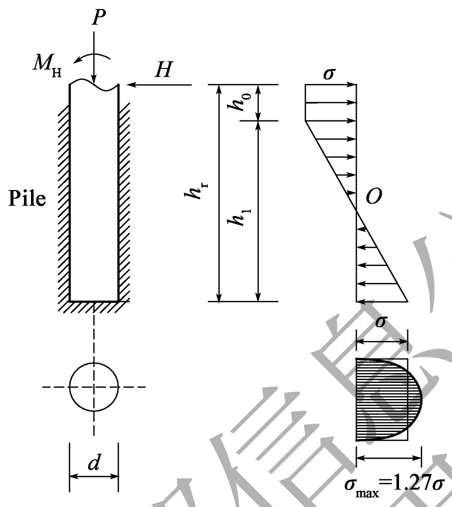


Fig. 6-3 Schematic Diagram of Pressure Distribution on Pile Side

1 For rectangular piles, according to the above assumptions, we can obtain:

$$\begin{cases} \sum H_{hol} = 0 \\ H = \sigma h_0 b \end{cases} \tag{6-4}$$

$$\begin{cases} \sum M_o = 0 \\ H\left(h_0 + \dfrac{h_1}{2}\right) + M_H = \sigma h0b\left(\dfrac{h_0}{2} + \dfrac{h_1}{2}\right) + \dfrac{1}{6}\sigma b h_1^2 \end{cases} \tag{6-5}$$

$$h_r = h_0 + h_1$$

$$\sigma_{\max} = 0.5\beta f_{rk}$$

By combining the above formulas, the following equations can be obtained:

$$h_0 = \frac{H}{\sigma b}$$

$$h_1 = h_r - h_0 = h_r - \frac{H}{\sigma b}$$

Substituting h_0, h_1 into Eq. (6-5), then:

$$H\left(\frac{H}{\sigma b} + \frac{h_r}{2} - \frac{H}{2\sigma b}\right) + M_H = H\left(\frac{H}{2\sigma b} + \frac{h_r}{2} - \frac{H}{2\sigma b}\right) + \frac{1}{6}\sigma b\left(h_r - \frac{H}{\sigma b}\right)^2$$

After simplification, we have:

$$\sigma^2 b^2 h_r^{\ 2} - 2h_r\sigma bH - 2H^2 - 6\sigma bM_H = 0$$

For rectangular piles, $\sigma = \sigma_{\max} = 0.5\beta f_{rk}$.

By solving the binary linear equation about the h_r, the minimum rock-socking depth can be obtained:

$$h_r = \frac{2\sigma H + \sqrt{4\sigma^2 b^2 H^2 + 8\sigma^2 b^2 H^2 + 24\sigma^3 b^3 M_H}}{2\sigma^2 b^2} = \frac{H + \sqrt{3\beta f_{rk} b M_H + 3H^2}}{0.5\beta f_{rk} b} \tag{6-6}$$

where: H——horizontal force at the top surface of the bedrock (kN);

M_H——bending moment at the top surface of the bedrock (kN · m);

h_r——socket depth (m);

h_0——depth of the rock in the socket rock segment when the allowable stress is reached (m);

h_1——depth of the elastic zone of the socket rock layer (m);

b——side length of the pile in the plane perpendicular to the bending moment (m);

σ_{max}——maximum compressive stress on the side of the pile (kPa);

β——reduction coefficient for converting the vertical compressive strength of the rock to the horizontal compressive strength, which is taken as 0.5 ~ 1.0, determined by the structure beside the rock stratum. The smaller value is taken for rock with developed joints, while the larger value is taken for rock with under-developed joints;

f_{rk}——Nominal value of uniaxial compressive strength of rock of saturated sample (kPa).

2 Circular pile

For circular piles, except that σ is equal to 1.27 times the average compressive stress on the side of the pile, the other assumptions are the same as those for rectangular piles. The minimum rock-socketed depth of the circular pile can also be obtained.

$$h_r = \frac{1.27H + \sqrt{3.81\beta f_{rk} d M_H + 4.894H^2}}{0.5\beta f_{rk}} \tag{6-7}$$

where:

d——pile diameter (m).

The rock-socketed depth of a rock-socketed pile is generally not greater than the recommended upper limit of the rock-socketed depth given in Table 6-3.

Table 6-3 Recommended Value for Socket Depth of Rock-socketed Pile

Rock mass rating of strength	The value range of recommended socket depth
Extremely weak rock	Drilled pile: 6 ~ 9d
	Bored pile: 3 ~ 5d
Weak rock	4 ~ 5d
Moderately weak rock	3 ~ 4d
Moderately strong rock	2 ~ 3d

The resistance of the rock-socketed pile is affected by the rock-socketed depth, which may be summarized as follows:

1) The distribution of skin friction in the rock-socketed segment affected by the rock-socketed depth. Under the action of general load, the non-linear distribution of skin friction in the rock-socketed segment is significant, generally the distribution will show a bimodal curve. When the rock-socketed depth is small, the non-linearity of the skin friction distribution in the rock-socketed segment is particularly obvious; when the rock-socketed depth is large, the peak at lower position will gradually degenerate. Generally, the skin friction peak at upper position is greater than that at the lower position. The location of the maximum peak of the skin friction has a trend to move downward with the increase of the ratio of rock-socketed depth to pile diameter.

2) As the ratio of rock-socketed depth to the pile diameter increases, the pile tip resistance will decrease significantly, that is, the ratio of the pile tip resistance to the pile top load decreases with the increase of the rock-socketed ratio. For rock-socketed piles in stiff rock, the decrease trend is more obvious.

The staticloading test data of 120 rock-socketed piles were summarized and their bearing characteristics were analyzed by considering the different strength of the bearing stratum rock at the pile tips. The conclusions are as follows:

① When the bearing stratum rock at the pile tip is extremely weak rock ($f_{rk} \leqslant 5$MPa), the maximum rock-socketed depth of the rock-socketed pile generally exceeds 7 ~ 12d and it may be taken as 10 ~ 15d in some special cases by considering the actual engineering situation, bearing capacity, economy, construction, and other factors. The recommended rock-socketed depth for bored rock-socketed piles is 6 ~ 9d, and for excavated piles is 3 ~ 5d.

②When the bearing stratum rock at the pile tip is weak rock (5MPa $< f_{rk} \leqslant$ 15MPa), the maximum socket depth generally exceeds 5 ~ 10d, and the optimal socket depth is recommended to be 4 ~ 5d.

③ When the bearing stratum rock at the pile tip is moderately weak rock (15MPa $< f_{rk} \leqslant$ 30MPa), the maximum socket depth generally exceeds 5-7d, and the optimal socket depth is recommended to be 3 ~ 4d.

④ When the bearing stratum rock at the pile tip is moderately strong rock (30MPa $< f_{rk} \leqslant$ 60MPa), the maximum socket depth generally exceeds 4 ~ 5d, and the optimal socket depth is recommended to be 2 ~ 3d.

6.3.9 It is known from the experiments that when the pile is pulled up, the soil around the pile can bulge upward relatively freely; while the soil around the pile is squeezed against each other when the pile is compressed and difficult for pile to be driven. Therefore, the skin friction is different in these two movements. The skin frictions of the soil to the side of the pile when the pile is pulled out is much smaller than that when the pile is compressed downward. According to research in China and other countries, the skin friction when the pile is pulled out is 0.6 ~ 0.8 times of the skin friction when the pile is compressed if the soil around pile is cohesive soil and silty soil; and it is 0.5 ~ 0.7 times if the soil around the pile is sand. For safety, it is uniformly taken as 0.6; after considering the safety factor, it is taken as 0.3 in the Eq. (6.3.9) of the *Specifications*.

For belled pile, when the ratio of pile length to pile diameter is $\sum li/d \leqslant 5$, the weight of the pile (soil) may be taken as the weight of the pile (soil) formed by the projection surface of the enlarged end cylinder. In this case, the perimeter of the failure body is πD, and the nominal value of the ultimate skin friction of a single pile when it is pull upward is still taken as that of the soil on the side of the pile.

For belled pile with $\sum li/d$, its uplift failure mode is affected by the compressibility of the soil, and the shear surface in the upper part of the pile will change to the pile-soil interface, that is, the diameter of the failure column is reduced from D to d. Therefore, the perimeters of the shear surfaces are calculated in segments, taking $\sum li/d = 5$ as the boundary.

6.3.10 There are *m method*, *constant coefficient method*, *c method*, *k method*, etc., for the internal force calculation of piles under horizontal load. A large number of tests and engineering practices show that the *m method* is suitable if the ground displacements do not exceed 10 mm. In the case of piles in deep and rapid flow water and subjected to horizontal loads, its displacement at the ground is usually greater than 10 mm and behave nonlinearity. However, the *m method* is still used in the *Specifications* considering that this method is already familiar by the majority of engineers and technical personnel, and it is convenient in usage due to there are ready-made dimensionless coefficient tables, moreover, its calculated result only has small deviation from the actual result when the horizontal load on the pile is small or the displacement of the pile at the ground does not exceed 10 mm. For large and major projects, *p-y* curve method may be used.

The calculation of elastic internal force and displacement of concrete-filled steel tube composite piles is determined based on experiences or experiments. In the absence of technical data, the pile body stiffness is calculated according to the stiffness superposition principle.

In the calculation of the concrete filled steel tube composite pile as an entire member, its horizontal rigidity is superimposed by the rigidity of steel tube and rigidity of concrete; in the calculation of the distribution of the internal force between the steel tube and core concrete, the steel tube is equivalent to reinforcement steel. Because the formula reflecting the combined effect is too complicated, the most conservative value is takenfor the sake of safety.

6.3.11 The failure forms of a pile group may be block failure or punching failure of a single pile. To avoid block failure, the pile group is considered as a block foundation and the end bearing resistance of the total area at the pile tip plane is checked in accordance with the provisions in the Appendix N of the *Specifications*. To avoid punching failure of a single pile, resistance of individual pile shall be checked.

6.3.12 The pile compressive deformation is calculated according to the actual skin friction distribution. When there is no relevant information, the following equation is used for estimation:

$$\text{Pile compressive deformation} \approx \frac{Pl}{2EA_p} \tag{6-8}$$

where:

P——load acting on the pile top (kN);

l——pile length (mm);

E——compressive modulus of concrete of the pile body (kN/mm^2);

A_P——sectional area of the pile body (mm^2).

7 Caisson Foundations

7.1 General

7.1.1 The caisson is a deep foundation with many advantages in the construction, such as relatively stable and reliable technics, easy and simple in construction. At the same time, the caisson foundation has good stability and can support large loads due to deeply buried. The degree of difficulty in caisson lowering will be increased dramatically if there are obstacles that are difficulty to be removed, such as boulders, logs of trees and existing bridge foundations. In addition, when the rockbed expected to be bearing stratum under the overlying soil of the riverbed has a steep slope, the construction difficulty will also be increased. Therefore, we should try to avoid using the caisson foundation in the above circumstances.

7.1.2 According to the stiffness of the soil, the caisson materials may be concrete reinforced by detailing requirement, reinforced concrete, steel shell concrete, and steel. Table 7-1 lists the materials of some caissons in large-scale bridge built in recent years in China. From the statistical results, it can be seen that the caissons are mainly made of reinforced concrete and steel shell concrete.

Table 7-1 Materials Investigation in Some Caissons of Large-scale Bridges

Project name	Plane size	Total height (m)	Height of each segment	Material
North Anchorage of Nanjing Fourth Bridge	69 m × 58 m	52.8	The first segment is 6 m high; the second to tenth segments are 5 m high; the last segment is 1.8 m	The first segment is Q235B steel shell concrete, the other segments are reinforced concrete
Anchorage of Maanshan Bridge	60.2 m × 55.4 m	48	The first segment is 8 m high; the second to seventh segments are 5 m high; the eighth segment is 5.5 m high; the ninth segment is 4.5 m high	The first segment is Q235B steel shell concrete, the other segments are reinforced concrete

continued

Project name	Plane size	Total height (m)	Height of each segment	Material
Sunk Well of Taizhou Bridge Middle Tower	58 m×44 m	76	The first segment is 8 m high; the second to thirteenth segments are 6 m high; the fourteenth segment is 8 m high	The first to seventh segments are Q235B steel shell concrete, the other segments are reinforced concrete
Taizhou Bridge North Anchor Caisson	67.9 m×52 m	57	The first segment is 8 m high; the tenth segment is 4 m high; the remaining segments are 5 m high	The first segment is Q235B steel shell concrete, the other segments are reinforced concrete
Anchor caisson of Jiangyin Bridge	69.2 m×51.2 m	58	The first segment is 8 m high; the second to tenth segments are 5 m high	The first segment is Q235B steel shell concrete, the other segments are reinforced concrete
Anchor Caisson of the Cockatoo Chau Bridge	Outer diameter 66 m, Inner diameter 41.4 m	43	The first segment is 6 m high; the second to sixth segments are 5 m high; the seventh to eighth segments are 6 m high	The first segment is steel shell concrete, the other segments are reinforced concrete
No. 3 main pier of Tongling Yangtze River Bridge	62 m×38 m	68	The first segment is 9.5 m high; the second to fifth segments are 7.5 m high; the sixth segment is 10.5 m high; the seventh to tenth segments are 4 m high; the eleventh segment is 2 m high	The bottom 50 m is steel shell concrete, the top 18 m is reinforced concrete
No. 29 caisson, middle tower of Hutong Bridge	86.9 m×58.7 m	115	Standard segment height is 6 m; bottom segment height is 8 m	The bottom 50 m is Q235B steel shell concrete, the rest is reinforced concrete

7.1.3 A caisson is directly set in the natural ground to be pier or abutment foundation, so that its bearing depth needs to comply with the relevant regulations of shallow foundations.

7.1.4 As a structural component, the caisson during the construction and in service stage shall be checked to comply with the relevant requirements for structures.

7.2 Detailing Requirements

7.2.1 The plane shape of a caissons may be circular, round-end, rectangular, etc. In the riverbed where the great scour may occurred, a cross-sectional form with smaller water resistance is preferred to be used. When the bridge foundation is of round-end or rectangular shapes, appropriate ratio of the long side length to the short side length needs to be adopted in order to maintain the stable of the caisson during its sinking.

The edges and corners of a caisson are made rounded or obtuse in order to ensure the caisson uniformly stressed when it behave as a plane frame, and to reduce the skin friction area of the caisson wall.

7.2.2 Caissons are generally sunk in segments. Considering the rigidity requirements for large caissons, the regulation that the segment height should not be higher than 5 m has been canceled in the *Specifications*. The outer surface of caissons may be made as column, stepped, or cone.

7.2.3 The thickness of the caisson wall is closely related to the segment length to be sunk, the skin friction of soil against sinking, and the construction methods. According to the design examples of large caissons, the upper limit of the caisson wall thickness is appropriately increased from 0.8 ~ 1.5m in 2007 version *Specification* to 0.8 ~ 2.2m in the *Specifications*.

7.2.4 In order to facilitate the dredging, the cutting edge slope of the caisson could be made as steep as possible on the premise that the flexural and shearing strength meet the requirements. Therefore, it is stipulated that the angle between the slope and the horizontal plane shall not be less than 45°.

To stipulate that the bottom surface of the diaphragm is at least 0.5 m higher than the bottom surface of the cutting edge is to reduce the resistance during sinking.

If a caisson with high and low cutting edges is used to stand on a sloping rock surface, sufficient drilling data is necessary for the designers to accurately understand the elevation changes of the rock surface, to design the cutting edges in steps or slopes to be compatible with the slope of the rock, and to make the cutting edges socketed into the rockbed to facilitate dredging and bottom cleaning without sand blowing.

7.2.5 It is stipulated in Clause 9.1.12 of the *Specifications for Design of Highway Reinforced Concrete and Prestressed Concrete Bridges and Culverts* (JTG3362-2018) that the minimum reinforcement ratio of eccentric compression members is 0.5%, and the minimum reinforcement rate of flexural members is $(45f_{td}/f_{td})\%$ and not be less than 0.2%, which is consistent with the provisions in the Code *for Design of Highway Reinforced Concrete and Prestressed Concrete*

Bridges and Culverts (JTG D62-2004), but some higher than that in the earlier version of the specifications. The provisions on the caisson reinforcement ratio shall not be less than 0.1% in the previous specifications (2007 Version) is followed in the *Specifications*. In the bottom segment of a caisson (including the cutting edge), the stresses of the cutting edge are difficult to be computed accurately, so that its minimum reinforcement ratio should not be too small. For thin-walled caissons, to satisfy by the minimum value for limitation is not enough, some larger reinforcement ratio should be adopted. For steel shell concrete caisson, the minimum reinforcement ratio may be 0.05%, and the caisson may be calculated as a reinforced concrete structure or a steel-concrete composite structure.

7.2.6 According to the actual engineering application in recent years, the requirements for concrete strength class of the cutting edge and well of a caisson is properly improved in the *Specifications*.

7.3 Analysis

7.3.1 In calculating the caisson as an block foundation, the shape and size of the caisson may be preliminary determined according to the load, hydrogeological conditions and engineering characteristics of each soil layer, and then the corresponding resistance, eccentricity, stability against sliding and overturning may be checked to find if they meet the design requirements. In the calculations, the constraint effect of soil on the caisson wall may be considered after deducted the scouring effect.

7.3.2 The sinking of a caisson is achieved by continuously dredging out soils from the open wells of the caisson, and the skin friction along the outside of the caisson wall and bearing resistance on the cutting edge are overcome by the total weight of the caisson and the ballast minus the buoyancy. Therefore, it is necessary to determine whether the caisson has enough self-weight acting to force the caisson lowering smoothly at first in the design. After the soil under the cutting edge, diaphragm and bottom beam are excavated out totally, the caisson sinking is only restrained by the skin friction on the outside wall, and the sinking factor of the caisson is calculated by Eq. (7.3.2-1) under such a working condition.

Besides the sinking shall be ensured to be smoothly, the stability of the caisson shall also be ensured and strength failure of the soil at the bottom of the caisson shall be avoided during the sinking. Therefore, it is necessary to use the Eq. (7.3.2-3) to check the stability during the sinking.

7.3.5 Typhoons, waves, and ocean currents have strong coupling effects on bridges crossing over bays or straits. In checking the caisson in construction and service period, the impact of wind-wave-current coupling effects on the caisson structure need to be considered in the most unfavorable conditions.

7.3.6 As the caisson is used as the bridge foundation, the bottom seal concrete bears not only the water pressure under it, but also the upward ground-soil reaction force generated by all loads of the caisson foundation. In case dewatering and dry construction after bottom sealing is required, if the design strength of concrete has not reached due to the insufficient age in dewatering, it is necessary to using the degraded strength grade of the concrete in calculation, to adopt the actual strength of the sealing concrete in dewatering for the calculation. If the caisson is filled after the bottom seal with granular materials such as sand and stone, it is beneficial to improve the stress of the bottom seal concrete in the service stage. From economic considerations, it is feasible to deduct the gravity effect of the filler from the reaction force of the seal.

7.3.7 Tipulating the inclination angle of a floating caisson not greater than 6^0 is to ensure its stability during its floating, which can also avoid unsafety in construction. The provisions on the stability check on floating caisson in the *Specifications* are stipulated by referring to the current *Technical Specifications for Construction of Highway Bridges and Culverts* (JTG/T F50).

8 Underground Diaphragm Walls

8.1 General

8.1.5 The inspection requirements for the construction quality of the diaphragm walls presented in design mainly include check or inspection of the materials, the production of reinforcement cages, the preparation and pouring of concrete, the setting of embedded components, the flatness and verticality of the side of the trenches, the quality of the connector of the trenches, and the integrity of the concrete walls, etc.

The requirements for environmental inspection presented in the design of diaphragm walls are associated with their function. The main requirements are described as follows:

For diaphragm walls as retaining structures, they include the inspection and monitoring of the foundation pits, the retaining structures and the surrounding environment, as well as the measures to be taken when abnormal conditions occur;

For diaphragm walls as foundations in service stage, they include the deformation inspection of diaphragm wall foundations;

For diaphragm walls as anchorage foundations of important bridges, they also include the long-term deformation inspection and monitoring, in order to grasp in time the deformation characteristics of the diaphragm wall foundations in service stage.

The fieldloading test of walls is usually carried out when it is necessary to evaluate accurately the bearing capacity or deformation characteristics of the diaphragm wall as foundation.

8.2 Retaining Structures

8.2.1 The requirements for calculating the strength, stability, and deformation in the design of retaining structures of foundation pits mainly include:

(1) Strength: the strength of the retaining structures, including the walls, internalbracing system or anchor rods (anchor cables), shall meet the design requirements for the resistance of corresponding components.

(2) Stability: the soil body around the foundation pits and the retaining structures shall be

stability, that is, no sliding failure of the soil, no flow of sand, no flow of soil and piping caused by seepage, and no instability of the retaining structures and internalbracing system will occur.

(3) Deformation: ground deformation caused by stratum movement and by changes of groundwater level due to excavation of the foundation pits do not exceed the allowable deformation value of the buildings and underground facilities around the foundation pits, and does not affect the construction of the underground structures.

8.2.2 The supporting system of straight diaphragm walls as retaining structures includes internal bracings (such as struts, horizontal brackets) and earth anchors (cables), etc. The supporting system of circular diaphragm walls as retaining structures includes the internal ring beams (including vertical ribs), inner linings, etc.

8.2.3 The deformation of the foundation pits with safety level 1 or 2 will affect the normal usage function of the retaining structures of the foundation pits, but the specific value of the deformation limit cannot be given at present. It may be determined according to factors such as the surrounding environment of the project in the local area.

8.2.4 For the safety in construction of the foundation pits and the stability of the soil around the bottom of the pits, the diaphragm walls are required to be inserted into the soil below the excavation surface of the foundation pits to a certain depth (also called the bearing depth). Generally, the bearing depth is preliminary determined by the limit equilibrium method, then comprehensively determined by referring to local engineering judgement after the checking of the stability and wall deformation.

8.2.7 This Clause specifies the detailings. The commentary are as follows:

1 Many techniques can be used for trench forming of diaphragm walls, such as excavators and trench milling machines. According to the design experiences and considering the feasibility and rationality of implementation, it is stipulated in the *Specifications* that the wall thickness should not be thinner than 600 mm. The largest wall thickness is mainly restricted by the ability of the trench forming equipment. At present, the largest thickness of trench in China is 1500 mm, which has been used in the foundations of the diaphragm walls of Wuhan Yangluo Bridge and Fourth Nanjing Yangtze River Bridge.

2 The wall thickness adopted in the calculation is directly influenced by the trench verticality of the diaphragm walls, especially for the circular diaphragm wall. At the same time, the anti-seepage effect of the wall and the construction of the connectors is also affected by the

trench verticality. The trench verticality of the diaphragm walls is closely affected by the trench forming equipment, trench depth, process technology and management level. Under normal circumstances, it can reach to a level of no more than 1/100. The maximum depth of the diaphragm walls of the south anchorage in Wuhan Yangluo Bridge is 60 m. The verticality required in the design was less than 1/300. This requirement was met in the actual construction, and the verticality even reached to 1/450 ~ 1/500 in some trench panels. Therefore, according to the current technology level in China, it is realistic to stipulate that the trench verticality of the diaphragm walls shall not be greater than 1/200.

3 In order to increase the durability of the structure, the thickness of the concrete cover of the primary reinforcement is specified in the Specifications, in which the factors such as difficult in control of the construction accuracy of the diaphragm wall and direct contact of the wall concrete with soil in its casting are considered. For reinforcement cages with L-, T-, Y- and polygonal shape, the concrete cover thickness needs to be appropriately increased because the concentration of mud for wall protection is heavy. If there is corrosive water or seawater, the concrete cover thickness needs to be appropriately increased.

4 Design and construction of wall connectors are the key issues for diaphragm walls. According to the used materials, the types of connectors may be divided into: steel pipe, steel plate, steel bar, shape steel and cast steel, precast concrete, man-made fiber cloth and rubber, etc. According to the structure types and construction methods, they may be divided into: drilling type, connector pipe, connector box, partition type, flexible connector, precast concrete components, etc. According to the forcing cases, it may be divided into: non-stressed connectors that only as a stopper for water and seepage, hinge connectors that can withstand shear, and rigid connectors that can withstand bending moment and shear force. The choice of connector type needs to meet the requirements for structural stresses and construction. Fig. 8-1a) ~ h) show several common connector types. The technology of connector pipe is mature and may be used under normal circumstances.

5 The vertical division of the reinforcement cages mainly depends on the lifting capacity. Considering that the position of connectors may form a weak part in the structure, in order to ensure safety, the position of connectors is required to be set at the place with less stress, and the connectors should be staggered each other as possible.

8.2.10 For cohesive soil with weak permeability, the groundwater is not easy to form buoyancy force on its soil particles. Therefore, in calculating the retaining structures, the water pressure and earth pressure may be considered at the same time by using the saturated weight and the total stress intensity index, and the effect of water pressure is involved in the calculation results. However, if a water head is formed between the retaining structure and the surrounding soil layer, the effect of water pressure still needs to be considered independently. For silty soil, sand, and gravelly soil below the groundwater level, groundwater can form buoyancy forces on the soil particles due to their strong permeability, so that the water and earth pressure are calculated independently. The water pressure may be calculated according to the hydrostatic pressure, and the influence of seepage action on the water pressure may also be considered according to the experience.

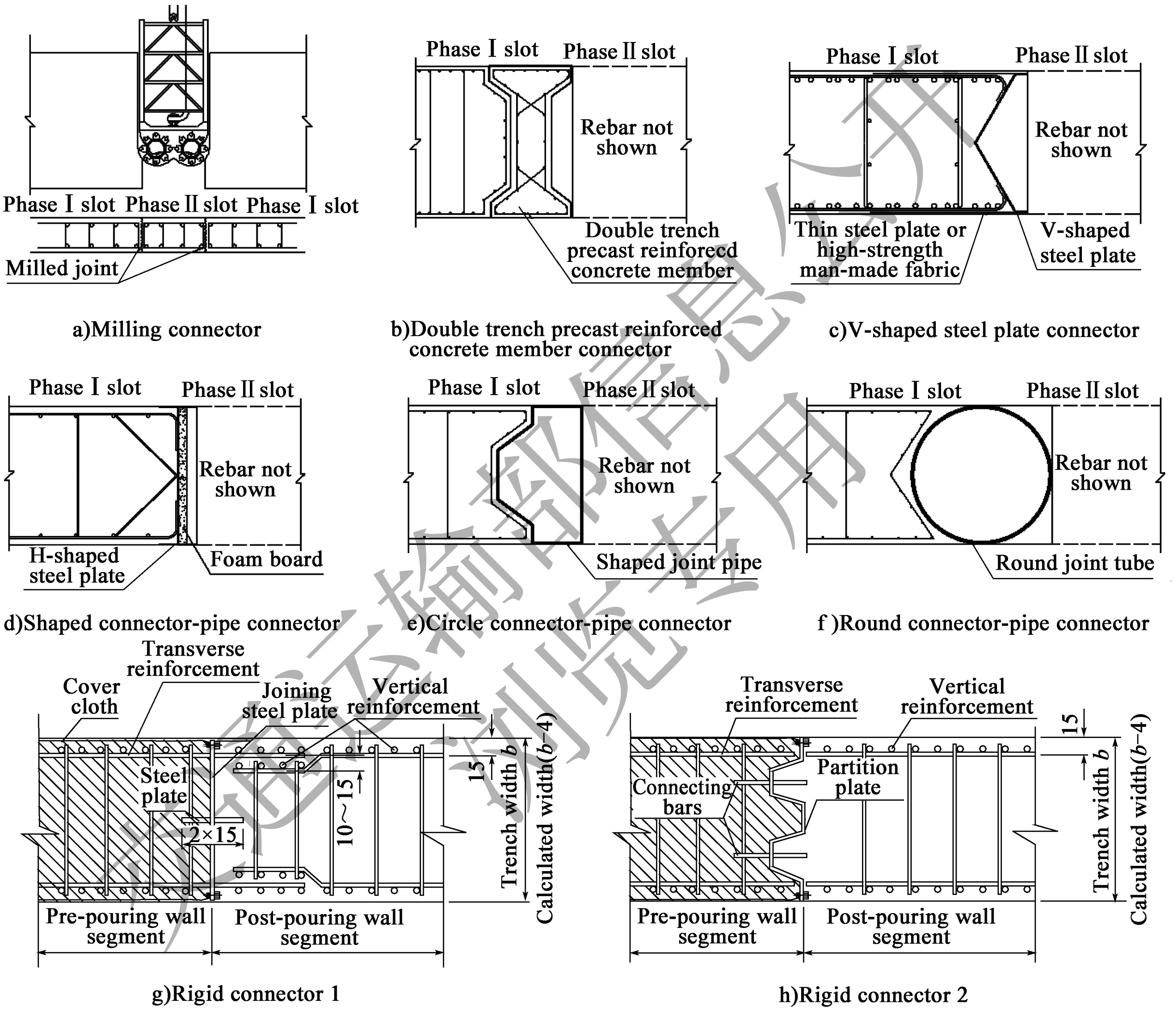


Fig. 8-1 Schematic Diagram of Several Types of Connectors (unit: cm)

(a) Milling connector; (b) Double trench precast reinforced concrete member connector; (c) V-shaped steel plate connector; (d) Shaped connector-pipe connector; (e) Circle connector-pipe connector; (f) Round connector-pipe connector; (g) Rigid connector 1 (h) Rigid connector 2.

8.2.12 This Clause mainly specifies the requirements for the calculation of straight diaphragm wall as retaining structure. Commentary of this Clause is as follows:

1 The stability of the retaining structure and the soil body include stability against overturning (stability for socket of diaphragm wall), overall stability against sliding, stability against basal heave, stability against groundwater seepage and surge, etc. These are the basic design concerns for foundation pit engineering and have mature calculation methods, so that they are not the main focus in the *Specifications*. The calculations may be implemented in accordance with the relevant provisions of the *Code for Design of Building foundation* (GB50007).

2 At present, the methods commonly used in the design of retaining structures in China may be divided into the elastic beam foundation method and limit equilibrium method. The influence of various working conditions and complex conditions on the stresses of the retaining structure during the construction of the foundation pit may be reflected better in the elastic beam foundation method. When the socket depth is reasonable and the stiffness of the elastic support is determined by test data or local experience, it is reasonable using this method to determine the internal forces and deformations of the retaining structures. Considering all the current calculation methods can obtain the reasonable results, the elastic beam foundation method is used to calculate the retaining structures in the *Specifications*.

8.2.13 The vertical axial force of a diaphragm wall is mainly caused by the self-weight of the wall and the internal bracing members, so that the wall is calculated as an eccentric compression member. But because the vertical axial force is generally small, the wall sometimes may be calculated as a flexural member conservatively. However, if the axial force is large, the wall should be calculated as an eccentric compression member.

8.2.14 A circular diaphragm wall as retaining structure is stressed differently from that of a straight diaphragm wall. A circular diaphragm has obvious spatial behaviors in the structural mechanism, so that it is calculated as a spatial structure. However, if the symmetricity of the structure, the hooping effects of the wall, the internal ring beam or the inner lining are accurately understood, and the uneven distribution and degree of water and earth pressure are also well known, the simple and intuitive calculation method may be adopted by computing a wall unit width as a vertical elastic beam foundation. This calculation method and its principle are similar to that for straight diaphragm wall as retaining structure, the only difference relies on that its calculation needs to consider the support stiffness of the ring effect for the wall, internal ring beam or inner lining.

The internal ring beams or inner linings may be calculated as rigid-frame ring beams in the plane. The nonuniform property of the load distribution has a great influence on the calculation results of the internal forces and deformations of the internal ring beams or the inner linings. This phenomena needs to be fully studied and accurately mastered. In the absence of data, the non-uniformity coefficients of loads may be taken as 1.1 ~ 1.2. For safety, the calculation is carried out according to the distribution along the diagonal quadrant. The restraining effect of the soil in the outer deformation area of the ring on the internal ring beam or the inner lining may be simulated by setting horizontal radial springs on the outer side.

8.3 Foundations

8.3.1 According to the connection type of the wall units, layout and functional use, diaphragm walls as foundation may be classified into the diaphragm wall as strip foundation, caisson foundation, and a part of other foundations.

1. Diaphragm wall as strip foundation

The diaphragm wall as foundation is composed of only one wall unit, or composed by more than one wall units separated or connected together but not closed. A unit has a plane length no less than 2.5 times of its width. Diaphragm walls as strip wall type foundations may be divided into the following sub-types:

(1) Single-wall type: one diaphragm wall unit works as a foundation [Fig. 8-2a)]. A diaphragm wall as single-wall type foundation can be seen as a special-shape cast-in-place pile (rectangular pile). Top plate may not be set.

(2) Parallel double-walls type: a diaphragm wall as foundation is formed by two or more separated diaphragm wall units in parallel, connected by the top plate [Fig. 8-2b)]. The stiffnesses along the bridge axis and perpendicular to bridge axis are quite different.

(3) Free double-walls type: a diaphragm wall as foundation is formed by two or more diaphragm wall units scattered in the plane and connected by the top plate [Fig. 8-2c)]. The units may be freely arranged according to the loading direction.

(4) Combined double-walls type: a diaphragm wall as foundation is formed by two or more diaphragm wall units connected by the top plate. This type diaphragm wall has various plane shapes, including T-, cross-, H-, I- and radial-shape [Fig. 8-2 d) ~ h)].

2. Diaphragm wall as caisson foundation

It is also called as diaphragm wall as closed foundation. It is a diaphragm wall as foundation composed of rigid connected multi wall segment units, or multi wall segment units rigid connected in outer wall while connector connected in inner wall. The foundation has closed section in plane and all walls are connected together by the top plate. The foundation may be divided into two sub-types, single-cell type and multi-cells type [Fig. 8-3(a), Fig. 8-3(b)].

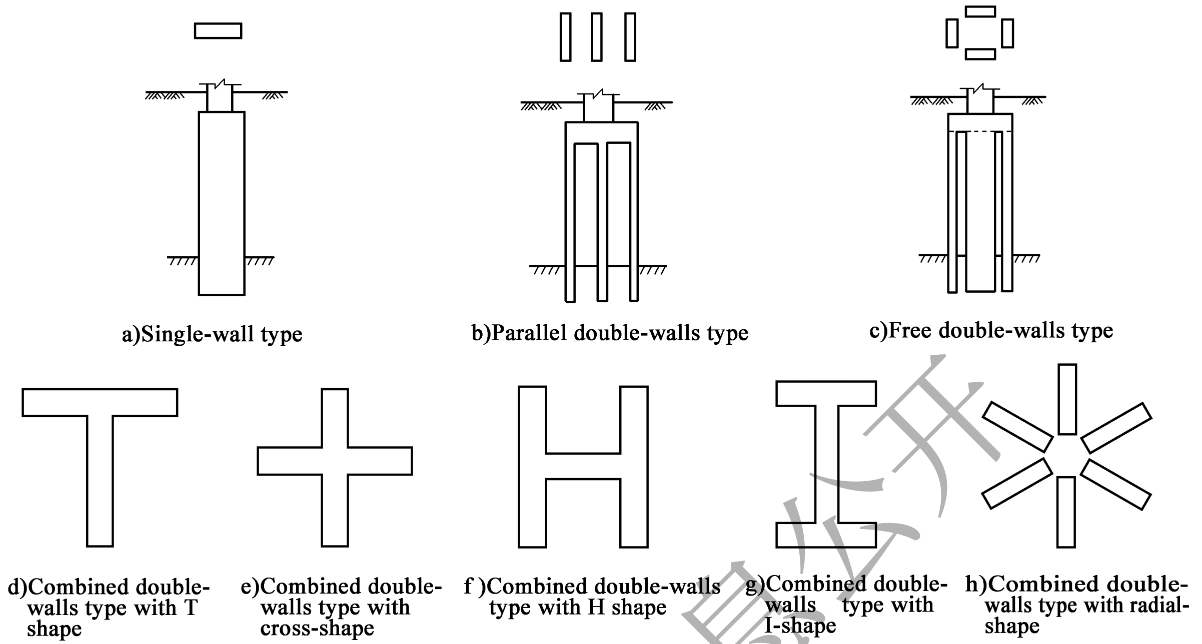


Fig. 8-2 Types of diaphragm walls as strip wall type foundations

a) Single-wall type; b) Parallel double-walls type; c) Free double-walls type; d) Combined double-walls type with T shape; e) Combined double-walls type with cross-shape; f) Combined double-walls type with H shape; g) Combined double-walls type with I-shape; h) Combined double-walls type with radial-shape

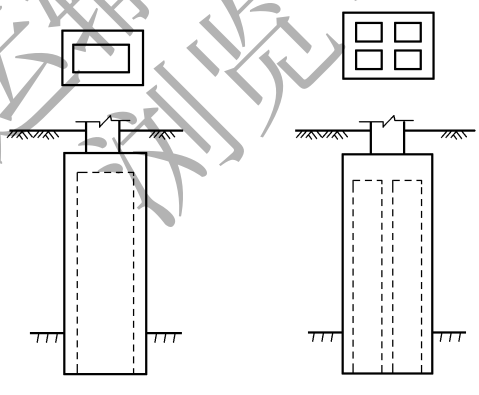
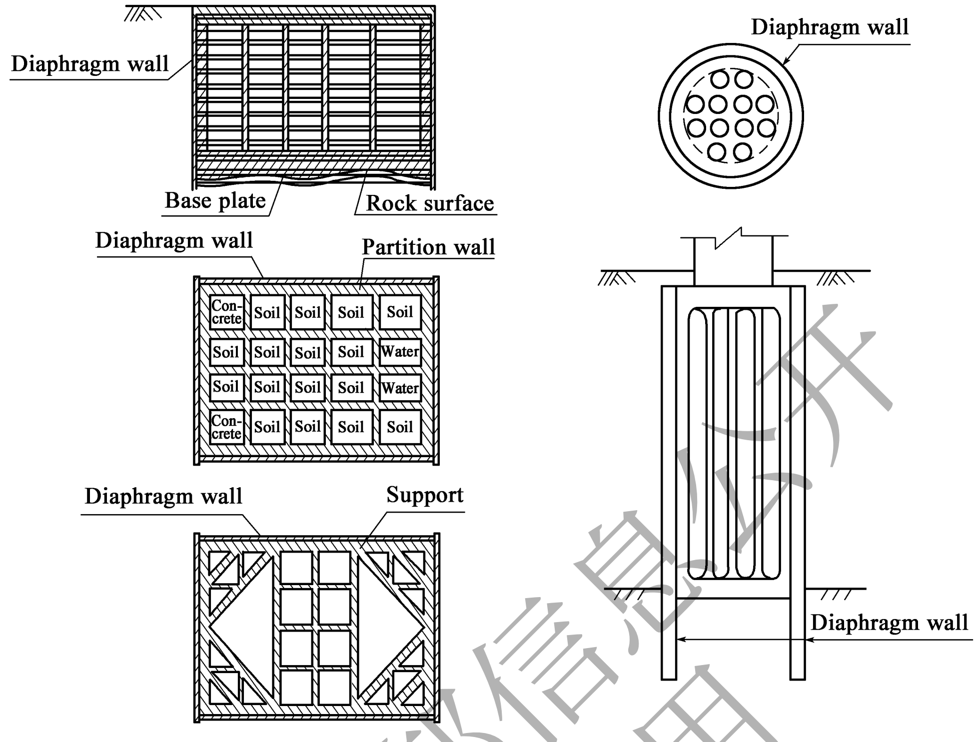


Fig. 8-3 Diaphragm Wall as Caisson Foundation

3 Diaphragm wall asa part of other foundations

A diaphragm wall is mainly used as the retaining structures during the excavation of the foundation pit, and is also served as a part of foundation of the bridge in service stage to participate in carrying the superstructure loads. The diaphragm wall is a reinforced concrete structure built inside the foundation pit after the soil inside the trench is excavated to the required depth. The plan layouts of the diaphragm walls have rectangular [Fig. 8-4(a)], circular [Fig. 8-4(b)] and hybrid special shape.



a)Rectangle　　b)Circular

Fig. 8-4　Diaphragm Wall as a Part of other Foundations

8.3.2　The vertical compressive resistance of a diaphragm wall as foundation is mainly composed of the frictional force of the side walls and the supporting resistance of the wall base. When the bearing stratum is a non-rock stratum, increase of the wall depth can significantly increase frictional force of the side walls and the supporting resistance of the wall base, which is more economic and easier in construction than increase of the wall size in plane. Therefore, it is stipulated in this Clause that increase of the bearing depth of the wall are more priority to increase the vertical compressive resistance.

8.3.3　The layouts of diaphragm wall as foundations have various forms and can be arranged flexibly in design. The layout of the trench segments in a diaphragm wall as caisson foundation may be arranged as one-cell section, two-cell section, or multi-cell section in plane.

8.3.4　The calculation of ground bearing capacity is an important issue in design of the structure of the diaphragm wall as foundation. The vertical ground bearing capacity of the diaphragm wall as strip foundation may be calculated with reference to that of the pile foundation. The calculation of the ground bearing capacity of the diaphragm wall as caisson foundation includes the vertical bearing capacity of the ground in the foundation base, the horizontal bearing capacity of the ground in front of the foundation, the horizontal shear bearing capacity of the ground at the side of the foundation, and the shear bearing capacity of the ground in the foundation base, etc. Its vertical bearing capacity includes the vertical foundation reaction force of the base foundation, the vertical frictional force of the side walls in the outer peripheral surface of the foundation, as well as the surrounding frictional force of the internal soil; the shear bearing capacity of of the ground in the foundation base includes the frictional force between the foundation body and the foundation soil and the frictional force among the internal soil.

8.3.7 This Clause is on the detailing requirements. Commentary of this Clause is as follows:

2 As an important load-carrying component, the foundation shall have a certain bearing capacity, so its minimum thickness is specified.

Considering the construction process and the influence of slurry, wall thickness may be divided into trench thickness, design thickness, and effective thickness. The trench thickness is the actual size of the trench formed by the excavator or the trench milling machine; the effective thickness is the design thickness minus the thickness of the slurry skin, the latter generally is 20 mm on each side, 40 mm in total. The design thickness is used in the stability calculation, and the effective thickness is used in the section check.

If the width of the single cell in a diaphragm wall as caisson foundation is too small, the construction will be difficult; while if the width is too large, it is not economic. It is stipulated that the minimum width shall not be less than 5 m and the maximum width shall not be greater than 10 m. The working-day cost of machines for trenching diaphragm walls is high. It is required to have the same thickness as possible for outer peripheral walls and diaphragms, in order to improve the efficiency of the trenching equipment, to reduce transformation steps in the construction process and to facilitate construction.

3 The top plate connects various wall panels into an integral diaphragm wall structure to bear the forces together, playing a similar function as cap beam for pile group. Therefore, top plate with sufficient rigidity is required for the diaphragm wall as foundation that is not a single wall type but is composed of multiple wall panels.

The diaphragm walls need to be built integral with the top plate, the walls need to be connected rigidly to the top plate, and their steel bars need to be extended into the top plate a certain length.

5 As anblock foundation, a diaphragm wall as caisson foundation shall have large overall rigidity. The outer peripheral wall supported by the inner diaphragm directly bears the external water and earth pressure, and is subjected to large bending moments and shear forces, so that rigid connectors are used for it. As the supports of the outer peripheral walls, the inner diaphragms mainly bear axial forces, therefore hinge connectors that cannot bear bending moment may be used in them, however, rigid connectors shall be used as much as

possible to increase the overall rigidity of the foundation.

8.3.8 A diaphragm wall as foundation is stressed complexly, and the soil-structure interaction shall be taken into account in its mechanical analysis. In designs, the structure may be taken as a spatial structure and calculated using reliable methods according to relevant data or experience.

9 Special Grounds and Foundations

9.1 Soft Grounds

9.1.1 ~ 9.1.2 When the bearing capacity of the soft ground in shallow foundation is insufficient or its settlement is greater than the allowable value, artificial reinforce treatment shall be adopted, and the treated ground is also called artificial ground.

Soft soil or soft ground generally refers to mud, muddy soil, hydraulically placed fills, plain fills, and miscellaneous fills, saturated soft clay, and other highly compressible soil layers with low shear strength, high natural moisture content, large natural void ratio, high compressibility, and low permeability.

To build structures on soft soil or ground, attention must be paid to the deformation and stability of the ground. The allowable bearing capacity of soft soil or ground under ordinary shallow foundation is about 60 ~ 80 kPa. It generally cannot meet the requirements for bearing capacity if it is not treated at all. There are many methods for ground treatment. The replacement layer of sandy gravel, sandy stone piles, and pre-pressed sand drain wells are commonly used in highway bridges. The treatment methods specified in the Specifications is based on those presented in the 2007 Version Specifications and adjusted according to recent developments. Other methods can refer to the current Technical Code for Ground Treatment of Buildings (JGJ 79-2012).

9.1.3 The materials of replacement layer of sandy gravel shall be obtained from local, and at the same time, shall comply with the requirements for strength.

9.1.4 The provisions in this Clause are derived from the relevant regulations of the *Technical Code for Ground Treatment of Buildings* (JGJ79-2012).

9.1.9 ~ 9.1.10 Preloading method is suitable for treating saturated clay ground composed of muddy soil, mud or hydraulically placed fills. It can be divided into two sub-methods: loading method and vacuum method. For loading method, it has the disadvantage that the preloading requires a large amount of stacking loads and long time for drainage and consolidation, therefore, before preloading, sand drain wells are often drilled into the ground, and then the ground is loaded, this is the so-called preloading method with sand drain wells. In this method, the sand drain well plays a function to shorten the drainage distance in the soft soil. The water in the soil is drained through the replacement layer of sand on the top of the sand drain well or through drainage ditches, thus the pore water pressure in the soft soil may be quickly released, and the consolidation process of the ground will be accelerated, and the strength of the ground can be improved quickly.

9.2 Collapsible Loess Grounds

Loess (collectively referred to as primary loess and secondary loess) is particularly developed in China. The engineering geology of the loess area has been paid great attention. Based on the relate provisions on treatment of loess grounds in the Section 4.6 of the 2007 Version Specifications, revisions are made in this chapter according to the relevant researches and practice achievements on loess ground in China in recent years.

The main references include:

1 Code for Design on Subsoil and Foundation of Railway Bridge and Culvert (TB10093-2017);

2 Practical Techniques for Design and Construction of Highway Foundation Treatment (Edited by Zhang Liujun and others, People's Communications Press, 2004);

3 Code for Building Construction in Collapsible Loess Regions (GB50025-2004);

4 Relevant specifications in building and construction industry.

9.2.1 ~ 9.2.5 Self-weight collapsible soil refers to the soil that will have collapse settlement when it is wetted by water and subjected to self-weight of the overlying soil; while non-collapsible soil under self-weight refers to the soil that will not have collapse settlement under the same conditions. The provisions in these Clauses are referenced to the relevant provisions in *Code for Building Construction in Collapsible Loess Regions* (GB50025-2004) and *Code for Design on Subsoil and Foundation of Railway Bridge and Culvert* (TB 10093-2017).

9.2.7 This Clause is compiled with reference to the Section 2 of Chapter 4 in the *Practical Techniques for Design and Construction of Highway Foundation Treatment*. The three methods, *i.e.*, replacement layer method, dynamic compaction method, and lime-soil compaction pile method, are mainly recommended in the *Specifications*, which are commonly used. In addition, vibro-flotation method (applicable to saturated loess) and high-pressure jet grouting method can also be used for treatment on the loess ground. Regarding the design and construction of loess ground treatment, in addition to the aforementioned relevant information for references, the current *Technical Code for Ground Treatment of Buildings* (JGJ79-2012) is also applicable.

Generally, the cost of the treatment is relatively high, and it needs to be selected through technical and economic comparisons; comprehensive measures with combination of improvement of the superstructure and foundation, treatment of the ground is favorable to be applied.

9.3 Steep Slope Grounds and Foundations

9.3.1 Referring to the definition in the road engineering, 1:2.5 is also taken as the threshold in definition of the deep slope ground and foundation for bridge and culvert in the *Specifications*. Such a definition is appropriate for bridge engineering, and can unify the definition of deep slope in road and bridge engineering, which is favorable for the field investigations.

9.3.2 For steep slope ground and foundation, not only the original steep slope shall be ensured to be stable, but also the steep slope after the foundation of the bridge or culvert is set in it shall be ensured to be stable. Therefore, it is necessary to analyze the stability and deformation of the slope subjected to the foundation loads. The consequences of instability of a slope where bridge and culvert located is more serious than that of a slope where road subgrade relied. Therefore, the safety factor of steep slope ground and foundation of bridge and culvert shall be higher than that of roadbed at the same highway. The values in Table 9.3.2 are the minimum safety factors specified in the *Specifications*. The actual safety factors may be larger than the values in Table 9.3.2, which may be determined by the designers based on the importance of the bridges and culverts and the local experiences.

The anti-sliding effect of pile foundations is generally not considered in the stability analysis of the steep slope. There are two reasons for this: on the one hand, once the steep slope slides, the landslide range may be much larger than the range that the pile foundation can retain, therefore, the resistance of the pile foundation resisting the landslide load is not considered in design unless it is a special design; on the other hand, the slope retaining structures and the bridge foundations are designed according to different reliability criteria, so that their designs are not mixed together except it is approved to be appropriate after an in-depth research.

If the slope where the bridge and culvert foundations located does not meet the safety requirements, the slope shall be reinforced by retaining methods, in order not to consider the influence of the sliding force in design of the pile foundation. In the case the displacement of the slope beside the pile cannot be avoided, an isolation method shall be used to separate the foundation side from the steep slope. Isolation may be achieved by placing a low-modulus flexible material between the side of the foundation and the steep slope.

9.3.3 To set a foundation in a steep slope area, the foundation will increase the loads on the slope, resulting a reduction of the slope stability. However, if the elevation of the foundation base is set below a certain depth of the slope surface, the adverse influence of the foundation loads on the slope stability may be ignored. The formula to calculate the minimum bearing depth of a foundation on steep slope is stipulated in this Clause. In the formula, the bearing depth H in soil ground corresponds to the depth when the vertical superimposed stress generated at the slope is less than $0.01p$, where p is the square load acting in the elastic semi-infinite space.

9.4 Karst Grounds and Foundations

9.4.1 For bridge foundations in karst regions, it is relatively simple to employ a design scheme of pile foundation. For example, the piles can be supported on stable bottom plate or integral bedrock to ensure the bearing capacity and deformation of the bridge foundations to meet the design requirements. Therefore, pile foundation schemes are widely used. However, in the case that the stable floor is buried deep, the piles need to pass through multi-layer karst caves to reach the stable floor, the pile foundation is difficulty for construction, and needs high cost and long construction period, thus the pile foundation scheme is not an optimal choice. The designers are encouraged to select an optimal foundation scheme by comprehensively considering the requirements for superstructure and substructure as well as the foundation conditions.

The difficulty of ground and foundation design for bridges and culverts in karst regions owes to the complexity of karst geological and hydrogeological conditions. The development and distribution of karst in a large area may have certain rules, but in terms of the site scope of a specific bridge and culvert, it is difficult to reveal the development and distribution of karst and the regularity of ground hydrological characteristics with limited survey work. Therefore, the design of foundations in karst regions is a work that needs experiences, understanding of local conditions and the characteristics of the specified case. The basic procedures and principles that shall be followed in the design of foundations of bridges and culverts in karst regions are stipulated in this Clause, namely:

1) Foundation is designed after evaluation of the stability of the karst cave roof;

2) Principles of dynamic design in the full process of the design work;

3) If foundation treatment is required, it is necessary to minimize the disturbance to the surface water and groundwater channels.

9.4.2 There are two methods for evaluation of karst cave roof: quantitative method and qualitative method. With regard of the quantitative evaluation of the stability of the karst cave roof, though there are some literatures can be taken as references, however, it is still in the exploratory stage. Because many factors can affect the karst stability. It is generally difficult to identify karst characteristics with current exploration methods. Thus, the evaluation of karst stability in present is still based on qualitative analysis and experiences. But no matter what evaluation method is used, qualitative or quantitative, stability shall be evaluated because karst collapse has a high impact on bridges and culverts. If the evaluation result indicates that the karst is not stable, the karst must be treated.

Filling the karst cave is a simple but rough treatment method. For a karst cave with the function of water passing, if it is filled, the surface water and groundwater cannot be drained normally, which will cause other problems. Therefore, in the treatment design of karst ground, not only the bearing capacity but also hydrological requirements must be considered.

9.4.3 It is a good choice to adopt shallow foundations for small bridges and culverts in karst areas where the karst is developed mainly in vertical direction. This method has significant economic and social benefits, and the construction is not difficult, compared with the pile foundation. Even if the ground bearing capacity of the spread foundation (rigid foundation) in the karst region is satisfied, in order to reduce the stress of the foundation and improve the stability when the karst may deform unevenly, the foundation shall be reinforced concrete monolithic slab footing.

9.4.4 When the pile foundation of a bridge in a karst region needs to be set in a certain depth of the karst cave roof, it is generally considered that the depth of the cave roof under the pile tip shall not be less than 3 times of pile diameter. If multiple piles are built on the same cave roof or the cave span is very large, the depth of the cave roof needs to be increased. The depth of the cave roof under the pile tip stipulated in this Clause is only the minimum requirement.

When the foundation piles are built on the cave roof, care shall be taken to protect the integrity of the cave roof under the pile tip. The rock-socketed depth of the piles shall be minimized on the premise of meeting the load requirements and minimum rock-socketed depth.

For bridges in karst region, multiple piles under the same pier or abutment may have different lengths, and the depths of the cave roof under the pile tips may also be different, and even the depths will be zero or negative for some piles. All these problems may lead to uneven load distribution among the piles and difference settlement at the pile tips. If these differences are not considered in the design, serious problems may be resulted in the engineering. Therefore, attention need to be paid for these problems in design of foundations in karst regions. To appropriately increase the resistance and rigidity of the pile cap or to adopt other structuralstiffness measures are all alternative solutions.

9.5 Foundation of Piles with Expanded Branches and Plates

9.5.1 Pile with expanded branches and plates are a new type pile invented in the 1990s. It has been widely used in industrial and civil construction, municipal administration and other fields, covering a variety of geological conditions, structures with various loading and deformation conditions in more than 20 provinces and cities in China, and the technology has become mature.

The provisions on the piles with expanded branches and plates in this section is formulated in conjunction with the provisions in the industrial standard of the Ministry of Transport *Pile with expanded branches and plates for Highway Bridge* (JT/T855- 2013) and the local standard in Zhejiang Province *Technical Specification for Pile with Expanded Branches and plates of Highway Bridge and Culverts Engineering*(DB33/T750-2009) as well as the summary of the development of the related technologies in recent years.

9.5.2 The provisions on the soil layers suitable for branches and plates in the *Specifications* are mainly referred to the relevant regulations of the industrial standard *Pile with expanded branches and plates for Highway Bridge*(JT/T855-2013). After this industrial standard was issued by China Ministry of Transport in 2013, the technology and construction methods, the extrusion capacity of equipment, and quality inspection methods of the piles with expanded branches and plates have been improved for perfection. The application scope of the branches and plates used in soil layers is expanded appropriately in the *Specifications*. Thick collapsible loess is widely distributed in western plateau in China, branches and plates may be set at the soil layer with a good bearing capacity and in a depth not influence by water, and the collapsibility of the soil may be improved through the expansion and compaction process.

The branch length and plate ring width are generally related to the characteristics of the soil layer, equipment capacity, and construction technology, a rough range on them is specified in the *Specifications* by referring to the *Pile with expanded branches and plates for Highway Bridge*(JT/T855-2013). In application of pile with expanded branches and plates in highway engineering in recent years, larger ring plate width and longer branch length can be designed owing to the improvement of the equipment and technology for expanding and compacting the branches and plates, which further enhances the bearing capacity of the piles and saves raw materials and investment.

According to the test results of the bearing performance of the branches and plates constructed by the expanding and compacting process, the reserved positions and numbers of the standby branches and plates need to be marked in design of the pile with expanded branches and plates, so as to achieve the purpose of regulating the pile resistance and controlling the pile rigidity.

9.5.3 The stress rationality among the plates in transverse direction and the construction reasonability of the spacing among the plates are considered in the provisions for the center-to-center distance between piles specified in this Clause. The plates are generally set in the pile body with small diameter for a variable diameter pile with branches and plates, which can better solve the problem of lateral plate spacing. At the same time, such an arrangement can provide an increase in the effective end bearing area of the plate ring.

9.5.4 The relevant formulas in this Clause follow the principles of the formula for calculating the resistance of pile foundations in the *Specifications*, and the calculations of end-bearing resistance at the end of the branches and plates is added, while the enhancing effect from compaction and expansion is not considered and is taken as a reserve.

Since various parameter information of the soil layers may be obtained in the process of expanding and compacting the branches and plates, Eqs. (9-1) and (9-2) are proposed after many years of accumulation and analysis. These two equations can be used in checking the characteristic values of the compressive resistance, and have been widely used in the industrial and civil construction industry, where the diameters of the pile and plate are smaller than those used in highway bridges. Therefore, when the piles are applied in a new region and new bridge type, it is recommended to carry out the staticloading test and internal force test for the branches and plates, in order to further optimize the parameters and to provide supplement and improvement of the design parameters for these two equations.

$$R_a = R/K \tag{9-1}$$

$$R = u\sum q_{ik} l_i + \sum A_{pj} q_{pkj} + A_p q_{pk} \tag{9-2}$$

where:

R——vertical ultimate bearing resistance of a pile;

K——safety factor, which may be 2 ~ 2.5;

q_{pkj}——nominal value of ultimate bearing resistance of the soil supported by the j^{th} branch and plate on the pile body (kPa). This value is taken from the actual measured bearing resistance of the soil at the end of the branch and plate. If actual measured value is unavailable, the nominal value of the ultimate bearing resistance proposed by the geological survey data is taken. If it is not provided in the geological survey, the value can be taken by referring to Table 9-1;

q_{pk}——ultimate value of pile tip resistance (kPa), refers to Table 9-1.

Table 9-1 Characteristic value of Ultimate Bearing Resistance of Soil at the Pile Tip, q_{pk} (kPa)

Soil name	First index	Second index	Third index	q_{pk}	
	State of soil	q_c	N	$5 < H \leqslant 20$	$H > 20$
Cohesive soil	$0.75 < I_L \leqslant 1.00$ $0.5 < I_L \leqslant 0.75$ $0.25 < I_L \leqslant 0.50$ $0 < I_L \leqslant 0.25$	1300 ~ 1800 1600 ~ 2500 2000 ~ 4500 4500 ~ 8000	$3 < N \leqslant 10$ $8 < N \leqslant 20$ $15 < N \leqslant 30$ $N > 30$	192 ~ 288 288 ~ 484 484 ~ 700 700 ~ 960	288 ~ 484 484 ~ 700 700 ~ 960 960 ~ 1650
Silty soil	$0.95 < e_0 \leqslant 1.05$ $0.85 < e_0 \leqslant 0.95$ $0.75 < e_0 \leqslant 0.85$	1300 ~ 2000 2000 ~ 5000 5000 ~ 8000	$5 < N \leqslant 12$ $10 < N \leqslant 35$ $N > 35$	192 ~ 288 484 ~ 600 580 ~ 960	288 ~ 484 600 ~ 960 960 ~ 1800
Silty sand, fine sand, medium sand, coarse sand	Slightly dense Medium dense Dense	3000 ~ 6000 6000 ~ 12000 > 12000	$10 < N \leqslant 25$ $20 < N \leqslant 50$ $N > 50$	480 ~ 720 960 ~ 1200 1160 ~ 1980	720 ~ 960 1100 ~ 1780 1980 ~ 2800

continued

Soil name	First index	Second index	Third index	q_{pk}	
	State of soil	q_c	N	$5 < H \leqslant 20$	$H > 20$
Round (breccia) egg (crushed) stone	Slightly dense Medium dense Dense	—	$10 < N_{63.5} \leqslant 25$ $20 < N_{63.5} \leqslant 50$ $N_{63.5} > 50$	1040 ~ 2260 2060 ~ 2800 2560 ~ 3500	
Completely-weathered, highly-weathered	Weak rock	—	$15 < N_{63.5} \leqslant 50$ $N_{63.5} > 50$	960 ~ 2220 2220 ~ 3500	
Moderately weathered, slightly weathered	Strong rock	—	—	2600 ~ 3650	

9.5.5 During the expansion and compaction process, various parameters of the soil layers may be obtained through indicators such as the compaction pressure and the equipment uplift value, so that the bearing performance of the branches and plates are inspected, which not only verifies the resistance of the pile foundations, but also verifies the geological survey data. If the inspection results show that the bearing capacity of the pile foundations cannot meet the design requirements, the number of branches and plates may be increased by using the reserved positions for the standby branches and plates, or the preset branches are changed into the plates in order to ensure the bearing resistance meeting the design requirements. The preset branches are the branches among the designed multiple branches that can be changed into plates.

9.5.6 Piles with expanded branches and plates were mostly used in underground anti-floating structures in former, such as basements, subway stations, and municipal water treatment pools. The formula in this Clause is determined based on the empirical formula in the industrial and civil construction industry, and comes from the local standard in Zhejiang Province *Technical Specification for Pile with Expanded Branches and plates of Highway Bridge and Culverts Engineering* (DB33/T750-2009).

9.5.7 The shallow geological layers often have poor geological conditions and cannot provide horizontal load-bearing capacities, therefore the horizontal forces of the branches and plates are not considered generally. When it is necessary to provide a large horizontal resistance to meet the requirements for seismic loading, a larger bearing branches may be set on the upper part of the pile body if the pile body itself cannot provide enough horizontal resistance to meet the design requirements, while there is a stiff ground in the shallow part of the soil layer. Generally the cross branches, six-star branches, and eight-star branches can be used, in which the horizontal load-bearing capacity can be determined through tests.

Appendix B
Essentials for Shallow Plate loading test

B. 0. 1 The 2007 Version *Specification* only specified the minimum area of the load plate for shallow plate loading tests on natural ground and soft soil ground. Due to the increasing applications of ground treatments in highway bridge and culvert engineering, it is necessary to standardize the size of the load plates for the ground loading test after treatment, so as to avoid using a too small load plate to obtain inaccurate results. For ground treated with composite ground, when determining the bearing capacity of the composite ground, the area of the load plate needs to cover the area reinforced by a pile.

B. 0. 7 In 2007 Version *Specifications*, no provisions was specified to deal with the case when the range of the measured values from the three testing points in the same soil layer exceeds 30% of the average measured value. It was found in practical engineering that when the range exceeds the requirement, sometimes the testing point number was simply increased and then the new and old tested data were merged together for analysis. Such a simply treatment can not conform to the particularity of the foundation engineering. In this Clause, it is stipulated that if the range of the measured values from the test points cannot meet the requirements, the cause shall be analyzed from a geological point of view in addition to find out the causes in the test method, operation, etc. If the soil layers in the testing points are different, the statistical units for the ground need to be redivided for evaluation.

Appendix H
Check for Stability against Frozen Uplift for Frozen Soil Ground

For the pier and abutment foundations (including pile foundations) setting in the regions ofseasonally frozen soil and permafrost soil, as shown in Fig. H. 0. 2 of the *Specifications*, there are upward tangential frost heave T, downward skin friction Q_{sk}, and downward freezing force Q_{pk} in the soil layers below the riverbed. The bearing depth of the foundations needs to meet the requirements for stability against frost heave (pulling) based on the stress condition. According to the investigations in Heilongjiang Province, many small bridges were damaged by frost heave, especially those hollow slab bridges with pile foundations. In a small bridge, the self-weight of the superstructure is light and the bearing depth of the foundation is shallow, the uplift force of frost heave is greater than the self-weight of the structure. In order to overcome the damage induced by the frost heave, one method is to increase the self-weight of the superstructure, and the other method is to increase the bearing depth of the foundation. But for a small bridge, it is difficult to increase self-weight of the superstructure. Therefore, properly increase of the foundation bearing depth is usually a preferable solution. In such a solution, the force equilibrium should be considered, the tangential frost heave and the tensile strength at the weak section of the foundations should be checked.

1 Tangential frost heave acting on shallow or pile foundations of pier and abutment

The nominal value of tangential frost heave inseasonally frozen soil listed in Appendix Table H. 0. 1, t_{sk}, is obtained by test and analysis results. In Anqing Freezing Science Test Site of Heilongjiang Transportation Research Institute, a total of 28 groups of tests in real situation on tangential frost heave of piles at the frost heave rates of 6% ~28% were carried out under different frost heave conditions, in which the piles with *d* 250 mm(*d* is the pile diameter) had 5 groups, with *d* 370 mm had 3 groups, with *d* 500 mm had 2 groups, with *d* 750 mm had 2 groups, with *d* 800 mm had 13 groups, with *d* 1000 mm had 2 groups, and with *d* 1250 mm had 1 group. Moreover, indoor model tests with three proportions were conducted, dozens of piles subjected to frozen uplifts were analyzed. Based the mass data from tests and analyses, five regression methods (straight line, logarithmic curve, power function curve, exponential curve, and hyperbola) were used for data analysis, and three check methods (correlation coefficient, residual sum of squares, and correlation index) were used to validate the equation. Finally, the best logarithmic equation was selected as follow:

$$\tau_{sk} = 63.45 \ln k_d - 2.38 \tag{H-1}$$

where:

τ_{sk}——nominal value of unit tangential frost heave (kPa);

k_d——frost heave rate of the ground soil (%).

The relationships between the pile diameter and the unit tangential frost heave are shown in Table H-1.

Table H-1 Relationship Between Pile Diameter and Unit Tangential Frost Heave

Pile diameter (mm)	500	750	1000	1250
Tangential frost heave (kPa)	60	58	56	58
The force ratio, taking the force when the pile diameter is 500 mm as the basis	1.00	0.97	0.93	1.00

It is shown by Table H-1 that the pile diameter has little effect on the tangential frost heave, so that the modification of the pile diameter on the tangential frost heave may not be considered in design calculation.

2 Stability forces against frozen uplift

The stability against frozen uplift of pier or abutment foundations (including strip foundation) on the ground ofseasonally frozen soil is calculated according to the Formula (H.0.1-1). Stability forces in resisting frozen uplift include the self-weight of the structure on the foundation, F_k, the self-weight of the foundation and the soil over it, G_k, and the skin friction of the thawing layer, Q_s. The stability against frozen uplift of pier or abutment foundations (including strip foundation) on permafrost ground is calculated according to the Formula (H.0.2-1). In addition to the above-mentioned F_k, G_k and Q_s, the frozen force of permafrost, Q_p, is also involved in the stability forces in resisting frozen uplift.

The stability against frozen uplift of piles (columns) on permafrost ground is calculated according to the Formula (H.0.3-1). Stability forces in resisting frozen uplift include the self-weight of structure on the top of the piles (columns), F_r, the self-weight of the piles (columns), G_k, the sum of the (columns) skin friction of the piles in all soil layers below the seasonal frost depth, Q_f. The nominal value of skin friction may be obtained from Tables 6.3.3-1 and 6.3.5-1 in the Specifications.

3 Unit tangential frost heave on strip foundations

Strip foundations have a relatively large length-to-width ratio (the length-to-width ratio is equal to or greater than 10). In former design, due to the lack of researches, tangential frost heave on strip foundations was generally substituted by that on the pile foundations. In fact, based on the field test observation and the theoretical analysis, the tangential frost heave on strip foundations is smaller than that on pile foundations under the same conditions.

If a length of D/2 is taken from the strip foundations (as shown in Fig. H-1), the length of the sidelength contact with the frozen soil is $2 \times D/2 = D$. Assuming that the diameter of a pile is d, the perimeter of the pile is πd. Let $\pi d = D$, that is, the perimeter of the pile is equal to the length of the two sides of the strip foundations. Assuming the frost depth is h, the constraint ranges of the strip foundations and the pile foundations to the frozen soil are equal, both are l. Then, the soil volumes V_1 and V_2 involved in frost heave in the design frost depth range are shown in Fig. H-1 and Eq. (H-2) and (H-3).

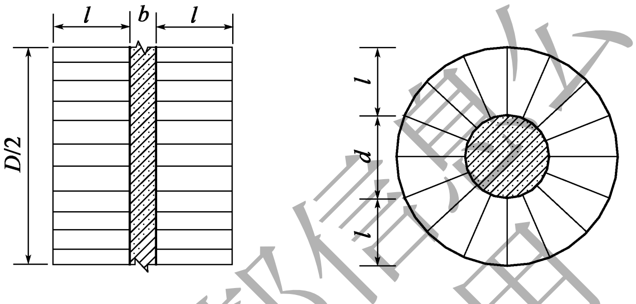


Fig. H-1 Plane Diagram of Tangential Frost Heave on Pile and Strip Foundations (frost depth h is not shown)
b——width of the strip foundation; $D/2$——cut-off length of the strip foundations; d——pile diameter, which is taken as $d = D/\pi$; l——restrained range of the strip or pile foundations on the frozen soil

Strip foundations:

$$V_1 = h \times 2l \times \frac{D}{2} = hlD \tag{H-2}$$

Pile foundations:

$$\begin{aligned} V_2 &= h\pi(2l + d)^2 \times \frac{1}{4} - h\pi d^2 \times \frac{1}{4} \\ &= h\pi(4l^2 + 2 \times 2ld + d^2) \times \frac{1}{4} - h\pi d^2 \times \frac{1}{4} \\ &= h\pi l(1 + d) = h\pi l^2 + h\pi ld \end{aligned} \tag{H-3}$$

Because $D = \pi d, V_2 = h\pi l^2 + hlD$

From the above Eq. (H-2) and (H-3), it can be seen that the volume of soil involved in frost heave in the pile foundations, V_2, has an additional term of πhl^2, compared with the volume in the strip foundations. Field tests have also shown that the frost heave range l affected by the pile foundations not only exceed the frost depth h but also is more than twice the pile diameter d, indicating that the tangential frost heave on pile foundations is larger than twice of the force in the strip foundations.

4 Nominal value of the pile (column) self-weight, Gk, and the skin friction of the piles (columns) in the thawing layer below the maximum seasonal frost depth line, Qs, in Eq. (H.0.3-1) are explained as follows:

The nominal values of the skin friction of the soil around the bored piles, q_{ik}, shown in Table 6.3.3-1 were measured under the test loads, in which the influence of pile self-weight on the friction was not included. In other words, before the test load was applied, the self-weight of the pile itself had caused resistance in the foundations. When the poured concrete was still in a fluid state, its self-weight was mainly resisted by the pile tip; when the concrete was hardened, the pile self-weight was mainly resisted by the pile skin friction. Therefore, the pile self-weight may be considered in checking the uplift of the pile; when the pile tip is below the water level and the soil under the pile tip is permeable, the buoyancy needs to be considered in calculating the pile self-weight.

The skin friction of the friction piles (columns) below the maximum seasonal frost depth line, Q_f, is the stability force in resisting frost heave, which is multiplied by a coefficient of 0.4 in Eq. (H.0.3-2). Taking real engineering projects as prototypes, various coefficients of 0.35, 0.40, and 0.50 were analyzed in comparison of the actual and computed bearing depths by the Heilongjiang Research Institute of Transportation. The results showed that the percentages of piles, not be pulled out by frozen uplifts, are 61%, 98% and 100% for the coefficients of 0.50, 0.40 and 0.35 in the analysis, respectively. Coefficient 0.40 was finally adopted.

5 To prevent or reduce the tangential frost heave, the following measures may be adopted:

1) Using coarse sand, gravel (pebbles), and other non-frost heaving materials to replace the frost heaving soil around the foundation. The replacement range is 0.5 ~ 1.0 m, and the replacement depth may be: 75% of the design frost depth for frost heave and strong frost heave ground, 90% for extra strong frost heave ground, 100% for extremely strong frost heave ground.

2) Making flat and smooth for the structural surfaces within the frozen layers, including the pier or abutment stems and the side of the foundations.

3) Coating asphalt, industrial Vaseline, or residual oil on the foundation sides within the frozen layers.

4) Making the foundation shape as regular trapezoidal block with inclined planes, in which the inclined plane slope (vertical to horizontal) is equal to or greater than 1:7 (see the 3rd Item of Clause 5.1.4 in Code for Design of Soil and Foundations of Building in Frozen Soil Regions (JGJ 118-2011).

Appendix L
Calculations of Horizontal Displacement and Action Effect for Elastic Piles by Using the *m Method*

Appendix L was rewritten by following the provisions of the 2007 Version *Specifications*. In rewriting this appendix, the *m method* for piles of $\alpha h > 2.5$ (elastic foundation) in Appendix 6 of the *Specification for Design of Foundations and Foundations of Highway Bridges and Culverts* (JTJ 024-85) was taken as the reference; the transformed *m method* for two-layer grounds and the modification method for the maximum bending moment of the pile body were improved, other contents remained unchanged with great simplification in the expression to make them be understood more clarity.

L.0.2 Transformation of equivalent *m* for multi-layer grounds:

(1) Accurate calculation method for displacement and internal force of piles in multi-layer grounds subjected to laterally load

As shown in Fig. L-1, the coefficients of ground resistance at the pile side increase proportionally with the depth. The proportionality factor of the first layer soil is m_1, the soil layer depth is h and the deformation factor of the corresponding pile is α_1; the proportionality factor of the second layer soil is m_2, the soil layer depth is h_2 and the deformation factor of the piles is α_2. If the pile diameter is constant, the deformation factor of the piles in different soil layers is:

$$\alpha = \sqrt[5]{\frac{m_1 b_1}{EI}} \tag{L-7}$$

where:

the subscript i —— the i^{th} layer of soil;

b_1 ——the effective width of the pile;

EI ——the flexural stiffness of the pile.

Then, the internal force and displacement of the piles in each soil layer are:

$$\left.\begin{aligned}
x_{iz} &= \alpha_{i0}A_1 + \alpha_{i1}B_1 + \alpha_{i2}C_1 + \alpha_{i3}D_1 \\
\frac{\varphi_{iz}}{\alpha_i} &= \alpha_{i0}A_2 + \alpha_{il}B_2 + \alpha_{i2}C_2 + \alpha_{i3}D_2 \\
\frac{M_{iz}}{\alpha_i^2 EI} &= \alpha_{i0}A_3 + \alpha_{i1}B_3 + \alpha_{i2}C_3 + \alpha_{i3}D_3 \\
\frac{Q_{iz}}{\alpha_i^3 EI} &= \alpha_{i0}A_4 + \alpha_{i1}B_4 + \alpha_{i2}C_4 + \alpha_{i3}D_4
\end{aligned}\right\} \tag{L-8}$$

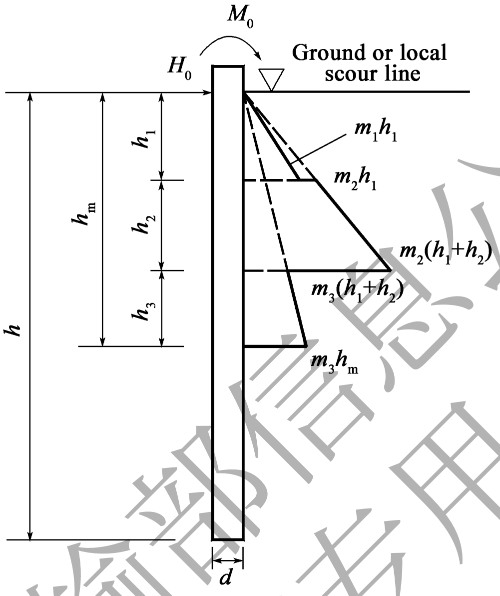


Fig. L-1 Schematic Diagram of Multi-layer Grounds

where:

$A_1 \sim D_4$ are dimensionless coefficients, and $a_{i0} \sim a_{i3}$ are the coefficients to be solved, which may be determined according to boundary conditions and continuous conditions. For a two layers ground, a linear equation group with 8 equations may be obtained, and the internal forces and displacements of any point of the pile may be obtained by simultaneous solution.

(2) The transformed method of m value and the calculation method of the maximum bending moment of the piles in the Specifications

In this *Specifications*, the equivalent *m* value is obtained from the weighted value of the upper and lower layers of soil according to the pile displacement deflection curve; then the pile in multi-layer is calculated as a pile in single layer by the calculation method and tables in the *Specifications*. Considering the influence of pile displacement to convert the *m* value is a more scientific method, the results obtained by the method specified in the *Specifications* as mentioned before shall be more accurate (see paper *Research about Simplified Calculation Method of Laterally-Loaded Piles in Double-layered Foundation* in 2006(12) of *Journal of Highway and Transportation Research and Development*).

L.0.6, L.0.7 (1) Tables L.0.6 and L.0.7 are applicable for calculations of the action effect and displacement in high capped vertical piles arranged symmetrically. Table L. 0. 6 is applicable for piles without lateral earth pressure, while Table L. 0. 7 is applicable for piles with lateral earth pressure. In Table L. 0. 7, only the piles in the outer row are subjected to lateral earth pressure, while other row piles are sheltered by the outer row piles, so that the lateral earth pressure is not considered to act on them; but if the piles are arranged in a quincuncial form, lateral earth pressure shall be considered for those piles that are not sheltered by the front piles (Fig. L. 0. 7).

(2) The action effect and displacement of high capped vertical piles arranged asymmetrically may be calculated according to the descriptions in Tables L. 0. 6 and L0. 7 and to the provisions in Sub-item 1 and Sub-item 2 in Item 3 of Clause L. 0. 7, respectively. If no lateral earth pressure acts on the piles, they may be calculated according to Eq. (L. 0. 7-1). If the piles are subjected to lateral earth pressure, they may be calculated according to Eq. (L. 0. 7-2).

(3) For a low capped pile foundation, i. e., the ground or the lowest scour line is above the pile cap bottom and the cap is buried in the soil, the soil around the cap can be regarded as an elastic medium. The action effect and displacement of the foundation are calculated according to provisions of Sub-item 3 in Item 3 of Clause L. 0. 7. Here, since the pile cap is buried in the soil, no lateral earth pressure is considered to act on the pile.

Appendix P
Calculations of Caisson Wall during Caisson Sinking Process

Appendix P was rewritten from the provisions and its corresponding commentary of Clause 6.3.2 in the 2007 Version Specifications, mainly the calculations of the caisson wall during the sinking process, where the main provisions and calculation methods were remained as before, while the text was adjusted accordingly in order to make the expressions clearer.

P.0.1 Due to the uncertainty of the supporting conditions, the most unfavorable supporting conditions that may be caused by different construction techniques is needed to be considered in the check of the caisson wall in the bottom segment.

1 When a caisson is lowered by excavation in the dry, the supporting positions of the caisson may be controlled in the most favorable range for structural stresses. For round-end or rectangular caisson, when the long side is greater than 1.5 times the short side, the supporting point may be set on the long side, and the distance between the two supporting points is equal to 0.7 times the side length (see Fig. P0.1-1). The absolute value of the negative bending moment generated at the support is approximately equal to the one of the positive bending moment generated at the midpoint of the long side, and the tensile strength of the concrete at the top or bottom of the caisson wall caused by the self-weight of the caisson is checked according to this condition.

2 When a caisson is lowered by excavation in water, because the supporting positions cannot be controlled, the bottom segment of the caisson may be checked as a beam under the unfavorable supporting conditions as assumed as follows:

(1) By assuming that the bottom segment of the caisson is only supported at points '2' at the midpoint of the long sides (see Fig. P0.1-2) and both ends are suspended in the air, the tensile strength of the concrete at the top of the caisson wall on the most unfavorable

vertical section near the midpoint of the long side caused by the self-weight of the caisson is checked.

(2) By assuming that the bottom segment of the caisson is supported at points '3' at both ends of the short sides (see Fig. P0.1-2), the tensile strength of the concrete on the bottom of the cutting edge at the midpoint of the short side caused by the self-weight of the caisson is checked.

P.0.2 When the caisson is sunk to the designed elevation and the soil under the cutting edge had been hollowed out, the upper part of the caisson wall may be clamped by the soil layer while the lower part of the caisson wall is suspended in air, thus the middle part of the caisson wall will be subjected to maximum vertical tension. The provision in this Section is only applicable to the case where the top of surface of the caisson is flush with the ground level. If the caisson is exposed above the ground, the acting position of the maximum tension force moves down and its value decreases. The calculation method is derived as follows:

(1) Caisson wall with constant sections:

Considering the most unfavorable condition for a caisson wall under vertical tension, it is assumed that the skin friction is distributed as shown in Fig. P-1.

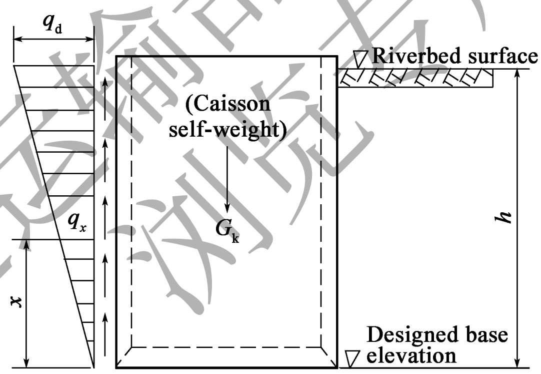


Figure P-1 Vertical Tension on the Caisson Wall with Constant Sections

Because $$G_k = \frac{1}{2} \cdot q_d \cdot h \cdot u$$

Therefore, $$q_d = \frac{2G_k}{hu}$$

Because again, $$\frac{q_x}{x} = \frac{q_d}{h}$$

Therefore, $$q_x = \frac{q_d}{h}x = \frac{2G_k}{hu} \times \frac{x}{h} = \frac{2G_k x}{h^2 u} \quad \text{(P-1)}$$

where:

G_k——self-weight of the caisson (kN);

μ——perimeter of the wall (m);

h——embedded depth of the caisson(m);

q_d——unit skin friction acting on the wall at the riverbed(kPa);

q_x——unit skin friction acting on the wall at the section with a height x from the base of the cutting edge(kPa).

The tension force of the caisson wall at x section, P_x, is equal to the result of its self-weight in x range minus the skin friction in x range, namely:

$$P_x = \frac{G_k x}{h} - \frac{q_x x u}{2} = \frac{G_k x}{h} - \frac{2G_k x}{h^2 u} \cdot \frac{xu}{2} = \frac{G_k x}{h} - \frac{G_k x^2}{h^2} \tag{P-2}$$

To find Pmax, set $\frac{dP_x}{dx} = 0$

Namely:
$$\frac{dP_x}{dx} = \frac{GK}{h} - \frac{2G_k x}{h^2} = 0$$

Then, we can obtain $x = \frac{h}{2}$. Substituting x into the Eq. (P-2):

$$P_{\max} = \frac{G_k}{h} \cdot \frac{h}{2} - \frac{G_k}{h^2}\left(\frac{h}{2}\right)^2 = \frac{G_k}{2} - \frac{G_k}{4} = \frac{1}{4}G_k \tag{P-3}$$

(2) Stepped caisson wall(Fig. P-2)

Because
$$G_{1k} + G_{2k} + G_{3k} + G_{4k} = 0.5 q_d h u$$

Therefore,
$$q_d = \frac{2(G_{1k} + G_{2k} + G_{3k} + G_{4k})}{hu}$$

From $\frac{q_x}{x} = \frac{q_d}{h}$, we have $q_x = \frac{x}{h} q_d$

The tension force of the caisson wall at x section is equal to the result of its self-weight in x range minus the skin friction in x range, namely:

$$p_x = G_x - \frac{1}{2} u q_x x \tag{P-4}$$

For a stepped caisson wall, the section tension force shall be calculated for each step segment, and then the maximum value shall be taken. Through calculations, we can find that the maximum tensile force occurs at every section where the step is changed.

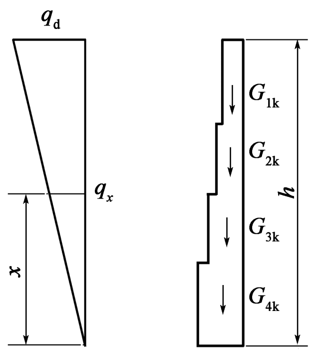


Fig. P-2　Stepped Caisson Wall Subjected to Vertical Tension Forces

P. 0. 3 When a caisson is sunk to the design elevation and the soil under the cutting edge is hollowed out, the caisson wall is subjected to the maximum horizontal force under the actions of water pressure and earth pressure. For this case, the caisson wall is taken as a plane frame, the bearing capacity of a caisson wall segment with a height equal to the wall thickness t as well as other sections of the caisson wall are checked in sequence.

Appendix Q
Calculations for Cutting Edge during Caisson Sinking Process

Appendix Q was rewritten from the provisions and the corresponding commentary of the Clause 6.3.3in the 2007 Version *Specifications*, mainly the calculations of the cutting edge during the caisson sinking process, where the main provisions and calculation methods were remained as before, while the text was adjusted accordingly in order to make the expressions clearer.

Q.0.1 During the sinking process of a caisson, its cutting edge is subjected to a very large force and the bearing capacity needs to be checked. For convenience in calculations, the cutting edge of a caisson is calculated as a cantilever beam and a plane frame, separately.

Q.0.2 ~ Q.0.3 When the cutting edge is regarded as a cantilever beam, there are two critical load cases. In the first case, the inside of the cutting edge is inserted into the soil by a certain depth, the cutting edge behaves as a cantilever beam deflecting outward. In the second case, the soil under the cutting edge has been hollowed out, the cutting edge behaves as a cantilever beam deflecting inward. In calculating the force, it shall be noted that the cutting edge is regarded as a cantilever beam and also a closed plane frame (see Q.0.4 of the *Specifications*), thud the horizontal force acting on the cutting edge side will be shared by the two different models. The distribution coefficients can be found in Clause Q.0.5 of the *Specifications*.

Q.0.5 On the one hand, the cutting edge of a caisson can be regarded as a cantilever beam fixed at the root of the cutting edge, and the beam length is equal to the height of the slope part of the outer wall of the cutting edge; on the other hand, the cutting edge may be regarded as a closed plane frame. In other words, the horizontal force acting on the cutting edge side will be shared by two different models, namely the cantilever beam and the plane frame. That is to say, one part of the horizontal force is carried by the root of the cutting edge vertically (cantilever effect), and the other part is carried by the plane frame (frame function). According to the deformation coordination relationship, the distribution coefficients α and β are derived. These formulas are applicable to the cases when the distance from the bottom surface of internal diaphragm to the bottom surface of the cutting edge is 0.5 m, or greater than 0.5m but strengthened by vertical supports. Otherwise, all the horizontal forces are carried by the cutting edge as a cantilever beam (*i. e.*, $\alpha = 1$).

Appendix R
Calculations of Horizontal Earth Pressure Based on Soil-Retaining Structure Interaction

When the retaining structure is designed according to the control principle of deformation, the earth pressure acting on the retaining structure is calculated according to the deformation condition.

The proportionality factor m for horizontal partial factor of ground that increases with the depth shall be determined through horizontalloading test as far as possible. If the test is unavailable, the value may be selected based on experience. If the test data are unavailable and the experience is lack, the value may be selected according to Table R-1.

Table R-1 '*m*' Value

Ground soil condition	m value (kN/m^4)
$I_L \geq 1$ cohesive soil, mud	1000 ~ 2000
$1.0 > I_L \geq 0.5$ cohesive soil, silt	2000 ~ 4000
$0.5 > I_L \geq 0$ cohesive soil, fine sand	4000 ~ 6000
$I_L < 0$ cohesive soil, coarse sand	6000 ~ 10000
Gravelly stone, gravelly sand, gravel, cobble	10000 ~ 20000

Note: 1. The I_L is the liquidity index of cohesive soil.

2. When the horizontal displacement of the diaphragm wall at the soil surface or the excavation surface is greater than 10 mm in the calculation, the smaller value in the table shall be taken.

Cohesive soil (especially for semi liquid or liquid clay) has a creep effect. The creep effect affects the value of earth pressure. Fig. R-1 shows the hysteresis of earth pressure caused by the creep effect of the cohesive soil. For a certain soil unit on the non-excavation side, if the displacement from A to B occurred in the previous stage, and then the soil unit moves to the opposite non-excavation side in the next stage, its earth pressure pattern shall be re-established, that is, the straight-line BC. The creep characteristics of cohesive soil are closely related to the factors such as foundation pit excavation and internalbracing construction process, soil stress level in the passive zone, and changes in soil moisture content. The creep effect of soil is difficult to be mastered accurately. In the calculations, the influence of the creep effect of earth pressure on the force and deformation of the retaining structures is considered according to a reliable method or experience.

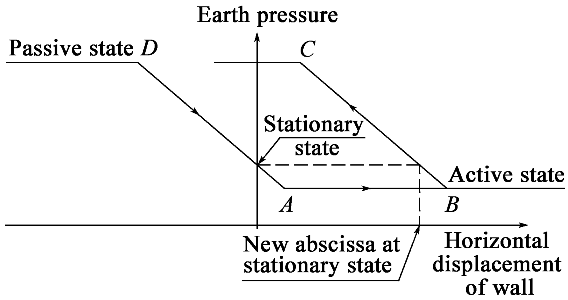


Fig. R-1 Schematic Diagram of Earth Pressure Hysteresis

Appendix S
Calculations of Straight Diaphragm Wall Retaining Structures

When the elastic beam foundation method was used to compute the earth pressure, internal force and deformation of the diaphragm wall, the following steps are carried out for the iterative processes:

(1) Assume the horizontal deformation of the walls to be 0 at the initial state, and compute the horizontal earth pressure on both sides of the walls using Eq. (R.0.1-1);

(2) Compute the horizontal deformation of the walls;

(3) Compute the horizontal earth pressure on both sides of the walls according to the Eq. (R.0.1-1) using the obtained horizontal deformation of the walls;

(4) Re-compute the internal force and deformation of the structure using the newly obtained horizontal earth pressure of the walls following the above computation steps;

(5) Repeat Step 3 and 4 until the difference between two adjacent computed deformations is small enough.

Appendix T
Calculations for Circular Diaphragm Wall Retaining Structures

T.0.2 The effective cross-sectional area, A_z, for one internal ring beam or inner lining is the effective ‘true ring’ cross-sectional area, which is the design cross-sectional area deducted by the construction deviation. The weakened section mainly refers to the reduction of the horizontal ring width of the internal ring beam or inner lining. The influence factors mainly include: (1) for the ‘polygonal’ diaphragm wall composed of multiple straight trenches, the diameter reduction of the theoretical ‘true circle’ of the internal ring beam or the outer edge of the inner lining; (2) the offset of the outer edges of the horizontal circular ring of the internal ring beam or the inner lining, which is induced by the mismatch between wall sections caused by the verticality deviation of trenches in construction of the diaphragm walls along vertical direction; (3) the diameter reduction of the theoretical ‘true circle’ caused by the plane construction errors of the internal ring beam or the inner lining itself.

T.0.3 The effective thickness d of the diaphragm wall panel is the effective ‘true circle’ thickness, which is the design thickness deducted the construction deviation. The influence factors mainly include: the reduction of the theoretical ‘true circle’ wall thickness caused by the ‘polygonal’ diaphragm wall composed of multiple straight trenches, the reduction of the wall thickness induced by the mismatch of adjacent wall sections caused by the verticality deviation of trenches in the construction of the diaphragm walls along vertical direction.

The reduction of the circumferential compression stiffness of the circular diaphragm wall by the clay coating between the wall sections is mainly considered in the modification coefficient α in Eq. (T.0.3). The concrete of the trenches is cast in stages. Due to the use of slurry for wall protection, when the second-stage trench section is cast, there must have a certain thickness of clay coating between the first and second-stage wall sections. When the foundation pit is excavated, the outside water and earth pressure causes the diaphragm wall to be compressed in the circumferential direction, causing the clay coating deformed and the circumferential rigidity of the wall to be weakened. For a circular diaphragm wall with large diameter, more connectors in the trench section and the greater the thickness of the clay coating will be used, and greater degree in weakening the stiffness will be resulted. The weaken effect is closely related to the technical level and experience of the contractors, how to consider the weaken effect in the design should be studied according to the specific conditions of the project. In calculation of the force of the circular diaphragm wall as retaining structure in the south anchorage foundation of the Wuhan Yangluo Bridge, the value of α was calculated using the recommended method by the French foundation company based on its years of experience. The computed results of the α was 0.417. According to the construction monitoring results, the force and deformation state of the wall were in good agreement with the calculated results. The outer diameter of the circular diaphragm wall as retaining structure of the south anchorage foundation of the Wuhan Yangluo Bridge reached 73 m, the wall thickness was 1.5 m, the maximum wall depth was about 61 m, and the maximum excavation depth was about 45 m, indicating it as a considerable scale diaphragm wall. Therefore, α as 0.4 is adopted as the low limit value in this Clause is applicable for circular diaphragm wall as retaining structures under normal circumstances. The high limit value of α is taken as 0.7, which is mainly referred from the provisions in the Design and Construction Technical Code for Diaphragm Wall Structure of Port Engineering (JTJ 303-2003).

Technical Terms in Chinese and English

<table>
<tr><th>序号</th><th>中文术语</th><th>英文术语</th></tr>
<tr><td colspan="3">B</td></tr>
<tr><td>1</td><td>半成岩</td><td>hypabyssal rock</td></tr>
<tr><td>2</td><td>崩解性岩石</td><td>dispersive rock</td></tr>
<tr><td>3</td><td>波浪力</td><td>wave force</td></tr>
<tr><td>4</td><td>不良地质</td><td>adverse geology</td></tr>
<tr><td>5</td><td>不排水抗剪强度</td><td>undrained shear strength</td></tr>
<tr><td colspan="3">C</td></tr>
<tr><td>7</td><td>侧摩阻力</td><td>skin friction</td></tr>
<tr><td>8</td><td>侧向压力</td><td>lateral pressure</td></tr>
<tr><td>9</td><td>插筋</td><td>steel dowel</td></tr>
<tr><td>10</td><td>沉降</td><td>settlement</td></tr>
<tr><td>11</td><td>沉井基础</td><td>caisson foundation</td></tr>
<tr><td>12</td><td>成槽竖直度</td><td>trench verticality</td></tr>
<tr><td>13</td><td>承台</td><td>pile cap</td></tr>
<tr><td>14</td><td>承载能力极限状态</td><td>ultimate limit state</td></tr>
<tr><td>15</td><td>持力层</td><td>bearing stratum</td></tr>
<tr><td>16</td><td>冲刷深度</td><td>scour depth</td></tr>
<tr><td>17</td><td>冲刷线</td><td>scour line</td></tr>
<tr><td rowspan="2">18</td><td>冲填土</td><td>hydraulically placed fill</td></tr>
<tr><td>素填土</td><td>plain fill</td></tr>
<tr><td>19</td><td>粗砂</td><td>coarse sand</td></tr>
<tr><td colspan="3">D</td></tr>
<tr><td>20</td><td>地面荷载</td><td>surcharge load</td></tr>
<tr><td>21</td><td>砂井</td><td>sand drain well</td></tr>
</table>

continued

序号	中文术语	英文术语
22	单向偏心受压	uniaxial eccentric compression
23	地层	stratum
24	地层结构	strata
25	地基	ground
26	地基承载力地基土承载力	ground bearing capacity bearing resistance of soil
27	地基土	foundation soil
28	地貌	landform
29	地下连续墙	diaphragm wall
30	地下水	groundwater
31	渗流	seepage
32	地形	topography
33	地质	geology
34	底梁	base beam
35	垫层	replacement layer
36	定倾半径	metacenter radius
37	冻深	frost depth
38	冻结线	depth of frost potential
39	冻土	frozen soil
40	冻胀	frostheave
41	陡坡地基	steep slope ground
42	端阻力	tip resistance
43	端承桩	end-bearing pile
44	多年冻土	permafrost
45	发育程度	development degree
F		
46	粉砂	silty sand
47	粉土	silty soil
48	粉质黏土	silty clay
49	浮重度	buoyant unit weight
50	负摩阻力	negativeskin friction
51	覆盖层	overlying layer 或 covering layer
G		
52	盖板	cover plate
53	钢筋/钢筋笼	reinforcing steel/reinforcement cage
55	高程	elevation

continued

序号	中文术语	英文术语
56	工程经验	engineering judgement
57	构造钢筋	auxiliary reinforcement
58	隔墙	diaphragm
	H	
59	红黏土	red cohesive clay
60	换算重度	transformedunit density
	换算截面	transformed section
	J	
61	基底	foundation base
62	基坑支护结构	retaining structure of foundation pit
63	基岩	bedrock
64	基桩	foundation pile
65	基础底面	base of foundation
66	极破碎	extremely fractured
67	极软岩	extremely weak rockveryweak rock
68	坚硬	stiffness
69	坚硬程度	rock mass rating of strength
70	坚硬岩	very strong rock
71	浆液灌注	grouting
72	角砾	breccia
73	较破碎	moderately fractured
74	较软岩	moderately weak rock
75	较完整	moderately intact
76	较硬岩	moderately strong rock
77	接头箱	box-typedconnectors
78	接桩	pile splicing
79	节理	joint
80	节理间距	spacing ofjoint
81	结构重要性系数	importance factor of structure
82	截面剪切刚度	sectional shear stiffness
83	截面抗弯刚度	sectional flexural stiffness
84	截面压缩刚度	sectionalcompression stiffness
85	襟边宽度	offset width
86	经常性流水	constant flow
87	井壁	caisson wall

continued

序号	中文术语	英文术语
88	井孔	dredge hole(Well hole)
89	井筒式地下连续墙基础	diaphragm wallas caisson (or closed) foundation
90	净高	vertical clearance
91	静力触探试验	static cone penetration test
92	静水压力	hydrostatic water pressure
	K	
93	勘察	survey
94	抗滑稳定性	stability against sliding
95	抗滑移稳定系数	stability factoragainst sliding
96	抗剪强度	shear strength
97	抗力系数	resistance factor
98	抗倾覆	against overturning
99	抗滑移	against sliding
101	抗压强度	compressive strength
102	可塑	plastic
103	孔隙比	void ratio
104	块石	block stone
105	扩底桩	belled pile
106	扩孔部分	bell-bottomed part
107	扩挤支盘桩	pile with expanded branches and plates/ under-reamed pile
108	扩孔	bell bottom
	L	
109	砾砂	gravelly sand
110	砾石	gravelly stone
111	裂缝	crack
112	裂隙	fissure
113	临水波浪作用	wave action near water
114	流砂	quicksand
115	流水压力	stream pressure
116	流塑	liquid
117	卵石	cobble
	M	
118	埋置深度	embedded depthor bearing depth
119	埋置式桥台	spill-through abutment

continued

序号	中文术语	英文术语
120	锚固长度	development length
121	锚固钢筋	anchorage reinforcement
122	密实度	density
123	面积抵抗矩	section modulus
124	摩擦桩	friction pile
125	摩阻力/摩擦力	frictional resistance/frictional force
	N	
126	内环梁	internal ring beam
127	内衬	inner lining
128	内支撑	internal bracing
129	泥炭	peat
130	泥炭质土	peaty soil
131	泥岩	mudstone
132	黏土岩	argillaceous rock
133	黏土/粘土	clay
134	黏性土	cohesive soil
	P	
135	排水	drainage
136	排水固结	drained consolidation
137	膨胀土	swelling soil
138	膨胀性岩石	swelling rock
139	漂石	boulder
140	平均常水位	average normal water level
141	破裂面	fracture plane
142	破碎	very fractured
143	破碎程度	fracture degree
144	破碎岩石地基	fractured rock foundation
	Q	
145	浅基础	shallow foundation
146	嵌固作用	socketed effect
147	嵌岩比	ratio of rock-socketed depth
148	嵌岩深度	rock-socketed depth
149	嵌岩桩	rock-socketed pile
150	强风化	highly weathered
151	强夯	dynamic compaction

continued

序号	中文术语	英文术语
152	墙体单元槽段	trench wall unit
153	切向冻胀力	tangential frost heave
154	倾斜锚杆	inclinedanchor rod
155	全风化	completely weathered
156	群桩	group piles
	群桩	Pile group
R		
157	刃脚	cutting edge
158	溶陷	karst depression
159	软化岩石	soft rock
160	软化系数	softening coefficient
161	软弱地基	soft ground
162	软塑	semi plastic
163	软土	soft soil
164	软岩	very weak rock
165	软粘土	soft clay
S		
166	砂井预压法	preloading method with sand drain wells
167	砂砾	sandy gravel
168	砂砾垫层	replacement layer of sandy gravel
169	砂石桩	sandy stone piles
170	砂土	sand/sandy soil
171	砂岩	sandstone
172	稍密	slightly dense
173	渗透性	permeability
174	湿陷系数	collapsibility coefficient
175	湿陷性	collapsibility
176	湿陷性黄土	collapsible loess soil
177	湿陷性土	collapsible soil
178	石灰桩	lime piles
179	实体基础	block foundation
180	受力钢筋	primary reinforcement
181	水泥搅拌桩	cement deep mixing piles
182	水文的	hydrological
183	松软土	soft soil

continued

序号	中文术语	英文术语
184	松散	loose
185	素填土	placed fill
186	塑料排水板	plastic vertical drains
187	塑性指数	plasticity index
188	碎石	gravel
189	碎石土	gravelly soil
	T	
190	台背填土	abutment backfill
191	弹性地基梁	elastic foundation beam
192	填土	fill/backfill
193	条壁式地下连续墙基础	diaphragm wall as strip foundation
194	土层锚杆(锚索)	earth anchor(cable)
194	土塞效应	soilplugging
	W	
196	挖孔桩	bored pile
197	完整	intact
198	微风化	slightly weathered
199	未风化	unweathered
	X	
200	系梁	tie-beam
201	细砂	fine sand
202	下沉	sinking
203	新近沉积黏性土	newly deposited cohesive soil
	Y	
204	压力扩散角	pressure spreading angle
205	压实填土	compacted fill
206	压缩模量	compressive modulus
207	压缩土层	compressible soil layer
208	压缩系数	compressive factor
209	压缩性	compressibility
210	严寒地区	extremely cold region
211	岩层	rock stratum
212	岩溶地基	karst grounds
213	岩石	rock
214	岩石地基	rock foundation

continued

序号	中文术语	英文术语
215	岩体节理发育程度	joint development degree of rock mass
216	盐胀	salt swelling
217	盐渍化岩石	salinized rock
218	盐渍土	salinized soil
219	阳坡	adret slope
220	液性指数	liquidity index
221	易溶性岩石	soluble rock
222	硬塑	stiff plastic
223	淤泥	mud
224	淤泥质土	muddy soil
225	圆砾	round gravel
226	圆形地下连续墙	circular diaphragm wall
227	圆形桩	circular pile
228	原状土	undisturbed soil
	Z	
229	杂填土	miscellaneous fill
230	振冲碎石桩	vibratory crushed stone piles
231	整体承载力	block bearing resistance
232	整体基础	block foundation 或 entire foundation
233	正常使用极限状态	serviceability limit state
234	支承系统	supporting system
235	直线形地下连续墙	straight diaphragm wall
236	中风化	moderately weathered
237	中密	medium dense
238	中砂	medium sand
239	重力密度	gravity density
240	桩侧摩阻力	skin friction of pile
241	桩端	pile tip
242	桩端阻力	tip resistance
243	桩基础	pile foundation
244	桩柱式(桥)墩	pile bent pier
245	最小边宽	minimal margin width
246	纵坡	longitudinal gradient
247	钻孔桩	drilled pile