NATIONAL STANDARD
OF THE PEOPLE'S REPUBLIC OF CHINA
中华人民共和国国家标准

Code for Seismic Design of Buildings

建筑抗震设计规范

GB 50011—2001

（英文版）

Beijing 2001

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NOTICE

The code is written in Chinese and English. The Chinese text shall be taken as the ruling one in the event of any inconsistency between the Chinese text and the English text.

Notice of Promulgation for the National Standard "Code for Seismic Design of Buildings"

Document JB[2001]No. 156

According to the requirements of "Notice of Printing and Distributing for the '1997 Preparation and Revision Plan of Engineering Construction Standards' (Document JB[1997] No. 108)-Ministry of Construction", that the "Code for Seismic Design of Buildings" has been revised by Ministry of Construction together with the relevant departments, after a joint examination of relevant departments, hence the new code has been approved as a national standard with a serial number of GB50011—2001, and it shall come into force upon January 1, 2002. Herein, clauses 1.0.2, 1.0.4, 3.1.1, 3.1.3, 3.3.1, 3.3.2, 3.4.1, 3.5.2, 3.7.1, 3.8.1, 3.9.1, 3.9.2, 4.1.6, 4.1.9, 4.2.2, 4.3.2, 4.4.5, 5.1.1, 5.1.3, 5.1.4, 5.1.6, 5.2.5, 5.4.1, 5.4.2, 6.1.2, 6.3.3, 6.3.8, 6.4.3, 7.1.2, 7.1.5, 7.1.8, 7.2.4, 7.2.7, 7.3.1, 7.3.3, 7.3.5, 7.4.1, 7.4.4, 7.5.3, 7.5.4, 8.1.3, 8.3.1, 8.3.6, 8.4.2, 8.5.1, 10.1.3, 10.2.5, 10.3.3, 12.1.2, 12.1.5, 12.2.1 and 12.2.9 are compulsory provisions, which must be enforced strictly. The former "Code for Seismic Design of Buildings" GBJ 11—89 and the "Proclamation on Partial Revision for National Standards of Engineering Construction" (No. 1) shall be abolished after December 31, 2002.

The Ministry of Construction is in charge of management and explanation of the mandatory clauses in the code, the China Academy of Building Research will be responsible for the explanation of specific technical contents and the Research Institute of Standards and Norms-Ministry of Construction will organize the China Architecture & Building Press to take on publishing and distributing works of this code.

Ministry of Construction of the People's Republic of China

July 20, 2001

PREFACE

The present code is the revised version of the former "Code for seismic design of buildings" GBJ 11—89. The revision is undertaken by China Academy of Building Research (CABR) together with other institutions related to design, reconnaissance, research, and universities in accordance with the Document JB [1997] No. 108 issued by the Ministry of Construction.

During the process of revision, the editorial team carried out studies on specific topics and some laboratory tests concerned. Experiences and lessons, learned from damages induced by strong earthquake having occurred in recent years home and abroad, are summarized, achievements of research in earthquake engineering are involved, the economic condition and construction practices in China are taken into account, comments from all aspects of design, reconnaissance, research, education and municipal authorities are widely collected nation wide. Through a multi-round discussion, revision, substantiation, and with pilot designs as well, the final version has been completed and reviewed by an expert panel.

The newly updated version consists of 13 chapters and 11 appendixes. The major contents of the revision can be clarified by follows: the seismic fortification classification for buildings has been adjusted; the seismic design is required to be based on the design basic acceleration of the ground motion; the near-field and far-field earthquakes employed by the previous code have been replaced by the design earthquake groups; provisions related to the site classification, liquefaction identification, seismic influence coefficient and torsion effect calculation have been modified; requirements for the conception design of irregular building structures, the seismic analysis of structures, the limit of inter-story seismic shear force and deformation have been added; the seismic measures of masonry and concrete structures, and masonry buildings with bottom R. C. frames have been improved; more contents related to active fracture, pile foundation, R. C. tube structure, steel structure, reinforced hollow block masonry structure, and non-structural components, as well as the provisions for base isolation and energy dissipation have been involved. In the meantime, the new code has abrogated some provisions dealing with inner-frame house and medium-sized block masonry structure, chimney and water tower, etc.

The new code could be partly modified in the future. The relevant information and details of clauses will be published on the magazine of "Engineering Construction Standardization".

Clauses marked with boldface type in this code are compulsory provisions and must be enforced strictly.

The Institute of Earthquake Engineering of CABR is responsible for the interpretation of this code. Hopefully, in the process of its enforcement, all institutions may sum up and

accumulate their experiences in practice. Any comment and advice is welcome to submit to the Code Panel by the address: IEE, CABR, No. 30, Bei San Huan Dong Road, Beijing 100013. (e-mail: ieecabr @ public3. bta. net. cn).

Chief Editorial Unit: China Academy of Building Research (CABR)

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Chapter 1 General

1.0.1 This code is prepared for the purpose of carrying out " The Law of Construction of People's Republic of China" and "The Law of Earthquake-prevention and Disaster Mitigation of People's Republic of China", and carrying out the policy of giving priority to the prevention of earthquake disasters. So that, when the buildings are made earthquake-fortification, the damages to buildings, loss of life and economic losses will be mitigated.

The seismic fortification objective of buildings, which designed and detailed in accordance with the requirements of this code, as follows:

When the place is subjected to frequently earthquake influence which intensity is lower than the local fortification intensity, the buildings will not be damaged or only slightly damaged, with continued service without repair.

When the place is subjected to local fortification intensity earthquake influence, the buildings will be damaged, with continued service after ordinary repair or without repair.

When the place is subjected to rarely earthquake influence which intensity is expected to higher than the local fortification intensity, the buildings will not collapse nor suffer damage that would endanger human lives.

1.0.2 Every building, which is situated on zones of fortification intensity 6 or above, must be designed to resist the effects of earthquake motions.

1.0.3 The design of seismic ordinary buildings and seismically isolated structures, which are situated on the zone of fortification intensity 6 to 9, shall be in accordance with this code.

When buildings are situated on zone where the fortification intensity is greater than 9, and/or industry buildings with specific professional requirements, the corresponding design of these buildings shall be in accordance with special provisions.

Note: The "fortification intensity" hereinafter usually referred as "intensity". For example, fortification intensity 6,7,8 and 9 is referred as intensity 6,7,8 and 9 respectively.

1.0.4 Fortification intensity of a region must be determined by documents (or maps) published by the authorized central government agency.

1.0.5 Normally, the local fortification intensity may be adopted the seismic basic intensity as provided in "the China Seismic Ground Motion Parameter Zonation Map" (or the intensity values corresponding to the design basic seismic acceleration in this code). If cities where a seismic fortification zonation has been drawn up, the approval fortification intensity or design ground motion parameters may be adopted.

1.0.6 Seismic design based on this code shall also be coordinated with provisions specified in other current compulsory design codes concerned.

Chapter 2 Definitions and notations

2.1 Definitions

2.1.1 Seismic Fortification Intensity

The seismic intensity approved by State authority, which is used as the basis for the seismic fortification of buildings in a certain region.

2.1.2 Seismic Fortification Criterion

The rule for judging the seismic fortification requirements, which depends on the seismic fortification intensity and importance of the building's using functions.

2.1.3 Earthquake Action

The structural dynamic action due to earthquake, including horizontal seismic action and vertical seismic action.

2.1.4 Design Parameters of Ground Motion

The seismic acceleration time-history curve, the response spectrum of acceleration, and the peak value acceleration used in seismic design.

2.1.5 Design Basic Acceleration of Ground Motion

The design value of seismic acceleration, that exceeding probability is 10% during the 50-years reference period.

2.1.6 Design Characteristic Period of Ground Motion

The period value corresponding to the starting point of reduced section of seismic influence coefficient curve, which describes the earthquake magnitude, the distance of epicenter and the site-classes etc.

2.1.7 Site

An area of a building group, usually it has similar characteristic in response spectrum. Its scope approximately equivalent to a factory area, a living quarter, a village or a plain area no less than $1.0km^2$.

2.1.8 Seismic Concept Design of Buildings

The process of making the general arrangement for the architectures and structures and of determining details, that based on the design fundamental principles and the ideas obtained from past experiences in earthquake disaster prevention and the constructional project.

2.1.9 Seismic Fortification Measures

The seismic design contents except seismic action calculation and member resistance calculation, and details of seismic design included.

2.1.10 Details of Seismic Design

All of detailed requirements, which are determined according to seismic concept design of buildings and no calculation is necessary.

2.2 Main Notations

2.2.1 Actions and effects

F_{Ek}, F_{Evk}——characteristic value of total horizontal and vertical seismic action of structure respectively.

G_E, G_{eq}——representative value of gravity load of structure (or member) and the total equivalent gravity load of a structure in earthquake respectively.

w_k——characteristic value of wind load.

S_E——seismic effect (bending moment, axial force, shear, stress and deformation).

S——base combination of seismic effect and other load effects.

S_k——effect corresponding to characteristic value of action or load.

M——bending moment.

N——axial force.

V——shear.

p——compression on bottom of foundation.

u——lateral displacement.

θ——rotation of story draft.

2.2.2 Resistance and Material Properties

K——stiffness of structure (or member).

R——resistant capacity of structural member.

f, f_k, f_E——design value, characteristic value and seismic design value of material strength (bearing capacity of subsoil included) respectively.

$[\theta]$——allowable rotation of story draft.

2.2.3 Geometric Parameters

A——cross-sectional area of structural member.

A_s——cross-sectional area of reinforcement.

B——total width of structure.

H——total height of structure, or column height.

L——total length of structure (or structural unit).

a——distance.

a_s, a'_s——distance from near extreme fiber of section to the point for resultant of force of all longitudinal reinforcement in tension and compression respectively.

b——width of cross section of member.

d——depth or thickness of soil, or diameter of reinforcement.

h——height of story, or height of cross section of member i.e. depth of member.

l——length or span of member.

t——thickness of seismic structural-wall or slab.

2.2.4 Coefficients of Calculation

α——horizontal seismic influence coefficient.

α_{max}——maximum value of horizontal seismic influence coefficient.

α_{vmax}——maximum value of vertical seismic influence coefficient.

$\gamma_G, \gamma_E, \gamma_w$——partial factor of gravity, earthquake and wind load respectively.

γ_{RE}——seismic adjusting factor for bearing capacity of member.

ζ——calculation factor.

η——amplification factor or adjusting factor of seismic effect (inner force or deformation).

λ——slenderness ratio of member, or proportional factor.

ξ_y——yield strength coefficient of structure (or members).

ρ——reinforcement ratio or ratio.

φ——stability factor of compressive member.

ψ——combination value coefficient, or effect factor.

2.2.5 Others

T——natural period of structure.

N——penetration resistance (in blow number).

I_{lE}——liquefaction index of subsoil under earthquake.

X_{ji}——mode coordinate of displacement (relative displacement of mass i-th of mode j-th in the x direction).

Y_{ji}——mode coordinate of displacement (relative displacement of mass i-th of mode j-th in the y direction).

n——total number, such as number of stories, masses, reinforcement bars, spans etc.

v_{se}——equivalent shear-wave velocity of soil.

Φ_{ji}——mode coordinate of rotation (relative rotation of mass i-th of mode j-th around the z axial direction).

Chapter 3 Basic requirements of seismic design

3.1 Classifications of seismic fortification and corresponding criterion

3.1.1 Every building shall be classified, according to the importance of their using functions, as a seismic fortification category A, B, C or D defined as follows:

Category A buildings are those, major buildings or the failure of which would result in severe secondary disaster.

Category B buildings are those which the continual function is necessary during earthquake or could be restored quickly after earthquake.

Category C buildings are those not assigned to either category A, B or D buildings.

Category D buildings are those less important buildings.

3.1.2 Each building shall be assigned to a fortification category in accordance with the current national standard "Standard for seismic fortification classification of buildings" GB 50223.

3.1.3 Fortification categories are used in this code to determine fortification criterion as follows:

1 For buildings assigned to category A, the earthquake action of which shall be higher than that of the local fortification intensity, that values shall be determined based on the site seismic safety appraisal results. When the fortification intensity is 6～8, the seismic measures shall be one grade higher than that of the local fortification intensity; However, the seismic measures shall be appropriately higher than that of fortification intensity 9, where zones belong to fortification intensity 9.

2 For buildings assigned to category B, the earthquake action of which shall be adopted as local fortification intensity. Normally When the fortification intensity is 6～8, the seismic measures shall be one grade higher than that of the local fortification intensity; However, the seismic measures shall be appropriately higher than that of fortification intensity 9, where zones belong to fortification intensity 9. And the seismic measures for foundation shall comply with relevant provisions.

For smaller buildings assigned to category B, only when their structural system changed into that with higher seismic capability, it is permitted to take seismic measures as that of local fortification intensity.

3 For buildings assigned to category C, the earthquake action and seismic measures shall be take as that of local fortification intensity.

4 For buildings assigned to category D, the earthquake action shall be take as that of local fortification intensity, and the seismic measures shall be appropriately lower than

that of local fortification intensity. However, seismic measures cannot be lowered where zones belong to fortification intensity 6.

3.1.4 If the buildings assigned to fortification category B, C and D are situated on zone where the fortification intensity is 6, except as specified in this code, earthquake action is permitted not to be calculated.

3.2 Seismic influences

3.2.1 The seismic influence for the buildings situated region shall be described by using the design basic acceleration of ground motion and design characteristic period of ground motion, or by using the design ground motion parameters as indicated in Clause 1.0.5 of this code.

3.2.2 The corresponding relationship between the fortification intensity and the design basic acceleration of ground motion as indicated in Table 3.2.2. Where the design basic acceleration of ground motion is 0.15g and 0.30g, except as specified in this code, the seismic design of buildings shall be adopted as that of fortification intensity 7 and 8 respectively.

Table 3.2.2 Relationship between the intensity and design basic acceleration of ground motion

Fortification intensity	6	7	8	9
Design basic acceleration of ground motion	0.05g	0.10 (0.15)g	0.20 (0.30)g	0.40g

Note: g is the gravitational acceleration.

3.2.3 The design characteristic period of ground motion shall be determined according to the design earthquake groups and Site-classes of location of each building. For Site-class Ⅱ, the design characteristic period of ground motion for 1 st, 2 nd, and 3 rd design earthquake group shall be taken as 0.35s, 0.40s and 0.45s respectively.

Note: The "design characteristic period of ground motion", hereinafter reference as "characteristic period".

3.2.4 The values of fortification intensity, design basic acceleration of ground motion and design earthquake groups for main cities and towns in China may be indicated in Appendix A of this code.

3.3 Site and subsoil

3.3.1 When selecting a construction place, a comprehensive assessment classified as favorable plat, unfavorable plat or hazardous plat to seismic fortification shall be made, according to seismicity of the region, and the geotechnical and geological data of site dependent to necessities of project. The favorable plats to seismic fortification shall be selected, while unfavorable plats shall be avoided except appropriate seismic measures have been taken. The buildings assigned to Fortification Category A, B and C. shall not be construct-

ed on the hazardous plats.

3.3.2 Only the construction field belong to Site-class Ⅰ, the seismic designed details is permitted taken as follows:

For building assigned to Fortification Category A or B, seismic detail requirements could be taken as that for local fortification intensity.

For building assigned to Fortification Category C, seismic detail requirements could be taken as that for intensity of one grade lowers than local fortification intensity, but shall not be reduced for local fortification intensity 6.

3.3.3 When design basic acceleration of ground motion is 0.15g and 0.30g, while construction field assigned to Site-class Ⅲ or Ⅳ, except as specified in this code, the seismic designed details should be taken as that of fortification intensity 8 (0.20g) and 9 (0.40g) respectively.

3.3.4 Design requirement of subsoil and foundation shall be as follows:

1 Foundation of same structural unit should not be posited on subsoil with entirely different characteristics.

2 Foundation of same structural unit should not consist of partly natural subbase and partly pile foundation.

3 For base-soil with layers consisted of soft clay, liquefied soil, recently back-filled soil, or soil with extremely non-uniform distribution, differential settlement and/or other harmful impact under earthquake shall be calculated in design and corresponding measures shall be taken.

3.4 Regularity of architectural design and structural design

3.4.1 The architectural design shall be made in accordance with the requirements of seismic concept design of buildings, a seriously irregular design scheme of building shall not be adopted.

3.4.2 The plan arrangement of architecture and lateral-force-resisting members should be regular and symmetrical, and shall have good integrity. The configuration and elevation of building should be regular, the lateral stiffness of structure should be changed equably, and the cross-sectional dimensions and its material strength of vertical lateral-force-resisting members should be reduced along whole structure from lower part to upper part gradually, to avoid sudden change in stiffness and bearing capacity of lateral-force-resisting system of the structure.

Buildings having one or more of the features listed in Table 3.4.2-1 shall be designated as having plan structural irregularity, buildings having one or more of features listed in Table 3.4.2-2 shall be designated as having vertical structural irregularity. Their design shall comply with the requirements in Clause 3.4.3 of this code.

Table 3.4.2-1 Plan structural irregulartties

Type of irregularity	Definitions
Torsion irregularity	Maximum story displacement or story drift, computed including accidental torsion, at one end of structure transverse to an axis is more than 1.2 times the average of the story displacement or story drift at two ends of the structure respectively, when diaphragms are not flexible
Re-entrant corners irregularity	Both projections of the structure beyond a re-entrant corner greater than 30% of the plan dimension of the structure in the given direction
Diaphragm discontinuity	Diaphragms with abrupt discontinuities or variations in stiffness, including those having cutout or open areas greater than 30% of the gross enclosed diaphragm area, or effective width less than 50% of total width of diaphragm, or more distinct from one floor slab to other at same story

Table 3.4.2-2 Vertical structural irregularities

Type of irregularity	Definitions
Stiffness irregularity	The lateral stiffness is less than 70% of that in the story or less than 80% of the average stiffness of the three stories above; the horizontal dimension less than 75% that of next story lower, except top story of building
Discontinuity in vertical anti-lateral-force members	The internal forces of vertical lateral-force-resisting members (columns, structural walls and braces) transfer to lower those members by using the horizontal transmission member (girders or trusses)
Discontinuity in bearing capacity	The story lateral shear capacity is less than 80% that of next story above. The story lateral shear capacity is the total capacity of anti-lateral-force members sharing the story for the direction under consideration

3.4.3 For the irregular structure, the analysis of horizontal seismic action and internal force adjustment of structure shall comply with the following requirements, and the effective seismic designed details of weak point shall be taken.

1 For plan irregular and vertical regular structure, the three-dimensional computed model shall be adopted, and comply with the following requirements:

1) Structure having torsion irregularity shall calculate the torsion effects, and the maximum story displacement or story drift at one end of structure transverse to an axis should not be more than 1.5 times the average of the story displacement or story drift at two ends of structure respectively;

2) Structure having re-entrant corner irregularity or diaphragm discontinuity shall calculate the possible deformation in plane of diaphragm and torsion effects where torsion irregularity exist also.

2 For vertical irregular and plan regular structure, the three-dimensional computed model shall be adopted, the seismic shear forces of weak stories shall be increased by factor 1.15, the elasto-plastic deformation analysis shall be made as required by this code, and

comply with following requirements:

1) Vertical lateral-force-resisting member having discontinuity, its seismic internal force shall be increased by factors 1.25~1.5 to transfer on the horizontal transmission members;

2) Story capacity having abrupt discontinuity, the shear capacity of week story shall not be less than 65% that of next story above.

3 The structure having plan and vertical irregularities shall comply with every requirements in point 1 and 2 of this Clause.

3.4.4 For masonry structures and single-story factory buildings, the plan and vertical irregularity shall comply with requirement in relevant chapters and sections of this code respectively.

3.4.5 The structures having complicate shapes or extreme irregular plan and elevations, isolation joints may be installed in appropriate point according to actual condition, to divide into some regular lateral-force-resisting structural units.

3.4.6 The width of isolation joint shall be enough large which is determined in accordance with seismic fortification intensity, the structural material, the structure system, the height and its difference of structural units; and two structural units divided by isolation joint shall be separated completely.

When the expansion joints or settlement joints have been installed, their width shall satisfy the requirements for isolation joints.

3.5 Structure system

3.5.1 The seismic structure system of a building shall be determined through comprehensive analysis for technical and economic conditions considering the following factors: fortification categories, fortification intensity, building height, site conditions, subsoil, structural material, and construction technology.

3.5.2 Seismic structure system shall comply with the following requirements:

1 The system shall have a clear computed model and reasonable transferred ways for seismic action.

2 The system shall have an ability to avoid loss of either seismic capacity or bearing gravity capacity of whole structure, that due to damage to some structural members or components.

3 It shall be provided with necessary seismic bearing capacity, adequate deformability, and better energy dissipation ability.

4 Some measures shall be taken to enhance earthquake resistance capacity of weak points.

3.5.3 Seismic structure system should also comply with the following requirements:

1 It should be installed with seismic multiple-defense lines.

2 It should be provided with reasonable distribution of stiffness and bearing capacity, to avoid existed weak point due to local weakening or abrupt changes so that the great concentrate of stress or deformation may be produced at weak points.

3 The dynamic characteristics of the structure in two principal axial directions should be similar.

3.5.4 The structural members shall comply with following requirements:

1 The masonry members shall be installed with the reinforced concrete ring-beams, tie-columns and core-columns in accordance with relevant provision, or shall be adopted with reinforced masonries.

2 The concrete members shall be selected with reasonable dimensions and be arranged with longitudinal bars and hoops, to avoid the shear failure occurring before flexural failure, the concrete crush occurring before reinforcements yielded, and the anchorage and cohesion failure of reinforcements occuring before member damaged.

3 In prestressed concrete members, sufficient non-prestress steel bars shall be arranged.

4 The steel members shall be controlled with reasonable dimensions to avoid the local instability or whole instability of members.

3.5.5 The connections of seismic structures shall comply with the following requirements:

1 The failure of connected nodes of members shall not occur before that of members it connects.

2 The anchorage failure of embedded parts shall not occur before that of members it connects.

3 The connections of prefabricated structures shall ensure the integrality of the structure.

4 Prestressing tendons of prestressed concrete members should be anchored at the exterior face of the core of joint or beyond.

3.5.6 The seismic bracing system of single-story fabricated factory shall ensure the stability of whole structure during an earthquake.

3.6 Structure analysis

3.6.1 The analysis for internal force and deformation of building structures under frequently earthquake shall be carried out, except otherwise provision in this code. In these analysis, it may be assumed that the structures and its members are working at elastic state, so the internal force and deformation may be calculated with the linear static /dynamic analyzing method.

3.6.2 For structures having irregularity and existing weak locations that may result in serious seismic damage, the elasto-plastic deformation analysis under rarely earthquake shall

be carried out according to relevant provisions of this code. In these analysis, the elasto-plastic static analyzing method or elastio-plastic time-history analyzing method may be adopted depending on the structural characteristics.

When there are specific provisions of this code, the simplified methods analyzing elasto-plastic deformations of the structures may be adopted also.

3.6.3 When the gravity additional bending moment due to seismic story drift is greater than 10% of original bending moment, the secondary effect of gravitation shall be taken into consideration.

Note: the gravity additional bending moment is the product by the total gravity load at and above this story and story drift; the original bending moment is the product by the seismic story shear and story height of the building.

3.6.4 In seismic analysis, the floor and roof shall be assumed as the rigid, semi-rigid or flexible diaphragm depending on deformation in slab plan, then the interaction behavior between lateral-force-resisting members may be determined by using above assumption, and then, the seismic internal forces of these members may be obtained.

3.6.5 The structure having rigid diaphragms and nearly symmetric distribution of masses and stiffness, as well as the structure with specific provisions of this code, may adopt plane structural model to carry out the seismic analysis. All other structures shall adopt three-dimensional structural models to carry out the seismic analysis.

3.6.6 The seismic analysis of structures by computers shall comply with following requirements:

1 The determination of computing model, necessary simplified calculation and technique for a structure shall comply with the actual performance of this structure.

2 The technical conditions of computer programmer shall comply with relevant provisions in this code and other design standards, and its contents and special technique shall also be clarified.

3 The analysis for internal force and deformation of complicate structures under frequently earthquake shall be adopted at least two different mechanic models, and the comparison shall be made for the calculation results of these models.

4 The rationality and validity of all the calculation results from the computer programmer shall be judged and affirmed, and after then it is permitted to use in the project design.

3.7 Nonstructural components

3.7.1 Nonstructural components of building including architectural, mechanical and electrical components permanently attached to structures, the supporting structures and attachments (hereinafter referred to as "components"), shall be designed and constructed to resist seismic action.

3.7.2 The seismic design of nonstructural components shall be carried out by those relevant professionals respectively.

3.7.3 The attached elements shall be reliably connected or anchored to relevant structural members, so that human injury or damage to important equipment due to their collapse can be avoided.

3.7.4 For the arrangement of exterior nonstructural walls and partition wall their unfavorable influnece on seismic performance of structure shall be considered; irrational arrangement of these walls that would cause damage to main structure shall be avoided.

3.7.5 Curtain wall and veneers shall be firmly adhered to main structure, so that human injury due to their falling can be avoided.

3.7.6 The attachments and supports of mechanical and electrical components permanently attached to structures shall satisfy the functional requirements under earthquake, and damages to relevant portions of structures shall be avoided.

3.8 Seismically isolation and energy-dissipating design

3.8.1 The seismically isolated and energy-dissipating structures shall mainly be applied to that buildings, which have special performance requirements or its fortification intensity is 8 degree or 9 degree.

3.8.2 The seismically isolated and energy-dissipating structures under the frequently, fortification and rarely earthquake influence shall meet the requirements higher than that in Clause 1.0.1 of this code.

3.9 Structural materials and construction

3.9.1 Specified material and construction requirements for seismic structures shall be clearly stated in the design documents.

3.9.2 Structural material property shall comply with minimum requirements as follows:

1 Material strengths of masonry structures shall meet the following requirements:

1) The strength grades of fired clay common bricks and perforatedbricks shall not be less than MU10, and the strength grades of mortar for such bricks shall not be less than M5;

2) The strength grades of small-sized concrete hollow blocks shall not be less than MU7.5, and the strength grades of mortar for such blocks shall not be less than M7.5.

2 Material property of concrete structures shall meet the following requirements:

1) The strength grades of concrete for framed beams, columns and joint-core of structure assigned to seismic Grade 1, as well as frame-supported beams and columns, shall not be less than C30; the strength grades of concrete for ring-

beams, tie-columns, core-columns and other members shall not be less than C20;

2) For the non-prestressed longitudinal reinforcements of framed structures assigned to seismic Grade 1or 2, the ratio of the actual ultimate tensile strength to actual tensile yield strength shall not be less than 1.25, and the ratio of actual tensile yield strength to characteristic strength shall not greater than 1.3.

3 Material property of steel structures shall meet the following required:

1) The ratio of actual ultimate tensile strength to actual tensile yield strength shall not be less than 1.2;

2) It shall have obvious yield steps, and the elongation rate shall be greater than 20%;

3) It shall have good weld-ability and qualified shock tenacity.

3.9.3 Structural material property should also comply with following requirements:

1 The non-prestress reinforcements having better elongation, weld-ability and tenacity should be given priority to selective using; the longitudinal non-prestress reinforcement bars should be selected grade HRB400 and HRB335 hot-rolling bars, the hoop bars should be selected grade HRB335, HRB400 and HPB235 hot-rolling bars.

Note: the checkout method of steel bars shall comply with the requirement of current national standard "Code for construction quality acceptance of concrete structure" GB 50204

2 The strength grades of concrete for concrete structures should not exceed C60 and C70 where fortification intensity 9 and 8 respectively.

3 The steel type of steel structures should be selected the Q235 grade B, C, D of carbon structural steel or Q345 grade B, C, D, E of low alloy and high strength structural steel; when there is reliable condition, other type and grade structural steel may also be selected.

3.9.4 In construction of concrete structures, if main longitudinal steel bars are necessary to be replaced by those with strength grade higher than that of original design, then shall be made as follow:

The conversion shall be made according to equal tensile capacity design values of such bars, and shall also comply with the requirements for serviceability limit states and seismic details.

3.9.5 For steel structures adopting welded connections, if the thickness of steel plate is not less than 40mm and tension along direction of thick, then contraction rate along direction of thick under tension test shall not be lower than allowable values of grade Z15. Such contraction rate is determined by current national standard "Thickness direction property of steel plate" GB 50313.

3.9.6 In construction of the tie-columns and core-columns of masonry structures, and framed columns which seismic-brick-walls of a masonry building with bottom-frame, the

masonry walls shall be laid before the such concrete column is cast.

3.10 Seismic response observation system of buildings

3.10.1 The high-rise buildings with heights exceeding 160m, 120m and 80m, for the seismic fortification intensity 7, 8 and 9 respectively, shall be installed the seismic response observation system, so the object design shall leave rooms for the observation equipment and relevant circuits.

Chapter 4 Site, Subsoil and Foundation

4.1 Site

4.1.1 When selecting a constructional place, the identification as favorable plat, unfavorable plat or hazardous plat to seismic fortification shall be made according to Table 4.1.1.

Table 4.1.1 Identification of plat

Plats	Geological, topogrphical and geomorphical description
Favorable to seismic fortification	Stable rock, stiff soil, or dense and homogeneous medium-stiff soil, which are in a wide-open area.
Unfavorable to seismic fortification	Soft soil, liquefied soil, stripe-protruding spur, Lonely tall hill, non-rocky steep slop, river bank or boundary of slop, Soil stratification having obviously heterogeneous distribution in plane and cause of formation, lithology, and state (such as abandoned and filled river beds, fractured zone of fault, and hidden swamp, creek, ditch and pit, as well as subsoil formatted with excavated and filled.)
Hazardous to seismic fortification	Places where landslide, avalanche, subsidence, ground fissure and debris flow are liable to occur during the earthquake, and where ground dislocation may be occur at active faulted zone.

4.1.2 The site class of building structures shall be classified according to the equivalent shear-wave velocity of soil and thickness of site overlying layer as guideline.

4.1.3 The measure of shear-wave velocity of soil shall comply with following requirements:

1 At the stage of primary investigation, for large areas of same geologic units, the number of borings for the shear-wave-velocity tests shall be 1/3 to 1/5 of the controlled boring numbers. For the mountain valleys or mountain slopes, such number may be reduced apropos but should not be less than 3.

2 At the stage of detailed investigation, for every building, the number of borings for shear-wave-velocity tests should not be less than 2; when the data varies significantly, the number can be increased apropos. In the case of close-set tall building groups in one subzone, which are built at the same geologic unit, such number may be reduced apropos but shall not be less than one for each tall building.

3 For buildings assigned to Category D or to Category C with less than 10 stories and no more than 30m in height, when the shear-wave velocity data are not available, appropriate shear-wave velocity values are permitted to be estimated by used the known geologic conditions. In these cases, the type of soil may be classified according to Table 4.1.3 by the geotechnical description of the soil, and then the shear-wave velocity of each soil layer may be estimated within the range as per set in Table 4.1.3 based on the local experiences.

Table 4.1.3 Classification of soil and range of shear-wave velocity

Type of soil	Geotechnical description	Shear-wave velocity of soil layer (m/s)
Stiff soil or rock	Stable rock, dense detritus	$v_s>500$
Medium-stiff soil	Medium dense or slightly dense detritus, dense or medium-dense gravel, coarse or medium sand, cohesive soil and silt with $f_{ak}>200$kPa	$500\geqslant v_s>250$
Medium-soft soil	Slightly dense gravel, coarse or medium sand, fine and mealy sand other than that which is loose, cohesive soil and silt with $f_{ak}\leqslant 200$kPa, fill land with $f_{ak}>130$kPa	$250\geqslant v_s>140$
Soft soil	Muck and mucky soil, loose sand, new alluvial sediment of cohesive soil and silt, fill land with $f_{ak}\leqslant 130$kPa	$v_s\leqslant 140$

Note: f_{ak} is the reference value of load-bearing capacity of soil; v_s is the shear-wave velocity.

4.1.4 The thickness of site overlaying layer shall be determined according to the following provisions:

1 In generally, the thickness of sife overlaying layer shall be determined according to the distance from the ground surface to a soil-layer level, under which the shear-wave velocity is more than 500m/s.

2 For a soil layer, which depth lower than 5 m underground and the shear-wave velocity is more than 2.5 times of that in above this soil layer and is not less than 400m/s, then the thickness of site overlaying layer may be adopted the distance from the ground surface to this layer.

3 The lone-stone and lenticular-soil with a shear-wave velocity greater than 500m/s shall be deemed the same as surrounding soil profile.

4 The hard volcanic inter-bedded rock in the soil profile shall be deemed as rigid body and its thickness shall be deducted from the thickness of site overlaying layer.

4.1.5 The equivalent shear-wave velocity of the soil profile shall be calculated according to the following equation:

$$v_{se}=d_0/t \tag{4.1.5-1}$$

$$t=\sum_{i=1}^{n}(d_i/v_{si}) \tag{4.1.5-2}$$

where: v_{se}——equivalent shear wave velocity, in m/s.

d_0——calculated depth, in m; it shall be taken as the minor of both the overlaying thickness and 20m.

t——the transmission time of the shear-wave from the ground surface to the calculated depth.

d_i——the thickness of the i-th soil layer within the range of calculated depth, in m.

v_{si}——the shear-wave velocity of the i-th soil layer within the calculated depth, in m/s.

n——number of soil layers within the range of calculated depth.

4.1.6 The construction sites shall be classified as four Site-classes defined in Table 4.1.6 depend on the equivalent shear-wave velocity and the overlaying thickness of soil profile. Only the values of the reliable shear-wave velocity and/or the overlaying thickness are near to the dividing line of the listed site values in Table 4.1.6, the design characteristic period value shall be permitted to determined by the interpolation method in calculating the seismic action.

Table 4.1.6 Overlaying thickness of soil profile for site classification, in m

Equivalent shear-wave velocity (m/s)	Site-classes			
	Ⅰ	Ⅱ	Ⅲ	Ⅳ
$v_{se}>500$	0			
$500\geqslant v_{se}>250$	<5	≥5		
$250\geqslant v_{se}>140$	<3	3~50	>50	
$v_{se}\leqslant 140$	<3	3~15	>15~80	>80

4.1.7 When seismogenic faults exist within the site, the impact assessment on the project for the fault shall be made, which shall comply with the following requirements:

1 If one of the following conditions can be satisfied, the impact on the building structures for the fault motion may be neglected:

1) For Intensity 6 and 7;

2) Not Holocene active faults;

3) For Intensity 8 and 9, the depth of overlaying soil for the hidden pre-Quaternary fault is greater than 60m and 90m respectively.

2 In the event that the situation does not conform to the provisions in item 1 of this clause, the main fault zone shall be avoided in the selection of site. The distance of avoidance should not be less than the minimum distance of avoidance in accordance with Table 4.1.7.

Table 4.1.7 Minimum distance of avoidance for seismogenic fault

Intensity	Fortification Category of buildings			
	A	B	C	D
8	Specification	300m	200m	—
9	Specification	500m	300m	—

4.1.8 When buildings assigned to Category C or A and B need to be constructed in unfavorable plats, which are such as stripe-protruding spur, lonely tall hill, non-rocky steep slop, river banks or boundary of slopes, that shall conform to follows:

Ensuring the stability of those buildings under earthquake. The amplification of design seismic parameters on the unfavorable plat shall also be taken into consideration, so that the maximum value of seismic influence coefficient shall be multiplied by the amplifying factor. The value of amplifying factor may be determined based on the actual condition of the unfavorable plat, but not exceed 1.6.

4.1.9 The geological investigation of the site shall be carried out as follows:

To classify the plats that are favorable, unfavorable or hazardous to seismic fortification, to provide the Site-classes of soil profile and to assess geotechnical stability under earthquake (whether landslide, avalanche, liquefaction or ground subsidence would occur) according to the actual condition.

For buildings that the time-history analysis is necessary, the relevant dynamic parameters and the overlaying depth shall also be provided as required by designer.

4.2 Natural subsoil and foundations

4.2.1 For the following buildings, the seismic bearing capacity check may not be carried out for the natural subsoil and foundation:

1 Masonry building structures;

2 Buildings without soft cohesive soil in the main load-bearing layer of the subsoil:

1) The ordinary single-story factory buildings and single-story spacious houses;

2) The ordinary civil framed buildings not more than 8 stories and 25m in height;

3) Multi-story framed factory buildings with foundation load equivalent to point 2).

3 Buildings that seismic check for the upper-structure is non-necessary in accordance with this code.

Notes: soft cohesive soil layer refers to the soil layer, which the soil bearing capacity characteristic values are less than 80, 100 and 120 kPa for Intensity 7, 8 and 9 respectively.

4.2.2 When seismic check needs to be done for natural subsoil foundations, the characteristic combination of seismic effect shall be adopted, and the seismic soil bearing capacity shall be determined by the soil bearing capacity characteristic value to multiply with the seismic adjusting factor of soil bearing capacity.

4.2.3 The seismic soil bearing capacity shall be calculated according to the following equation:

$$f_{aE} = \zeta_a f_a \tag{4.2.3}$$

where: f_{aE}——seismic soil bearing capacity.

ζ_a——the seismic adjusting factor of soil bearing capacity, which shall be taken in accordance with Table 4.2.3.

f_a——soil bearing capacity characteristic values after depth and width adjustment, that shall be determined according to the current national standard "Code for foundation design of buildings" GB50007.

Table 4.2.3 Seismic adjusting factor of soil bearing capacity

Name and characteristic of rock and soil	ζ_a
Rock, dense detritus, dense gravel, course and medium sand, cohesive soil and silt with $f_{ak} \geqslant 300$kPa	1.5
Medium dense and slightly dense detritus, medium dense gravel, course and medium sand, dense and medium dense fine and mealy sand, cohesive soil and silt with 150kPa$\leqslant f_{ak} <$ 300kPa, and hard loess	1.3
Slightly dense fine and mealy sand; cohesive soil and silt with $100 \leqslant f_{ak} < 150$, plastic loess	1.1
Mud, silty soil, loose sand, fill land, newly piled loess and streamed loess	1.0

4.2.4 When checking the vertical seismic bearing capacity of natural subsoil, the mean pressure of foundation bottom and maximum pressure on the foundation edge according to the seismic effect characteristic combination shall conform to the requirement of the following equation:

$$p \leqslant f_{aE} \tag{4.2.4-1}$$

$$p_{max} \leqslant 1.2 f_{aE} \tag{4.2.4-2}$$

where: p——mean pressure on foundation bottom according to seismic effect characteristic combination

p_{max}——maximum pressure on the foundation bottom edge according to seismic effect characteristic combination.

For high-rise buildings with height-width ratio greater than 4, the foundation bottom shall have no tensioned stress under the earthquake; for other buildings, the area of zero stress zone between the subsoil and the foundation bottom shall not exceed 15% of the foundation bottom area.

4.3 Liquefaction and soft subsoil

4.3.1 For Intensity 6, the consequences of liquefaction of the saturated soil (loess not included) and the mitigation measures need not be considered for the ordinary buildings, but buildings assigned to Category B which is sensitive to the settlement caused by liquefaction, that may be carried out as intensity 7. For the buildings assigned to Category B, the assessment of consequences of the liquefaction and the mitigation measures may be considered by referring to the original intensity for Intensity 7 to 9.

4.3.2 For the subsoil inside which saturated sand and saturated silt(loess not included) exist, the assessment of liquefaction shall be made, except where is Intensity 6; when there is liquefaction, corresponding mitigation measures shall be taken depend on the Fortification Category, the liquefaction grade and other actual condition.

4.3.3 If one of the following condition is satisfied, the saturated sand and saturated silt (loess not included) may be primarily discriminated as non-liquefaction or consequences of liquefaction need not be considered:

1 The geochron of soil is Epipleistocene of Quaternary (Q3) or earlier, it may be discriminated as non-liquefied soils for Intensity 7 to 9.

2 The percentage of contain of clay particles (diameter less than 0.005mm) in silt is not less than 10%, 13% and 16% for Intensity 7, 8 and 9 respectively, it may be discriminated as non-liquefied soils.

Note: The clay particles contain shall be determined by use of sodium hexametaphosphate as the dispersant. When other methods are be used, it shall be correspond conversed according to relative provisions.

3 For buildings with natural subsoil, the consequences of liquefaction need not be considered when the thickness of the overlaying non-liquefiable soils and the elevation of groundwater table comply with one of following conditions:

$$d_u > d_0 + d_b - 2 \qquad (4.3.3\text{-}1)$$

$$d_w > d_0 + d_b - 3 \qquad (4.3.3\text{-}2)$$

$$d_u + d_w > 1.5d_0 + 2d_b - 4.5 \qquad (4.3.3\text{-}3)$$

where: d_w——elevation of groundwater table (in m), for which the mean annual highest elevation during the reference period should be used, or the annual highest elevation in recent years may also be used.

d_u——thickness of the overlaying non-liquefiable layer (in m), in which the thickness of mud and silt seams should be deduced.

d_b——foundational depth (in m), when it is less than 2m, shall equal 2m.

d_0——reference depth of liquefaction soil (in m), it may be taken as Table 4.3.3.

Table 4.3.3 Reference depth of liquefaction soil (m)

Type of saturated soil	Intensity 7	Intensity 8	Intensity 9
Silt	6	7	8
Sand	7	8	9

4.3.4 When the sequence discriminated liquefaction need be considered base on the primary discrimination, the standard penetration tests shall be performed, in which the discriminated depth shall be taken as 15m underground, but shall be taken as 20m underground for the pile foundation or for foundation buried depth greater than 5m.

When the measured value of standard penetration resistance (in blow-number, and bar-length-modification is not included) is less than the critical value of that, the saturated soil shall be discriminated as liquefied soil; and other methods, if already proved successful, may also be used.

Within the depth of 15m underground, the critical value of standard penetration resistance (in blow-number) for liquefaction discrimination may be calculated according to the following equation:

$$N_{cr} = N_0[0.9 + 0.1(d_s - d_w)] \sqrt{3/\rho_c} \quad (d_s \leqslant 15) \qquad (4.3.4\text{-}1)$$

Within the depth of 15～20m underground, the critical value of standard penetration

resistance (in blow-number) for liquefaction discrimination may be calculated according to the following equation:

$$N_{cr} = N_0(2.4 - 0.1d_w)\sqrt{3/\rho_c} \quad (15 \leqslant d_s \leqslant 20) \qquad (4.3.4\text{-}2)$$

where: N_{cr}——critical value of standard penetration resistance (in blow-number) for liquefaction discrimination;

N_0——reference value of standard penetration resistance (in blow-number) for liquefaction discrimination, it shall be taken from Table 4.3.4;

d_s——depth of standard penetration for saturated soil (in m);

ρ_c——percentage of clay particle content; when it is less than 3% or when the soil is sand, the value shall equal 3%.

Table 4.3.4 Reference value of standard penetration resistance (in blow-number)

Design seismic group	Intensity 7	Intensity 8	Intensity 9
Group 1st	6 (8)	10 (13)	16
Group 2nd or 3rd	8 (10)	12 (15)	18

Note: Values in the brackets are used for the design basic acceleration of ground motion is 0.15g and 0.30g.

4.3.5 For the subsoil with liquefied soil layers, the level and thickness of soil layer shall be explored and the liquefaction index shall be calculated by the following equations, and then the liquefaction grades shall be comprehensively classified according to Table 4.3.5:

$$I_{lE} = \sum_{i=1}^{n}\left(1 - \frac{N_i}{N_{cri}}\right)d_i W_i \qquad (4.3.5)$$

where: I_{lE}——liquefaction index.

n——total number of standard penetration test points in each bore within the discriminated depth under the ground surface.

N_i, N_{cri}——measured value and critical value of standard penetration resistance (in blow-number) at the i-th point respectively, when the measured value is greater than the critical value, shall take as equal critical value.

d_i——thickness of soil layer (in m) at the i-th point, it may be taken as half of the difference in depth between the upper and lower neighboring standard penetration test points; but the upper point level shall not be less than the elevation of groundwater table, and the lower point level not greater than the liquefaction depth.

W_i——weighted function value of the i-th soil layer (in m^{-1}), which is considered the effect of the layer portion and level of the unit soil layer thickness. For discrimination depth is 15m underground, such value is equal 10 when the depth of the midpoint of the layer is less than 5m, is zero when it equals 15m, and linear interpolation when it is between 5 m and 15m. For discrim-

ination depth is 20m underground, such value is equal 10 when the depth of the midpoint of the layer is less than 5m, is zero when it equals 20m, and linear interpolation when it is between 5m and 20m.

Table 4.3.5 Grade of liquefaction

Grades of liquefaction	Light	Moderate	Serious
Liquefaction index for discrimination depth is 15m	$0<I_{lE}<5$	$5<I_{lE}<15$	$I_{lE}>15$
Liquefaction index for discrimination depth is 20m	$0<I_{lE}<6$	$6<I_{lE}<18$	$I_{lE}>18$

4.3.6 For the even liquefied soil layer, the liquefaction mitigation measures shown in Table 4.3.6 may be selected; and the effect of the gravity for structural system may also be considered to adjust the above liquefaction mitigation measures.

The liquefied soil layer, which mitigation measure such as ground stabilization has not made, should not be used as bearing stratum of the footings.

Table 4.3.6 Liquefaction mitigation measures

Building category	Grades of liquefaction subsoil		
	Light	Moderate	Serious
B	Differential settlement due to liquefaction to be partially eliminated, or adjusted design of foundation and structural systems	Differential settlement due to liquefaction to be completely eliminated, or partially to be eliminated together with adjusted design of foundation and structural systems	Differential settlement due to liquefaction to be completely eliminated
C	Adjusted design of foundation and structural systems, or mitigation measures may not be taken	Adjusted design of foundation and structural systems, or other more strict measures may be taken	Differential settlement due to liquefaction to be completely eliminated, or partially to be eliminated together with adjusted design of foundation and structural system
D	Mitigation measures may not be taken	Mitigation measures may not be taken	Adjusted design of foundation and structural systems, or other low-cost measures may be taken

4.3.7 The measures to eliminate completely the differential settlement due to liquefaction shall comply with the following requirements:

1 When pile foundation is used, the length (pile-tip not included) of the pile driven into the stable soil layer below the liquefaction depth shall be determined by calculation. And for detritus, gravel, coarse and medium sand, stiff cohesive soil, and dense silt, such length shall not be less than 500 mm; for other non-rocky soil, that should not be less than 1.5 m also.

2 When deep foundations are used, the depth of the foundational bottom embedded in the stable soil layer below the liquefaction depth shall not be less than 500mm.

3 When a compaction method is used for strengthening (e.g. vibrating impact, vibrating compaction, sand or detritus pile compaction, and strong ramming), compaction

shall be carried out down to the lower margin of liquefaction depth. After strengthen the measured value of the standard penetration resistance (in number) of soils shall be greater than the corresponding critical value of provision in Clause 4.3.4 of this code.

4 Removal of all the liquefaction soil layers, and then fills non-liquefiable soils.

5 When the compaction method or removal of liquefiable soil methods are adopted, the strengthened width outside the foundation edge shall exceed 1/2 of its depth under the foundation bottom; moreover, the width shall not be less than 1/5 of the foundation width.

4.3.8 Measures to eliminate partially the differential settlement due to liquefaction shall comply with the following requirements:

1 The compacted or removal shall be carried out to a depth so that the liquefaction index of the subsoil shall be not greater than 4 for discriminated depth 15m and than 5 for discriminated depth 20m. For individual footings and strip footings, such depth shall also not be less than the reference depth of liquefaction soil measured from the bottom of the footing or the width of the foundation, which is greater.

2 In the range of the above depth, the liquefaction soil layers shall be strengthened by compaction, so that the measured value of the standard penetration resistance (in blow-number) of the compacted soil layer should be more than the corresponding critical value of provision in Clause 4.3.4 of this code.

3 The strengthened width outside the foundation shall conform to the requirements in Item 5 of Clause 4.3.7 of this section.

4.3.9 For design of the foundation and the structural system to reduce the influence of liquefaction, the following mitigation measures can be taken based on comprehensive considerations:

1 Select the appropriate buried depth of foundation.

2 Adjust the foundation bottom area to reduce the eccentricity of the foundation.

3 Increase the integrality and rigidity of the foundation, for example, the use of box-shape foundation or raft footing, the use of cross-strip footing, adding foundation ring beams or connecting beams.

4 Decrease the load, increase the integrality, uniformity and symmetry of the structural system, install rational settlement joints, and avoid the use of a structural configuration that is vulnerable to differential settlement.

5 At locations where pipelines pass through the building, sufficient space shall be left beforehand for the pipelines, or flexible connections shall be used.

4.3.10 For abandoned and filled river beds, modern riverside and beaches where the liquefaction grade is moderate or serious, when there is the possibility of lateral expansion or now-sliding due to liquefaction, no permanent buildings should be constructed within 100 m of the normal waterline. Otherwise, anti-sliding checks must be carried out, and anti-sliding measures for the earth and anti-cracking measures for the structure must be taken.

Note: the normal waterline may also be adopted according to the mean annual highest level during the reference period or the annual highest level in recent years.

4.3.11 If the main load-bearing soil of building inside which the soft cohesive soil or collapsible loess exists, the measures such as adoption of pile foundation, strengthening of subsoil or those measures in Clause 4.3.9 of this section, shall be taken based on the comprehensive consideration the actual conditions. And corresponding measures may also be taken based on the estimation of the settlement under earthquake.

4.4 Pile foundation

4.4.1 For the following buildings, the pile foundation, with low pile caps and mainly resisting vertical load but without liquefaction soil, mud, silt soil or backfill soils with characteristic value of load-bearing capacity not greater than 100 kPa, may not be checked for the seismic bearing capacity:

1 Buildings as specified in Item 1 and 3 of Clause 4.2.1 of this chapter.

2 The following buildings for Intensity 7 and 8:

1) Ordinary single-story factory buildings or single-story spacious buildings;

2) Ordinary framed civil buildings with not more than 8 stories and 25m in height;

3) Multi-story framed factory with foundation load equivalent to that in point 2).

4.4.2 The seismic check of pile foundation with low pile caps in non-liquefaction soil shall comply with the following requirements:

1 The seismic bearing capacity characteristic values in the vertical and horizontal direction of a single pile may both increase by 25% than that is required for non-seismic designs.

2 When the dry density of tamped backfill around the pile cap satisfying the requirements for backfill provided in "Code for design of building foundations", the pile and its front backfill soil may together resist the horizontal seismic action. However, the friction between the bottom surface and the subsoil shall not be taken into account.

4.4.3 The seismic check of pile foundation with low pile cap in the liquefaction soil shall comply with the following requirements:

1 For the ordinary shallow foundations, the resistance of the pile cap surrounding soil and the horizontal seismic-resistance of the rigid ground floor should not be taken into account.

2 When there is non-liquefaction soil or non soft soil layer with the thickness of 1.5m and 1.0m in the upper and lower pile cap, the seismic check may be carried out according to the following two cases, and the design shall be carried out according to unfavorable conditions:

1) The pile designed to resist all the seismic action, and the bearing capacity of the pile shall be determined from Clause 4.4.2 of this section, but the friction and

the horizontal resistance of the pile in liquefaction soil shall all multiplied by the reduction factor defined in Table 4.4.3.

Table 4.4.3 Reduction factor for the liquefaction effect of soil layer

Ratio of measured value of standard penetration resistance (in blow-number) to critical value of that	depth d_s(m)	Reduction factor
≤0.6	$d_s \leqslant 10$	0
	$10 < d_s \leqslant 20$	1/3
>0.6~0.8	$d_s \leqslant 10$	1/3
	$10 < d_s \leqslant 20$	2/3
>0.8~1.0	$d_s \leqslant 10$	2/3
	$10 < d_s \leqslant 20$	1

2) The seismic action shall be determined according to 10% of the maximum value of horizontal seismic influence coefficient, which provided in this code. And the pile bearing capacity shall be determined from Clause 4.4.2 of this section, but all of the friction, in the liquefaction soil and in the non-liquefaction soil within the depth of 2m under the pile cap, shall be deducted.

3 For hammered and other driving piles, when the pile center-to-center mean spacing is 2.5 to 4 times of pile diameters and the pile group is not less than 5×5, the compacting effect for driving piles and the favorable affect of the pile on restricting the soil deformation may be taken into consideration. When the standard penetration resistance (in blow- number) of soil around pile meet the requirements for non-liquefaction, the bearing capacity of single pile may not be reduced, however, when checking the capacity of the pile-tip bearing stratum, the diffusion angel outside the pile group shall be taken as zero. The standard penetration resistance (in blow-number) on the soil around the pile after pile driving should be determined by tests, and may also be determined according to the following equation:

$$N_1 = N_p + 100\rho(1 - e^{-0.3N_p}) \qquad (4.4.3)$$

where: N_1——the standard penetration resistance (in blow-number) after pile driving;

ρ——ratio of the area conversion for piles to soils,

N_p——the standard penetration resistance (in blow-number) before pile driving.

4.4.4 The area around the pile cap in liquefaction soils should be tamped with non- liquefiable soil; if sand or silty-soil are used, the standard penetration resistance (in blow- number) of the soil layer shall not be less than the critical value for liquefaction of provisions in Clause 4.3.4 of this chapter.

4.4.5 The reinforced range of the pile in liquefaction soil shall cover from the top of the pile to depth under the liquefaction soil, comply with the required depth for eliminating completely the different settle ment due to liquefaction. In within, the longitudinal reinforcement shall be the same as the top of the pile, the stirrups shall be densified.

4.4.6 In plats where exist lateral movement due to liquefaction, the pile foundation with a distance less than 100m from the normal waterline, besides satisfying other provisions set forth in this section, shall also take the lateral-force due to soil-flow into consideration. Moreover, the area bearing lateral-force shall be calculated according to the distance between the both of side-piles.

Chapter 5 Seismic action and seismic checking for structures

5.1 General

5.1.1 Seismic action of every building structure shall be considered and complied with the following principles:

1 In generally, horizontal seismic actions may be considered and checked separately along the two orthogonal major axial directions of the building structure; and that horizontal seismic action shall be resisted by all of the corresponding direction lateral-force-resisting members.

2 Structures having the oblique direction lateral-force-resisting members and the oblique angel to major orthogonal axes is greater than 15°, the horizontal seismic action along the direction of each lateral-force-resisting member shall be considered respectively.

3 Structures having obviously asymmetric mass and stiffness distribution, the torsion effects caused by both two orthogonal horizontal direction seismic action shall be considered; and other structures, it is permitted that a simplified method, such as adjusting the seismic effects method, to consider their seismic torsion effects.

4 Large-span structures and long-cantilevered structures for Intensity 8 or 9, and tall buildings for Intensity 9, vertical seismic action shall be considered.

Note: For building structures adopting seismic-isolated designs for Intensity 8 and 9, the vertical seismic action shall be calculated according to relevant provisions in this code.

5.1.2 The following methods shall be taken for seismic computation of any building structure:

1 For structures, which is not higher than 40m with deformations predominantly due to shear and a rather uniform distribution of mass and stiffness in elevation, or for structures modeled as a single-mass system, a simplified method, such as the base shear method, may be used.

2 For building structures other than those as stated in the above clause, the response spectrum method for modal analysis should be used.

3 For buildings having extremely irregular configuration, buildings assigned Fortification Category A, and tall buildings within the height range given in Table 5.1.2-1, a time-history analysis method under frequently earthquake shall be used as an additional computation. And then the greater value between the average value of the time-history calculation results and the result of the response spectrum method may be adopted.

When the time-history method is adopted in the analysis, at least 2 sets of strong earthquake records and 1 set of artificial acceleration time-history curve shall be selected based on

the intensity, the Design seismic group and Site-class. Their average seismic influence coefficient curve shall be in conformity with the seismic influence coefficient curve when adopting the response spectrum method; the maximum value for its acceleration time-history may be adopted according to Table 5.1.2-2. When adopting elastic time-history analyzing method, the structure base shear force obtained from each time-history curve shall not be less than 65% of that from the response spectrum method, and the average value from several time-history curve shall not be less than 80% of that from the response spectrum method.

Table 5.1.2-1 Building height range for using time-history analysis method

Intensity and Site-class	Range of building height(m)
Intensity 7, Intensity 8 with Site-class Ⅰ、Ⅱ	>100
Intensity 8 with Site-class Ⅲ、Ⅳ	>80
Intensity 9	>60

Table 5.1.2-2 Maximum values for the seismic acceleration of ground motion used in time-history analysis(cm/s²)

Seismic action	Intensity 6	Intensity 7	Intensity 8	Intensity 9
Frequently earthquake	18	35 (55)	70 (110)	140
Rarely earthquake		220 (310)	400 (510)	620

Note: Values in the brackets are used that the design basic acceleration of ground motion is 0.15g and 0.30g respectively.

4 When calculating the deformation of the structure under rarely earthquake, the simplified elasto-plastic analyzing method or elasto-plastic time-history analyzing methods shall be adopted in accordance with the provisions in Section 5.5 of this chapter.

Note: The seismic-isolation and the energy-dissipating design of the building structures shall adopt the calculation method provided in Chapter 12 of this Code.

5.1.3 In the computation of seismic action, the representative value of gravity load of the building shall be taken as the sum of characteristic values of the weight of the structure and members plus the combination values of variable loads on the structure. The combination coefficients for different variable loads shall be taken from Table 5.1.3.

Table 5.1.3 Combination coefficient

Type of variable load	Combination coefficient
Snow load	0.5
Dust load on the roof	0.5
Active load on the roof	Not considered
Active load on the floor, calculated according to actual state	1.0

Table 5.1.3 (continued)

Type of variable load		Combination coefficient
Active load on the floor, calculated according to equivalent uniform load	Library, Archives	0.8
	Other civil buildings	0.5
Gravity for hanging object of crane	Cranes with hard hooks	0.3
	Cranes with flexible hooks	Not considered

Note: When the hanging weight of crane with hard hooks is bigger, the combination coefficient shall be adopted according to the actual condition.

5.1.4 Seismic influence coefficient of a building structure shall be determined according to the Intensity, Site-class, Design seismic group, and natural period and damping ratio of the structure. The maximum value of horizontal seismic influence coefficient shall be taken from Table 5.1.4-1; the characteristic period shall be taken as Table 5.1.4-2 according to Site-class and Design seismic group, that shall be increased 0.05s for rarely earthquake of Intensity 8 and 9.

Notes: 1. The seismic influence coefficient shall be studied specifically for building structures with natural period greater than 6.0s;

2. Only the cities that have established seismic fortification zonation, it shall be permitted to adopt correspondent seismic influence coefficient according to the approved design seismic parameters.

Table 5.1.4-1 Maximum value of horizontal seismic influence coefficient

Earthquake influence	Intensity 6	Intensity 7	Intensity 8	Intensity 9
Frequently earthquake	0.04	0.08 (0.12)	0.16 (0.24)	0.32
Rarely earthquake	—	0.50 (0.72)	0.90 (1.20)	1.40

Note: the values in the brackets are separately used for where the design basic seismic acceleration is 0.15g and 0.30g.

Table 5.1.4-2 Characteristic period value(s)

Design earthquake group	Site-class 1	Site-class 2	Site-class 3	Site-class 4
1 st Group	0.25	0.35	0.45	0.65
2 nd Group	0.30	0.40	0.55	0.75
3 rd Group	0.35	0.45	0.65	0.90

5.1.5 The damping adjusting and forming parameters on the building seismic influence coefficient curve (Fig.5.1.5) shall comply with the following requirements:

1 The damping ratio of building structures shall select 0.05 except otherwise provided, the damping adjusting coefficient of the seismic influence coefficient curve shall select 1.0, and the coefficient of shape shall conform to the following provisions:

1) Linear increase section, whose period(T)is less than 0.1 s;

2) Horizontal section, whose period from 0.1s thought to characteristic period, shall select the maximum value (α_{max});

3) Curvilinear decrease section, whose period from characteristic period thought to 5

times of the characteristic period, the power index(γ)shall choose 0.9.

4) Linear decrease section, whose period from 5 times characteristic period thought to 6s, the adjusting factor of slope(η_1)shall choose 0.02.

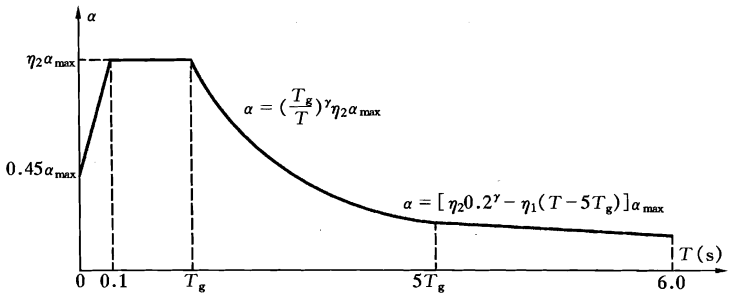

Figure 5.1.5 Seismic influence coefficient curve

2 When the damping ratio of building structures is not equal to 0.05 according to relevant provisions, the damping adjusting and forming parameters on the seismic influence coefficient curve shall comply with the following requirements:

1) The power index of the curvilinear decrease section shall be determined according to the following equation:

$$\gamma=0.9+\frac{0.05-\zeta}{0.5+5\zeta} \tag{5.1.5-1}$$

where: γ——the power index of the curvilinear decrease section;

ζ——the damping ratio.

2) The adjusting factor of slope for the linear decrease section shall be determined from following equation:

$$\eta_1=0.02+(0.05-\zeta)/8 \tag{5.1.5-2}$$

where: η_1——the adjusting factor of slope for the linear decrease section, when it is less than 0, shall equal 0.

3) The damping adjustment factor shall be determined according to the following equation:

$$\eta_2=1+\frac{0.05-\zeta}{0.06+1.7\zeta} \tag{5.1.5-3}$$

where: η_2——the damping adjustment factor, when it is smaller than 0.55, shall equal 0.55.

5.1.6 The seismic check of the building structure shall comply with the following requirements:

1 Only the buildings assigned to fortification intensity 6 (except higher buildings built on Site-class Ⅳ), unfired earth house and wood house, the seismic checking of cross section of structural members shall be permitted to not carry out. However the relevant seismic detail requirements for those buildings shall be satisfied.

2 For building structures other than those as stated in point 1 of this clause, seismic checking of cross section of structural members shall be carried out under frequently earthquakes.

Note: The seismic check of building structures adopting seismic-isolation design shall comply with relevant provisions.

5.1.7 For structures conforming to the provisions in Section 5.5 of this chapter, besides carrying out cross section seismic check, corresponding deformation check shall also be carried out.

5.2 Calculation of horizontal seismic action

5.2.1 When the base shear force method is used, only one degree of freedom may be considered for each story; the characteristic value of ho rizontal seismic action of the structure shall be determined by the following equations (Fig.5.2.1):

$$F_{Ek} = \alpha_1 G_{eq} \tag{5.2.1-1}$$

$$F_i = \frac{G_i H_i}{\sum_{j=1}^{n} G_j H_j} F_{Ek}(1 - \delta_n)(i = 1,2\cdots n) \tag{5.2.1-2}$$

$$\Delta F_n = \delta_n F_{Ek} \tag{5.2.1-3}$$

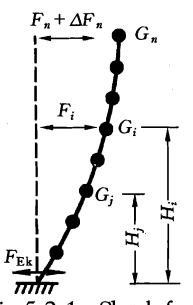

Fig.5.2.1 Sketch for Computation of the horizontal seismic action

where: F_{Ek}——characteristic value of the total horizontal seismic action of the structure.

α_1——horizontal seismic influence coefficient corresponding to the fundamental period of the structure, which shall be determined by using Clause 5.1.4. For multi-story masonry buildings and multi-story brick buildings with bottom-frames or inner-frames, the maximum value of horizontal seismic influence coefficient should be taken.

G_{eq}——equivalent total gravity load of a structure. When the structure is modeled as a single-mass system, the representative value of the total gravity load shall be used; and when the structure is modeled as a multi-mass system, the 85% of the representative value of the total gravity load may be used.

F_i——characteristic value of horizontal seismic action applied on mass i-th.

G_i, G_j——representative values of gravity load concentrated at the masses of i-th and j-th respectively, which shall be determined by Clause 5.1.3.

H_i, H_j——calculated height of mass i-th and j-th from the base of the building respectively.

δ_n——additional seismic action factors at the top of the building; for multi-story reinforced concrete buildings, it may be taken using Table 5.2.1; for multi-story brick buildings with inner-frames, a value of 0.2 may be used; no need to con-

sider for other buildings.

ΔF_n——additional horizontal seismic action applied at top of the building.

Table 5.2.1 Additional seismic action factors at top of the building

T_g(s)	$T_1 > 1.4T_g$	$T_1 \leqslant 1.4T_g$
$\leqslant 0.35$	$0.08T_1+0.07$	0.0
$>0.35\sim0.55$	$0.08T_1+0.01$	0.0
>0.55	$0.08T_1-0.02$	0.0

Not: T_1 is the fundamental period of the structure.

5.2.2 When the response spectrum method is used for model analysis, if the torsion effect of a structure is not considered, the seismic action and its effect may be calculated according to the following requirements:

1 The characteristic value of horizontal seismic action on mass i-th of the structure, corresponding to j-th mode, shall be determined by the following equations:

$$F_{ji} = \alpha_j \gamma_j X_{ji} G_i \quad (i = 1,2,\cdots n,\ j = 1,2,\cdots m) \tag{5.2.2-1}$$

$$\gamma_j = \sum_{i=1}^{n} X_{ji} G_i \Big/ \sum_{i=1}^{n} X_{ji}^2 G_i \tag{5.2.2-2}$$

where: F_{ij}——characteristic value of horizontal seismic action of mass i-th corresponding to mode j-th.

α_j——seismic influence coefficient corresponding to the natural period of mode j-th of the structure, determined by clause 5.1.4 of this chapter.

γ_j——mode participation factor of mode j-th.

2 Total effect of the horizontal seismic action (bending moment, shear, axial force, or deformation) shall be determined by the following equation:

$$S_{Ek} = \sqrt{\Sigma S_j^2} \tag{5.2.2-3}$$

where: S_{Ek}——characteristic value of horizontal seismic effect.

S_j——effect caused by the horizontal seismic action of mode j-th, and only the first two or three modes may be taken. When the fundamental natural period is greater than 1.5s, or the height-width-ratio of the building exceeds 5, number of modes used shall be increased in the computation.

5.2.3 When considering the torsion effect of horizontal earthquake actions, structural seismic action and its effect shall be calculated according to the following requirements;

1 When no coupled torsion calculation is to be carried out for regular structures, the seismic effect of the two side trusses parallel to the earthquake action direction shall be multiplied by the amplifying factor. In one word, the short side may select the factor 1.15, and the longer side may choose the factor 1.05; when the torsion rigidity is smaller, it is appro-

priate to select the factor no less than 1.3.

2 When calculating according to coupled torsion method, three degrees of freedom may be selected for each floor, including two orthogonal horizontal deformations and a rotation around the vertical axis, and the seismic action and its effect shall be calculated according to the following equations. When there are sufficient reasons, other simplified methods may also be used in determining the seismic effect.

1) The horizontal seismic action characteristic value in the floor i-th for mode j-th of natural vibration of structure shall be determined according to the following equation:

$$F_{xji} = \alpha_j \gamma_{tj} X_{ji} G_i$$

$$F_{yji} = \alpha_j \gamma_{tj} Y_{ji} G_i \quad (i = 1,2,\cdots n, j = 1,2,\cdots m)$$

$$F_{tji} = \alpha_j \gamma_{tj} r_i^2 \varphi_{ji} G_i \tag{5.2.3-1}$$

where: $F_{xji}, F_{yji}, F_{tji}$——the seismic action characteristic value of floor i-th for mode j-th of natural vibration of structure in direction of x, y and rotation respectively.

X_{ji}, Y_{ji}——the horizontal relative displacement of the center of floor i-th for mode j-th of natural vibration of structure in direction of x, y respectively.

φ_{ji}——relative rotation angle of floor i-th for mode j-th of natural vibration of structure.

r_i——the rotating radius of floor i-th, which is the square root of that the rotating moment of inertia around the center of floor i-th divided by the mass of this floor.

γ_{tj}——mode participation factor of mode j-th considering rotation effect, that may be determined in accordance with the following equation:

When only the seismic action in x direction is considered

$$\gamma_{tj} = \sum_{i=1}^{n} X_{ji} G_i \Big/ \sum_{i=1}^{n} (X_{ji}^2 + Y_{ji}^2 + \varphi_{ji}^2 r_i^2) G_i \tag{5.2.3-2}$$

When only the seismic action in y direction is considered

$$\gamma_{tj} = \sum_{i=1}^{n} Y_{ji} G_i \Big/ \sum_{i=1}^{n} (X_{ji}^2 + Y_{ji}^2 + \varphi_{ji}^2 r_i^2) G_i \tag{5.2.3-3}$$

When the seismic action oblique with x direction is considered

$$\gamma_{tj} = \gamma_{xj} \cos\theta + \gamma_{yj} \sin\theta \tag{5.2.3-4}$$

where: γ_{xj}, γ_{yj}——the participation factors defined by equations (5.2.3-2) and (5.2.3-3) respectively.

θ——angle between the seismic action direction and x direction.

2) The torsion effect of single direction horizontal seismic action may be determined in accordance with following equation:

$$S_{Ek} = \sqrt{\sum_{j=1}^{m}\sum_{k=1}^{m}\rho_{jk}S_jS_k} \tag{5.2.3-5}$$

$$\rho_{jk} = \frac{8\zeta_j\zeta_k(1+\lambda_T)\lambda_T^{1.5}}{(1-\lambda_T^2)^2 + 4\zeta_j\zeta_k(1+\lambda_T)^2\lambda_T} \tag{5.2.3-6}$$

where: S_{Ek}——torsion effect caused by the seismic action characteristic value.

S_j, S_k——the effect caused by seismic action of modes j-th and k-th respectively, the first 9～15 modes may be selected.

ζ_j, ζ_k——the damping ratio of modes j-th and k-th respectively; and in Eq. (5.2.3-6) shall taken as $\zeta_j = \zeta_k$.

ρ_{jk}——coupling factor of modes j-th and k-th.

λ_T——ratio between the natural periods of modes k-th and j-th.

3) The torsion effect of double direction horizontal seismic action shall be determined according to the greater value calculated from the following two equations:

$$S_{Ek} = \sqrt{S_x^2 + (0.85S_y)^2} \tag{5.2.3-7}$$

or

$$S_{Ek} = \sqrt{S_y^2 + (0.85S_x)^2} \tag{5.2.3-8}$$

where: S_x, S_y——the torsion effect caused by horizontal seismic action along x and y directions determined accordance with Formula (5.2.3-5) respectively.

5.2.4 When the base shear method is used, the seismic effect of penthouse, parapet and chimney on the roof should be multiplied by an amplifying factor of 3; such increase part of effect shall only be assigned to the members of roof, not to the lower part of the structure. When modal analysis method is used, the projecting part may be considered as one mass; the amplifying factor of seismic effect of the projecting skylight frame of a single-story factory building shall comply with relevant provisions in Chapter 9 of this code.

5.2.5 The horizontal seismic shear force at each floor level of the structure shall comply with the requirement of the following equation:

$$V_{Eki} > \lambda \sum_{j=1}^{n} G_j \tag{5.2.5}$$

where: V_{Eki}——the floor i-th shear corresponding to horizontal seismic action characteristic value.

λ——shear factor, it shall not less than values in Table 5.2.5; for the weak location of vertical irregular structure, these values shall be multiplied by the amplifying factor of 1.15.

G_j——the representative value of gravity load in floor j-th of the structure.

Table 5.2.5 Minimum seismic shear factor value of the floor level

Structures	Intensity 7	Intensity 8	Intensity 9
Structures with obvious torsion effect or fundamental period is less than 3.5s	0.016 (0.024)	0.032 (0.048)	0.064
Structures with fundamental period greater than 5.0s	0.012 (0.018)	0.024 (0.032)	0.040

Notes: 1. The values may be selected through interpolation method for structures whose fundamental period is between 3.5s and 5s.

2. Values in the brackets are used at the regions with basic seismic acceleration as 0.15g and 0.30g respectively.

5.2.6 The horizontal seismic shear force at each floor level of the structure shall be distributed to the lateral-force-resisting members (such as walls, columns, and seisuic braces) according to the following principles:

1 For buildings with rigid diaphragms, such as cast-in-place and monolithic-prefabricated concrete floors and roofs, the distribution should be done in proportion to the equivalent stiffness of the lateral-force-resisting members.

2 For buildings with flexible diaphragms, such as wood roof and wood floors, the distribution should be done according to the ratio of gravity load representative value on the areas which are subordinated the lateral-force-resisting members.

3 For buildings with semi-rigid diaphragms, such as ordinary prefabricated concrete roof and floors, the distributed may select the average value of the above two methods.

4 The above distribution results may be adjusted in accordance with the relevant provisions in this code, to consider the space interaction of the lateral-force-members, deformation of diaphragms, elasto-plastic deformation of the wall, and torsion response.

5.2.7 In the seismic computation of a structure, in general, the subsoil-structure interaction may be ignored; if the subsoil-structure interaction of reinforced concrete tall buildings for Intensity 8 and 9 and with Site-class Ⅲ or Ⅳ is need to consider, that shall comply with following requirements:

The tall building structures shall be with box-type or a relatively rigid raft foundation or box-pile foundation; and the fundamental period of the structure is within the scope of 1.2 to 5 times of the characteristic period of Site.

If the subsoil-structure interaction is considered for those structures, the horizontal seismic shear forces assumed for rigid base may be reduced in accordance with the following provisions, and the story drift may be calculated according to the reduced story shear force.

1 For structures with height-width-ratio less than 3, the reduction factor of horizontal seismic shear of each floor may be determined by following equation:

$$\psi = \left(\frac{T_1}{T_1 + \Delta T}\right)^{0.9} \tag{5.2.7}$$

where: ψ——seismic shear reduction factor considering the subsoil-structure interaction.

T_1——the fundamental period of the structure, which determined by assumption of the rigid base (in s).

ΔT——the additional period after considering the subsoil-structure interaction (in s), that may be determined from Table 5.2.7.

Table 5.2.7 Additional period(s)

Intensity	Site-class Ⅲ	Site-class Ⅳ
8	0.08	0.20
9	0.10	0.25

2 For structures whose height-width-ratio is not less than 3, the seismic shear of the structural bottom may be reduced according to point 1 of this clause ,that of the top may not reduced, and that of the middle floors may be reduced according to the linear interpolation values.

3 The reduced horizontal shear of all floors shall satisfy the requirements of Clause 5.2.5 of this chapter.

5.3 Calculation of vertical seismic action

5.3.1 For tall buildings for Intensity 9, the characteristic value of vertical seismic action shall be determined by the following equations (Fig. 5.3.1). The effects of vertical seismic action at the floor level may be distributed in proportion of the representative value of gravity load acting on the members, which should multiply with the amplifying factor 1.5.

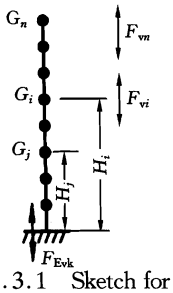

Fig. 5.3.1 Sketch for the computation of vertical seismic action

$$F_{Evk} = \alpha_{vmax} G_{eq} \tag{5.3.1-1}$$

$$F_{vi} = \frac{G_i H_i}{\Sigma G_j H_j} F_{Evk} \tag{5.3.1-2}$$

where: F_{Evk}——characteristic value of the total vertical seismic actions of the structure.

F_{vi}——characteristic value of vertical seismic action at the level of mass i-th.

α_{vmax}——maximum value of vertical seismic influence coefficient, which may be taken as 65% of the maximum value of the horizontal seismic influence coefficient.

G_{eq}——equivalent total gravity load of the structure, which may be taken as 75% of the representative value of the total gravity load of the structure.

5.3.2 For a flat lattice truss roof and for trusses with a span greater than 24m, the characteristic value of vertical seismic action should be taken as the product of the representative value of the gravity load and the coefficient of vertical seismic action. Values for the coefficient of vertical seismic action may be determined using Table 5.3.2:

Table 5.3.2 Coefficients of vertical earthquake action

Type of structure	Intensity	Site-class		
		Ⅰ	Ⅱ	Ⅲ, Ⅳ
Flat lattice truss, Steel truss	8	not considered(0.10)	0.08(0.12)	0.10(0.15)
	9	0.15	0.15	0.20
Reinforced Concrete truss	8	0.10(0.15)	0.13(0.19)	0.13(0.19)
	9	0.20	0.25	0.25

Note: the values in the brackets are used to design for regions where the basic seismic acceleration is 0.30g.

5.3.3 For long cantilever and other large-span structures for Intensity 8 and 9, the vertical seismic action characteristic value may be taken as 10% and 20% of the gravity load representative values of structure or member respectively. When the design basic seismic acceleration is 0.30g, which may be taken as 15% of the gravity load representative value of structure or member.

5.4 Seismic check for load-bearing capacity of structural members

5.4.1 The combination of seismic effect, which is considered as dominating, with other loads effects on structural members shall be determined by the following equation:

$$S = \gamma_G S_{GE} + \gamma_{Eh} S_{Ehk} + \gamma_{Ev} S_{Evk} + \psi_w \gamma_w S_{wk} \qquad (5.4.1)$$

where: S **——design value of combination of inner forces in a structural member, including design value of combination of bending moment, axial force and shear force.**

γ_G **——partial factor of gravity load, which shall be taken as 1.2 in ordinary conditions; when the effect of gravity load is favorable to the bearing capacity of the member, such factor shall not greater than 1.0.**

γ_{Eh}, γ_{Ev} **——partial factors for horizontal and vertical seismic action respectively, which shall be determined from Table 5.4.1.**

γ_w **——partial factor for wind load, which shall be taken as 1.4.**

S_{GE} **——effects for representative value of gravity load, in which shall be included the characteristic value of the all hanging weight of the crane.**

S_{Ehk} **——effects for characteristic value of seismic action in horizontal direction, that shall be multiplied with the relevant amplifying factor or adjusted factor.**

S_{Evk} **——effects for characteristic value of seismic action in vertical direction, that shall be multiplied with the relevant amplifying factor or adjusted factor.**

S_{wk} **——effects for characteristic value of wind load.**

ψ_w **——factor for combination value of wind load, which shall be taken as 0.0 for ordinary structures, and taken as 0.2 for tall building structures that the wind load is control load.**

Note: Subscripts representing the horizontal seismic action direction are generally omitted in this code.

Table 5.4.1 Seismic action partial factors

Seismic action	γ_{Eh}	γ_{Ev}
Only horizontal seismic action	1.3	0.0
Only vertical seismic action	0.0	1.3
Both horizontal and vertical seismic action	1.3	0.5

5.4.2 The checking seismic resistance of cross section of structural members shall be made using the following design expression:

$$S \leqslant R/\gamma_{RE} \tag{5.4.2}$$

where: γ_{RE}——seismic adjusting factor for load-bearing capacity of the structural member, which shall be determined from Table 5.4.2 except having another requirements.

R——design value of load-bearing capacity of the structural member.

Table 5.4.2 Seismic adjusting factor of load-bearing capacity

Material	Type of structural member	Stress type	γ_{RE}
Steel	Column, beam		0.75
	Brace		0.80
	Panel, connecting bolt		0.85
	Connecting weld		0.90
Masonry	Walls with tie-columns or core-columns at both ends	Shear	0.90
	Other seismic structural wall	Shear	1.0
Concrete	Beam	Bending	0.75
	Columns with axial force ratio <0.15	Eccentric compression	0.75
	Columns with axial force ratio $\geqslant 0.15$	Eccentric compression	0.80
	Wall	Eccentric compression	0.85
	All types of member	Shear, eccentric tension	0.85

5.4.3 When vertical seismic action is only considered, the seismic adjusting factor should be taken as 1.0 for all structural members.

5.5 Seismic check for deformation

5.5.1 Seismic deformation checks shall be made for all types of structures as per listed in Table 5.5.1, and the maximum elastic story drift shall comply with the following requirement:

$$\Delta u_e \leqslant [\theta_e] h \tag{5.5.1}$$

where: Δu_e——elastic story drift caused by the characteristic value of the frequently earthquake action. The story drift may be computed as the largest difference of the horizontal displacements along any of the edges of the structure at the top and bottom of the story under consideration, except the structures having predominating flexural deformation. When calculating, all of partial fac-

tors of various actions shall be taken as 1.0, and elastic stiffness may be used for reinforced concrete members.

$[\theta_e]$——limit value of elastic story drift rotation which may be taken in accordance with Table 5.5.1.

h——height of calculated story.

Table 5.5.1 Limit value of elastic story drift

Material	Type of structure	$[\theta_e]$
Reinforced concrete	Frame	1/550
	Frame-wall, slab-column-wall, frame-core-tube	1/800
	Wall, tube-in-tube	1/1000
	Frame-supporting stories of structure	1/1000
steel	Multi-story and tall structures	1/300

5.5.2 Elasto-plastic deformation check shall be made for the weak stories (or locations) of the structure under the rarely earthquake, and shall comply with the following requirements:

1 Elasto-plastic deformation check shall be done for the following structures:

1) Bent-frames of single-story reinforced concrete column factory with higher columns and larger spans for Intensity 8 with Site-class Ⅲ and Ⅳ, or for Intensity 9;

2) Reinforced concrete frame structures for Intensity 7～9, when the yield strength coefficient of the story is less than 0.5;

3) Steel structures with height more than 150m;

4) Reinforced concrete structures and Steel structures, which assigned to Fortification Category B for Intensity 9 and to Fortification Category A;

5) Structures adopting seismic-isolation and energy dissipating designs.

2 Elasto-plastic deformation check should be done for the following structures:

1) Tall building structures within the height scope of as per listed in Table 5.1.2-1 and having also the vertical irregular types as per listed in Table 3.4.2-2;

2) Reinforced concrete and steel structures assigned to Fortification Category B for Intensity 7 with Site-class Ⅲ and Ⅳ, or Intensity 8;

3) Slab-column-wall structures and masonry buildings with bottom-frame;

4) Tall steel structures with height not more than 150m.

Note: The yielding strength coefficient of a story is the ratio between the shear bearing capacity and the elastic seismic shear force at such story under rarely earthquake. For the bent-frame, it refers to the ratio between the bending capacity and elastic seismic moment of such cross section of structure under rarely earthquake. The shear bearing capacity of a story can be determined according to the actual reinforcement area and material characteristic strength of members and the axial force of columns and walls.

5.5.3 Elasto-plastic deformation of a weak story (or location) of a structure under the rarely earthquake may be determined by the following methods:

1 For framed structures that do not exceed 12 stories in height and with no abrupt change of story stiffness as well as single-story factory buildings with reinforced concrete column, the simplified method in Clause 5.5.4 may be used.

2 For structures outside those in point 1 of this clause, the static elasto-plastic analysis method and the time-history analysis method may be used.

3 Regular structures may adopt bending shear model or planar line-members system model, irregular structures as per listed in Section 3.4 of this code shall adopt spacious structure models.

5.5.4 The simplified calculation method for elasto-plastic story drift in the weak story (or location) of a structure should comply with the following requirements:

1 The weak story (or location) may be identified as follows:

1) For structures with a uniform distribution of story yield strength coefficient along the height of the structure, the first story of the building may be identified as the weak story;

2) For structures with non-uniform distribution of story yield strength coefficient along the height of the structure, the story (location) with minimum, or smaller story yield strength coefficient may be identified as the weak story. However, in general, no more than two or three stories (locations) may be identified as weak stories;

3) For single-story factory buildings, the weak location is at the upper portion of the columns.

2 The elasto-plastic story drift may be calculated by the following equations:

$$\Delta u_p = \eta_p \Delta u_e \tag{5.5.4-1}$$

or

$$\Delta u_p = \mu \Delta u_y = \frac{\eta_p}{\xi_y} \Delta u_y \tag{5.5.4-2}$$

where: Δu_p——elasto-plastic story drift.

Δu_y——yield story drift.

μ——story ductility factor.

Δu_e——elastic story drift under the rarely earthquake.

η_p——amplifying factor for elasto-plastic story drift. When the yield strength coefficient of the weak story (location) is not less than 80% of the average value of coefficients of the neighboring stories (location), it may be taken from Table 5.5.4. When the yield strength coefficient is not more than 50% of the above-mentioned average value, it may be taken the corresponding values in the Table 5.5.4 multiplied by 1.5. When the yield strength coefficient is between the above two cases, it may be determined

by interpolation.

ξ_y——yield strength coefficient of story.

Table 5.5.4 Amplifying factor for elasto-plastic story drift

Type of structure	Total Stories or location	ξ_y		
		0.5	0.4	0.3
Multi-story frame structure with uniform elevation	2~4	1.30	1.40	1.60
	5~7	1.50	1.65	1.80
	8~12	1.80	2.00	2.20
Single-story factory building	Upper portion of column	1.30	1.60	2.00

5.5.5 The elasto-plastic story drift in the weak stories (locations) of a structure shall comply with the following requirement:

$$\Delta u_p \leqslant [\theta_p] h \tag{5.5.5}$$

where: $[\theta_p]$——limit value of elasto-plastic story drift rotation, which can be taken from Table 5.5.5. For reinforced concrete frame structures, the values may be increased. Such as, it may be increased 10% where axial-force-ratio is less than 0.40; it may be increased 20% where the stirrup characteristic value along the full height of the column is 30% greater than those provisions in Table 6.3.12. But its total increase shall not be greater than 25%.

h——height of the weak story (location) or the height of the upper portion of column in single-story factory building.

Table 5.5.5 Limit values for elasto-plastic story drift

Material	Type of structure	$[\theta_p]$
Reinforced concrete	Bent-frame of Single-story factory building with reinforced concrete columns	1/30
	Frame	1/50
	Frame-wall of brick building with bottom-frame	1/100
	Frame-wall, slab-column-wall , frame-core-tube	1/100
	Wall, tube-in-tube	1/120
Steel	Multi-story and tall Structures	1/50

Chapter 6 Multi-story and tall reinforced concrete buildings

6.1 General

6.1.1 In this chapter, the structural type and applicable maximum height of cast-in-situ reinforced concrete buildings shall comply with the provisions in Table 6.1.1. For structures assigned to irregular plan and elevation or structures sat on Site-class IV, the applicable maximum height shall be reduced appropriately.

Note: Inhere "wall(i.e. seismic structural wall)" is refer to "shear-wall" in the national standard "Code for Design of Concrete Structures" GB50010.

Table 6.1.1 Applicable maximum height for cast-in-situ reinforced concrete buildings(m)

Type of structures	Intensity 6	Intensity 7	Intensity 8	Intensity 9
Frame structure	60	55	45	25
Frame-wall structure	130	120	100	50
Wall structure	140	120	100	60
Frame-support wall structure	120	100	80	No applicable
Frame-core-tube structure	150	130	100	70
Tube-in-tube structure	180	150	120	80
Slab-column-structural wall structure	40	35	30	No applicable

Notes: 1 The height of building refers to the height from the outdoor ground level to the main roof level of a building (which locations exceeding roof level are not included);

2 Frame-core-tube structure refers to the structure composed of the perimeter frames and the core tube;

3 Frame-support- wall structure refers to the wall structure having some discontinuous wall supported by frames in the first story or the first and second stories at the bottom of the building;

4 Applicable maximum height for buildings assigned Fortification Category B may be determined according to the local fortification intensity;

5 When the height of a building exceeding that as provisions in this table, special research and demonstration shall be carried out and effective seismic measures shall be taken.

6.1.2 For the seismic design of reinforced concrete buildings, different measure grade shall be adopted according to the Intensity, the structural type, the building height and its Fortification Category; and shall satisfy the corresponding requirements of the calculation and design details. The measure grade of buildings assigned to Fortification category C shall be determined in accordance with the Table 6.1.2.

6.1.3 The determination for the seismic measure grades of reinforced concrete structures shall also comply with the following requirements:

1 For frame-wall structures subjected to the fundamental mode seismic action, when the seismic overturning moment distributed to frame parts is more than 50% of the total

seismic overturning moment of the structure, the measure grade of frame parts for such structure shall be determined as that of the framed structures.

Table 6.1.2 Grade of cast-in-situ concrete structures

Type of structures		Intensity 6		Intensity 7		Intensity 8		Intensity 9
Frame structure	Height (m)	≤30	>30	≤30	>30	≤30	>30	≤25
	Frame	4th	3rd	3rd	2nd	2nd	1st	1st
	public building with large span such as theatre and stadium	3rd		2nd		1st		1st
Frame- wall structure	Height (m)	≤60	>60	≤60	>60	≤60	>60	≤50
	Frame	4th	3rd	3rd	2nd	2nd	1st	1st
	wall	3rd		2nd		1st		1st
Wall Structure	Height (m)	≤80	>80	≤80	>80	≤80	>80	≤60
	wall	4th	3rd	3rd	2nd	2nd	1st	1st
Frame-support-wall structures	Seismic-structural wall	3rd	2rd	2nd	2nd	1st		
	Frame of the frame-support stories	2nd	2nd	2nd	1st	1st		
Frame-core-tube structure	Frame	3rd		2nd		1st		1st
	Core-tube	2nd		2nd		1st		1st
Tube-in-tube structure	Exterior tube	3rd		2nd		1st		1st
	Interior tube	3rd		2nd		1st		1st
Slab-column wall structures	Column	3rd		2nd		1st		
	wall	2nd		2nd		2nd		

Notes: 1 On Site-class I, the grades of design details shall be permitted reducing one degree which list in this Table unless Intensity 6, but the requirements for calculation shall not be reduced.

2 When the height of building is close to or equal to the height dividing line, the measure grade shall be permitted adjusted appropriately in consideration of the degree of the building irregularity, the site and subsoil conditions.

3 For frame-support-wall structures, the measure grade of wall that above strengthen-regions of the bottom of structure shall be permitted taken as in accordance with the wall structures.

2 When the podium is connected with the main building, the measure grades shall be determined with the podium itself and shall not be lower than that of the main building. And design details of the main building shall be strengthened appropriately at the level of the top of podium and adjacent upper and lower level of that. When the main building and the podium are separated, the measure grades shall be determined according to the provision of the podium.

3 When the top-slab of the basement is used as the fixing location in the structural system analysis, the measure grade of the first story underground shall be the same as the structural system. And the measure grade for other stories lower than first story underground may be adopted grade 3-rd or lower grade based on actual conditions. For parts of the basement without corresponding structural system, the measure grade may be taken as

grade 3-rd or lower grade.

4 For buildings assigned to Fortification Category A, B and D, the measure grades may be determined according to the provisions in Clause 3.1.3 of this code and in Table 6.1.2. When the height of a building assigned to Fortification Category B for Intensity 8 exceeds the height listed in Table 6.1.2, it shall be especially studied and more effective seismic measures than that of grade 1-st shall be taken.

Note: The "measure grade 1-st, 2-nd, 3-rd and 4-th" hereinafter refer to "Grade 1, 2, 3, and 4" respectively.

6.1.4 Tall reinforced concrete buildings should avoid to adopting the building structure having irregular plans and vertical configuration listed in Section 3.4 of this code, and no seismic joints are necessary to be installed. When seismic joints are necessary, that shall satisfy the following requirements:

1 The minimum clear width of the isolation joint shall comply with the following requirements:

1) For framed building structures, when the building height is not more than 15 m, a width of 70 mm may be used. And when the framed building height is more than 15 m, then for the Intensity 6, 7, 8 and 9, the width shall be added by 20 mm for every 5m, 4m, 3m and 2m increase in height respectively;

2) For frame-wall building structures, this width may be taken as 70% of the values as provision in point 1); for wall building structure, this width may be taken as 50% of the values as provision in point 1); besides, neither shall be smaller than 70mm;

3) When the structural systems at the two sides of the seismic joint are different, such width shall be determined according to the structural system needed wider and the lower building height.

2 For Intensity 8 and 9, when the total height, stiffness or story height of frame structures at the two sides of the seismic joint are significantly different, the retaining wall may be installed at structural edges of the two side of the seismic joint. Such retaining wall shall comply with following requirements:

1) Along the overall height of the structure and orthogonal to the seismic joint;

2) The number of retaining walls on each side shall not be less than two segments, that should be arranged symmetrically, and the length of this wall may be no more than the span of columns;

3) The internal force of the frame and the retaining wall shall be analyzed according to the two cases of having and no-having retaining walls, and then make the choice according to the unfavorable situation;

4) For the end columns of the retaining wall and the side columns of the frame, the spacing of hoops of the column shall be densified along the overall height of the

building.

6.1.5 For frame or frame-wall structures, its frames or walls shall be arranged in two orthogonal directions. The centerlines of beam-to-column or those of the column-to-wall should coincide with each other, and eccentricity between the centerlines of beam-to-column or column-to-wall should not be greater than 1/4 of the column width.

6.1.6 For frame-wall structures and slab-column-wall structures, the aspect ratio of the diaphragm, which there are no large openings between two adjacent-walls, should not exceed those set forth in Table 6.1.6. If the aspect ratio is greater than those in Table 6.1.6, the influence of in-plane deformation of the diaphragm shall be considered.

Table 6.1.6 Aspect ratio of the diaphragm between two adjacent-walls

Type of floors and roofs	Intensity 6	Intensity 7	Intensity 8	Intensity 9
Cast-in-situ or lapped beam and slab	4	4	3	2
Fabricated floors	3	3	2.5	Should not adopt
Floor of frame-support-story or slab-column-wall structures	2.5	2.5	2	Shall not adopt

6.1.7 When precast floor or roof members are used in frame-wall structures, it shall be satisfied that integrality of diaphragms and reliable connection between diaphragm and wall; when cast-in-situ toppling with reinforcement is adopted for such purpose, the thickness shall not be less than 50mm.

6.1.8 Installation of the walls in frame-wall structures should comply with the following requirements:

1 The walls should be built through the overall height of the building, and the transverse walls should be connected with the longitudinal walls.

2 The walls should be placed at locations where large opening on wall is needless.

3 When the building is relatively long, the longitudinal wall with bigger stiffness should not be installed at the end bay of the building.

4 The openings of the wall should be aligned from the upper to the lower part; the distance from the edge of the opening to the end column should not be less than 300mm.

5 For the coupling beam of the wall for Grade 1 and Grade 2, that span-to-depth ratio should not be greater than 5, and the depth of the coupling beam should not be less than 400mm.

6.1.9 Installation of the walls in wall structures and frame-support-wall structures shall comply with the following requirements:

1 A relatively long-wall should be divided uniformly into several wall-segments by installing of coupling beams that the span-to-depth ratio should greater than 6, and the height-to-width ratio of wall-segments shall be not less than 2.

2 The length of walls along overall height of a structure should not be cutout. The relatively large openings of the walls and the openings of bottom of wall for Grade 1 or Grade 2, the openings should be aligned from the upper to the lower part.

3 For frame-support-wall structures with rectangular plan, stiffness of frame-support story shall be not less than 50% of the stiffness of the adjacent upper first story. The spacing of the wall continued to the ground shall not be greater than 24 m, and the plan arrangement of lateral-force-resisting members of frame-support story should also be symmetric and should be installed the tubes.

6.1.10 For the frame-support-wall structures, the height of the strengthening portion at the bottom may be taken as the greater of the height of the two stories above the frame-support story and 1/8 of the total wall height, neither greater than 15m. For other wall structures, the height of the strengthening portion at bottom may be taken as the greater of the height of the first two stories and 1/8 of the total wall height, neither greater than 15m.

6.1.11 In one of the following cases for framed structure, foundation tie beams shall be installed along both principal axial directions:

1 Frames assigned to Grade 1 or to Grade 2 with Site-class IV.

2 The representative values of gravity load on each column footing are differing greatly.

3 The buried depths of foundations or their relative difference are greatly.

4 In the range of the main bearing stratum of the subsoil, that exist weak cohesive soil layers, layers liable to liquefaction, and seriously heterogeneous layers.

5 Connection between the one and other pile caps.

6.1.12 The foundations of the wall in frame-wall structures and of the wall which continuing to ground in the frame-support-wall structures shall have excellent integrality and anti-rotational capability.

6.1.13 If the podium is connected with the main building and adopted together the natural base, besides satisfying the requirements of the provisions in Clause 4.2.4 of this code, zero stress zones should not occur in the foundation bottom face of the main building under seismic action.

6.1.14 When the top slab of basement is used as the fixing location of the structural system, the opening of large holes on this diaphragm shall be avoided, at the same time, this floor shall adopt the cast-in-situ structure and shall comply with following requirements:

1) The thickness of the slab shall not be less than 180mm, the concrete strength grade should not be less than C30; the double layer and two way reinforcements shall be arranged, moreover, the ratio of reinforcement shall not be less than 0.25%;

2) The lateral stiffness of the basement structure should not less than 2 times that of the first story of this building;

3) The area of longitudinal bars on each side of the basement column section, besides satisfying the necessary of the calculation, shall not be less than 1.1 times of that on the corresponding column in the first story of this building;

4) The bending moment design values of the framed columns and of the bottom section of the walls in the first story shall comply with the provisions in Clause 6.2.3, 6.2.6, and 6.2.7 of this code;

5) The actual bending capacity of column at upper face in the joint of the top of basement columns shall be less than sum of actual bending capacity of beam at the left-and right-faces in that joint and of the column at the lower-face in that joint.

6.1.15 The filling wall of the frame shall comply with the provisions in Chapter 13 of this code.

6.1.16 The seismic design of high strength concrete structures shall comply with the provisions in Appendix B of this code.

6.1.17 The seismic design of prestressed concrete structures shall comply with the provisions in Appendix C of this code.

6.2 Essentials in calculation

6.2.1 The design values of seismic effects of reinforced concrete structural members shall be adjusted in accordance with the provisions in this section, and the story drift shall satisfy the requirements in Section 5.5 of this code.

If not specified in this chapter and related Appendices, the checking of members shall be made in accordance with the current codes for relevant structural design, but design value of the non-seismic bearing capacities of members shall be divided by the seismic adjusting factors provided in this code.

6.2.2 At the faces of each joint of the beam-column for assigned to Grade 1, 2 and 3, the combinatory moment design value of the column shall comply with following equation; unless the joints of column in top story or with axial-force-ratio less than 0.15 and of the supporting-columns.

$$\Sigma M_c = \eta_c \Sigma M_b \qquad (6.2.2\text{-}1)$$

For Grade 1 framed-structures or Intensity 9, it shall also comply with

$$\Sigma M_c = 1.2 \Sigma M_{bua} \qquad (6.2.2\text{-}2)$$

where: ΣM_c——sum of combinatory moment design values in clockwise or counter-clock-wise direction of column at the faces of the joint; in general case, the moments at the upper and lower faces of the joint may be distributed by elastic analysis.

ΣM_b——sum of combinatory moment design values in counter-clockwise or clock-

wise direction of beam at the faces of the joint. For Grade 1 frame, the moments at the left and right faces of the joint are negative together, that of the smaller absolute value may be taken as 0.0.

ΣM_{bua}——sum of moments in counter-clockwise or clockwise direction at the faces of the joint corresponding to actual seismic bending capacity of normal cross section of beams framing into that joint, which may be determined by actual reinforcement area and characteristic value of the material strength.

η_c——amplifying factor of the moment of column end, taken as 1.4 for Grade 1, taken as 1.2 for Grade 2, and taken as 1.1 for Grade 3.

When the zero-moment-point is not in the range of a story height of the column, the combinatory moment design value of the column may be multiplied with the above amplifying factor.

6.2.3 The combinatory moment design values of the lower end of column at the first story of Grade 1, 2, and 3 frames shall be multiplied by an amplifying factor of 1.5, 1.25 and 1.15 respectively. The longitudinal reinforcement of the column at the first story shall be arranged according to unfavorable conditions in its upper and lower ends.

6.2.4 For frame beams of Grade 1, 2 and 3 and coupling beam with span-to-depth ratio greater than 2.5 in wall, the shear force design value of the beam end shall be adjusted according to the following equations:

$$V = \eta_{vb}(M_b^l + M_b^r)/l_n + V_{Gb} \tag{6.2.4-1}$$

For Grade 1 framed-structures or Intensity 9, it shall also comply with

$$V = 1.1(M_{bua}^l + M_{bua}^r)/l_n + V_{Gb} \tag{6.2.4-2}$$

where: V——combinatory shear force design value of the beam-ends.

l_n——clear span of the beam.

V_{Gb}——shear force design value of the beam end obtained in the analysis based on simply supported beams, which subjected to the representative value of gravity load (for the Intensity 9, it shall include also the characteristic value of vertical seismic action for tall buildings).

M_b^l, M_b^r——the combinatory bending moment design values in counter-clockwise or clockwise direction of the beam end assigned to left and right respectively. For Grade 1 frame, the bending moments at the left and right end of beams are negative together, which of the smaller absolute value may be taken as 0.0.

M_{bua}^l, M_{bua}^r——the bending moments in counter-clockwise or clockwise direction corresponding to actual seismic bending capacity of normal cross section of the beam assigned to left and right ends respectively, that may be determined by actual

reinforcement area and characteristic value of the material strength.

η_{vb}——the amplifying factor of shear force for the beam end, taken as 1.3 for Grade 1, taken as 1.2 for Grade 2, and taken as 1.1 for Grade 3.

6.2.5 The combinatory shear force design values of Grade 1, 2 and 3 frame column and frame-support-column shall be adjusted according to the following equations:

$$V = \eta_{vc}(M_c^b + M_c^t)/H_n \tag{6.2.5-1}$$

For Grade 1 framed-structures or Intensity 9, it shall also comply with

$$V = 1.2(M_{cua}^b + M_{cua}^t)/H_n \tag{6.2.5-2}$$

where: V——combinatory shear force design value of cross-sections at the column ends; for the frame-support-column, which shall also be complying with the provisions in Clause 6.2.10 of this chapter.

H_n——clear height of the column.

M_c^b, M_c^t——combinatory moments design value in clockwise or counter-clockwise direction of the column assigned to upper and lower end respectively, which shall comply with the provisions in Clause 6.2.2 and 6.2.3. For the frame-support-column, that shall also comply with the provisions in Clause 6.2.10 of this chapter.

M_{cua}^b, M_{cua}^t——bending moment values in the clockwise and counter-clockwise directions corresponds to the normal cross section seismic bending capacity of the eccentric compression column assigned to upper and lower ends respectively; and they shall be determined by the actual reinforcement area, the material strength characteristic value and the axial compressive force etc.

η_{vc}——amplification factor of the column shear force, taken as 1.4 for Grade 1, taken as 1.2 for Grade 2, and taken as 1.1 for Grade 3.

6.2.6 For corner columns of Grade 1, 2 and 3 frames, the combinatory bending moment design values shall also be multiplied by the amplifying factor of not less than 1.10.

6.2.7 The combinatory moment design value for all limb wall section of the wall shall be adopted according to the following provisions:

1 In strengthened portion at the bottom and its first-upper story of Grade 1 wall, such design value may be adopted according to the combinatory moment design value of the wall at the bottom face. For other portions, the combinatory moment design value of the wall shall be multiplied by the amplifying factor, which may be taken as 1.2.

2 For the frame-support-wall structures, the small eccentric tensioning should not occur on the wall continued to ground.

3 In the double-limb coupling wall, the small eccentric tensioning should not occur on limb-wall; when that is occur on anyone limb-wall, the combinatory shear force design value

and moment design value of the another limb-wall shall be multiplied by an amplifying factor 1.25.

6.2.8 For Grade 1, 2, and 3 wall, the combinatory shear force design value of the strengthening portion at the bottoms shall be adjusted according to the following equations:

$$V = \eta_{vw} V_w \tag{6.2.8-1}$$

For the Intensity 9, it shall also comply with

$$V = 1.1 \frac{M_{wua}}{M_w} V_w \tag{6.2.8-2}$$

where: V——the combinatory shear force design value of the wall at the strengthening portion of the bottom.

V_w——calculated combinatory shear force value of the wall at the strengthening portion of the bottom.

M_{wua}——the moment corresponding to the actual seismic bending capacity of the bottom face of the wall, which shall be determined by the actual reinforcement area, the material strength characteristic value and the axial compressive force. When the flange exists, the longitudinal bars within one thickness of flange at the two sides of web shall be taken into account.

M_w——combinatory moment design value of the bottom face of the wall.

η_{vw}——shear force amplifying factor of the wall, taken as 1.6 for Grade 1, taken as 1.4 for Grade 2, and taken as 1.2 for Grade 3.

6.2.9 The combinatory shear force design value of cross-sections at the ends of beams, columns, walls and its coupling beams shall comply with the following requirements:

For beams and coupling beams with span-to-depth ratio greater than 2.5 and columns and walls with shear-span-ratio greater than 2:

$$V \leqslant \frac{1}{\gamma_{RE}} (0.20 f_c b h_0) \tag{6.2.9-1}$$

For coupling beams with span-to-depth ratio not greater than 2.5, and the columns and walls with shear span-to-depth ratio not greater than 2, such as the supporting-columns, supporting-beams and the strengthened portion at bottom on the walls continued to ground of the frame-support-wall structures:

$$V \leqslant \frac{1}{\gamma_{RE}} (0.15 f_c b h_0) \tag{6.2.9-2}$$

The shear span-to-depth ratio shall be calculated according to the following equation:

$$\lambda = M^c / (V^c h_0) \tag{6.2.9-3}$$

where: λ——shear span-to-depth ratio , which shall be taken the greater value between both calculation values from the upper end and lower end of member; When the zero-moment-point is in the range of a story height of the column, it may

be taken as the ratio of clear column height to double column depth in considered direction.

M^c——combinatory moment calculation value of members.

V^c——combinatory shear force calculation value of members.

V——combinatory shear force design value of the beam ends, column ends, or wall section, that shall be determined in accordance with the provision of the Clauses 6.2.4, 6.2.5, 6.2.6, 6.2.8 and 6.2.10.

f_c——design value of axial compressive strength of concrete.

b——cross-sectional width of the beam, column or wall, and it may be calculated by using the equal area of square section for the circular section.

h_0——effective depth of member or effective height of cross-section, for wall, it may be taken as the lateral dimension.

6.2.10 The supporting-columns of frame-support-wall structure shall also satisfy the following requirements:

1 The minimum seismic shear forces distributed to the supporting-column shall comply with as follows: When the number of supporting-columns is more than 10, the sum of seismic shear force of the supporting-columns shall not be less than 20% of the seismic shear force of the same story. When the number of supporting-columns is less than 10, the seismic shear force of each supporting-column shall not be less than 2% of the seismic force of the same story.

2 The additional axial force produced by the seismic action in Grade 1 and 2 supporting-columns shall be multiplied by the amplifying factors 1.5 and 1.2 respectively; but when calculating the axial-force-ratio, the additional axial force may not be multiplied by such amplifying factors.

3 For the upper end of the top story and lower end of first story of the supporting-columns for Grade 1 and 2, the combinatory moment design values shall be multiplied by the amplifying factors 1.5 and 1.25 respectively. And the middle joints of the supporting-column shall satisfy the requirements in Clause 6.2.2 of this section.

4 The centerline of the supporting-beams should be coincided with the centerline of cross section of the supporting-columns.

6.2.11 The strengthening portion at the bottom of walls continued to ground on the frame-support-wall structures assigned to Grade 1 shall also satisfy the following requirements:

1 When calculating the seismic shear capacity of wall, the concrete areas should not be considered. If it is considered, than the ties with diameter not less than 8mm shall be arranged between two layers of web reinforcement beyond the boundary elements. And ties spacing in vertical and horizontal directions shall be not greater than the smaller value of the twice times of the reinforcement spacing respectively and 400mm.

2 When no basement building with the bottom of the limb-wall has eccentric tensions, the additive anti-sliding diagonal bars should be placed at intersection surface of the limb-wall and the foundation. The tension resisted by the anti-sliding diagonal bars may be taken as 30% of the shear force design value at the intersection surface.

6.2.12 The transference diaphragms of frame-supported-story in frame-support-wall structures shall conform to the provisions in Appendix E.1 of this code.

6.2.13 Seismic calculation for the reinforced concrete structures shall also comply with following requirements:

1 For frame-wall structures having even distribution of lateral stiffness along vertical configuration, the seismic shear force resisted by the frame-part of any story shall not be less than follows:

The 20% of the total seismic action of the structure, and 1.5 times of maximum seismic shear force in all stories in the frame-part according to structural analysis, whichever smaller.

2 The stiffness of the coupling-beam of the wall may be reduced, which shall not be less than 0.50.

3 When calculating the interior force and deformation of the wall structure, the frame-support-wall structure, the frame-wall structure, tube structure, and slab-column-wall structure, the interaction of web and flange shall be considered. The effective flange widths of flanged sections shall extend from the face of the web a distance equal to the smallest of one-half the distance to an adjacent wall web, to edges of opening of flange and 15 percent of the total wall height.

6.2.14 The following equation shall be used in the seismic checking for constructional joints of walls assigned to Grade 1:

$$V_{wj} \leqslant \frac{1}{\gamma_{RE}}(0.6 f_y A_s + 0.8N) \tag{6.2.14}$$

where: V_{wj}——combinatory shear force design value of constructional joints in the wall.

f_y——specified tensile strength design value of the vertical reinforcement.

A_s——sum of cross-sectional area of vertical distribution reinforcements and vertical inserted reinforcements in the wall web as well as longitudinal reinforcements of the bounding members (not including flanges on both sides) of the wall at the constructional joint.

N——combinatory axial force design value at the constructional joint, positive value is taken for compression and negative for tension.

6.2.15 The seismic check for the frame nodes, which referred to the portion of structure common to intersecting beams and columns, shall conform to the following requirements:

1 The seismic capacity of nodes for frames assigned to Grade 1 or 2 shall be checked; and need not be checked for frames assigned to Grade 3 or 4, but these shall comply with

the requirements of design details.

2 Method for checking of seismic capacity for the nodes of frame shall comply with the provisions in Appendix D of this code.

6.3 Details of seismic design for framed structures

6.3.1 Dimensions of cross section of beams shall comply with the following requirements:

1 Width of beams should not be less than 200mm.

2 The depth-to-width ratio beams should not be greater than 4.

3 Ratio of clear span to depth of beams should not be less than 4.

6.3.2 When the flat beam with the width more than the width of column has be adopted, that floor shall be cast in-situ, the centerlines of the beams and columns should be coincided, and the flat beam shall be arranged in double principal axial directions. And such flat beam should not be used for framed structures assigned to Grade 1.

The cross-sectional dimension of the flat beam shall comply with the following equations; and the provisions controlling deflection and cracked width of beam in governed current codes shall also be satisfied:

$$b_b \leqslant 2b_c \tag{6.3.2-1}$$

$$b_b \leqslant b_c + h_b \tag{6.3.2-2}$$

$$h_b \geqslant 16d \tag{6.3.2-3}$$

where: b_c——width of the column ; for circular columns , that taken as 0.8 times of the diameter.

b_b, h_b——the width and depth of the beam respectively.

d——diameter of longitudinal bars in the column.

6.3.3 Arrangement of reinforcement in beams shall comply with the following requirements:

1 The reinforcement ratio of tensile longitudinal bars at the both ends of beam shall not exceed 2.5%. And also the ratio of the neutral axis depth, which the compressive reinforcements may be taken into consideration, to the effective depth in the section shall not exceed 0.25 for frames assigned to Grade 1, and 0.35 for frames assigned to Grades 2 or 3.

2 The ratio of the reinforcement amounts between the bottom and top of each beam end shall be not less than 0.5 for frames assigned Grade 1 and 0.3 for frames assigned to Grade 2 or 3, nor less than the calculated necessary ratio.

3 Length arranged densified hoops, maximum spacing and minimum diameter of hoops at both ends of beam shall be taken in accordance with Table 6.3.3. When the reinforcement ratio of the tensile longitudinal bars at the beam end exceeds 2 %, the minimum diameter of hoops in the Table shall be increased by 2 mm.

Table 6.3.3 Length of the densified regions, maximum spacing and minimum diameter of hoops in a beam

Grade of frames	Length arranged densified hoops (the greater value shall be taken) (mm)	Maximum spacing of hoops (smallest value shall be taken) (mm)	Minimum diameter of hoops (mm)
Grade 1	$2h_b$, 500	$h_b/4$, 6d, 100	10
Grade 2	$1.5h_b$, 500	$h_b/4$, 8d, 100	8
Grade 3	$1.5h_b$, 500	$h_b/4$, 8d, 150	8
Grade 4	$1.5h_b$, 500	$h_b/4$, 8d, 150	6

Note: d referred to the diameter of longitudinal bars; h_b referred to the depth of the beam.

6.3.4 Arrangement of the longitudinal bars in beams shall also comply with the following requirements:

1 Continuously longitudinal reinforcements at the top and the bottom of beams shall be not less than 2ϕ14, and also not less than 1/4 of the greater amount of top or bottom longitudinal beam bars at both ends for frames assigned to Grade 1 or 2. And that shall be not less than 2ϕ12 for frames assigned to Grade 3 or 4.

2 Diameter of each longitudinal beam reinforcement extending through a mid-column-beam joint for frames assigned to Grade 1 or 2, shall not be greater than 1/20 of the column dimension parallel to the beam reinforcement for column with rectangular section. For circular columns, the diameter shall not be greater than 1/20 of the chord length of the column section where such beam reinforcement locates.

6.3.5 The distance between the crossties in the densified regions of hoops at both beam-ends shall be not greater than follows:

For frames assigned to Grade 1, not greater than 200 mm and 20 times of the hoop diameter; for frames assigned to Grade 2 or 3, not greater than 250mm and 20 times of the hoop diameter; for frames assigned Grade 4, not greater than 300mm.

6.3.6 Dimension of cross section of columns shall comply with the following requirements:

1 The shortest cross-sectional dimension of the column should all not be less than 300mm; and the diameter of circular column should not be less than 350mm.

2 The shear span-to-deplh ratio should be greater than 2.

3 The ratio of the longest cross-sectional dimension to the perpendicular dimension should not be greater than 3.

6.3.7 Axial-force ratio of the column should not exceed the limit values as shown in Table 6.3.7; for tall building structures built on Site-class IV, this axial-force-ratio shall be reduced accordingly.

Table 6.3.7 Limit value for the axial-force-ratio of column

Structural systems	Grade 1	Grade 2	Grade 3
Frame structures	0.7	0.8	0.9

Table 6.3.7 (continued)

Structural systems	Grade 1	Grade 2	Grade 3
Frame-wall, slab-column-wall and tube structures	0.75	0.85	0.95
Frame-support-wall structures	0.60	0.70	—

Notes: 1 Axial-force-ratio refers to the ratio of the combinatory axial compressive force (including seismic effects) design value of column to the product of the column cross-sectional area and the specified concrete compressive strength. As for structures, which need not be seismic checked, may be taken as combinatory axial compressive force design value without seismic effects.

2 The limit values in the table are applicable to columns whose shear span-to-depth ratio is greater than 2 and the strength grade of concrete are not higher than C60. For columns whose shear span-to-deplh ratio is not greater than 2, the limit values of axial-force-ratio shall be reduced by 0.05; for columns with shear span-to-deplh ratio less than 1.5, the limit value of axial-force-ratio shall be studied especially and special details shall be taken.

3 The limit values in the table may be increased 0.10 in following three cases, but that hoop characteristic factors of the three cases shall comply with the provision of Clause 6.3.12 according to the enlarged axial-force-ratios:

The compound hoops are adopted along overall height of the column and the distances between the crossties or legs is not greater than 200mm, the spacing of hoops is not greater than 100mm and the diameter is not less than 12mm. The compound spiral hoops are adopted along overall height of the column, and the spiral spacing is not greater than 100mm, the distance between the crossties or legs is not greater than 200mm and the diameter of the hoops is not less than 12mm. The continuous compound rectangular spiral hoops are adopted along overall height of the column, the clear spiral distance is not greater than 80mm, the distance between the crossties or legs is not greater than 200mm and the diameter is not less than 10mm.

4 When an additive core-longitudinal-bars are arranged in the middle of the cross section of the column, that the total cross-sectional area of the additive longitudinal bars shall not be less than 0.8% of the cross section area of column, the limit values in the table may be increase 0.05. When these longitudinal reinforcements is adopted together with the hoops provided in note 3, the limit values in the table may be increase 0.15, but the hoop characteristic factors may be determined in according with the provision of that the axial-force-ratio is increase 0.10.

5 The axial-force-ratio of columns shall not be grater than 1.05 in every case.

6.3.8 The arrangement of reinforcement in columns shall comply with the following requirements:

1 Minimum total reinforcement ratio of longitudinal bars in columns shall be adopted as shown in Table 6.3.8-1, and the total reinforcement ratio in each side shall not be less than 0.2%; for tall building structures built at Site-class IV, the values in the Table shall be increased by 0.1.

Table 6.3.8-1 Minimum total reinforcement ratios of longitudinal bars in columns (%)

Type of columns	Grade 1	Grade 2	Grade 3	Grade 4
Middle columns and exterior columns in frames	1.0	0.8	0.7	0.6
Corner Columns in frames, supporting-columns	1.2	1.0	0.9	0.8

Note: When HRB400 hot rolling steel reinforcements are adopted, the values in table may be reduced by 0.1; when the strength grade of the concrete is higher than C60, the value shall increase by 0.1.

2 The hoops of column shall be densified in the provision regions; the spacing and diameter of such hoops shall comply with following requirements:

1) In general, maximum spacing and minimum diameter of hoops shall be taken in accordance with Table 6.3.8-2.

Table 6.3.8-2 Maximum spacing and minimum diameter for hoops in the column hoop densified regions

Grade of frames	Maximum spacing of hoops (smaller value shall be taken)/(mm)	Minimum diameter of hoops (mm)
Grade 1	6d, 100	10
Grade 2	6d, 100	8
Grade 3	8d, 150 (in column bottom 100)	8
Grade 4	8d, 150 (in column bottom 100)	6 (in column bottom 8)

Note: *d* refers to the minimum diameter of the longitudinal bar in the column; the column bottom refers to the fixing section of the low end of column on the first story.

2) For frames assigned to Grade 2, when the diameter of hoops is not less than 10mm and distance between crossties or legs of compound hoop not greater than 200 mm, unless the column bottom, the maximum spacing of hoops shall be permitted to adopt 150mm. For frames assigned to Grade 3, when the cross-sectional dimension of columns is not greater than 400mm, the minimum diameter of the hoop shall be permitted to adopt 6mm. For frames assigned to Grade 4 with the shear span-to-depth ratio not greater than 2, the diameter of the hoops shall not be less than 8mm.

3) For supping-columns of frame-support-wall structures and columns with the shear span-to-depth ratio not greater than 2, spacing of hoops shall be not greater than 100 mm.

6.3.9 Arrangement of longitudinal bars in the column shall also comply with the following requirements:

1 They should be arranged symmetrically.

2 Spacing of longitudinal bars should not be greater than 200 mm for columns that the cross-sectional dimension is greater than 400 mm.

3 Total reinforcement ratio of the column shall not be greater than 5%.

4 For the columns with shear span-to-clepth ratio not greater than 2 and assigned to Grade l, the reinforcement ratio on each side should not be greater than 1.2%.

5 When small eccentric tension occurs at the side column, the corner column and the end column of the wall under the seismic action, the total cross-sectional area of longitudinal bars of such columns shall increase 25% of that calculated necessary value.

6 The binding wire splices of longitudinal bars in the column shall be permitted only within the non-densified regions of the hoops of the column.

6.3.10 The regions of densified hoops in the column shall comply with the following provisions:

1 For the all of ends of the columns, the length from each joint face shall not be less than the largest of the depth of column at the joint face, 1/6 of the clear column height and 500mm.

2 For column in the first story, the length from fixed point of column shall not be less than 1/3 of the clear column height; when rigid ground surface exists, the length shall also be taken from 500mm upper to 500mm lower of the rigid ground surface.

3 For columns with shear span-to-depth ratio not greater than 2, and columns with ratio of the clear column height to depth not greater than 4 which is caused by the filling wall etc., shall taken as the overall height of column.

4 For supporting-columns of frame-support-wall structures, shall be taken as the overall height of column.

5 For the corner columns of frames assigned to Grade 1 or 2, shall be taken as the overall height of column.

6.3.11 The distance between the crossties or legs in the densified regions of hoops of columns should not exceed 200mm for frames assigned to Grade 1, not exceed the larger of 250mm and 20 times of the hoop diameter for Grade 2 or 3, and not exceed 300mm for Grade 4.

The crossties or legs should be arranged in two directions for every other longitudinal bars for confinement; when the compound hoops are used, the crossties or legs should hook together with the longitudinal bar and the hoop.

6.3.12 The volumetric ratio of spiral or hoop reinforcement in the densified regions of the column shall comply with the following equation:

$$\rho_v \geqslant \lambda_v f_c / f_{yv} \tag{6.3.12}$$

where: ρ_v——volumetric ratio of spiral or hoop reinforcement in the densified regions of the column; which shall not be less than 0.8% for frames assigned to Grade 1, 0.6% for frames assigned to Grade 2, and 0.4% for frames assigned to Grade 3 and 4 respectively. When calculating the volumetric ratio for overlapping hoops, the volume in the overlapping parts shall be reduced;

f_c——specified compressive strength of concrete; when the strength grade is lower than C35, the calculation shall be done according to C35;

f_{yv}——specified tensile strength of hoop; when exceeds 360N/mm^2, equal to 360N/mm^2;

λ_v——the minimum hoop characteristic value, which should be taken as according to Table 6.3.12.

Table 6.3.12 Minimum hoop characteristic value

Grades	Type of hoops	Axial-force-ratio								
		≤0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.05
Grade 1	Ordinary hoop, compound hoop	0.10	0.11	0.13	0.15	0.17	0.20	0.23		
	Spiral hoop, compound or continuous compound rectangular spiral hoop	0.08	0.09	0.11	0.13	0.15	0.18	0.21		

Table 6.3.12 (continued)

Grades	Type of hoops	Axial-force-ratio								
		≤0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.05
Grade 2	Ordinary hoop, compound hoop	0.08	0.09	0.11	0.13	0.15	0.17	0.19	0.22	0.24
	Spiral hoop, compound or continuous compound rectangular spiral hoop	0.06	0.07	0.09	0.11	0.13	0.15	0.17	0.20	0.22
Grade 3	Ordinary hoop, compound hoop	0.06	0.07	0.09	0.11	0.13	0.15	0.17	0.20	0.22
	Spiral hoop, compound or continuous compound rectangular spiral hoop	0.05	0.06	0.07	0.09	0.11	0.13	0.15	0.18	0.20

Notes: 1 Ordinary hoop refers to single rectangular hoops and circular hoops. compound hoops refer to hoops formed by rectangular hoops and rhombic, polygonal, circular hoops or crossties. compound spiral hoops refer to hoops formed by spiral hoop and rectangular, rhombic, polygonal, circular hoops or crossties; continuous compound rectangular hoops refer to all of spiral hoops that are made of one steel bar;

2 Supporting-columns should adopt compound spiral hoops or compound hoops, the minimum hoop characteristic factors shall increase 0.02 than the provisions in this table; and the volumetric ratio shall not be less than 1.5%;

3 Columns with shear span-to-depth ratio not greater than 2 should adopt compound spiral hoops or compound hoops, its volumetric ratio shall not be less than 1.2%, and shall not be less than 1.5% for Intensity 9;

4 When calculating the volumetric ratio of compound spiral hoops, the volume of its non-spiral hoops shll be multiplied by the reducing factor 0.8.

6.3.13 The volumetric ratio of stirrups in the non-densified regions of hoop of the column should be not less than 50% of that in the densified regions. And also spacing of stirrups shall be not greater than 10 times the longitudinal bar diameter for frames assigned to Grade 1 or 2 and 15 times for frames assigned to Grade 3 or 4 respectively.

6.3.14 The maximum spacing and minimum diameter of hoops at a node of the frame should conform to the provision in Table 6.3.8. The hoop characteristic values at the node of frames assigned Grade 1, 2 and 3 should not be less than 0.12, 0.10 and 0.08, and the hoop volumetric ratio should not be less than 0.6%, 0.5%, and 0.4% respectively. For columns with shear span-to-depth ratio not greater than 2, the hoop characteristic values in the node of the frame should not be less than the greater of the upper and lower column ends of that node.

6.4 Details of seismic design for the wall structures

6.4.1 The thickness of a wall shall not be less than 160 mm and/or 1/20 of the story height for structures assigned to Grade 1 or 2, and 140mm and/or 1/25 of the story height for structures assigned to Grade 3 or 4. For the strengthening portion at the bottom of the wall assigned to Grade 1 or 2, the thickness of wall should not be less than 200mm and/or 1/16 of the story height; and 1/12 of the story height where walls without end-columns or flanges.

6.4.2 When the thickness of the wall is greater than 140mm, the vertical and lateral distribution web reinforcements shall be arranged in two layers, spacing of ties between two layers shall not exceed 600 mm, and the diameter shall not be less than 6mm. For the

strengthening portion at the bottom, spacing of the tie bars beyond the boundary elements shall be densified accordingly.

6.4.3 The vertical and horizontal distributed web reinforcements in a wall shall comply with the following requirements:

1 The minimum reinforcement ratio for the vertical and horizontal distributed reinforcements of the wall shall not be less than 0.25 % for structures assigned to Grade 1, 2, and 3; and 20% for Grade 4. The maximum reinforcement spacing each way in the wall shall not exceed 300 mm, and the minimum diameter of the reinforeements bar shall not be smaller than 8 mm.

2 At the strengthening portion at the bottom of the wall for frame-support-wall structures, the reinforcement ratio each way shall not be less than 0.3 %, and the reinforcement spacing each way shall not exceed 200mm.

6.4.4 The diameter of vertical and horizontal distributed reinforcement bars should not be greater than 1/10 of the wall thickness.

6.4.5 The axial-force-ratio of the strengthening portion at bottom of the wall subjected to the gravity load representative value should not be greater than 0.4 for structures assigned to Grade 1 with Intensity 9, 0.5 for structures assigned to Grade 1 with Intensity 8, and 0.6 for structures assigned to Grade 2.

6.4.6 The two ends and opening sides of the wall shall be installed boundary elements according to the following provisions:

1 For wall structure assigned to Grade 1 or 2, the strengthening portion at bottom and its adjacent upper one story shall be installed confining boundary elements in accordance with the provision of the Clause 6.4.7. But if the axial-force-ratio at bottom surface of the wall subjected to the gravity load representative value is smaller than the provision value in Table 6.4.6, the ordinary boundary elements may be installed according to Clause 6.4.8 of this chapter.

Table 6.4.6 Maximum axial-force-ratio of walls for installing the ordinary boundary elements

Grade and/or Intensity	Grade 1with Intensity 9	Grade 1 with Intensity 8	Grade 2
Axial-force-ratio	0.1	0.2	0.3

2 For the wall continued to ground of frame-support-wall structures assigned to Grade 1 or 2, the strengthening portion at bottom and its adjacent upper one story shall be installed the flanges wall or end-columns which comply with confining boundary elements, and the opening sides shall be installed confining boundary elements. For the discontinuous walls of frame-support-wall structures assigned to Grade 1 or 2, the strengthening portion at bottom and its adjacent upper one story shall be installed confining boundary elements.

3 At other portion of wall assigned to Grade 1 or 2 and all of wall assigned to Grade 3

or 4, ordinary boundary elements shall be installed according to Clause 6.4.8 of this chapter.

6.4.7 The confining boundary elements of the wall include the hidden column, the end-column, and the flange wall (figure 6.4.7). The length of the confining boundary elements measured from extreme compression fiber toward to neutral axis of web and the hoop characteristic value should comply with the requirements in Table 6.4.7. And the reinforcement ratio of longitudinal bars within the range of hoops (the shadowy part of figure 6.4.7) shall not be less than 1.2% and 1.0% for Grade 1 and 2 respectively.

Table 6.4.7 confining boundary elements length l_c and hoop characteristic value λ_v

Item	Grade 1 with Intensity 9	Grade 1 with Intensity8	Grade 2
λ_v	0.2	0.2	0.2
l_c(hidden column)	$0.25h_w$	$0.20h_w$	$0.20h_w$
l_c(flange wall or end-column)	$0.20h_w$	$0.15h_w$	$0.15h_w$

Notes: 1 When the width of the flange wall is less than 3 times of the flange thickness or when the least cross-sectional dimension of end-column is less than 2 times of the web thickness, it shall be deemed as no flange wall or no end-column.

2 l_c refers to the length of the confining boundary elements along web; and it shall not be less than the greatest of the values in the table, $1.5b_w$ and 450mm, when there is flange wall and end-column, it shall not be less than the thickness of the flange wall or the depth of the end-column plus 300mm.

3 λ_v refers to the hoop characteristic value of the confining boundary element; when calculating the volumetric ratio of hoops, if the strength design value of the hoop or crossties exceed 360N/mm^2, shall equal to 360N/mm^2; the vertical spacing of hoops or crossties should not be greater than 100mm for structures assigned to Grade 1, and than 150mm for structures assigned to Grade 2.

4 h_w refers to the lateral length of section for the wall segment of wall.

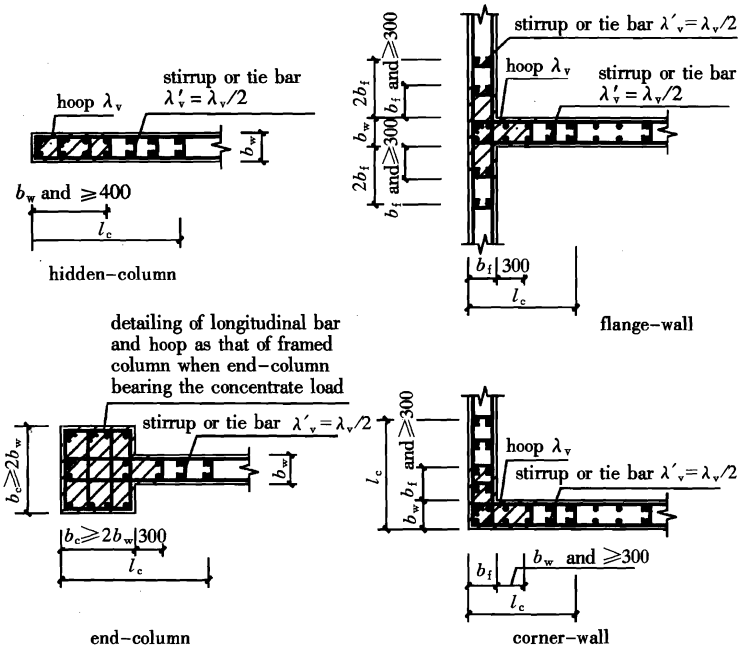

Figure 6.4.7 confining boundary element of wall

6.4.8 The range of ordinary boundary elements of the wall shall be adopted according to figure 6.4.8; the arrangement of reinforcement for the ordinary boundary elements shall satisfy the requirements for bending capacity as well as the requirements in Table 6.4.8.

Table 6.4.8 Reinforcement requirements for ordinary boundary elements

Grade	Strengthening portion on bottom			Other portion		
	Minimum amount of longitudinal bars (greater value shall be taken)	Minimum stirrup diameter (mm)	Maximum stirrup spacing (mm)	Minimum amount of longitudinal bars	Minimum tile diameter(mm)	Maximum tile spacing(mm)
Grade 1	$0.010A_c$, 6ϕ16	8	100	6ϕ14	8	150
Grade 2	$0.008A_c$, 6ϕ14	8	150	6ϕ12	8	200
Grade 3	$0.005A_c$, 4ϕ12	6	150	4ϕ12	6	200
Grade 4	$0.005A_c$, 4ϕ12	6	200	4ϕ12	6	200

Notes: 1 A_c refers to the calculated area of the hidden column or end-column of the longitudinal bars of the boundary elements, i.e. the shadowy part of Figure 6.4.8.

2 For other portions, the horizontal spacing of ties shall not be greater than 2 times of spacing of the longitudinal bars, hoops should be used at the corners.

3 When the end-column subjected to concentrating loads, the diameters of its longitudinal bars, hoops, and spacing shall satisfy corresponding requirements of the column.

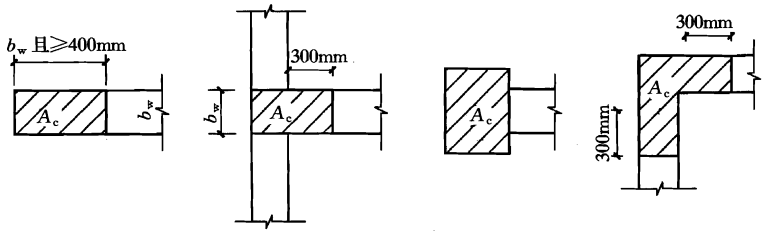

Figure 6.4.8 Range of ordinary boundary element of wall

6.4.9 When the largest cross-sectional dimension of the wall piers is not greater than 3 times of the thickness, such wall piers design shall be carried out according to the requirements for columns; and stirrups shall be densified through overall height of the wall piers.

6.4.10 The coupling beams with span-to-depth ratio not greater than 2 and width not less than 200mm in the wall structures assigned to Grade 1 or 2, besides ordinary hoops, shall be reinforced with additive two intersecting groups of diagonally placed bars symmetrical about the midspan.

6.4.11 Within the developed length of longitudinal reinforcement bars in the coupling beam of the top story, hoops shall be arranged.

6.5 Details of seismic design for frame-wall structures

6.5.1 Thickness of a wall shall be not less than 160 mm, and also not less than 1/20 of the story height; thickness of wall at the strengthening portion of the bottom shall not be

less than 200mm, and also not less than 1/16 of the story height.

In the perimeter of the wall panel, a boundary frame formed by beams (or hidden beams) and end-columns shall be made. The cross-sectional dimensions of the end-column should be the same as that of the frame column in the same story and shall also satisfy the requirements for framed columns in Section 6.3 of this chapter.

For the end-columns at the strengthening portion at the bottom of the wall and the end-columns immediately next to the opening, stirrups should be densified according to the requirements for framed columns and through overall height of end-columns.

6.5.2 The vertical and horizontal distribution reinforcement ratios in web of a wall shall not be less than 0.25%, and the reinforcements shall be arranged in two layers, spacing of ties between two layers shall not exceed 600 mm, and the diameter shall not be less than 6mm.

6.5.3 The other design details for frame-wall structures shall be in accordance with the requirements of frame and wall of Section 6.3 and 6.4 in this chapter respectively.

6.6 Seismic design requirements for slab-column-wall structures

6.6.1 The design details of the wall for slab-column-wall structures shall comply with relevant provisions in Section 6.4 of this chapter. The confined boundary elements of wall shall be arranged at the strengthening portion at the bottom and the adjacent upper one story according to the requirements of Clause 6.4.7 of this chapter. And the ordinary boundary elements of other portions of wall shall be arranged according to Clause 6.4.8 of this chapter. The design details for the columns and end-columns of wall shall satisfy relevant provisions regarding frame columns in Section 6.3 of this chapter.

6.6.2 The framed beams shall be arranged at the perimeter of the floor and of the opening for the stairs and the elevators.

6.6.3 The drop panel or column capitals shall be installed for the slab-column-wall structures for Intensity 8. The thickness (including thickness of slab) of the drop panel and of the root of column capital shall not be less than 16 times of the longitudinal reinforcement diameter. The length of each direction for the drop panel or the column capital should not be less than the sum of the 4 times of the slab thickness and cross-sectional dimension of the column in that direction.

6.6.4 The roof slab of the building and the first underground story of the slab-column-wall structures should adopt the beam-slab structural system.

6.6.5 The wall of the slab-column-wall structures shall resist all the seismic action of the structure; the slab-column portion in each story shall satisfy the requirements of the calculation, and shall resist a least 20% of the total seismic action of that story.

6.6.6 When the slab-column-wall structure under the earthquake is analyzed by using the equivalent plane-framed model, the width of the equivalent beam should adopt 50% of the

span length measured from center-to-center of supports in orthogonal to the equivalent plane-frame direction.

6.6.7 The ordinary hidden beams of slabs should be arranged on the column strip without column capitals. The width of the hidden beam may be taken as the width of the column plus 1.5 times of the thickness of slab on the both sides of the column. The reinforcement area in the top of the hidden beam at supporting face shall not be less than 50% of that of the column strip, and the reinforcement area in the bottom of the hidden beam shall not be less than 1/2 that of the top.

6.6.8 The lap spliced of reinforcements in the slab bottom at the column strip without column capital should be beyond over 2 times of the developed length of longitudinal bars measured from the column face; and should have hooks orthogonal to the slab face at the reinforcement ends.

6.6.9 The total section areas of the continuous reinforcements at the slab bottom, that pass the column core along the both principal axial directions, shall comply with the following equation:

$$A_s \geqslant N_G / f_y \tag{6.6.9}$$

where: A_s——total cross-sectional area of the continuous reinforcements at the bottom of the slab.

N_G——the column axial compression subjected to the gravity load representative value of the same story.

f_y——the specified tensile strength of reinforcements.

6.7 Seismic design requirements for tube structures

6.7.1 Frame-core tubular structures shall comply with the following requirements:

1 Floors between the core tube and the perimeter frame should be adopted the beam-slab structural system.

2 When the Intensify is lower than 9 and the stories strengthening with outrigger members is arranged, the outrigger girder or trusses shall be run-through to the wall of the core tube, and the joints of outrigger girder or trusses and perimeter frame columns should be adopted hinges or semi-rigid connections.

3 The analysis of the integer structure shall be taken into consideration of the effect due to the deformation of the stories strengthening with outrigger members.

4 The stories strengthening with outrigger members shall not be adopted for Intensity 9.

5 The construction procedure and connection details shall be adopted to reduce the effect to the stories strengthening with outrigger members under the vertical deformation caused by temperature and the axial compresses of the structure.

6.7.2 The walls in the core tube of the frame-tube structure and in the inner tube of the

tube-in-tube structure shall comply with relevant provisions in Section 6.4 of this chapter. Moreover, the thickness of the wall, the vertical and horizontal distribution reinforcements of web shall comply with the provisions in Section 6.5 of this chapter. In the strengthening portion at the bottom of the tube and its adjacent upper one story, the thickness of walls shall not be changed. The boundary elements at the corner of structures assigned to Grade 1 or 2 shall be strengthened according to the following requirements:

For the confined boundary elements in the strengthening portion at bottom, its length shall be taken as 1/4 of the lateral length of the wall segment, and its transverse reinforcement shall only adopt hoops. For the scope of the overall height above the strengthening portion at the bottom, the confined boundary elements should be arranged according to corner wall as provisions in Figure 6.4.7 of this chapter, its length shall still taken as 1/4 of the lateral length of the wall segment.

6.7.3 The door opening on the inner tube should not be placed near the corner of tube.

6.7.4 The floor girders should be neither arranged by concentrating on the corner of the inner tube or core tube nor arranged on the coupling beam; hidden columns should be arranged in locations supporting the floor girders within the inner tube or core tube.

6.7.5 The coupling beams with span-to-depth ratio not greater than 2 for core tube or inner tube structures assigned to Grade 1 or 2 shall comply with following requirements:

When the width of the coupling beam is not less than 400mm, that shall be reinforced with additive two intersecting groups of diagonally placed embedded columns composed by the longitudinal bars and stirrups symmetrical about the midspan. All the shear force shall be resisted by the reinforcements of the hidden column, and ordinary stirrups shall be arranged in accordance with the framed beams also.

When the width of the beam cross section is less than 400mm but not less than 200mm, besides ordinary stirrups, that shall be reinforced with additive two intersecting groups of diagonally placed bars symmetrical about the midspan.

6.7.6 The seismic design of the transference story of tube structure shall comply with the provisions in Appendix E.2 of this code.

Chapter 7 Multi-story Masonry Buildings and Multi-story Brick Buildings with Bottom-frame or Inner-frame

7.1 General

7.1.1 This chapter is applicable to multi-story fired clay brick buildings, to multi-story concrete small hollow block buildings, as well as to multi-story brick buildings with frame-wall in the first story or first and second stories(hereinafter refer to multi-story brick buildings with bottom-frame)and to multi-story brick buildings with inner-multi-column frames (hereinafter refer to "multi-story brick buildings with inner-frame").

The seismic design of reinforced wall structures using concrete small hollow block shall conform to the provision in Appendix F of this code.

Notes: 1 In this chapter, "fired common clay brick, fired clay perforated brick, concrete small hollow block" hereinafter refer to "common brick, perforated brick, and small block" respectively. For masonry buildings using other fired bricks and autoclaved bricks, the material property of unit shall have reliable testing data. When the shear strength of the masonry is not less than that of the clay brick masonry, it shall be carried out according to relevant provision for clay brick buildings of this charter.

2 For masonry buildings using autoclaved sand-lime bricks or fly-ash-lime bricks for Intensity 6 and 7, when the shear strength of the masonry is not less than 70% that of the clay bricks, the seismic design shall comply with follows:

The maximum number of stories shall reduce by one story than that of clay brick buildings, the maximum total height shall reduce by 3m, and the tie-columns shall be installed according to one story more for corresponding clay brick buildings, other requirements shall comply with the provisions for clay brick buildings.

3 The "multi-story brick buildings with frame-seismic-structural wall in the first story or first two stories" hereinafter refer to "multi-story brick buildings with bottom-frame"; and the "multi-story brick buildings with inner-multi- column frames".

7.1.2 The total height and number of stories of multi-story buildings shall comply with the following requirements:

1 For usual masonry buildings, the total height and number of stories shall not exceed the limits in Table 7.1.2.

2 For multi-story buildings with rather less transverse walls, as well as hospitals or schools, the limits value of total height shall be decreased by 3m from the values in Table 7.1.2, and of the stories shall be decreased by one. For multi-story buildings with a few of transverse walls, the total height and the stories shall be reduced based on actual condition.

Note: building with rather less transverse walls refer to that rooms with span greater than 4.20m take up more than 40% of the areas in the same story.

3 For multi-story brick living buildings with rather less transverse walls, when the strengthened measures has be taken according to the provision and the seismic capacity of wall is sufficient, the total height and number of stories shall be permitted to adopt the limits in Table 7.1.2.

Table 7.1.2 Limit values of total height and number of stories

Type of building		Min. wall thickness	Intensity							
			6		7		8		9	
			Height	Stories	Height	Stories	Height	Stories	Height	Stories
Multi-story masonry	Common brick	240mm	24m	8	21m	7	18m	6	12m	4
	Perforated brick	240mm	21m	7	21m	7	18m	6	12m	4
	Perforated brick	190mm	21m	7	18m	6	15m	5	—	—
	Small block	190mm	21m	7	21m	7	18m	6	—	—
With bottom-frame		240mm	22m	7	22m	7	19m	6	—	—
With inner-frame		240mm	16m	5	16m	5	13m	4	—	—

Notes: 1 Total height of the building refers to the height from the ground level to top of the main roof slab. For semi-basement, the height is counted from the indoor ground of the basement; for basement and semi-basement with better fixing conditions, the height shall be counted from outdoor ground; for slope roof with garret, the height shall be counted to 1/2 height of the gable.

2 When the indoor and outdoor height difference is greater than 0.6m, the total height of the building shall be permitted to increase appropriately but cannot be more than 1m.

3 The small block buildings in this table do not includ the reinforced small block buildings.

7.1.3 For the common brick, perforated brick and small block buildings, the story-height shall not exceed 3.6m. For the bottom stories of the brick buildings with bottom-frame and the brick buildings with inner-frame, the story-height shall not exceed 4.5m.

7.1.4 Maximum ratio of the total height to total width for multi-story masonry buildings should conform to Table 7.1.4

Table 7.1.4 Maximum ratio of total height to total width for buildings

Intensity	6	7	8	9
Max. ratio	2.5	2.5	2.0	1.5

Notes: 1 The total width of buildings with an external corridor does not includ the width of the corridor;

2 When the plain of the building is close to square, the ratio shall be reduced accordingly

7.1.5 Maximum spacing of adjacent transverse walls in buildings shall not exceed the requirements in Table 7.1.5.

Table 7.1.5 Maximum spacing of adjacent transverse walls (m)

Type of building and type of floor or roof		Intensity			
		6	7	8	9
Multi-story masonry	Cast-in-situ or precast-monolithic reinforced concrete	18	18	15	11
	Fabricated reinforced concrete	15	15	11	7
	Timber	11	11	7	4
Multi-story masonry with bottom-frame building	All masonry stories above the framed stories	Same as multi-story masonry			—
	First story or first and second frame stories	21	18	15	—
Multi-story masonry with inner-frame building		25	21	18	—

Notes: 1 For top story of multi-story masonry buildings, the maximum spacing requirement of transverse walls shall be permitted loosened.

2 The provision for timber floor and roof in the table is not applicable to small block buildings.

7.1.6 The limitation of local dimension for masonry wall should comply with the requirements in Table 7.1.6:

Table 7.1.6 Limitation of local dimension for masonry wall (m)

Location	Intensity6	Intensity7	Intensity8	Intensity9
Min. width of a bearing wall between windows	1.0	1.0	1.2	1.5
Min. distance from a bearing exterior wall end to the edge of the door or window opening	1.0	1.0	1.2	1.5
Min. distance from a non-bearing exterior wall end to the edge of the door or window opening	1.0	1.0	1.0	1.0
Min. distance from the salient angle of inter wall to the edge of the door or window opening	1.0	1.0	1.5	2.0
Max. height of parapet without anchorage (not at entrance)	0.5	0.5	0.5	0.0

Notes: 1 The strengthening measures shall be taken when the local scales are insufficient;

2 Parapet in exit and/or entrance shall be anchored;

3 The width of an exterior longitudinal wall between windows for multi-story brick buildings with inner-frames shall not be less than 1.5m.

7.1.7 The structure system of multi-story masonry buildings shall comply with the following requirements:

1 The structure system of bearing by transverse wall or of bearing by both longitudinal and transverse walls shall be adopted with priority.

2 The arrangement of transverse and longitudinal walls should be symmetrical, even, and aligned in-plane, shall be continued from footing to top, and the widths of all wall-segments between windows in an axis should be equivalent.

3 The seismic joints should be installed if the building has one of the following cases as well; The walls shall be arranged on both sides of the joint, and the joint clear width shall be determined dependent on the fortification intensity and the height of the building, that may be taken as 50～100mm.

1) The height difference in elevation of the building is greater than 6 m;

2) The building having staggered-floor with significant level differences;

3) The stiffness and mass of every parts of a structure are completely different.

4 The staircase should not be arranged at the end and corner of the building.

5 The flue, air duct, and refuse chute shall not weaken the walls; in case the wall is weakened, the strengthening measures shall be taken. For chimney without vertical reinforced, which is adhered to wall or having height exceeding roof surface, should not be adopted.

6 The precast reinforced concrete eaves plate without anchorage shall not be used.

7.1.8 The structural arrangement of brick buildings with bottom-frames shall comply with the following requirements:

1 All of the masonry walls above the bottom-frame story shall be or shall basically be level by the frame-beams or walls at the bottom.

2 A certain number of walls shall be installed along both the longitudinal and transverse directions at the bottom of the building, and the walls shall be arranged symmetrically or basically symmetric. If multi-story brick buildings with framed first story, that not more than 5 stories and for Intensity 6 and 7, the masonry walls at the first story may be adopted, but the additional axial force and shear force of frame caused by masonry wall shall be taken into consideration. The reinforced concrete walls shall be adopted for other cases.

3 In the longitudinal and transversal directions of multi-story brick building with framed first story, the lateral rigidity ratio of the second story to the first story shall not be greater than 2.5 for Intensity 6 and 7, and than 2.0 for Intensity 8; both shall not be less than 1.0.

4 In the longitudinal and transversal directions of multi-story brick building with framed first and second stories, the lateral rigidity ratio of third story to second story shall not be greater than 2.0 for Intensity 6 and 7, and than 1.5 for Intensity 8; both shall not be less than 1.0. And the lateral rigidity of the first to the second story shall be approximately equivalent.

5 The strip foundation, raft foundation or pile foundation shall be adopted for the walls of brick buildings with bottom-frames.

7.1.9 The structural arrangement for multi-story brick buildings with inner-frames shall comply with the following requirements:

1 Rectangular plane should be adopted for the building, and the elevation should be regular; the transversal wall of the stair should be installed through the full width of the building.

2 For the spacing of adjacent transversal wall is greater than 18m for Intensity 7 and greater than 15m for Intensity 8, the composite columns shall be installed at longitudinal external walls between windows.

3 The strip foundation, raft foundation or pile foundation shall be adopted for the walls of brick buildings with inner-frames.

7.1.10 The seismic design of reinforced concrete structural parts of brick buildings with bottom-frames and inner-frames shall comply with both the provisions in this Chapter and relevant requirements in Chapter 6 of this code. Meanwhile, the seismic measure grades for brick buildings with bottom-frame, the frame and the wall shall be taken as Grade 3, 2, and 1 for Intensity 6, 7, and 8 respectively. And the seismic measure grades for brick buildings with inner-frames shall be taken as Grade 4, 3, and 2 for Intensity 6, 7, and 8 respectively.

7.2 Essentials in calculation

7.2.1 The base shear method may be used in the seismic calculation for multi-story masonry buildings, and brick buildings with bottom-frame or inner-frame, and the seismic effects shall be adjusted in accordance with the provisions of such section.

7.2.2 For masonry buildings, the seismic checking of walls may only be made that with greater subordinating areas or with less vertical stress.

7.2.3 When carrying out seismic shear force distribution and seismic checks, the story equivalent lateral stiffness of the masonry wall shall be determined according to the principle as follows:

1 For the calculation of stiffness, the influence on height-width ratio of wall-segment shall be taken into consideration. When this ratio is less than 1, only shear deformation of wall needs to be taken into account. When this ratio is not greater than 4 and not less than 1, both the bending and shear deformation shall be taken into consideration. And this ratio is greater than 4, the equivalent lateral stiffness may be taken as 0.0.

Note: The height-width ratio refers to the ratio between the height of story and the lateral-length of wall. In the case of spacing-walls between the opening of the door and the windows, it refers to the ratio between the clear height of the opening and the wall width on the sides of the opening.

2 The wall-segments should be divided according to the openings of the door and the

windows. For stiffness calculated accordingte gross wall surface in the small opening wall sections, the calculation may multiply the opening reduced factors which depend upon the opening rate and as per set in Table 7.2.3:

Table 7.2.3 Opening reduced factors

Opening rate	0.10	0.20	0.30
Reduced factor	0.98	0.94	0.88

Note: the opening rate is the ratio between the opening area and the gross area of the wall; when the height of window opening is greater than 50% of the story height, it shall be treated as door opening.

7.2.4 Seismic effects of brick buildings with bottom-framed shall be adjusted according to the following provision:

1 For brick buildings with framed first story, the first story longitudinal and transversal seismic shear force design value shall be multiplied by an amplifying factor. The value of this amplifying factor shall be permitted to be selected in the range from 1.2 to 1.5 according to the lateral stiffness ratio between the second story and the first story.

2 For brick buildings with framed first and second stories, the longitudinal and transversal seismic shear force design value of the first story and the second story shall all be multiplied by an amplifying factor. The value of this amplifying factor shall be permitted to be selected in the range from 1.2 to 1.5 according to the lateral stiffness ratio.

3 All of longitudinal and transversal seismic shear force design value of the first story and the second story shall be resisted by the wall of corresponding direction separately, and the distribution shall be made according to the lateral stiffness ratio of every wall.

7.2.5 Seismic effect of frames in brick buildings with bottom-frame shall be determined by the following method:

1 The seismic shear force and axial force of the bottom-framed columns should be adjusted according to the following provision:

1) Design value of seismic shear force resisted by framed columns may be determined in proportion to the effective lateral stiffness of every lateral-force-resisting member. The value of the effective lateral stiffness may not be reduced for the frame, and may be multiplied by 0.30 for the reinforced concrete wall, and may be multiplied by 0.20 for the clay brick wall.

2) Additional axial force caused by the seismic overturning moment shall be considered in the calculation of the axial force of the framed column. The seismic overturning moment carried by the elements in all axes may be determined in proportion of the lateral stiffness of walls and frames in the bottom approximately.

2 When calculating the seismic combinatory inner force for the reinforced concrete

spandrel girder for brick buildings with bottom-frames, proper calculation figure shall be adopted. If the composite effect of the upper walls and its spandrel girder may be taken into consideration, the unfavorable influence caused by the cracking of wall during earthquake shall also be taken into consideration, relevant bending moment factors and axial factors shall also be adjusted.

7.2.6 Design value of seismic shear force for columns in multi-story brick buildings with inner- frames should be determined in accordance with the following equation:

$$V_c = \frac{\psi_c}{n_b \cdot n_s}(\zeta_1 + \zeta_2 \lambda) V \tag{7.2.6}$$

where: V_c——design value of seismic shear force of column.

V——design value of seismic shear force of story in spacing of transverse walls.

ψ_c——factor of type of columns, and may be taken as 0.012 for interior reinforced concrete columns, 0.0075 for composite brick columns in the exterior wall, and 0.005 for plain brick columns(or walls).

n_b——number of bays in spacing of transverse walls.

n_s——number of spans in inner-frame.

λ——the ratio between the spacing of transverse walls and the total width of the building, and may be taken as 0.75 when the ratio is less than 0.75.

ζ_1, ζ_2——factors for calculation, and may be taken in accordance with Table 7.2.6.

Table 7.2.6 Factors for calculation

Total number of stories of building	2	3	4	5
ζ_1	2.0	3.0	5.0	7.5
ζ_2	7.5	7.0	6.5	6.0

7.2.7 The design value for seismic shear strength along the ladder shaped damage of various masonry structures shall be determined according to the following equation:

$$f_{vE} = \zeta_N f_v \tag{7.2.7}$$

where: f_{vE}——the design value for seismic shear strength along the ladder shaped damage of masonry.

f_v——the design value for shear strength along the ladder shaped damage of masonry.

ζ_N——normal stress iufluence factors for the seismic shear strength of masonry, and shall be taken as from Table 7.2.7.

Table 7.2.7 Normal stress influence factor of masonry strength

Type of masonry	σ_0/f_v							
	0.0	1.0	3.0	5.0	7.0	10.0	15.0	20.0
Common brick, perforated brick	0.80	1.00	1.28	1.50	1.70	1.95	2.32	
Small block		1.25	1.75	2.25	2.60	3.10	3.95	4.80

Note: σ_0 refer to the mean pressure of the masonry cross section corresponding to gravity load representative value.

7.2.8 The seismic shear capacity for walls of common bricks and perforated bricks shall be checked according to the following provisions:

1 Generally, the check shall be made according to the following equation:

$$V \leqslant f_{vE}A/\gamma_{RE} \tag{7.2.8-1}$$

where: V——shear of wall of masonry structures.

f_{vE}——design value for seismic shear strength along the ladder shaped damage of masonry.

A——cross-sectional area of wall, the gross area of cross section for perforated brick wall.

γ_{RE}——seismic adjusting factor for shear bearing capacity, for bearing wall shall be taken as from Table 5.4.2 of this code, for self-bearing wall shall be taken as 0.75.

2 When the checking according to equation (7.2.8-1) fails to satisfy the requirements, the improving effect on the seismic shear capacity by the tie-columns may be taken according to the simplified methods as follows. However, such tie-columns are installed in the middle of the wall, cross-sectional areas not less than 240mm×240mm and spacing not greater than 4m.

$$V \leqslant \frac{1}{\gamma_{RE}}[\eta_c f_{vE}(A - A_c) + \zeta f_t A_c + 0.08 f_y A_s] \tag{7.2.8-2}$$

where: A_c——the total cross-sectional areas of middle tie-columns; for transversal and inner longitudinal wall, when $A_c > 0.15A$, taken as $0.15A$; for the external longitudinal wall, when $A_c > 0.25A$, taken as $0.25A$.

f_t——specified concrete tensile strength of tie-column in middle.

A_s——the total areas of reinforcements of tie-column in middle, that the steel bar ratio is not less than 0.6%. when it is greater than 1.4%, taken as 1.4%.

f_y——design value of reinforcement tensile strength.

ζ——the participation factor of tie-column in the middle; for only one tie-column,

taken as 0.5; for other cases, taken as 0.4.

η_c——the confined factor of wall; generally the factor adopts 1.0; when the spacing of the tie-columns is not greater than 2.8m, adopts 1.1.

7.2.9 For horizontal reinforced common brick and perforated brick walls, the seismic shear bearing capacity shall be checked in accordance with following equation:

$$V \leqslant \frac{1}{\gamma_{RE}}(f_{vE}A + \zeta_s f_y A_s) \tag{7.2.9}$$

where: A——cross-sectional area of wall, the gross area of cross section for perforated brick wall.

f_y——design value of reinforcement tensile strength.

A_s——the total areas of horizontal reinforcements in height of a story, the rein forcment ratio shall not be less than 0.07% and not greater than 0.17%.

ζ_s——the participation factor of reinforcement, may be taken as from Table 7.2.9.

Table 7.2.9 Participation factor of reinforcement

Ratio of height to width for wall	0.4	0.6	0.8	1.0	1.2
ζ_s	0.10	0.12	0.14	0.15	0.12

7.2.10 For small blocks wall, the seismic shear bearing capacity shall be checked in accordance with following equation:

$$V \leqslant \frac{1}{\gamma_{RE}}[f_{vE}A + (0.3f_t A_c + 0.05f_y A_s)\zeta_c] \tag{7.2.10}$$

where: f_t——specified concrete tensile strength of core-column.

A_c——total cross-sectional area of core-columns.

A_s——total cross-sectional area of reinforcements in core-column.

ζ_c——the participation factor of core-column, may be taken as from Table 7.2.10.

Note: when both core-columns and tie-columns are installed together, the cross-sectional areas of the tie-column may be treated as the cross-sectional areas of the core-column, and the reinforcement of the tie-column may also be treated as that of the core-column.

Table 7.2.10 Participation factor of core-column

Hole filling rate ρ	$\rho<0.15$	$0.15 \leqslant \rho<0.25$	$0.25 \leqslant \rho<0.5$	$\rho \geqslant 0.5$
ζ_c	0.0	1.0	1.10	1.15

Note: Hole filling rate refer to the ratio of number of core-columns to total hole numbers.

7.2.11 For multi-story brick building with framed first story, when walls with common bricks filled in the frames satisfy the detail requirements in Clause 7.5.6 of this chapter, the seismic check shall comply with the following provisions:

1 The axial and shear force of the frame columns in the first story shall take into consideration of the additional axial and shear force according to the following equations:

$$N_f = V_w H_f / l \qquad (7.2.11\text{-}1)$$

$$V_f = V_w \qquad (7.2.11\text{-}2)$$

Where: V_w——the shear force design value distributed to the wall; for walls existed on both sides of the column, the value may be taken as the greater one.

N_f——additional axial pressure design value of the frame column.

V_f——additional shear force design value of the frame column.

H_f, l——the story height and span of the frame respectively.

2 The seismic bearing capacity of the wall made with common bricks filled in the frame and the frame columns at the two ends of wall shall be checked according to the following equation:

$$V \leqslant \frac{1}{\gamma_{REc}} \Sigma (M_{yc}^{u} + M_{yc}^{l}) / H_0 + \frac{1}{\gamma_{REw}} \Sigma f_{vE} A_{w0} \qquad (7.2.11\text{-}3)$$

where: V——the shear force design value of the filled common brick wall and the frame columns at the two ends of wall.

A_{w0}——calculated horizontal sectional area of the brick wall. When wall has no opening, take as 1.25 times of the actual sectional area; when wall has opening, take as the net sectional area, but the sectional area of the wall, which width is less than 1/4 of the opening height, is not considered.

M_{yc}^{u}, M_{yc}^{l}——non-seismic bending bearing capacity design values at the upper and lower end of the frame columns in the first story respectively, and it may be determined by the provision in the current national standard "Code for design of concrete structure" GB 50010.

H_0——calculated height of first story frame column; when there are brick walls on the both sides of the column, take as 2/3 of clear height of the column; in other cases, take as the clear height of the column.

γ_{REc}——seismic adjusting factor for first story frame column bearing capacity, and may be taken as 0.8;

γ_{REw}——seismic adjusting factor for filled common brick wall bearing capacity, and may be taken as 0.9.

7.2.12 The seismic check for composite brick column at external wall of multi-story buildings with inner-frames shall be done according to the provision in Clause 9.3.9 of this code.

7.3 Details of seismic design for multi-story clay brick buildings

7.3.1 The cast in-situ reinforcement concrete tie-columns (hereinafter referred to as tie-column) for multi-story clay brick and perforated brick buildings shall be installed in accordance with the following requirements:

1 The location installed of tie-column shall comply with the requirements in Table 7.3.1 in generally.

2 For multi-story gallery-type or one-sided corridor buildings, the tie-columns shall be installed in accordance with the Table 7.3.1, but the building assumed with one more story, and the longitudinal walls on the both sides of the one-sided corridor shall be regarded as exterior walls.

3 For buildings with rather less transversal walls such as schools and hospitals, the tie-columns shall be installed in accordance with the Table 7.3.1, but the building assumed with one more stories. When such buildings adopt the gallery-type or one-sided corridor, that shall also comply with provision in Point 2 of this clause; but the following building assumed with two more stories: does not exceed 4 stories for Intensity 6, or 3 stories for Intensity 7, or 2 stories for Intensity 8.

Table 7.3.1 Requirements for arrangement of tie-columns for brick buildings

<table>
<tr><th colspan="4">Number of stories in building</th><th colspan="2" rowspan="2">Location of installation</th></tr>
<tr><th>Int. 6</th><th>Int. 7</th><th>Int. 8</th><th>Int. 9</th></tr>
<tr><td>4,5</td><td>3,4</td><td>2,3</td><td></td><td rowspan="3">Four corners of the exterior wall; Intersections of the transversal wall in the slit-level portion and the exterior longitudinal wall; Both sides of bigger openings; Intersections of interior wall and exterior longitudinal walls at large rooms</td><td>Four corners of the staircase and elevator shaft for Intensity 7 and 8; intersections of each 15m or the unit transversal wall and exterior longitudinal wall</td></tr>
<tr><td>6,7</td><td>5</td><td>4</td><td>2</td><td>Intersections of every other transversal wall (axis) and exterior wall; intersections of gable and interior walls; four corners of staircase and elevator shaft for Intensity 7 to 9</td></tr>
<tr><td>8</td><td>6,7</td><td>5,6</td><td>3,4</td><td>Intersections of interior wall and exterior wall, smaller piers of the interior wall, four corners of the staircase and elevator shaft of Intensity 7~9; Intersections of interior longitudinal and transversal wall for Intensity 9</td></tr>
</table>

7.3.2 The tie-columns of the multi-story common brick and perforated brick buildings shall comply with the following requirements:

1 The minimum crosssection for the tie-column may adopt 240mm × 180mm, the longitudinal bars should adopt 4ϕ12; spacing of the stirrups shall not be greater than 250mm, besides, in the upper and lower ends of the tie-column, the spacing of stirrups shall be reduced accordingly. When exceeding 6 stories for Intensity 7, exceeding 5 stories for Intensity 8, and for Intensity 9, the longitudinal bars of the tie-column shall adopt 4ϕ14, and the spacing of stirrups shall not exceed 200mm. For the tie-columns in the corners of the building, cross section and stirrups shall be increased accordingly.

2 The connection of the tie-column and the adjacent walls shall be built into horse-toothed joints, the 2ϕ6 tie bars shall be arranged in spacing each 500mm along the height of the wall, the length extending into the wall at each side should not be less than 1 m.

3 At the connection of the tie-column and the ring-beam, the longitudinal bars of the tie-column shall be through the ring-beam to ensure the continuation of longitudinal bars in the tie-column.

4 The tie-columns may not establish individual footing, but they shall extend to 500mm into the underground level, or shall be connected with the foundation ring-beam, whose buried depth less than 500mm underground.

5 When the building height and the number of stories are close to the limit values of Table 7.1.2, the spacing of tie-columns within the longitudinal and transversal walls shall also comply with the following requirements:

1) For the transversal wall, the spacing of tie-columns should not be greater than 2 times of the story height, and this spacing of tie-columns in lower 1/3 of stories should be reduced accordingly;

2) For the longitudinal walls, when the bays of building are greater than 3.9m, the exterior longitudinal walls shall adopt strengthening measures; the spacing of tie-columns of the interior longitudinal wall should not be greater than 4.2m.

7.3.3 The cast-in-situ reinforced concrete ring-beam of multi-story common brick and perforated brick buildings shall be installed in accordance with the following requirements:

1 For the buildings with precast reinforced concrete or timber floors and roof, ring-beams shall be installed as follows:

When the buildings assigned to bearing transversal wall system, ring-beams shall be installed according to the requirements in Table 7.3.3; when assigned to bearing longitudinal wall system, ring-beams shall be installed at each story and their spacing on the transversal wall shall be reduced accordingly.

2 Only the building with cast-in-situ or assembly-monolithic reinforcement concrete floors and roof that have reliable connection with the walls, the ring-beams shall be permitted not installed. But the strengthened reinforcements of in-situ slabs shall be arranged along the wall perimeters and shall be reliably connected with corresponding tie-columns.

Table 7.3.3 Requirements for installation of cast-in-situ reinforcement concrete ring-beam in brick buildings

Type of wall	Intensity		
	6,7	8	9
Exterior walls and interior longitudinal wall	At roof level, each floor level	At roof level, each floor level	At roof level, each floor level
Interior transversal wall	Ditto; the spacing at roof shall not be greater than 7m; spacing at the floor shall not be greater than 15m; corresponding location of the tie-column	Ditto; along all transversal wall at roof and the spacing shall not be greater than 7m; the spacing at floor shall not be greater than 7m; corresponding location of the tie-column	Ditto; all transversal walls at roof and each floor

7.3.4 The details of cast-in-situ reinforced concrete ring-beam in multi-story common and perforated brick buildings shall comply with the following requirements:

1 The ring-beam shall be enclosed; at the location of opening, the ring-beam shall be spliced with two limbs along the upper and lower of opening. The ring-beams should be installed in the same level of the precast slabs or immediate next to the bottom of the slab.

2 For no transversal wall exists within of ring-beam spacing required by Table 7.3.3, the reinforcements in the floor girder or the joint between precast slabs shall be used for the replacement of ring-beam.

3 The cross-sectional height of the ring-beam shall not be less than 120mm, and the reinforcements shall comply with the requirements in Table 7.3.4. The ring-beams added according to the requirements of point 3 of Clause 3.3.4 of this code, the cross-sectional height shall not be less than 180mm, and the reinforcement bar shall not be less than 4ϕ12.

Table 7.3.4 Requirements for reinforcement arrangement in ring-beam of brick buildings

Reinforcement	Intensity		
	6,7	8	9
Min. longitudinal bar	4ϕ10	4ϕ12	4ϕ14
Max. stirrup spacing (mm)	250	200	150

7.3.5 Roof and floors of multi-story common brick and perforated brick buildings shall comply with the following requirements:

1 The length for cast in-situ reinforced concrete roof or floor slabs extending to the transversal and longitudinal walls shall not be less than 120mm.

2 For precast reinforcement concrete floor or roof slab and the ring-beam is not installed at the same level of the slab, the length for the slab end extending into the exterior wall shall not be less than 120mm; into interior wall, than 100mm; and in to beam, than 80mm.

3 For the span of the precast slab is greater than 4.8m and is parallel to the exterior wall, the side of the precast slab next to the exterior wall shall be tied with the exterior wall or ring-beam.

4 The precast slabs of the large room at the end of the building, which assigned to the roof for Intensity 8 or to the floors and roof for Intensity 9, shall be tied with one another, as well as with the beam, wall or ring-beam, when the ring-beam is installed at the bottom of the slab.

7.3.6 The reinforced concrete girders or trusses of the roof or floor system shall be reliably connected with the wall, column (including tie-column) or ring-beam. The connection of the girder and the brick column shall not weaken the cross section of the brick column. For the independent brick columns, the top of each story shall have reliable connection in two

directions.

7.3.7 For the rooms with length greater than 7.2m of Intensity 7 or for Intensity 8 and 9, in the corners of exterior wall and intersection of exterior and inner wall, the tie bars of 2ϕ6 shall be installed in each 500mm along the height of the wall. Besides, the tie bars should be extended into the walls on each side with length not less than 1 m.

7.3.8 The staircase shall comply with the following requirements:

1 For the transversal wall and exterior wall of the staircase at top story for Intensity 8 and 9, 2ϕ6 reinforcement bars shall be installed overall length of wall and installed in each 500mm along the height of the wall. For Intensity 9, a 60mm thick reinforced concrete strip or a reinforced brick course shall be installed at the landing platform or middle level of the story in other stories of the staircase. For reinforced brick course, the strength grade of mortar shall not be less than M7.5, and the longitudinal reinforcement bars shall not be less than 2ϕ10.

2 For Intensity 8 and 9, the supporting length of the girder, which is at the staircase or the salient angle of the interior wall for the vestibule, shall not be less than 500mm, and the girder shall be connected with the ring-beam.

3 The precast waist slabs shall be reliably connected with the beam of the landing platform; the stairs with the cantilevered steps tread from wall or the steps riser interposed the walls shall not be adopted, and the plain brick railing shall not be adopted.

4 For staircase or elevator shaft exceeding the roof level, the tie-column shall extend to the wall top and shall connect with the ring-beam of the wall top. And its intersection of the interior and exterior walls, 2ϕ6 tie-bars shall be installed in each 500mm along the height of the wall; more over, the length for each side to extend into the wall shall not be less than 1 m.

7.3.9 The trusses of pitch roof shall be reliably connected with the ring-beam of the top story of building; the purlines and the roof slabs shall be connected with the walls or trusses. The tiles of eaves course at the entrance and exit of the building shall be anchored to the roof members. For Intensity 8 and 9, the stepwise piers at the top of longitudinal interior wall of the top story should be built up to support the gables.

7.3.10 The plain brick lintels shall not be adopted at the door or window openings. The supporting length of lintel shall not be less than 240mm from Intensity 6 through Intensity 8, and shall not be less than 360mm for Intensity 9.

7.3.11 The precast balcony slabs shall be reliably connected with the ring-beam and the cast-in-situ strip of the precast floor slab.

7.3.12 The post-built non-bearing partition wall shall comply with relevant provision of Section 13.3 in this code.

7.3.13 The foundation (included the pile capping) of the same structural unit should adopt foundation of the same type. The bottom of foundation shall be buried at the same lev-

el; otherwise, added foundation ring-beams shall be installed, and foundation shall be stepped on a slope 1:2.

7.3.14 For the total height and number of stories of multi-story common brick and perforated brick living buildings exceed the limit values listed in Table 7.1.2, the strengthening measures shall comply with following provisions:

1 The size of the largest bay in the building shall not be greater than 6.6m.

2 Within the same structural unit, the number of staggered-axis transversal wall should not exceed 1/3 of the total number of walls; more over, successive staggered-axis walls should not exceed two. The added tie-columns shall be installed at fall of intersection of the staggered-axis walls and longitudinal walls, and the floors and roof shall adopt in-situ reinforced concrete slabs.

3 The width of opening in the transversal wall and the interior longitudinal walls should not be greater than 1.5m; the width of opening in the exterior longitudinal wall should not exceed 2.1m or 50% of the bay dimension. More over, the locations of these opening on the interior and exterior walls shall not affect the integral connections between the interior and/or exterior longitudinal walls and transversal walls.

4 The in-situ strengthening reinforcement concrete ring-beam shall be installed for each transversal and longitudinal wall in the floors and roof. The cross-sectional height of the ring-beam should not be less than 150mm, the upper and lower longitudinal reinforcement bars shall not be less than 3ϕ10, the stirrup diameter shall not be less than ϕ6, and the spacing of stirrup shall not be greater than 300mm.

5 In the intersections of all transversal and longitudinal walls as well as the middle of the transversal walls, the added tie-columns shall be installed in accordance with following requirements:

The column spacing within the transversal wall should not be greater than the story height, the spacing of column within the longitudinal walls should not be greater than 4.2m;

The minimum cross section of tie-columns should not be less than 240mm × 240mm; the reinforcements should comply with the requirements in Table 7.3.14.

Table 7.3.14 Requirements for longitudinal bars and stirrups in the added tie-column

Location	Longitudinal bars			Stirrup		
	Max. steel ratio (%)	Min. steel ratio (%)	Min. diameter(mm)	Scope of densified zone (mm)	Spacing in densified zone (mm)	Min. diameter(mm)
Corner column	1.8	0.8	14	Full height	100	6
Side column	1.8	0.8	14	Upper end 700 Lower end 500	100	6
Middle column	1.4	0.6	12			

6 The floors and roof of the same structural unit should been installed at the same level.

7 At the windowsill level of the top and first story of the building, the cast-in-situ reinforced concrete horizontal strip should be installed along overall length of the transversal walls and longitudinal walls. The cross-sectional height of this strip shall not be less than 60mm, the width shall not be less than 240mm, and the longitudinal bars shall not be less than 3ϕ6.

7.4 Details of seismic design for multi-story small-block buildings

7.4.1 The reinforced concrete core columns (hereinafter refer to core-column) for small-block buildings shall be installed in accordance with the requirements of Table 7.4.1. For buildings with rather less transversal walls such as hospital and school, core-columns shall be installed in accordance with the Table 7.4.1, but the building assumed with one more stories.

Table 7.4.1 Requirements for core-columns installed in small-block buildings

Number of stories			Location of core-columns	Number of core-columns (filled holes)
Int. 6	Int. 7	Int. 8		
4, 5	3, 4	2, 3	Corner of exterior wall, four corners of staircase intersection of interior and exterior walls in large rooms, intersections of each 15m or the unit transversal wall and exterior longitudinal wall	Corners of the exterior wall, 3 holes shall be filled; intersection of interior and exterior walls, 4 holes
6	5	4	Corner of exterior wall, four corners of staircase intersection of interior and exterior walls in large rooms, intersection of the interior wall and the gable, intersection of other bay transversal wall (axis) and exterior longitudinal wall	
7	6	5	Corner of exterior wall, four corners of staircase, intersection of all interior and exterior walls; for Intensity 8, intersection of interior longitudinal wall and transversal wall (axis), both sides of bigger openings	Corners of the exterior wall, 5 holes shall be filled; intersection of interior and exterior walls, 4 holes; intersection of interior walls, 4 or 5 holes; both sides of opening, 1 holes
	7	6	Ditto; The spacing of the transversal wall core-column shall not be greater than 2m	Corners of the exterior wall, 7 holes shall be filled; intersection of interior and exterior walls, 5 holes; intersection of interior walls, 4 or 5 holes; both sides of opening, 1 hole

Note: In locations such as the corners of the exterior wall, intersection of the interior and exterior wall, and corners of staircase, it shall be permitted adopted tie-columns to replace corresponding core-columns.

7.4.2 The core-columns of multi-story small-block buildings shall comply with following requirements:

1 The cross section of the core-column shall not be less than 120mm×120mm.

2 The concrete strength grade of the core-column shall not be less than C20.

3 The longitudinal bars of the core-column shall be through overall wall and connect with the ring-beam; the longitudinal bar shall not be less than 1ϕ12, and than 1ϕ14 for the building exceeds 5 stories at Intensity 7 and exceeds 4 stories at Intensity 8.

4 The core-column shall extend to 500mm underground level or connect with foundation ring-beam with a buried depth less than 500mm.

5 The core-columns which improve the seismic shear capacity of wall should be distributed in the wall evenly, and the maximum clear spacing shall not be greater than 2.0m.

7.4.3 The tie-columns used to replace core-columns in small-block buildings shall comply with the following requirements:

1 The minimum cross section of the tie-column may adopt 190mm×190mm, the longitudinal bars shall adopt 4ϕ12, the spacing of stirrups shall not be greater than 250mm and shall be densified at the upper and lower end of the column accordingly. When the building exceeds 5 stories for Intensity 7, exceeds 4 stories for Intensity 8, the longitudinal bars of the tie-column should adopt 4ϕ14, and the spacing of stirrups shall not be greater than 200mm. For the tie-columns at the corners of the exterior wall, the cross section and the reinforcement amount may be increased accordingly.

2 The connection of the tie-column and the adjacent block walls shall be built into horse-toothed joints, the adjacent block hole with the tie-column should be filled for Intensity 6, and shall be filled for Intensity 7, and shall be filled and dowel reinforcements for Intensity 8. The tie reinforcing fabric shall be installed in each 600mm along the height of the wall, and the length extending into the each side-wall should not be less than 1 m.

3 At the connection of the tie-column and the ring-beam, the longitudinal bars of the tie-column shall be through the ring-beam to ensure the continuation of longitudinal bars in the tie-column.

4 The tie-columns may not establish individual footing, but they shall extend to 500mm into the underground level, or shall connect with the foundation ring-beam, which buried depth less than 500mm underground.

7.4.4 The cast-in-situ ring-beam of small-block building shall be installed in accordance with the requirements in Table 7.4.4, the width of the ring-beam shall not be less than 190mm, the reinforcement shall not be less than 4ϕ12, and the spacing of stirrups shall not be greater than 200mm.

7.4.5 The reinforcement fabrics shall be installed at the intersection of the block walls or the intersection of the core-column and the walls. The fabric may be made through spot welding by using the diameter 4mm bars, and shall be installed in each 600mm along the

wall height, and the length extending to each side of the wall should not be less than 1 m.

Table 7.4.4 Requirements of installation for ring-beam of small-block building

Type of wall	Intensity	
	6,7	8
Exterior walls and interior longitudinal wall	At roof level, each floor level	At roof level, each floor level
Interior transversal wall	Ditto; along all transversal wall at roof; spacing at the floor shall not be greater than 7m; corresponding location of the tie-column	Ditto; all transversal walls at roof and each floor; corresponding location of the tie-column

7.4.6 For the small-block building having 7 stories at Intensity 6, exceed 5 stories at Intensity 7 and exceed 4 stories at Intensity 8, the in-situ reinforced concrete horizontal strip should be installed at the windowsill level of the top and first story of the building. This strip shall run through overall length of the transversal and longitudinal walls, the cross-sectional height shall not be less than 60mm, the longitudinal bar shall not be less than 2ϕ10, and tie bars shall also be arranged. More concrete strength grade shall not be less than C20.

7.4.7 Other seismic design details for multi-story small-block buildings shall comply with relevant requirements in Clauses 7.3.5 through 7.3.13 of this chapter.

7.5 Details of seismic design for multi-story buildings with bottom-frame

7.5.1 For the multi-story brick structures above the bottom-frame story, the reinforced concrete tie-columns shall be installed and comply with following requirements:

1 The location of the tie-column shall be installed according to the provision in Clause 7.3.1 of this code, base on the total stories of the building. For the transitional story, tie-columns shall also be installed at the corresponding point of the frame column at the bottom.

2 The cross section of the tie-column shall not be less than 240mm×240mm.

3 The longitudinal bars of the tie-column shall not be less than 4ϕ14, and the spacing of stirrups shall not be greater than 200mm.

4 For the transitional story, the longitudinal bars of tie-column shall not be less than 4ϕ16 at Intensity 7, and than 6ϕ16 at Intensity 8. Generally, the longitudinal bars shall be developed to the frame column, when the longitudinal bars are developed to the frame beam, the corresponding location of the frame beam shall be strengthened.

5 The tie-column shall be connected with ring-beams in each story, or shall be reliably tied with the cast-in-situ slabs.

7.5.2 The centerline of the upper wall of the multi-story masonry building with bottom-frame should be coincided with the axis of the frame beam or the wall of bottom-story, and the tie-column should be continuous to the frame column.

7.5.3 The floors of the multi-story masonry building with bottom-frame shall comply

with following requirements:

1 The transference floor shall adopt cast-in-situ reinforced concrete slab. This slab thickness shall not be less than 120mm; the openings in slab shall be cut down or small; when the dimension of the opening exceeds 800mm, boundary beams shall be installed along the perimeters of the opening.

2 For other stories, when precast reinforced concrete slabs are adopted, cast-in-situ ring-beams shall be installed; only in-situ reinforced concrete slabs are adopted, ring-beams shall be permitted not installed, but the strengthened reinforcements of in-situ slabs shall be arranged along the wall perimeters and shall be reliably connected with corresponding tie-columns.

7.5.4 The reinforced concrete spandrel girder of buildings with bottom-frame shall comply with the following requirements:

1 The cross sectional width of the girder shall not be less than 300mm, and the cross sectional height shall not be less than 1/10 of the span.

2 The diameter of the hoops shall not be less than 8mm, and the spacing of hoops shall not be greater than 200mm. At the girder end within 1.5 times of girder height and not less than 1/5 of the clear span, and the both sides for the opening of the upper wall within 500mm and not less than the girder height, the spacing of hoops shall not be greater than 100mm.

3 The space bars shall be arranged along the girder height, the amount shall not be less than 2ϕ14, and the spacing shall not be less than 200mm.

4 The main reinforcements and spacer bars of the girder shall be developed to the column according to the requirements for tensile bars; besides, the developed length of the upper longitudinal bars of girder in the support shall comply with relevant requirements for reinforced concrete frame-supporting girders.

7.5.5 The reinforced concrete wall at the bottom of buildings shall comply with the following requirements:

1 In the perimeter of the wall panel, a boundary frame formed by side beams (or hidden beams) and end columns shall be installed. The cross-sectional width of the side beams should not be less than 1.5 times of the wall panel thickness, the cross-sectional height should not be less than 2.5 times of the wall panel thickness. The cross-sectional height of the end column should not be less than 2 times of the wall panel thickness.

2 The thickness of the wall panel shall not be less than 160mm, nor less than 1/20 of the clear height of the wall panel. The wall should be installed openings to form several short wall-segments, and the height-width ratio of each wall-segment shall not be less than 2.

3 The reinforcement ratio for both vertical and horizontal distributed reinforcements of the wall shall not be less than 0.25%, which shall be arranged in two layers. The spac-

ing of tie bars for two layers shall not be greater than 600 mm, and the diameter of the tie bar shall not be less than 6 mm.

4 The boundary elements of the wall may be installed according to the requirements for general portions in Section 6.4 of this code.

7.5.6 The common brick walls, which used for the first story of the buildings with framed first story, shall conform to the following requirements:

1 The thickness of brick wall shall not be less than 240mm, the strength grade of the mortar used shall not be lower than M10, and the wall shall be built first and then cast the frame.

2 The tie bars of 2ϕ6 shall be arranged in each 500mm along the frame columns, and shall be installed along overall length of the brick wall; reinforcement concrete horizontal strip connecting with the both frame columns shall be installed at half height level of the wall height.

3 When the length of wall is greater than 5m, added tie-column shall be installed within the wall.

7.5.7 The strength grade of the materials used for multi-story masonry buildings with bottom-frame shall conform to the following requirements:

1 The concrete strength grade of the frame column, wall and spandrel girder shall not be lower than C30.

2 The mortar strength grade for masonry wall of transitional story shall not be lower than M7.5.

7.5.8 Other design details for multi-story masonry buildings with bottom-frame shall comply with relevant requirements from Clauses 7.3.5 through 7.3.14 of this chapter.

7.6 Details of seismic design for multi-story buildings with inner-frames

7.6.1 For the multi-story buildings with inner-frames, the reinforced concrete tie-columns shall be installed and conform to following requirements:

1 The tie-columns shall be installed in the following locations:

1) Four-corners of the exterior wall, staircase and elevator shaft; supporting location of the beam for landing slab of stair;

2) The both ends of the wall, locations corresponding to the axis of the inner-frame column at the exterior longitudinal and transversal wall that without composite columns.

2 The cross-sectional dimension of the tie-column shall not be less than 240mm $\times$ 240mm.

3 The longitudinal bars of the tie-column shall not be less than 4ϕ14, and the spacing of the stirrups shall not be greater than 200mm.

4 The tie-column shall be connected with ring-beams in every story, or be tied reli-

ably to the cast-in-situ floor slabs.

7.6.2 For the multi-story buildings with inner-frames, the floors and roof shall adopt cast-in-situ cast or precast-monolithic reinforced concrete slabs. When cast-in-situ reinforced concrete slabs are adopted, ring-beams shall be permitted not installed, but the strengthened reinforcements of in-situ slabs shall be arranged along the wall perimeters and shall be reliably connected with corresponding tie-columns.

7.6.3 The supporting length for the inner-frame beam on the exterior longitudinal and transversal walls shall not be less than 300mm, besides, the ends of the inner-frame beam shall be connected with the ring-beam or composite column or tie-columns reliably.

7.6.4 Other design details for multi-story brick buildings with inner-frames shall conform to relevant requirements from Clause 7.3.5 through 7.3.13 of this chapter.

Chapter 8 Multi-story and tall steel structure buildings

8.1 General

8.1.1 In this chapter, the structural type and applicable maximum height of steel structural civil buildings shall comply with the requirements in Table 8.1.1. For structures assigned to irregular plan and elevation or structures sat on Site-class IV, the applicable maximum height shall be reduced accordingly.

Note: The seismic design of multi-story steel structural factory shall confirm to the requirements in Appendix G of this code.

Table 8.1.1 Applicable maximum height of steel structural building (m)

Type of structures	Intensity 6	Intensity 7	Intensity 8	Intensity 9
Frame structure	110	110	90	50
Frame-brace(or wall)	220	220	200	140
Tube (framed tube, tube by tube, truss tube, tubes) and great frame	300	300	260	180

Note: 1 The height of building refers to the height from the outdoor ground level to the main roof level of a building (which locations exceeding the roof level are not included);

2 When the height of a building exceeding that as provisions in this Table, special researches and demonstration shall be carried out and effective seismic measures shall be taken.

8.1.2 The maximum height-to-width ratio of steel structural civil buildings applicable to this chapter should not exceed the limits in Table 8.1.2:

Table 8.1.2 Maximum height-to-width ratios of steel structural civil building

Intensity	6	7	8	9
Maximum height-to-width ratio	6.5	6.5	6.0	5.5

Note: the height for calculating the height-to-width ratio shall be counted from the outdoor ground level.

8.1.3 The seismic design of steel structure buildings shall adopt different seismic effect adjustment factors and different seismic design details based on the fortification intensity, the structural type and the height of the buildings.

8.1.4 The steel structure buildings should avoid adopting the building structure having irregular plans and vertical configuration listed in section 3.4 of this code, and no seismic joints are necessary to be installed. When seismic joints need to be installed, the minimum clear width of the seismic joint shall not be less than 1.5 times of that for reinforced concrete buildings.

8.1.5 The steel structure buildings with not exceeding 12 stories in height may use frame, frame-brace, or other structural types. For Intensity 8 and 9, the buildings with exceeding

12 stories in height shall adopt frame brace structure with, eccentric braces, reinforced concrete wall plane with vertical separators, and reinforced concrete wall plane with hidden steel brace or other energy-dissipating braces as well as tube structures.

8.1.6 The steel frame-braced structures shall comply with the following provisions:

1 The arrangement of the braced frame shall be basically symmetrical in two directions, the length-to-width ratio of the floor between two adjacent braced frames shall not be greater than 3.

2 For steel structure with not exceeding 12 stories in height should adopt epicenter brace; where the condition permits, energy-dissipating braces such as eccentric brace may also be used. When the steel structure with exceeding 12 stories in height adopts eccentric braces, the top story may adopt epicenter braces.

3 Epicenter braced frames should adopt crosswise braces, and may also adopt the inverted-V shape braces or single diagonal braces, but no K-shape braces should be used. The centriod axis of the brace shall converged at the intersection point of the beam centriod axis and column centriod axis; when this is difficult, the eccentricity shall not exceed the width of the braced member, and the additional bending moment caused by which shall be taken into consideration.

4 Each brace of the eccentric braced frame shall have at least one end connecting with the framed beam, and thus form an energy-dissipating beam-segment that length is from the brace-to-beam intersection to the beam-to-column joint or is between both brace-to-beam intersection within the same span.

8.1.7 The floors of the steel structure should adopt composite floor with cast-in-situ concrete and fluted plate or non-composite floor. For buildings with not exceeding 12 stories in height, precast-monolithic reinforced concrete slab may also be used, as well as precast floor slabs or other types of lightweight floors; for steel structures with exceeding 12 stories in height, horizontal braces in floor may be installed if necessary.

The composite floor with fluted plate and concrete or cast-in-situ concrete floors shall be reliably connected with the steel beam. For the precast floor slabs, precast-monolithic floor slabs, or other types of lightweight floors, the embedded parts of the floor shall be welded with the steel beam, or taking other details to ensure the integrity of the floor.

8.1.8 For the steel frame-tube structures with exceeding 12 stories in height, a strengthened story composed by the outrigger trusses or the outrigger trusses and the perimeter trusses may be installed according to necessary.

8.1.9 When basement is installed for the steel frame-brace (or wall) structures, the vertical continuously arranged brace or wall shall extend to the foundation, and the frame column shall least extend to the first floor underground.

8.1.10 The steel structures with exceeding 12 stories in height shall have basement. For the undisturbed subsoil foundation such as footings or mats, its buried depth shall not be less

than 1/15 of the total building height; for the pile foundation, the buried depth of the pile cap shall not be less than 1/20 of the total height of the building.

8.2 Essentials in calculation

8.2.1 Steel structures shall adjust the seismic effect according to the provisions of this section, and the story drift shall comply with relevant requirements in Section 5.5 of this code.

If no provisions in this chapter, the seismic checking of the members and its connections shall be made in accordance to current codes for relevant structure design, but the non-seismic design value for the bearing capacity of members shall be divided by the specified seismic adjustment factors for bearing capacity of members in this code.

8.2.2 The damping ratio of the steel structure under frequently earthquake, for steel structures with not exceeding 12 stories in height, may be taken as 0.035; for steel structures with exceeding 12 stories in height, may be taken as 0.02. For structural analysis under the rarely earthquake, the damping ratio may be taken as 0.05.

8.2.3 Interior force and deformation analysis of the steel structures under seismic action shall comply with the following provisions:

1 The steel structures shall take into account of the gravity secondary effect according to the provisions in Clause 3.6.3 of this code. For the design of frame beams, the interior force may be not taken as that of the intersecting point of beam axis and column axis, but taken as that of the supported face of the beam. For the I-shaped section column, the effect of joint-panel shear deformation on the structure lateral displacement should be taken into consideration; but for epicenter braced frame and steel structures with not exceeding 12 stories in height, thus effect on the story drift may be omitted.

2 For the analysis of steel frame-brace structure, the brace may be assumed to the pin-connection at the end, and the frame part shall be resisted at least of the smaller value between the 25% total seismic shear of structure and 1.8 times of calculated maximum story shears of frame part. And so the calculated seismic shear force of the frame part shall be multiplied by the adjustment factor.

3 When the eccentricity between the epicenter brace axial line and intersection joint of the beam and column axial lines does not exceed the width of the brace member, the analysis may be done assumed to epicenter braced frame; but the added bending moment caused by which shall be taken into consideration. The interior force design value of the inverted-V and V-shaped braces shall be multiplied by the amplifying factor, which value may be taken as 1.5.

4 For the eccentric brace framd structures, the interior force design value of members shall be adjusted according to the following requirements:

1) The axial force design value of the brace member shall be taken as the product of the amplifying factor and brace axial force that corresponding to the shear bearing

capacity of the connecting energy-dissipating beam-segment. This factor value shall not be less than 1.4 for Intensity 8 or below Intensity 8 and not less than 1.5 for Intensity 9.

2) The interior force design value of frame beam locating at the energy-dissipating beam-segment shall be taken as the product of the amplifying factor and frame beam interior force which corresponding to the shear bearing capacity of the connecting energy-dissipating beam-segment. This factor value shall not be less than 1.5 for Intensity 8 or below Intensity 8 and not less than 1.6 for Intensity 9.

3) The interior force design value of the frame column shall be taken as the product of the amplifying factor and column interior force that corresponding to the shear bearing capacity of the connecting energy-dissipating beam-segment. This factor value shall not be less than 1.5 for Intensity 8 or below Intensity 8 and not less than 1.6 for Intensity 9.

5 The calculation of the reinforced concrete wall plane with hidden steel brace and the reinforced concrete wall plane with vertical separators shall be carried out according to relevant provisions. The reinforced concrete wall plane with vertical separators may only resist the shear caused by horizontal load, but not bear the vertical load.

6 The seismic interior force of frame columns supporting the transfer members of the steel structure shall be multiplied by the amplifying factor, this value may be taken as 1.5.

8.2.4 When the upper flange of the steel frame beam with shear-connectors such as shear-stubs is connected to composite floor slab, the checking of integral stability of beams under seismic action may be omitted.

8.2.5 The seismic check of the steel frame members and connections shall comply with the following provisions:

1 The plastic capacity for the right and left ends of beam and the upper and lower ends of the column at the a joint shall comply with the requirements of following equation:

$$\Sigma W_{pc}(f_{yc} - N/A_c) \geqslant \eta \Sigma W_{pb} f_{yb} \qquad (8.2.5\text{-}1)$$

where: W_{pc}, W_{pb}——the plastic section modulus of the column and beam respectively.

N——the axial force design value of the column.

A_c——the cross-sectional area of the column.

f_{yc}, f_{yb}——the specified steel yield strength of the column and the beam respectively.

η——the strong-column amplifying factor, for steel structures with exceeding 6 stories in height, taken as 1.0 for Intensity 6 with Site-class IV or Intensity 7, taken as 1.05 for Intensity 8, and taken as 1.15 for Intensity 9.

Exception: The following three cases may not be checking with equation (8.2.5-1):

The shear capacity of the story greater than 25% of that of the adjacent upper story;

The ratio of the column axial force design value to the product of the column whole section area and the specified steel tensile strength less than or equal 0.4;

The stability of the axial compressive member under two times of the seismic force is ensured.

2 The yield capacity for the joint-panel of beam and column shall comply with the requirements of the following equations:

$$\psi(M_{pb1} + M_{pb2})/V_p \leqslant (4/3)\ f_v \tag{8.2.5-2}$$

For I-shaped section column

$$V_p = h_b h_c t_w \tag{8.2.5-3}$$

For box-shaped section column

$$V_p = 1.8 h_b\ h_c t_w \tag{8.2.5-4}$$

3 The joint-panel of I-shaped and box-shaped section column shall be checked according to the following equations:

$$t_w \geqslant (h_b + h_c)/90 \tag{8.2.5-5}$$

$$(M_{b1} + M_{b2})/V_p \leqslant (4/3) f_v/\gamma_{RE} \tag{8.2.5-6}$$

where: M_{pb1}, M_{pb2}——the full plastic bending capacity of the beam on the both sides of the joint-panel.

V_p——volume of the joint-panel.

ψ——deducting factor, taken as 0.6 for Intensity 6 with Site-class IV or Intensity 7, taken as 0.7 for Intensity 8 or 9.

h_b, h_c——the web depth of beam and column respectively.

t_w——the web thickness of joint-panel of the column.

M_{b1}, M_{b2}——the bending moment design value of the beams at the two sides of the joint-panel.

γ_{RE}——the seismic adjustment factor of joint-panel bearing capacity, taken as 0.85.

Exception: when the web thickness of the joint-panel of the column is not less than 1/70 of the sum of depth of the beam and column section depth, the stability check for the joint-panel may be omitted.

8.2.6 The seismic capacity check of the epicenter braced frame member shall comply with the following provisions:

1 The compressive capacity of the braced member shall be checked with the following equations:

$$N/(\phi A_{br}) \leqslant \psi f/\gamma_{RE} \tag{8.2.6-1}$$

$$\psi = 1/(1 + 0.35\lambda_n) \tag{8.2.6-2}$$

$$\lambda_n = (\lambda/\pi)\sqrt{f_{ay}/E} \tag{8.2.6-3}$$

where: N——axial force design value of the braced member.

A_{br}——cross-sectional area of the braced member.

ϕ——stability factor of the axis compressive member.

ψ——strength reducing factor under cycling load.

λ_n——the normal slenderness ratio of the braced member.

E——elastic modulus of the braced member.

f_{ay}——the specified steel yield strength of the braced member.

γ_{RE}——the seismic adjustment factor of brace bearing capacity.

2 The horizontal beam of inverted-V shape and the V-shaped brace shall keep continuous at the connection joint of the brace. This horizontal beam shall resist the interior force transmitted from the brace, which the analysis of beam assumed to simple support beam under the gravity load and that does not take the supporting effect of the brace, or of beam under the non-balanced force due to brace buckling.

Exception: the beams of the top story and the penthouse may not follow the provisions in this item.

8.2.7 The seismic check of the eccentric braced frame members shall comply with the following provisions:

1 The shear capacity for the energy-dissipating beam-segment of eccentric braced frame shall be checked according to the following equation:

for $N \leqslant 0.15Af$

$$V \leqslant \varphi V_l / \gamma_{RE} \tag{8.2.7-1}$$

$V_l = 0.58A_w f_{ay}$ or $V_l = 2M_{lp}/a$, taken as smaller value

$A_w = (h - 2t_f)t_w$

$M_{lp} = W_p f$

for $N > 0.15Af$

$$V \leqslant \varphi V_{lc} / \gamma_{RE} \tag{8.2.7-2}$$

$V_{lc} = 0.58A_w f_{ay}\sqrt{1 - [N/(Af)]^2}$ or $V_{lc} = 2.4M_{lp}[1 - N/(Af)]/a$, taken as the smaller value

where: φ——calculation factor, taken as 0.9.

V, N——the shear force design value and the axial force design value of the energy-dissipating beam-segment respectively.

V_l, V_{lc}——the shear capacity and that with axial force affection of the energy-dissipating beam-segment respectively

M_{lp}——full plastic bending capacity of the energy-dissipating beam-segment.

a, h, t_w, t_f——the length, depth, web thickness and flange thickness of the energy-dissipating beam-segment respectively.

A, A_w——the total sectional area and web sectional area of the energy-dissipating beam-segment respectively.

W_p——plastic section modulus of the energy-dissipating beam-segment.

f, f_{ay}——the tensile strength design value and the specified yield strength of the energy-dissipating beam-segment respectively.

γ_{RE}——the seismic adjustment factor of capacity of the energy-dissipating beam-segment, taken as 0.85.

Note: the energy-dissipating beam-segment of the eccentric braced frame refers to the portion from the intersection of the bracing and beam to the column or the portion between both adjacent intersection of braces and beam at same beam-span. During an earthquake, the energy-dissipating beam-segment yields and leaves other portions of beam in the elastic state.

2 The bearing capacity of the connection between the bracing and the energy-dissipating beam-segment shall not be smaller than the bearing capacity of the bracing. For the brace having the bending moment, such connection shall be designed according to combinatory flexure and axial force.

8.2.8 Elastic design of the connection of steel structural members shall be carried out according to the combinatory seismic interior force, and the ultimate limit bearing capacity shall also be checked as follows:

1 When the connection of beam and column adopts elastic design, the end section of the top and bottom flanges of beam shall satisfy the requirements for elastic design, the shear force and the bending moment shall be taken into consideration for the design of the beam web. The ultimate bending and shear capacity of the connection of beam and column shall comply with the following requirements:

$$M_u \geqslant 1.2M_p \tag{8.2.8-1}$$

$$V_u \geqslant 1.3(2M_p/l_n) \text{ and } V_u \geqslant 0.58h_w t_w f_{ay} \tag{8.2.8-2}$$

where M_u——the ultimate bending capacity of the fusion V-weld in the top and bottom flanges of beam.

V_u——ultimate shear capacity of the connection of beam web; and for the connection subjected to the shear orthogonal the fillet weld, that may be increased by 1.22 times.

M_p——the full plastic bending capacity of the beam (or column where beam is continually).

l_n——clear span of the beam (or clear height of the column where beam is continually).

h_w, t_w——the depth and width of the beam web.

f_{ay}——the specified steel yield strength of the member.

2 The ultimate limit capacity for the connection of brace and frame or the splicing of brace shall comply with the requirements of the following equation:

$$N_{ubr} \geqslant 1.2\ A_n f_{ay} \tag{8.2.8-3}$$

where: N_{ubr}——the ultimate capacity of the bolt connection and gusset plate connection along the brace axial line direction.

A_n——net area of the brace section.

f_{ay}——the specified steel yield strength of the brace.

3 When the splicing of the beam or column members adopts elastic design, the design for web shall be taken into account of the bending moment, and the shear capacity of web shall not be less than 50% of the total shear capacity of the member. The ultimate limit capacity of the splicing shall comply with the following requirements:

$$V_u \geqslant 0.58 h_w t_w f_{ay} \tag{8.2.8-4}$$

without axial force $$M_u \geqslant 1.2\, M_p \tag{8.2.8-5}$$

with axial force $$M_u \geqslant 1.2\, M_{pc} \tag{8.2.8-6}$$

where: M_u, V_u——the ultimate bending and shear bearing capacity of the member splicing respectively.

M_{pc}——the total section bending capacity when the member with axial force.

h_w, t_w——the depth and thickness of web of the splicing of member.

f_{ay}——the specified steel yield strength of spliced member.

The splicing with bolts shall also be comply with the following requirements:

flange $$nN_{cu}^b \geqslant 1.2 A_f f_{ay} \quad \text{and} \quad nN_{vu}^b \geqslant 1.2 A_f f_{ay} \tag{8.2.8-7}$$

web $$N_{cu}^b \geqslant \sqrt{(V_u/n)^2 + (N_M^b)^2} \quad \text{and} \quad N_{vu}^b \geqslant \sqrt{(V_u/n)^2 + (N_M^b)^2} \tag{8.2.8-8}$$

where: N_{vu}^b, N_{cu}^b——the ultimate shear capacity of a bolt and the ultimate compressive capacity of the corresponding plate.

A_f——effective section area of the flange.

N_M^b——the maximum shear force of a bolt caused by bending moment in splicing web.

n——the number of bolts on one side for flange splicing or web splicing.

4 The calculation for the whole section bending capacity of beams or columns with axial force shall be done according to the following equations:

for major axial of I-shaped section and box section

when $N/N_y \leqslant 0.13$ $$M_{pc} = M_p \tag{8.2.8-9}$$

when $N/N_y > 0.13$ $$M_{pc} = 1.15\,(1 - N/N_y) M_p \tag{8.2.8-10}$$

for minor axial of I-shaped section

when $N/N_y \leqslant A_w/A$ $$M_{pc} = M_p \tag{8.2.8-11}$$

when $N/N_y > A_w/A$ $$M_{pc} = \{1 - [(N - A_w f_{ay})/(N_y - A_w f_{ay})]^2\} M_p \tag{8.2.8-12}$$

where: N_y——the axial yield capacity of the member, taken as $N_y = A_n f_{ay}$

5 The ultimate bearing capacity of the welding joint shall be calculated by following equations:

for the tensile splice weld

$$N_u = A_f^w f_u \tag{8.2.8-13}$$

for the shear fillet weld

$$V_u = 0.58 A_f^w f_u \tag{8.2.8-14}$$

where: A_f^w——the effective bearing area of the welding joint.

f_u——the minimum value of tensile strength of the welded members.

6 The ultimate shear capacity of the connection with high-tensile bolt shall take the smaller one of the results calculated by the following equations:

$$N_{vu}^b = 0.58 n_f A_e^b f_u^b \tag{8.2.8-15}$$

$$N_{cu}^b = d \Sigma t f_{cu}^b \tag{8.2.8-16}$$

where: N_{vu}^b、N_{cu}^b——the ultimate shear capacity of a high-tensile bolt and the ultimate compressive capacity of corresponding plate respectively.

n_f——number of shearing faces of bolt connection.

A_e^b——the effective section area of threaded portion of bolt.

f_u^b——lowest value of steel tensile strength of bolt.

d——the diameter of bolt rod.

Σt——total thickness of steel plates corresponding to loading direction.

f_{cu}^b——the ultimate compressive strength of the bolt connecting plate, taken as $1.5 f_u$.

8.3 Details of seismic design for steel framed structures

8.3.1 The slenderness ratio of frame columns shall conform to the following provisions:

1 For the framed structures with height not exceeding 12 stories, the slenderness ratio of the column shall not be greater than 120 $\sqrt{235/f_{ay}}$ for Intensity 6 through Intensity 8, and not be greater than 100 $\sqrt{235/f_{ay}}$ for Intensity 9.

2 For the framed structures with height exceeding 12 stories, the slenderness ratio of the column shall comply with the provisions of Table 8.3.1:

Table 8.3.1 Limit value of slenderness ratio for framed structures exceeding 12 stories

Intensity	6	7	8	9
Slenderness ratio	120	80	60	60

Note: the values listed in the table is applicable to Q235 steel, other steel types shall multiply by $\sqrt{235/f_{ay}}$

8.3.2 The width-to-thickness ratio of the elements of beam and column shall comply with the following provisions:

1 For the framed structures with height not exceeding 12 stories, the width-to-thickness ratio of the elements of beam and column shall conform to the requirements in Table 8.3.2-1:

Table 8.3.2-1 Width-to-thickness ratio of the element of beam and column for not exceeding 12 stories

Name of the element		Intensity 7	Intensity 8	Intensity 9
Column	Flanges of I-shaped	13	12	11
	Plates of box-section	40	36	36
	Web of I-shaped	52	48	44
Beam	Flanges of I-shaped or plate projecting from box-section	11	10	9
	Flanges between both webs of box-section	36	32	30
	Webs of I-shaped or box-section:			
	($N_b/Af<0.37$)	85-120N_b/Af	80-110N_b/Af	72-100N_b/Af
	($N_b/Af\geqslant 0.37$)	40	39	35

Notes: the values listed in the table is applicable to Q235 steel, other steel types shall multiply by $\sqrt{235/f_{ay}}$. N_b refers to axial force of beam.

2 For the framed structures with height exceeding 12 stories, the width-to-thickness ratio of the elements of beam and column shall conform to the requirements in Table 8.3.2-2:

Table 8.3.2-2 Width-to-thickness ratios of the elements of beam and column for exceeding 12 stories

Name of the element		Intensity 6	Intensity 7	Intensity 8	Intensity 9
Column	Flanges of I-shaped	13	11	10	9
	Plates of box-section	39	37	35	33
	Web of I-shaped	43	43	43	43
Beam	Flanges of I-shaped or plates projecting from box-section	11	10	9	9
	Flanges between both webs of box-section	36	32	30	30
	Webs of I-shaped or box-section:	85-120N_b/Af	80-110N_b/Af	72-100N_b/Af	72-100N_b/Af

Note: the values listed in the table is applicable to Q235 steel, other steel types shall be multiply by $\sqrt{235/f_{ay}}$. N_b refers to axial force of beam.

8.3.3 Lateral brace of beam and column members shall comply with the following requirements:

1 For the sections where plastic hinges occur on the flanges of beam or column, lateral braces shall be installed at the top and bottom flanges of this section.

2 The slenderness ratio of the member between both adjacent lateral braced points shall comply with relevant provisions of the plastic design of national standard "Code for design of steel structures" GB50017.

8.3.4 The details of connection of the beam and the column shall comply with the following requirements:

1 The connection of the beam and the column should adopt the form of continued col-

umn.

2 When the columns adopt rigid connection with beam in two orthogonal directions, the box section columns should be adopted. When the columns adopt rigid connection with the beam only one direction, the I-shaped columns should be adopted, and the column web shall be placed within the rigid connection plane of the frame.

3 When the I-shaped and box-section columns adopt rigid connection with the beam, the following requirements shall be observed (Figure 8.3.4-1), other details may also be used when there is sufficient reason.

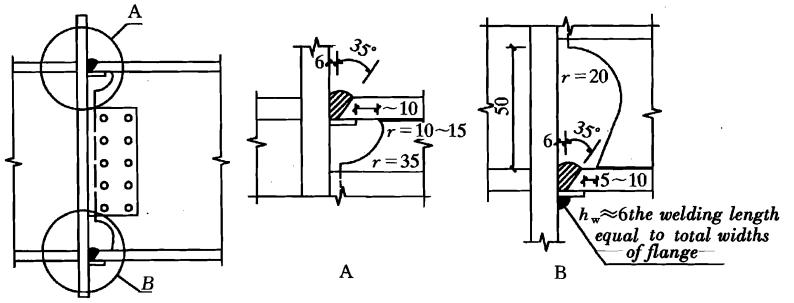

Figure 8.3.4-1 On-site connections of frame beam and column

1) The fusion V-weld joints shall be adopted between the beam flange and the column flange; for Intensity 8 with the Category B and for Intensity 9, the shock-tenacity of the V-weld joint shall be checked, and its Charpy shock-tenacity shall not be lower than 27J at −20℃;

2) When horizontal stiffening ribs are used in the column at the corresponding locations of the beam flanges, the thickness of the stiffening ribs shall not be less than the thickness of the beam flanges;

3) The web of the beam should adopt friction-type high-tensile bolt to connect with the column through gusset plates. At the corner of the web, sector cutting should be used, and its edge shall be separated with the fusion weld joint of the beam flange;

4) When the plastic section modulus of the beam flange is smaller than 70% of the plastic section modulus of total beam section, the number of bolts for the connection of the beam web and the column shall not be less than two rows. When only one row is needed based on the result of calculation, still two rows shall be arranged, and the total number of bolts shall not be less than 1.5 times of the calculated value;

5) For Intensity 8 with Site-class Ⅲ and Ⅳ and for Intensity 9, the dog-bone connection should be adopted to move the plastic hinges outward from the beam supported faces.

4 When the frame column adopts rigid connection with the cantilever beam segment

(Fig.8.3.4-2), the cantilever beam segment and the column shall be connected with fusion weld at facilities, and at in-situ, the beam may be connected by using flange welding and web bolts (a) or completely connected by bolts (b).

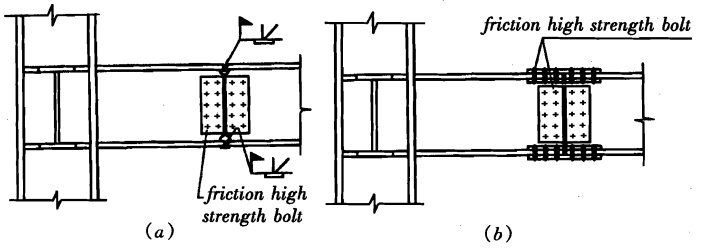

Figure 8.3.4-2 connections of the frame beam and column through suspending beam

5 For the box-section column, the separator shall be installed in corresponding location of the beam flange and connected with plates of column shall adopt fusion butt-weld. For the I-shaped section column, the connection of the horizontal stiffening ribs with the flange of column shall adopt fusion butt-weld, and connection with the web of column may adopt fillet weld.

8.3.5 When the volume of the joint-panel cannot satisfy that the provisions in point 3 of Clause 8.2.5, the thickness of joint-panel shall be increased or add-stiffening plate shall be welded. The thickness of the add-stiffening plate and the welding joint shall be designed according to the requirements for transferring the shear-force-resistant of add-stiffening plat.

8.3.6 When the beam and the column adopt rigid connections, within the range of 500mm in the upper and lower of flanges of beam, the welding joints between the flanges and the webs of I-shaped column and between the plates of box-section column shall use the fusion V-weld.

8.3.7 The splice joints of the framed column should locate at nearby 1.3m of the upper frame beam. The splice of the upper and lower columns shall adopt completely fusion weld joints. Within the range of 100 mm in the splice of the upper and lower part of the column, the welding joints between the flanges and the web of I-shaped column and between the corner plates of box-section column shall adopt the completely fusion weld.

8.3.8 The rigid connection column foots of steel structures with exceeding 12 stories in height should adopt the buried type foots; for Intensity 6 and 7, overall boundary coverage type foots may also be used.

8.4 Details of seismic design for steel frame-epicenter-braced structures

8.4.1 When the epicenter brace frame adopts the single tensile diagonal, both series of brace diagonal pointing to different directions shall be installed together; and the difference between the horizontal projection areas of diagonal in two directions cannot be greater than 10%.

8.4.2 The slenderness ratio and the width-to-thickness ratio of the epicenter diagonal brace shall comply with following requirements:

1 The slenderness ratio of the epicenter diagonal brace shall not be greater than the limit values in Table 8.4.2-1.

Table 8.4.2-1 slenderness ratio limit values for the epicenter brace of steel structures

Type		Intensity 6 and 7	Intensity 8	Intensity 9
Not exceed 12 stories	Design follow compress struts	150	120	120
	Design follow tensile struts	200	150	150
Exceed 12 stories in height		120	90	60

Note: the values listed in the table is applicable to Q235 steel, other steel types shall multiply by $\sqrt{235/f_{ay}}$

2 The width-to-thickness ratio of the element of diagonal brace shall not be greater than the limit values in Table 8.4.2-2. When gusset plate is used for the connection, attention shall be paid to the strength and stability of the gusset plate.

Table 8.4.2-2 Limit values for the width-to-thickness ratio of the epicenter brace for steel structure

Name of element	Not exceeding 12 stories			Exceeding 12 stories			
	Int.7	Int.8	Int.9	Int.6	Int.7	Int.8	Int.9
Flanges with one free edge	13	11	9	9	8	8	7
Web of I-shaped	33	30	27	25	23	23	21
Plates of box-section	31	28	25	23	21	21	19
External diameter-thickness ratio of tube				42	40	40	38

Note: the values listed in the table is applicable to Q235 steel, other steel types shall multiply by $\sqrt{235/f_{ay}}$, except the diameter-thickness ratio where shall multiply by $235/f_{ay}$.

8.4.3 The detail of the epicenter brace joints shall satisfy the following requirements:

1 When the structure with exceeds 12 stories in height, the brace struts should be made using the rolled H-shaped steels, its two ends and the frame may use rigid connection, and the connection part of the beam and column with the brace shall install stiffener ribs. When adopting welding I-shaped brace for Intensity 8 or 9, the connection of its flange and the web should adopt completely fusion weld joints.

2 At the location of connection between the brace and the frame, the diagonal ends should be made into arc shape.

3 At the intersecting place of beam with the inverted-V and V-shaped braces, lateral braces shall be installed. And the lateral slenderness ratio (λ_y) of beam segment from the laterally braced point to the supporting of the beam and its bearing capacity shall comply with the provisions in the national standard "Code for steel structure design" GB50017.

4 When the structure with not exceeding 12 stories in height, the gusset plate, which using in connection of the brace and the frame, shall have an angel not less than 30° on each side of the diagonal, see the provision of national standard "Code for design of steel structures" GB50017. And the distance from the end of the brace diagonal to the fixing point of

the gusset plate shall not be smaller than two times of the gusset plate thickness.

8.4.4 For the epicenter brace frame, when height does not exceed 100m and the seismic-forces distributed to the frame part is not greater than 25% of the total seismic shear, the detail requirements for the frame part may be reduced by one degree for Intensity 8 and 9. Other details shall comply with the provisions for framed structures in Section 8.3 of this chapter.

8.5 Details of seismic design for steel frame-eccentric-braced structures

8.5.1 The specified yield strength of the steel material in the eccentric braced frame energy-dissipating beam-segment shall not be greater than 345 MPa. For the energy-dissipating beam-segment and other beam segment for the same span, the width-to-thickness ratio of the elements shall not be greater than the limit values in Table 8.5.1.

Table 8.5.1 Limit values for width-to-thickness ratio of element

Name of element		Width-thickness ratio
Flanges with one free edge		8
Webs	$N/(Af) \leqslant 0.14$	$90[1-1.65N/(Af)]$
	$N/(Af) > 0.14$	$33[2.3-N/(Af)]$

Note: the values listed in the table is applicable to Q235 steel, other steel types shall multiply by $\sqrt{235/f_{ay}}$.

8.5.2 The slenderness ratio of the brace in eccentric braced frame shall not be greater than $120\sqrt{235/f_{ay}}$, and the width-thickness ratio of the element shall not exceed the limit value of width-to-thickness ratio for members subject to axial compression through the centroidal axis at elastic design, which provisions in the national standard "Code for design of steel structures" GB50017.

8.5.3 The detail of the energy-dissipating beam-segment shall comply with the following requirements:

1 When $N > 0.16Af$, the length of the energy-dissipating beam-segment shall comply with the following provisions:

when $\rho(A_w/A) < 0.3$, $\quad a < 1.6M_{lp}/V_l$ (8.5.3-1)

when $\rho(A_w/A) \geqslant 0.3$,

$$a \leqslant [1.15 - 0.5\rho(A_w/A)]1.6\, M_{lp}/V_l \quad (8.5.3\text{-}2)$$

$$\rho = N/V \quad (8.5.3\text{-}3)$$

where a——the length of the energy-dissipating beam-segment.

ρ——ratio of axial force design value to the shear force design value for the energy-dissipating beam-segment.

2 The add-stiffening plate welded to increase the thickness or opening shall not be arranged on the web of the energy-dissipating beam-portion.

3 At the connection of the energy-dissipating beam-segment and the brace, stiffening ribs shall be installed on the two sides of the web, the depth of the stiffening ribs shall be

the same as the that of the beam. The width of the stiffening ribs on one side shall not be smaller than $(b_f/2 - t_w)$, and the thickness shall not be smaller than $0.75t_w$ and 10mm, whichever is bigger.

4 On the web of the energy-dissipating beam-segment, the intermediate stiffening ribs installed shall comply with the following requirements:

1) When $a \leqslant 1.6M_{lp}/V_l$, the spacing of the stiffening ribs shall not be greater than $(30t_w - h/5)$;

2) When $2.6\ M_{lp}/V_l < a \leqslant 5M_{lp}/V_l$, the stiffening ribs shall be installed at $1.5b_f$ from the end of the energy-dissipating beam-segment, and the spacing of the stiffening ribs shall not be greater than $(52t_w - h/5)$;

3) When $1.6M_{lp}/V_l < a \leqslant 2.6M_{lp}/V_l$, the spacing of stiffening ribs shall be interpolated linear between the two cases above;

4) When $a > 5\ M_{lp}/V_l$, stiffening ribs may not be installed;

5) The intermediate stiffening ribs shall be the same depth as the web of the energy-dissipating beam-segment; when the depth of the energy-dissipating beam-segment is not greater than 640mm, the stiffening ribs may be installed at only one-sided of web; when the depth of the energy-dissipating beam-segment is greater than 640mm, stiffening ribs shall be installed at the both sides of the web; the width of the stiffening rib installed at only one-side shall not be less than $(b_f/2 - t_w)$, and the thickness shall not be less than t_w and 10mm.

8.5.4 The connection of the energy-dissipating beam-segment with the column shall comply with the following requirements:

1 When the energy-dissipating beam-segment connects with the column, its length shall not be greater than $1.6\ M_{lp}/V_l$, and shall also comply with the provision in Clause 8.2.7.

2 The connection between the energy-dissipating beam-segment flange and the column flange shall adopt the completely fusion V-weld joint, the connection between energy-dissipating beam-segment web and the column shall adopt the fillet-welding joints. And the bearing capacity of the fillet-welding joint shall not be less than the axial tensile capacity, the shear capacity and the bending capacity of the web of the energy-dissipating beam-segment.

3 When the energy-dissipating beam-segment is connected with the column web, completely fusion V-weld joint shall be adopted for connection between the flange of the energy-dissipating beam-segment and the gusset plates; the fillet-welding joint shall be adopted for connection between the web of beam-segment and the column. The bearing capacity of the fillet joint shall not be less than the axial tensile capacity, the shear capacity and the flexure capacity for the web of the energy-dissipating beam-segment.

8.5.5 Lateral braces shall be installed on the upper and lower flange for ends of the ener-

gy-dissipating beam-segment. The axial force design value of the lateral brace shall not be less than 6% of the axial tensile capacity design value of the flange of the energy-dissipating beam-segment, i.e. $0.06\ b_f t_f f$.

8.5.6 Lateral braces shall be installed on the upper and lower flange of the non-energy-dissipating beam-segment of the eccentric brace framed beam, the axial force design of the lateral brace shall not be less than 2% for the axial tensile capacity of the beam flange, i.e. $0.02 b_f t_f f$.

8.5.7 For the frame-eccentric-brace structures, when the height does not exceed 100m and the seismic-forces distributed to the frame part is not greater than 25% of the total seismic shear, the requirements for detail requirements of frame part may be reduced by one degree for Intensity 8 and 9. Other detail requirements shall comply with the provisions for framed structures in Section 8.3 of this chapter.

Chapter 9 Single-story factory buildings

9.1 Single-story factory buildings with reinforced concrete columns

(Ⅰ) General

9.1.1 Layout for the fabricated factory building shall comply with the following requirements:

1 Height and length of each span in multi-span factory buildings should be equal.

2 Auxiliary building of a factory building should not be arranged at the corners of the building or close-by the seismic joint.

3 The seismic joint should be installed in factory buildings with an irregular configuration or with attached buildings. The clear width of the seismic joint may be taken as 100～150mm at the junction of the longitudinal and the transversal factory units, at the large bay factory and at no-brace factory; and may be taken as 50～90mm at other cases.

4 For the transitional span between two main factory buildings, the seimic joints shall be installed to be separated with the main factory building on at least one side.

5 The steel ladder for getting on the crane within the factory building shall not be installed near the seismic joint; the steel ladder for getting on the crane in multi-span factory building shall not be installed near the same transversal axial line.

6 The operation platform should be separated with the main structure of the factory building.

7 Within the same structural unit of the factory building, the different structural systems shall not be used, not the loading masonry gable wall but the roof truss shall be installed at the end of the factory building, and load-bearing compounded by transversal masonry wall and R.C.bent shall be avoided.

8 The lateral stiffness of each longitudinal row of the columns in the factory building shall be even.

9.1.2 Installation of the skylight truss of the factory building shall comply with the following requirements:

1 The skylight should adopt the shelter type skylights rising small out of the roof, the basin-type skylights should be adopted when conditions are sat or for Intensity 9.

2 The convex-type skylights should adopt steel skylight; for Intensity 6 through Intensity 8, the reinforced concrete skylight with rectangular section elements may be adopted

3 For Intensity 8 and 9, the first skylight-truss should be installed on the third bay from the end to middle of the factory buildings.

4 The roof and perimeter walls of skylight should be made of lightweight materials.

9.1.3 Installation of the roof truss for factory buildings shall comply with the follow requirements:

1 Factory buildings should adopt steel truss, or pre-stressed concrete and reinforced concrete truss with lower center of gravity.

2 When the span is less than or equal 15m, reinforced concrete roof girders may be used.

3 When the span is greater than 24m, or for Intensity 8 with Site-class Ⅲ and Ⅳ or Intensity 9, it shall be predominated that steel truss is to be adopted.

4 When the spacing of the columns is 12m, prestressed concrete spandrel truss (or girder) may be adopted, and for steel truss, the steel spandrel truss (or girder) may also be used.

5 The roof truss with convex-type skylight shall not adopt prestressed concrete or reinforced concrete opening-web truss.

9.1.4 Installation of columns in factory building shall comply with the following requirements:

1 For Intensity 8 and 9, the rectangular, I-shaped section column or double limb columns with inclined web-bar should be adopted; but the thin-web I-shape columns, the opening web I-shape columns, the precast web I-shape columns and the tube columns should not be adopted.

2 Rectangular sections should be used from the bottom of the column to the scope of 500mm above the ground floor level, or the upper column of stepped columns.

9.1.5 The arrangement and details for the enclosure wall and the parapet wall of the factory building shall comply with relevant provisions of non-structural members in Section 13.3 of this code.

(Ⅱ) Essentials in calculation

9.1.6 For the factory buildings assigned to single-span or multi-span with equal height, the column height less than or equal to 10m, both gable wall has installed, and for Intensity 7 with the Site-class Ⅰ and Ⅱ, the seismic analysis of transversal and longitudinal directions need not be carried out. But the seismic details of factory building shall comply with provisions of this code.

9.1.7 The following methods shall be used for seismic calculation in transversal direction of a factory building:

1 For the factory buildings with reinforced concrete roof with or without purlins, the multi-mass spacious structure analysis considering the influence of transversal planar elastic deformation of the roof should be used generally. When satisfying the conditions in Appendix H of this code, it may be calculated by using planar bent analyzed method, and corresponding adjusting factors for seismic shear and bending moment of the bent as set forth in Appendix H in this code may be used.

2 For the factory buildings with a lightweight roof and the column bays are same, the planar bent analyzed method may be used.

Note: Lightweight roof is refers to a roof with purlins that made of fluted steel sheets, or corrugated iron sheets, or asbestos sheets.

9.1.8 The following methods shall be used for seismic calculation in longitudinal direction of a factory building:

1 For the factory buildings with concrete roof with or without purlin as well as with lightweight roof that have considerably complete bracing system, the following methods may be adopted:

1) Generally, it should be analyzed as multi-mass spacious structures, considering the influence of longitudinal planar elastic deformation of the roof, effective stiffness of enclosure walls and partition walls, as well as the torsion due to non-symmetry.

2) For factory buildings with single-span or multi-spans with equal height whose column height is less than or equal to 15m and the average span is less than 30m, it should be analyzed by using the modification rigidity method for the provision in Appendix J of this code.

2 For single-span factory buildings and multi-span factory buildings with lightweight roof and longitudinal wall arranged symmetrically, it may be analyzed by using the longitudinal bent consisting of each column rows separately.

9.1.9 The transversal seismic calculation for convex-type skylight may use the following methods:

1 The base shear method may be used for seismic calculation of reinforced concrete or steel three-hinged skylight truss with struts. When the span of the skylight truss is greater than 9m or for Intensity 9, the seismic effect of the concrete skylight truss shall be multiplied by an amplifying coefficient, which value may be taken as 1.5.

2 In other cases, the horizontal seismic action of the skylight truss may adopt the mode analysis method with response spectrum.

9.1.10 The longitudinal seismic calculation for convex-type skylight may be carried out according to the following methods:

1 The spacious structure analysis method may be used for seismic calculation of skylight truss in the longitudinal direction, considering planar elastic deformation of the roof and effective stiffness of the longitudinal walls.

2 Base shear method may be used in the calculation of longitudinal seismic action for skylight truss in the single-span or multi-span with equal height factory building, which reinforced concrete roof without purlin is used and the height of columns does not exceed 15m. But the seismic effect of the skylight truss shall be multiplied by the following amplifying coefficients:

1) For the roof in a single span or an exterior span, or in the mid-span having longitudinal interior partition walls

$$\eta = 1 + 0.5n \tag{9.1.10-1}$$

2) For the roof in other mid-spans

$$\eta = 0.5n \tag{9.1.10-2}$$

where: η——amplifying coefficient of seismic effect.

n——number of spans in the factory building; if number of spans exceeds 4, shall equal 4.

9.1.11 For a factory building with the column spacing which in both direction is not less than 12m, and without a bridge-type crane and column braces, the horizontal seismic action in two major axis directions shall be considered simultaneously in seismic checking of cross-section. And the P-delta effect shall be considered also.

9.1.12 In factory buildings with unequal heights, the section area of the longitudinal tensile reinforcement in the column bracket, supporting the roof on the lower span, shall be determined according to the following equation:

$$A_s \geqslant \left(\frac{N_G a}{0.85 h_0 f_y} + 1.2\frac{N_E}{f_y}\right)\gamma_{RE} \tag{9.1.12}$$

where: A_s——section area of the longitudinal tensile reinforcement.

N_G——design value of compression on the column bracket subject to the representative value of gravity load.

a——distance from the action axis of the gravity load to the near edge of the lower column bracket; when it is less than $0.3h_0$, shall equal $0.3h_0$.

h_0——effective depth of the largest vertical section of the bracket.

N_E——design value of horizontal tension in the seismic combination on the top of the column bracket.

γ_{RE}——seismic adjusting factor for loading capacity, it may be taken as 1.0.

9.1.13 The seismic effect of the crosswise column braces and the seismic check for connection of the brace with the column may be carried out according to the provisions in Appendix J of this code.

9.1.14 For Intensity 8 and 9, the out-of-plane seismic check shall be made for the wind-resistant column of tall gables.

9.1.15 When the wind-resisting column connects with the bottom chord of roof truss, the intersecting point shall be arranged at the node of the transverse brace of the bottom chord, and seismic check shall be made for the section of the transverse brace and its connection.

9.1.16 When the working deck and rigid separation wall connect with the main structural members of the factory building, the structure calculation model shall comply with the actu-

al working state of this factory, and the additional seismic effect of that on this factory shall be taken into consideration. And the columns of bent frames whose deflection is restricted and shear span-to-depth ratio not more than 2, the shear capacity shall be checked according to the provisions in the current national standard "Code for design of concrete structures" GB50010, and corresponding seismic details shall be taken.

9.1.17 For Intensity 8 with Site-class Ⅲ and Ⅳ and for Intensity 9, the arched and polygonal bowstring truss with small posts adjusting roof pitch or the truss with longer top chord portion and larger bow-height, the torsion capacity check should be made for the top chord of these trusses.

(Ⅲ) Details of seismic design

9.1.18 Connection of members and arrangement of braces in the roof with purlin shall comply with the following requirements:

1 The purlins shall be welded tightly with the roof truss (roof girder), and sufficient supported length of purlins on truss shall be provided.

2 Both purlins of the double-ridge roof shall be tied with each other at 1/3 of the span.

3 The fluted steel sheets shall be connected firmly with purlins, and the corrugated iron sheets and asbestos sheets shall be tied with purlins.

4 Arrangement of braces shall comply with the requirements in Table 9.1.18.

Table 9.1.18 Braces arrangement for roof with purlin

<table>
<tr><th colspan="2">Braces</th><th>Intensity 6 and 7</th><th>Intensity 8</th><th>Intensity 9</th></tr>
<tr><td rowspan="4">Braces of the roof truss</td><td>Transverse braces at the top chord</td><td>One row in each end bay of a factory building unit</td><td>One row in each end bay of a factory unit, the column-brace bay of a unit with length greater than 66m;
Local both end bays of the opening zone caused by the skylight</td><td rowspan="3">One row in each end bay of a factory unit, the column-brace bay of a unit with length greater than 42m;
Local both end bays of the opening zone caused by the skylight</td></tr>
<tr><td>Transverse braces at bottom chord</td><td colspan="2">Same as in the non-seismic design</td></tr>
<tr><td>Vertical brace in the middle of span</td><td colspan="2">Same as in the non-seismic design</td></tr>
<tr><td>Vertical brace at the end of span</td><td colspan="3">For the end vertical member with height greater then 900mm of a truss, one row in each end bay and column-brace bay of a factory unit</td></tr>
<tr><td rowspan="2">Braces of the skylight</td><td>Transverse braces at the top chord</td><td>One row in each end bay of skylight</td><td rowspan="2">One row in each end bay of skylight and in each 30m spacing</td><td rowspan="2">One row in each end bay of skylight and in each 18m spacing</td></tr>
<tr><td>Vertical brace at two sides</td><td>One row in each end bay of skylight and in each 36m spacing</td></tr>
</table>

9.1.19 Connection of members and arrangement of braces in the roof without purlins shall comply with the following requirements:

1 Large-size roof panels shall be welded firmly with the roof truss (roof girder), and

length of the welding seam in the connection of the roof panel adjacent to the row of columns and the roof truss (roof gird) should be not less than 80 mm.

2 For the end bay of a factory unit with a skylight of Intensity 6 and 7, and for each bay of Intensity 8 and 9, adjacent large-size roof panels which are orthogonal to the roof truss should be welded with each other at their top surfaces.

3 For Intensity 8 and 9, angle steel should be used as embedded parts at the bottom of the end of the large-size roof panel, and it shall be welded firmly with the main reinforcement in the panel.

4 For non-standard roof panel, precast monolithic joint should be used, or all of the corners of the panel should be cut and then welded firmly with the roof truss (girder).

5 Anchorage bars of the embedded parts on the top of the end of the roof truss (roof girder) should be not less than 4ϕ10 and 4ϕ12 for Intensity 8 and 9 respectively.

6 Arrangement of braces should be comply with the requirements in Table 9.1.19-1 and Table 9.1.19-2 for the basin-type skylight. When the roof gird with the span not greater than 15m for Intensity 8 and 9, it may be installed only the vertical bracing at each end of a factory unit.

Table 9.1.19-1 Braces arrangement for roof without purlin

<table>
<tr><th colspan="3">Braces</th><th>Intensity 6 and 7</th><th>Intensity 8</th><th>Intensity 9</th></tr>
<tr><td rowspan="7">Braces of the roof truss</td><td colspan="2">Transverse braces at the top chord</td><td>Same as non-seismic design when the span is less than 18m.
One row in each end bay of a factory unit when the span ≥18m.</td><td colspan="2">One row in each end bay of a factory unit, and the column-brace bay.
Local end bay of the opening zone caused by the skylight.</td></tr>
<tr><td colspan="2">Horizontal tie rod through all factory at top chord</td><td>Same as non-seismic design</td><td>One row along the bay in each 15m, but non-necessary for precast monolithic roof;
And non-necessary at truss end when in-situ ring-beam of curtain wall installed at top chord level</td><td>One row along the bay in each 12m, but non-necessary for precast monolithic roof;
And non-necessary at truss end when in-situ ring-beam of curtain wall installed at top chord level</td></tr>
<tr><td colspan="2">Transverse braces at the bottom chord</td><td>Same as non-seismic design</td><td>Same as non-seismic design</td><td>Same as transverse braces at top chord</td></tr>
<tr><td colspan="2">Vertical brace in the middle of span</td><td>Same as non-seismic design</td><td>Same as non-seismic design</td><td>Same as transverse braces at top chord</td></tr>
<tr><td rowspan="2">Vertical brace at ends of truss</td><td>end height ≤900mm</td><td>Same as non-seismic design</td><td>One row in each end bay of a factory unit</td><td>One row in each end bay of a factory unit and in each 48m spacing</td></tr>
<tr><td>end height >900mm</td><td>One row in each end bay of a factory unit</td><td>One row in each end bay of a factory unit and the column-brace bay</td><td>One row in each end bay of a factory unit, the column-brace bay and in each 30m spacing</td></tr>
</table>

Table 9.1.19-1 (continued)

Braces		Intensity 6 and 7	Intensity 8	Intensity 9
Braces of the skylight	Vertical brace at two sides	One row in each end bay of skylight and in each 30m	One row in each end bay of skylight and in each 24m	One row in each end bay of skylight and in each 18m spacing
	Transverse braces at the top chord	Same as non-seismic design	For span of skylight is not less than 9m, one row in each end bay of skylight and the column-brace bay	One row in each end bay of skylight and the column-brace bay

Table 9.1.19-2 Braces arrangement of basin-type skylight roof without purlin

Braces		Intensity 6 and 7	Intensity 8	Intensity 9
Transverse braces at the top chord and bottom chord		One row in each end bay of a factory unit	One row in each end bay of a factory unit and the column-brace bay	
Horizontal tie rod through factory at top chord		In the joints of the top chord of the truss within the opening zone caused by skylight		
Horizontal tie rod through factory at bottom chord		In both sides of skylight and in the joints of the bottom chord of the truss within the opening zone caused by skylight		
Vertical brace in the middle of span		In bay with transverse braces at the top chord, their location corresponds to the tie rod through over factory at bottom chord		
Vertical brace at both ends of the roof truss	End height ≤ 900mm	Same as non-seismic design		Bay with the top chord transverse braces, and spacing ≤ 48m
	End height > 900mm	One row in each end bay of a factory unit	Bay with the top chord transverse braces, and spacing ≤ 48m	Bay with the top chord transverse braces, and spacing ≤ 30m

9.1.20 The braces of the roof shall also conform to the following requirements:

1 Within the opening zone caused by skylight, a horizontal compressive rod at top chord shall be installed through on the ridge of the truss.

2 The spacing of vertical braces at middle of span shall not be greater than 15m for Intensity 6 through Intensity 8, and greater than 12m for Intensity 9. When only one row is installed, the location shall be the ridge of the truss; when two rows are to be installed, they shall be distributed evenly.

3 The relative segments of horizontal tie rod of the top and bottom chord through factory should be treated as the component part of the vertical brace of the truss.

4 For factory with column bays not less than 12m and the truss spacing 6m, bottom chord longitudinal horizontal braces shall be installed in the bay of spandrel truss (beam) and its next bays.

5 The brace members of the roof truss should adopt shaped steel.

9.1.21 For the reinforced concrete convex-type skylight, the wall panels on both sides should be connected with vertical post of skylight by using bolts.

9.1.22 The cross section and reinforcement of the reinforced concrete roof truss shall comply with the following requirements:

1 For the first portion for the top chord of the truss and the end vertical post of the

trapezoid-shaped truss, they reinforcement should be not less than 4ϕ12 for Intensity 6 and 7, and not less than 4ϕ14 for Intensity 8 and 9.

2 The cross-sectional width of the end vertical post of the trapezoid-shaped truss should be the same as that of the top chord.

3 For the small posts supporting the roof panels at the end of the top chord of the arched and polygonal bowstring truss, the cross section dimension should not be less than 200mm×200mm, and the height should not be greater than 500mm. Major reinforcements in the small post shall be Π-shape and not less than 4ϕ12 for Intensity 6 and 7, or 4ϕ14 for Intensity 8 and 9; the stirrups in small post may be used ϕ6, and the spacing should be 100mm.

9.1.23 The stirrups in a column of the factory shall be comply with the following requirements:

1 The stirrups shall be densified in the range below:

1) The column top, for a length of 500 mm from the top of the column, and also not less than the dimension of the longer side of the column cross section;

2) Upper column, from the bracket surface to a distance of 300mm above the crane beam surface of a stepped column;

3) Bracket (the column shoulder), through over height;

4) The column foot, from the bottom of the lower column to a distance of 500mm above the indoor ground level;

5) The connection of the column and its brace, such as the locations that the column displacement is constrained by the platform or others, a distance 300mm above and below of the joint.

2 The spacing of stirrups in the densified zone shall not be greater than 100 mm, and the spacing between limbs and the diameter of the stirrup shall comply with the provisions in Table 9.1.23.

Table 9.1.23 Maximum spacing for limbs and minimum diameter of stirrups in the densified zone

Intensity and Site class		Intensity 6, Intensity 7 with Site-class Ⅰ, Ⅱ	Intensity 7 with Site-class Ⅲ and Ⅳ, Intensity 8 with Site-class Ⅰ and Ⅱ	Intensity 8 with Site-class Ⅲ and Ⅳ, Intensity 9
Maximum spacing for limbs of stirrup (mm)		300	250	200
Min. diameter of stirrup	Common column top and foot	6 mm	8 mm	8 (10) mm
	Column top of corner column	8 mm	10 mm	10 mm
	Bracket of upper column and column root with brace	8 mm	8 mm	10 mm
	Column top with brace and location at which column deformation is constrained	8 mm	10 mm	10 mm

Note: values in the brackets are used for the column foot.

9.1.24 Reinforcement of wind-resisting column of the gable shall comply with the follow-

ing requirements:

1 For the stirrups within the scope of 300mm below the top of the wind-resisting column and 300mm above the brackets (column shoulder), the diameter should not be less than 6mm, the spacing should not be greater than 100mm, and the spacing of limbs should not be greater than 250mm.

2 Longitudinal tensile reinforcements should be installed at the brackets (column shoulder) of the variation section of the wind-resisting column.

9.1.25 The section and reinforcement of column for factory with large-spacing of column shall comply with the following requirements:

1 The column section should adopted the square or nearly square rectangular shapes, and the depth should not be less than 1/18～1/16 of the total height of the column.

2 The axial-force-ratio of column for heavy weight roof factory, should not be greater than 0.8 for Intensity 6 and 7;0.7 for Intensity 8;or 0.6 for Intensity 9.

3 Longitudinal steel bars should be arranged symmetrically along the perimeters of the cross section, the spacing should not be greater than 200mm; and the steel bars with greater diameter shall be arranged at the corner of section.

4 The stirrups in the column top and column foot shall be densified and comply with the following requirements:

1) For the column foot, the densified length from the bottom of column to a distance of 1m above the indoor ground level, and this distance shall not be less than 1/6 of the total height of the column. For the column top, 500mm below the top of the column and the distance shall not be less than the larger dimension of the column cross section;

2) The spacing, diameter and the limb spacing of the stirrups shall conform to the provisions in Clause 9.1.23 of this chapter.

9.1.26 The arrangement and details of the column-braces in the factory building shall comply with the following requirements:

1 The arrangement of column-braces shall comply with the following provisions:

1) Generally, column-braces shall be installed for the upper and lower columns in the middle of the factory building unit, and the braces of the lower column and the upper columns shall be installed correspondingly;

2) When there are cranes or for Intensity 8 and 9, added braces of upper columns shall be installed at the both ends bay of the factory-building unit;

3) When the factory building unit is long or for Intensity 8 with Site-class Ⅲ and Ⅳ or for Intensity 9, two rows of column-braces may be arranged in 1/3 length scope measured from both ends of the factory unit.

2 The column-braces shall adopt shaped steel, the brace type should be of the cross-wise type, the angel between the struts and the horizontal level should not be greater than

55°.

3 The slenderness ratio of the brace strut should not exceed the provisions in Table 9.1.26.

4 The location of the lower joint for the brace of lower column shall ensure to transfer the seismic action to the foundation. For Intensity 6 and 7, if cannot transfer the seismic action directly to the foundation, that unfavorable affection of the brace on the column and the foundation shall be considered.

5 The gusset plate shall be installed at the intersecting point of the crosswise brace struts, and the plate thickness shall not be less than 10mm. The struts shall be welded to the gusset plate of intersection point of crosswise brace and should be welded to the gusset plate of end joint.

Table 9.1.26 Maximum slenderness ratio of crosswise brace diagonals

Intensity and Site class	Intensity 6, Intensity 7 with Site-class Ⅰ and Ⅱ	Intensity 7 with Site-class Ⅲ and Ⅳ, Intensity 8 with Site-class Ⅰ and Ⅱ	Intensity 8 with Site-class Ⅲ and Ⅳ, Intensity 9 with Site-class Ⅰ and Ⅱ	Intensity 9 with Site-class Ⅲ and Ⅳ
Brace of upper column	250	250	200	150
Brace of lower column	200	200	150	150

9.1.27 The horizontal compression bars should be installed along the top level of the middle column rows for multi-span factory building with a span not less than 18m at Intensity 8, or along the top level of all columns rows for multi-span factory building at Intensity 9. This compression bar may be taken instead of the horizontal tie rod installed at the supports of the trapezoid-shape roof truss. The clearance between the end of the reinforced concrete tie rod and the roof truss shall be filled with concrete.

9.1.28 The connection of the structural members in a factory building shall comply with the following requirements:

1 The roof truss (girder) should be connected with columns on top by bolts for Intensity 8 and by hinge plate for Intensity 9, but bolts may also be used for Intensity 9. The thickness of the bearing plate at the end of the roof truss (girder) should not be less than 16 mm.

2 The anchorage bars of embedded parts on top of a column should be 4ϕ14 for Intensity 8 and 4ϕ16 for Intensity 9. For columns with braces, shear plate shall be installed additionally other than embedded parts.

3 The embedded plate shall be put on the top of the wind-resisting column in a gable wall, so that top of the column can be reliably connected with the top chord of the end truss (or top flange of the roof girder). The connected point shall be installed at the node of top chord transverse brace and roof truss; if not, the transverse brace may install additionally sub-web or shaped steel beam to transfer the horizontal seismic force to this node.

4 The embedded parts in the bracket (column shoulder) of the middle-column sup-

porting the lower-span roof shall be welded with the longitudinal reinforcement subject to the calculated horizontal tension in the bracket (column shoulder). And the welded reinforcements shall not be less than 2ϕ12 for Intensity 6 and 7, than 2ϕ14 for Intensity 8, and than 2ϕ16 for Intensity 9.

5 For Intensity 8 with Site-class Ⅲ or Ⅳ and for Intensity 9, the steel angles together with end plate should be used as anchors of the embedded parts at the joint of the brace and the column. And in other cases, the steel bars of grade HRB335 or HRB400 may be used as anchors, but the developed length shall be not less than 30 times the diameter of the anchor bar or end plate shall be adopted.

6 The crane corridor panel, the small roof panel filled the space between the end roof truss and gable, valley plate, and the fascia masonry underneath the skylight end plate and side plate etc. shall have reliable connection with its supporting structural members.

9.2 single-story steel structure factory buildings

(Ⅰ) General

9.2.1 This section is applicable to single-story factory buildings with single-span or multi-span of equal height and bearing steel columns and roof trusses, but is not applicable to single-story lightweight steel structure factory buildings.

9.2.2 The plan configuration of the factory building and the detailing requirements of the reinforced concrete roof panels may refer to relevant provisions for single-story factory buildings with reinforced concrete column in Section 9.1 of this code.

9.2.3 The structural system of the factory building shall conform to the following requirements:

1 The transverse lateral-force-resisting system of the factory building may adopt the frame which the beam and column are rigid-jointed or hinged, gabled frame, cantilever column, and other structural systems. The longitudinal lateral-force-resisting system should adopt column-braces; rigid-frame structures may also be used when the braces setting is not conditioned.

2 Within the maximum stress zone where member may occur plastic hinge, welded junctions shall be avoided; for members with larger thickness that bolt connection is impossible, the equal-strong-butt-weld joints may be adopted.

3 When the transverse roof girder and the top of the column adopt hinged joint, bolt connection should be adopted. For the rigid-frame, the gusset plate of the truss top chord and the column shall not have plastic deformation under seismic action. When the transverse girder is of a solid web type, the ultimate bending and shearing strength of the beam-to-column connection or beam-to-beam splicing shall not be less than 1.2 times of the yield bearing and shearing strength of the gross cross section of the beam respectively.

4 The whole piece material shall be adopted for the column-braces, when exceeding

the normal maximum length of the material, the equal-strong-splicing such as butt-welding connections may be adopted. The capacity of the connection of column-to-braces and the members shall not be less than 1.2 times of the plastic capacity of the brace.

(Ⅱ) Essentials in calculation

9.2.4 In the seismic calculation for factory buildings, the single-mass, double-mass and multi-mass models shall be adopted respectively based on the height differences of the roof levels and the arrangement of the crane.

9.2.5 In the calculation of seismic action for factory buildings, the weight and stiffness of the enclosed wall shall comply with the following provisions:

1 For lightweight panels or precast reinforced concrete panels that are flexibly connected with columns, the overall weight of the wall shall be considered, but the influence on stiffness may not be considered.

2 For masonry walls nestling closely to the outside face of column and tied with the column, the overall weight of walls shall be considered, but only 40% of stiffness in the direction parallel to the wall may be considered in the calculation.

9.2.6 The following methods may be adopted for seismic calculation in transversal direction of the factory building:

1 The spacious structure analysis considering the influence on transversal planar elastic deformation of the roof should be used generally.

2 When lightweight roof is adopted, the calculation may be made according to planar bents or frame.

9.2.7 The following methods may be adopted for seismic calculation in longitudinal direction of the factory building:

1 For factory buildings adopts lightweight wall panel or large-sized wall panel that is flexibly connected with columns may be calculated by using the single mass model, the seismic action of all column rows shall be distributed according to the following principles:

1) For reinforced concrete roof without purlins, the distribution may be made according to the rigidity ratio of the column rows;

2) For lightweight roof, the distribution may be made according to the gravity load representative value of the column rows;

3) For reinforced concrete roof with purlins, the average value of the above mentioned two results may be adopted.

2 In case the factory building adopts clay brick nestling closely to the outside face of column, may refer to the provisions in Clause 9.1.8 of this code.

9.2.8 The web-bars of the vertical brace trusses for the roof shall be able to resist and transfer the horizontal seismic action of the roof, the bearing capacity of its connection shall be greater than the interior force of the web-bars, and shall comply with the details requirements.

9.2.9 The seismic check of the crosswise column-brace may be made according to the provisions in Appendix J.2 of this code, where tensile strut and the effect of the compressive strut shall be considered. For connection of the ends of diagonal brace, the strength reduction shall be considered for single angle steel brace that shall not be used for Intensity 8 and 9. When one strut is non-whole piece in the diagonal brace, the gusset plate at the intersecting point shall be strengthened, and the bearing capacity of the gusset plate shall not be less than 1.1 times of strut bearing capacity.

(Ⅲ) Details of seismic design

9.2.10 The arrangement of roof braces should conform to relevant requirements in Section 9.1 of this code.

9.2.11 The slenderness ratio of the column shall not be greater than $120\sqrt{235/f_{ay}}$.

9.2.12 The width-to-thickness ratio limit value of the flanges and webs for single-story frame column and beam, besides complying with relevant provisions of elastic design in "Code for design of steel structures" GB50017, shall also conform to the provisions in Table 9.2.12

Table 9.2.12 Width-thickness ratio limit values of plates of single-story steel factory

Member	Name of plate	Intensity 7	Intensity 8	Intensity 9
Column	Flanges of H-section	13	11	10
	Middle-flanges of box-section	38	36	36
	Webs of H-and box-section ($N_c/Af<0.25$)	70	65	60
	($N_c/Af\geqslant 0.25$)	58	52	48
	External diameter-thickness ratio of tube	60	55	50
Beam	Flanges of H-section	11	10	9
	Middle-flanges of box-section	36	32	30
	Webs of H-and box-section ($N_b/Af<0.37$)	$85-120\rho$	$80-110\rho$	$72-100\rho$
	($N_b/Af\geqslant 0.37$)	40	39	35

Notes: 1. The values listed in the table are applicable to Q235 steel, for other type of steel, multiply by $\sqrt{235/f_{ay}}$.

2. N_c、N_b refer to the axial force of the column and the beam separately, A refers to the cross-sectional area of corresponding members, f refers to the tensile strength design value of the steel material.

3. ρ refers to N_b/Af.

4. The requirements of width-thickness ratio of web may be reduced by installed longitudinal stiffening ribs.

9.2.13 The column foot shall adopt the inserting or embedding type that shall ensure to transfer the bearing capacity of the column. Exposed rigid column foot may also be adopted for Intensity 6 and 7, but the combined moment design value of the column foot bolt shall be multiplied by the amplifying factor 1.2.

The buried depth for solid web steel column that adopts inserting column foot shall not be less than 2 times of the steel column depth and satisfy requirements in the following equa-

tion:

$$d \geqslant \sqrt{6M/b_f f_c} \tag{9.2.13}$$

where: d——buried depth of the column foot.

M——limit bending moment when the column foot yield.

b_f——width of flange of the column at the bending direction.

f_c——sepecified compressive strength of concrete of foundation.

9.2.14 The crosswise column-braces shall comply with the following requirements:

1 When cranes need to be installed, in the middle of the factory unit, column-braces shall be arranged at the upper and the lower columns, and the upper column-brace shall be added at the two ends of the factory unit. When the length of the structural unit exceeds 120m for Intensity 7, or exceeds 90m for Intensity 8 and 9, two rows of braces for the upper and lower columns should be arranged at 1/3 length scope measured form both ends of the factory unit.

2 The slenderness ratio of the crosswise column-braces, the angel between the brace strut and the horizontal level, the thickness of the gusset plate at the intersecting point of the struts shall conform to relevant provisions in Clause 9.1.26 of this code.

3 Energy-dissipating struts may be used if the condition is available.

9.3 single-story factory buildings with brick columns

(Ⅰ) General

9.3.1 This section is applicable to the following medium and small-size single-story factory buildings with fired common clay brick columns (piers):

1 The workshop and warehouse with single-span or multi-span of equal height, but without bridge crane.

2 For Intensity 6 through Intensity 8, the factory with a span not greater than 15m and an elevation at top of the column is not greater than 6.6m.

3 For Intensity 9, the factory with a span not greater than 12m and an elevation at top of the column is not greater than 4.5m.

9.3.2 The plan and vertical configuration of a factory building should be comply with the relevant provisions in Section 9.1 regarding as factory of reinforced concrete columns, but the arrangement of seismic joint shall conform to the following requirements:

1 The seismic joint need not be installed for a factory building with a lightweight roof.

2 A seismic joint should be installed between the factory building with a reinforced concrete roof and auxiliary building, and clear width of the joint may be taken as 50～70 mm.

3 Double columns or double walls shall be installed at the seismic joint.

Note: the lightweight roof refers to a roof that made of wood, or fluted steel sheets, or corrugated iron sheets, or asbestos sheets etc.

9.3.3 Load-bearing gables shall be installed at the both ends of the factory building. The skylight shall not penetrate to the end bays of the factory unit, and brick bearing walls shall not adopted at the end of skylight.

9.3.4 The structural system of a factory building shall also comply with the following requirements:

1 Lightweight roof should be used for Intensity 6 through Intensity 8 and shall be used for Intensity 9.

2 The cross-shaped section plain brick columns may be used for Intensity 6 and 7; the composite brick columns shall be used for Intensity 8 and 9, and reinforced concrete middle columns should be used for Intensity 8 with Site-class Ⅲ and Ⅳ and for Intensity 9.

3 The brick walls, resisting longitudinal seismic action and having the same height as the column, may be installed between brick columns in the longitudinal row of a factory building. Brick walls shall be set in staggered joint courses with the brick column and shall have a foundation. A through horizontal compressive bar shall be installed on top of the column without brick walls.

4 Transversal and longitudinal interior partition-walls should be designed as seismic structural walls. Lightweight-walls should be used as non-bearing transversal partition-walls or longitudinal partition-walls which height is less than that of column. When normal-weight walls are used, additional seismic shear induced by the partition-wall on the column and its connection with the roof truss shall be considered. For independent transversal and longitudinal partition-walls shall be designed to ensure the stability of the wall outside the plane, and cast in-situ reinforced concrete coping beams shall be installed on the top of the walls.

(Ⅱ) Essentials in calculation

9.3.5 For Intensity 7 with Site-class Ⅰ or Ⅱ, the single-story buildings with brick columns which satisfy the following requirements, the seismic analysis and check of transversal or longitudinal directions need not be carried out, but the seismic details shall comply with provisions of this section:

1 For the factory units assigned to single-span or multi-span with equal height, the column height less than or equal 4.5m, both gable walls installed, the seismic check in transversal and longitudinal directions need not be carried out.

2 For the factory units assigned to single-span, the column height not greater than 6.6m, both gable walls installed and both longitudinal walls which thickness not less than 240mm and the opening horizontal sectional area not exceed 50% gross area, longitudinal directions need not be checked.

9.3.6 The following methods may be used for the seismic calculation in the transversal di-

rection for factory buildings:

1 Lightweight roof factory buildings may be calculated according to planar bents.

2 The factory building assigned to reinforced concrete roof or wood roof with fully covered sheathings may be calculated according to planar bents that the space interaction has been considered, and the seismic effect shall be adjusted according to Appendix H of this code.

9.3.7 The following methods may be use for the seismic calculation in longitudinal direction for the factory building:

1 For the factory buildings assigned to reinforced concrete floor, the modal-analysis response-spectrum method may be adopted.

2 For the factory buildings assigned to reinforced concrete floor and multi-span with equal height, the modification rigidity method as set forth in Appendix K in this code may be adopted.

3 For single-span factory buildings and multi-span factory building with lightweight roof that longitudinal walls arranged symmetrically, the method to calculate independently according to the arrangement of column rows may be adopted.

9.3.8 The longitudinal and transversal seismic calculation for skylight rising out the roof shall conform to the provisions in Clause 9.1.9 and 9.1.10 of this chapter.

9.3.9 For eccentric compressive brick-columns, the seismic check shall comply with the following requirements:

1 For plain brick columns, the eccentricity of the seismic combinatory axial force design value shall not exceed 0.9 times of the distance from the cross-sectional centroid to the section edge in the direction where the axial force locates, and the seismic bearing capacity adjustment factor may adopt 0.9.

2 The reinforcement amount for composite brick columns shall be determined through calculation, and the seismic bearing capacity adjustment factor may adopt 0.85.

(Ⅲ) Details of seismic design

9.3.10 Arrangement of bracing in a wood roof should comply with the requirements as set forth in Table. 9.3.10; the roof with steel trusses, corrugated iron sheets or asbestos tiles may be regarded as roof without sheathings; and the horizontal tie bar of bottom chord connected with gable wall at end bay shall not be installed.

The bracing shall be connected with the roof truss or the skylight truss by bolts. The side columns of the wood skylight truss shall adopt full length wood clamping plate or steel plate and with bolts to strengthen the connection between the side column and the top chord of the truss.

9.3.11 The purlins shall be connected reliably with the coping-beam along the roof levels at the gable; when condition is available, the roof-structure for purlins extended to out-gable may also be adopted.

Table 9.3.10 Arrangement of braces in a wood roof

Braces		Intensity 6, 7	Intensity 8			Intensity 9	
		All type of roof	In full sheathing		Scattered or no sheathing	In full sheathing	Scattered or no sheathing
			Without skylight	With skylight			
Roof truss	Transverse braces at the top chord	Same as in the non-seismic design		One row in both end bay of skylight scope	When the span of roof truss is greater than 6m, a row in 2nd end bay and in each 20m	When the span of roof truss is greater than 6m, a row in 2nd end bay	When the span of roof truss is greater than 6m, a row in 2nd end bay and in each 20m
	Transverse braces at the bottom chord	Same as in the non-seismic design				Same as in the non-seismic design	When the span of roof truss is greater than 6m, a row in 2nd end bay and in each 20m
	Vertical brace at middle of span	Same as in the non-seismic design				Same as in the non-seismic design	In each other bay and plus horizontal tie bar at bottom chord through allover length
Skylight truss	Vertical brace at two sides	One row in each end bay of skylight				One row in each end bay of skylight and in each 20m	
	Transverse braces at top chord	For skylight with greater spans, the arrangement of brace may refer to the truss without skylight					

9.3.12 The deigned details for reinforced concrete roofs shall conform to the related provisions as set forth in Section 9.1 of this code.

9.3.13 A cast-in-place closed ring-beam shall be installed along the exterior wall and the bearing interior walls at the top level of the column; for Intensity 8 and 9, the ring-beams shall be also added in each 3～4m along the height of the wall. Depth of the ring-beam shall not be less than 180mm and its reinforcement shall not be less than 4ϕ12. Foundation ring-beam shall also be installed, when the subsoil is weak cohesive soil, liquefaction-soil, newly filled soil or a seriously non-uniformly distributed soil layer. When the ring-beam also serves as the lintel or resisting uneven-settlement, besides satisfying the above detail requirements, its cross-sectional dimension and reinforcements shall also be determined based on calculation for actual action.

9.3.14 A cast-in-place reinforced concrete coping-beam along the roof levels shall be installed at the gable wall and anchored with the roof members. The cross-sectional area and reinforcements of the buttress of the gable wall should not be less than those of the bent-column. The buttress shall be extended through to top of the gable wall and connected with the coping-beam at the gable wall or other roof members.

9.3.15 The ring-beam on top of the wall or bearing plate on top of the column shall be connected firmly with roof truss (girder) by bolts or welding. Thickness of the bearing plate on top of the column shall be not less than 240 mm, and two layers of reinforcement mesh

with a diameter of not less than 8mm and the spacing not greater than 100mm shall be installed in the plate. The ring-beam on top of the wall and the bearing plate on top of the column shall be poured at the same time. For Intensity 9 and scope in a distance of 500mm on both sides of the bearing plate, the spacing of stirrups in the ring-beam shall be not greater than 100mm.

9.3.16 The details of brick columns shall conform to the following requirements:

1 The brick strength grade shall not be less than MU10, the strength grade of mortar shall not be less than M5; the concrete strength grade for composite brick column shall adopt C20.

2 The damp-proof course of the brick column shall adopt waterproof mortar.

9.3.17 Factory buildings with brick columns and reinforced concrete floor panel, the horizontal sectional area of the opening on the gable should not exceed 50% of the gross cross-sectional area. For the Intensity 8, the reinforced concrete tie-columns shall be installed at the two ends of the gable and transverse wall. For Intensity 9, the reinforced concrete tie-columns shall be installed at the two ends of the gable, the transverse wall and tall opening of doors.

The cross-sectional dimension of the reinforced concrete tie-column may adopt 240mm $\times$240mm; but for Intensity 9 and the thickness of the gable and transverse wall is 370mm, the cross-sectional width of tie-column should select 370mm. The vertical reinforcements of the tie-column shall not be less than 4ϕ12 for Intensity 8, or 4ϕ14 for Intensity 9; the stirrups may adopt diameter 6mm, and the spacing of stirrup should select 250$\sim$300mm.

9.3.18 The details of brick wall shall comply with the following requirements:

1 For brick column factory buildings with reinforced concrete roof without purlins of Intensity 8 and 9, 1ϕ8 vertical reinforcement should be embedded in each 1m along the length of the brick protection wall and extended into ring-beam of the top of the wall.

2 For the wall height is greater than 4.8m of Intensity 7 or for Intensity 8 and 9, at the corner of the exterior wall and the intersection of the bearing interior transverse wall and the exterior longitudinal wall, the strengthened reinforcement shall be arranged, when no tie-column is installed. In which, the reinforcements 2ϕ6 shall be arranged in each 500mm along the height of the wall, and the length for each side extended into the wall should not be less than 1m.

3 The seismic detail requirements for parapet with rising out the roof shall conform to relevant provisions in Section 13.3 of this code.

Chapter 10 Single-story spacious buildings

10.1 General

10.1.1 This chapter is applicable to public buildings consisted of considerable spacious single-story hall and annex buildings.

10.1.2 The seismic joint shall not be installed between a hall and its ante-hall or stage, and also may not be installed between a hall and its attached rooms on both sides, but their connection shall be strengthened.

10.1.3 The load-bearing structures for the hall of single-story specious buildings shall not adopt plain brick columns in one of the following situations:

1 The building assigned to for Intensity 8 with Site-class Ⅲ and Ⅳ or for Intensity 9.

2 There is a cantilever platform in the hall.

3 For Intensity 8 with Site-class Ⅰ and Ⅱ or for Intensity 7 with Site-class Ⅲ and Ⅳ, the span of the hall is greater than 15m or the column height is greater than 6m.

4 For Intensity 7 with Site-class Ⅰ and Ⅱ or for Intensity 6 with Site-class Ⅲ and Ⅳ, the span of the hall is greater than 18m or the column height is greater than 8m.

10.1.4 The load-bearing structure of the roof at longitudinal wall for the hall of single-story spacious buildings, besides the provisions in Clause 10.1.3, may also add composite buttress with reinforced concrete and brick at the supports of roof, but plain brick buttress shall not be adopted.

10.1.5 The lateral stiffness in the transverse direction of the antehall shall be increased by the structural member layout, the gatepost and independent columns in the antehall shall adopt reinforced concrete columns.

10.1.6 The lateral stiffness of transverse walls, which in the connection of the antehall and the hall, as well as the hall and the stage, shall be increased, and a certain number of reinforced concrete walls shall be established.

10.1.7 For other requirements regarding the hall can refer to Chapter 9 of this code, and attached rooms of the hall shall comply with relevant provisions in this code.

10.2 Essentials in calculation

10.2.1 In seismic calculation of the single-story spacious buildings, which may be divided into several separate units, such as the ante hall, the stage, the hall, and attached rooms, each unit is calculated according to relevant structural system provisions in this code, but one another influnece for those units shall be considered.

10.2.2 The seismic calculation for single-story spacious building may adopt the base shear method, the seismic influence coefficient may be taken as the maximum value.

10.2.3 The horizontal longitudinal seismic action characteristic value of halls may be calculated according to the following equation:

$$F_{Ek} = \alpha_{max} G_{eq} \tag{10.2.3}$$

where: F_{Ek}——the characteristic value of horizontal longitudinal seismic action on the longitudinal wall or column row in one side of the hall.

G_{eq}——equivalent gravity load representative value; it includs the half of weight for the roof of hall, the half of weight for roof of adjacent attached rooms, 50% of the snow load characteristic value, and the conversed weight of the longitudinal wall or column row in one side of the hall.

10.2.4 The transversal seismic calculation for hall should comply with the following principles:

1 In case of hall without attached rooms on the two sides, the parts with and without cantilever platform may choose a typical bay for the calculation respectively; when the provisions in Chapter 9 of this code are satisfied, the space structural model may also be taken into consideration.

2 In case of hall with attached rooms on the two sides, proper calculation methods shall be selected based on the structural types of the attached rooms.

10.2.5 The seismic checking of out-of-plane for the buttress of the tall gable wall shall be carried out for Intensity 8 and 9.

10.3 Details of seismic design

10.3.1 The roof details of the hall shall comply with the provisions of Chapter 9 in this code.

10.3.2 The reinforced concrete column and composite brick-column of the hall shall comply with the following requirements:

1 The reinforcement bars of composite brick-columns shall be developed to the reinforced concrete ring-beam at the bottom of the truss. The longitudinal reinforcements of each side for the composite brick-column, besides to be determined by calculation, shall not be less than 4ϕ 14 for Intensity 6 with Site-class Ⅲ or Ⅳ and Intensity 7 with Site-class Ⅰ or Ⅱ, or 4ϕ 16 for Intensity 7 with Site-class Ⅲ or Ⅳ and Intensity 8 with Site-class Ⅰ or Ⅱ.

2 The reinforced concrete column shall be designed according to Grade 2 frame columns, and the reinforcement ratio shall be determined by calculation.

10.3.3 The transversal wall connecting the antehall with hall or the hall with stage shall comply with the following requirements:

1 The tie-columns or reinforced concrete columns shall be installed on the two ends

of the transverse wall, the supports of longitudinal girders and both edges of large openings of the walls.

2 Some transversal walls that built-in plane of the reinforced concrete frame columns shall be designed according to Grade 2 reinforced concrete seismic wall.

3 The reinforced concrete structure shall be adopted for girder and columns of the front of stage. For the bearing masonry wall above the girder in front of the stage, the pillars spacing not exceeding 4m and ring-beams spacing not exceeding 3m shall be installed. And their cross-sectional dimension, reinforcement ratio, and connection with the masonry wall shall conform to the requirements for multi-story masonry buildings.

4 Brick wall on the girder in front of the stage shall not be used for bearing loads for Intensity 9.

10.3.4 A cast-in-place ring-beam shall be installed at the top of the column (wall) of the hall, and an additional ring-beam shall be installed in each about 3m along the height of the wall. When the height of the end of the trapezoid-shaped truss greater than 900mm, the additional ring-beam shall also be installed at the level of the top chord. The depth of ring-beam should be not less than 180mm, and the width of ring-beam should be the same as thickness of the wall, the reinforcements should not be less than 4ϕ 12, and stirrup spacing should not exceed 200mm.

10.3.5 When no seismic joint is installed between the hall and the attached rooms, a closed ring-beam shall be installed at the same level and tied throughout the intersection place of the hall and the attached rooms. And at the intersection of walls, 2ϕ6 tie bars shall be installed in each 500mm along the height of the wall, and the ties should be extended into the wall on both sides with length not less than 1m.

10.3.6 The cantilever platform shall be reliably anchored, and details shall be taken to prevent its overturning.

10.3.7 The reinforced concrete coping-beams along the roof levels shall be installed at the gable wall and shall be tied with the roof members. The tie-columns or composite brick-columns shall be installed in the gable wall, howerer, their cross-sectional area and reinforcements shall not be less than those of the bent column or composite brick-column in the longitudinal wall respectively. These columns shall extend to the corresponding tops of the gable wall and be connected with the coping-beam on the gable wall.

10.3.8 The operation platform or floor diaphragm shall be used as the horizontal support of the tall gables, which are the back wall of the stage and the junction of the hall and the antehall.

Chapter 11 Earth, wood and stone houses

11.1 Unfired earth houses in villages and towns

11.1.1 This section is applicable to the unfired earth-houses with bearing adobe, lime-earth and rammed earth walls, as well as earth-caves and earth-arch houses for Intensity 6 through Intensity 8.

Note: 1. Lime-earth walls refer to walls made of earth added with lime (or other cementing materials) or walls made of adobe added with lime

2. Earth caves include both caves cut from undisturbed earth cliff and caves with an arch roof laid by adobe.

11.1.2 The unfired earth-houses should be of one story only; the two story houses with lime-earth-walls may be built which total height should be not exceeding 6 m for Intensity 6 and 7. For the single-story unfired earth-houses, the height of eave level should not exceed 2.5m, and the span of a room should not exceed 3.2m. The clear span of the earth-cave should not exceed 2.5m.

11.1.3 Transverse wall shall be installed in each bay of an unfired earth-house. The beams should not be laid on earth walls without bearing plate. The bearing walls made of different kinds of materials should not be used in the same house.

11.1.4 The lightweight roofing materials shall be used. For houses with purlins supported at gable wall, double-pitch roof or arch-shaped roof should be adopted, and bearing wood plate shall be installed to support purlins. On the top of wall, the wood ring-beam or bearing wood plate shall be installed; the purlins on gable wall shall extend out of the wall; the purlins on the interior wall shall be overlapped entirely or shall be connected by wood splice pieces or dovetail- dowel-connection. Members of the wood roof shall be connected each other by nails, cramp iron and galvanized wire.

11.1.5 The interior and exterior earth walls of unfired earth-house shall be laid by zigzag joint or rammed simultaneously in each layer. In the four corners of the exterior wall and the intersection of the exterior wall and the interior wall, a layer of tying materials should be laid at an interval of 300 mm along the height of the wall. And the tying materials may use bamboo bundles, timber battens or twigs of the chaste tree etc.

11.1.6 For all types of unfired earth-houses, the subsoil shall be rammed firmly, the brick or stone footing shall be adopted, and the damp-proof course should be put beneath the wall.

11.1.7 The adobe shall be molded by wet-process, using cohesive soil as raw material and straw and reed shall be added in the soil as tying materials. Adobe shall be built in flat

course and mud paste or mud-lime mortar should be used in laying walls.

11.1.8 For the houses with lime-earth walls, the ring-beam shall be installed in each story and shall pass through all transversal walls; and step-wise buttress should be added at each side of the gable wall up to the top of the interior longitudinal walls.

11.1.9 For the earth-arch houses, the multi-span arch structure shall be adopted and arranged continuously, the each arch footing shall be supported on the stable natural ground or artificial earth-walls. Thickness of the earth-arch should be 300～400 mm; the arch shall be laid by use formwork and not be laid by inclined bonds without formwork. For the exterior artificial earth-walls and the earth-arch, the doors or windows shall not be arranged.

11.1.10 For the construction of earth-caves, sites where land-slides or slumping are liable to occur shall be avoided; the soil of the earth cliff, from which the cave is cut, shall be dense and stable, and the cliff shall be smooth in slope, with no obvious vertical joints. For front of the cliff-cave, no adobe wall or walls of other materials shall be laid as the front wall; caves at different elevations should not be made, otherwise sufficient spacing between each other shall be kept for such caves, and they shall not be in line vertically

11.2 Wood houses

11.2.1 This section is applicable to houses with mortised timber frames, houses with timber trusses and wood columns, and houses with wood columns and wood beams.

11.2.2 Irregular configuration shall be avoided in the plan of wood houses. In a house, wood columns and brick columns or brick walls shall not be used simultaneously for bearing purpose.

11.2.3 The houses with timber trusses and wood columns, as well as houses with mortised timber frames, should not exceed two stories and total height of the house should not exceed 6m. The houses with wood columns and beams should be of single-story only and the height of the house should not exceed 3m.

11.2.4 For spacious houses of considerable large span, such as auditoriums, theaters and cereal warehouses, three-bay timber frame with four columns should be used.

11.2.5 The arrangement of roof brace of the timber truss shall comply with the related requirements in Section 9.3 of this code, but the roof braces on both ends of the house shall be installed in the end bay.

11.2.6 The top of the column shall be mortised to the bottom chord of the truss and connected by means of U-shaped cramp iron. For Intensity 8 and 9, foot of the column shall be anchored in the foundation by use of iron joining means.

11.2.7 The diagonal brace shall be installed between the wood column and the truss (or beam) in spacious houses; the diagonal brace shall be installed also in non-seismic partition walls in dwelling houses with considerable transverse walls; but braces need not be used in houses with mortised timber frames. Diagonal braces should be made of both timber plates

and extend to the top chord of the truss.

11.2.8 For the houses with mortised timber frames, along both longitudinal and transversal directions of houses, a piece of wood shall be installed at both upper and lower ends of the timber column, passing through and connected with the other column. And one or two diagonal column braces shall be installed in each row of columns in the longitudinal direction of house.

11.2.9 The diagonal braces and roof bracing members shall be connected with main structural members by bolts. Except for the mortised timber members, the connection of other wood members should be adopted bolts.

11.2.10 The connectied part of the rafters and the purlins shall be nailed completely to ensure the integrity of the roof. In the case of wood structures, vertical diagonal bracing should be installed above the eaves along the longitudinal direction of the houses, and the aim is to improve the longitudinal stability of the houses.

11.2.11 The wood members shall comply with the following requirements:

1 The tip diameter of the wood column should not be less than 150mm. The slotting on the same level of the column in both longitudinal and horizontal directions shall be avoided; besides, the slotting area on the same section shall not exceed 1/2 of the total section area.

2 The whole piece wood shall be adopted for the columns.

3 The penetrating purlins shall be set throughout all columns of the wood structure.

11.2.12 The enclosure walls shall be reliably tied to the wood structure; masonry enclosure walls with adobes and bricks shall not enclose the wood column completely and should be sat outside of the wood column.

11.3 Stone buildings

11.3.1 This section is applicable to masonry buildings with coursed square stone bearing walls (with or without chips of stone) laid by mortar for Intensity 6 to Intensity 8.

11.3.2 The total height and the number of stories of multi-story stone building shall not exceed the values as set forth in Tab 11.3.2:

Table 11.3.2 Limit Value for Total Height (m) and Number of stories of Multi-story stone Building

Type of masonry	Intensity 6		Intensity 7		Intensity 8	
Fine and semi-fine stone masonry (without chips)	16 m	5 stories	13 m	4 stories	10 m	3 stories
Course and gross stone masonry (with chips)	13 m	4 stories	10 m	3 stories	7 m	2 stories

Note: Determination for total height of building is specified in Note of Table 7.1.2 in this code.

11.3.3 The story height of multi-story stone masonry buildings shall not exceed 3m.

11.3.4 The spacing of the wall of multi-story stone masonry buildings shall not exceed the values as set forth in Table 11.3.4.

Table 11.3.4 Spacing for transverse walls in multi-story stone buildings(m)

Type of roofing	Intensity 6	Intensity 7	Intensity 8
Cast-In-situ or precast monolithic reinforced concrete slab	10	10	7
Precast reinforced concrete slab	7	7	4

11.3.5 Multi-story stone buildings should adopt cast in-situ or precast monolithic reinforced concrete roof and floors.

11.3.6 The seismic check of stone wall may refer to Section 7.2 of this code; and its shear strength shall be determined according to data obtained from tests.

11.3.7 Reinforced concrete tie-columns shall be installed in the following positions in multi-story stone buildings:

1 Four corners of the exterior wall and of the staircase.

2 The intersections of interior walls and exterior walls in every other bay for Intensity 6.

3 The intersections of interior walls and exterior walls of each bay for Intensity 7 and 8.

11.3.8 Horizontal sectional area of the opening in a seismic transverse wall shall be not greater than one-third of the gross sectional area.

11.3.9 Ring-beams shall be installed on the longitudinal and transverse walls in each story, the depth of which shall not be less than 120mm, and the width should be the same as the wall thickness. Longitudinal reinforcements in the ring-beam shall be not less than 4ϕ 10, and the spacing of stirrups shall not exceed 200mm.

11.3.10 At the intersection of the longitudinal wall and transverse wall, where no tie-columns are installed, the dressed rags shall be laid without chip of stone, and steel mesh shall be laid at intervals of about 500mm along the height of the walls. The mesh of intersection should be extended into the walls on each sides with length not less than 1m.

11.3.11 Other relevant detail requirements refer to provisions in Chapter 7 of this code.

Chapter 12 Seismic-isolation and seismic energy-dissipating design

12.1 General

12.1.1 This chapter is applicable to designs for seismic-isolated buildings that set up a seismic-isolation story between the upper structure and its foundation to isolate seismic energy, as well as to designs for seismic-energy-dissipated buildings that set up energy-dissipating devices in the lateral-force-resisting structures to dissipate the seismic energy.

The seismic-isolated and energy-dissipated building structures shall comply with the provisions in Clause 3.8.1 of this code, and the fortification subject shall comply with the provisions in Clause 3.8.2 of this code.

Notes: 1. The design of seismic-isolated building in this chapter refers to the seismic-isolation story, which consists of the rubber isolator units and the damper etc, established at the bottom of the building. The purpose for establishing the layer is to increase the vibration period and the damping, reduce the seismic energy introduced to the upper structures, and satisfy the expected seismic requirements.

2. The design of energy-dissipated building refers to install energy-dissipating components, which can provide additional damping by partial deformation, dissipate the seismic energy, which introduced to the upper structure, and satisfy the expected seismic requirements.

12.1.2 The designs for seismic-isolated and energy-dissipated building structures shall be determined after the technical and economical feasibility comparison analysis, which shall be given consideration to the fortification category, the fortification intensity, the conditions of the sites, the building structural plans and the use requirements.

12.1.3 The multi-story masonry and reinforced concrete frame building structures, which need to reduce the seismic action, shall comply with the following requirements when seismic-isolation designs are to be adopted:

1 For the structures with relevant regularity, the base-shear method provided in Clause 5.1.2 of this code can be used along two axial directions of structure and the calculated fundamental period of the structure shall be less than 1.0s when it is non-isolated. For irregular building structures need to adopt seismic-isolation designs, it shall be determined after the model tests.

2 The buildings shall be assigned to Site-class Ⅰ, Ⅱ, or Ⅲ; and the stable foundation types shall also be selected.

3 The total horizontal force produced by wind load and other non-seismic action shall not exceed 10% of the total structural gravity.

4 The seismic-isolatied story shall provide necessary load bearing capacity, lateral stiffness and damping; the pipelines and circuits of equipment crossing the seismic-isolation

story shall adopt flexible connection and other effective measures to withstand the horizontal displacement during rarely earthquakes.

12.1.4 The steel and reinforced concrete building structures, which need to reduce seismic horizontal displacement, should adopt energy-dissipating designs.

The energy-dissipating components shall be able to provide sufficient additional damping, and shall respectively comply with the requirements of corresponding structural types for the provisions in this code.

12.1.5 For the design of seismic-isolated and energy-dissipated buildings, the seismic-isolator units and energy-dissipating devices shall comply with the following requirements:

1 The durability and the design parameters of the seismic-isolator units and energy-dissipated devices shall be determined by testing.

2 The location installing seismic-isolator units and energy-dissipating devices, besides to be determined by calculation, measures for the convenience of check and renewal shall also be taken.

3 The property requirements for seismic-isolator units and energy-dissipating devices shall be stated clearly in the design document. The sampling check shall be carried out on prototype parts of each predominant type and size before installation, the number of each type and size shall not be less than 3, and the quality rate of the sampling check shall be 100%.

12.1.6 The designs of seismic-isolated and energy-dissipated buildings shall also comply with the provisions of corresponding specification.

12.2 Essentials in design of seismic-isolation buildings

12.2.1 The design of seismic-isolation buildings shall select appropriate isolation story consisted of the isolator units (including the dampers) and components with the primary stiffness to resist the pulsation of the ground and the wind load, based on the expected horizontal seismic-reduced factor and the requirements for seismic displacement control.

The load bearing capacity check and horizontal displacement check under rare earthquake shall be carried out for the seismic-isolator units.

The horizontal seismic action of lateral-force-resisting system above the isolation story shall be determined according to the horizontal seismic-reducing factor. The characteristic values of vertical seismic action for that shall not be less than 20% and 40% of the total gravity load representative values of the structure above the seismic-isolation story for Intensity 8 and 9.

12.2.2 The calculation analysis of the design of seismic-isolation building structures shall comply with the following requirements:

1 The calculation model of the seismic-isolation building structures may adopt the shear type model (Figure 12.2.2). When the gravity center of the structure above the iso-

lation story and the rigidity center of the isolation story are not in a line, the influence of torsion deformation shall be taken into consideration. For the reinforced concrete seismic-isolated structures, the beam-slab structures above the isolation interface shall be deemed as a part of the structure above the isolation story for calculation and design.

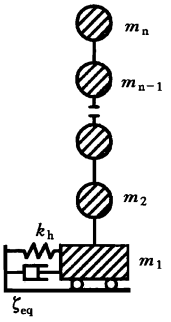

Figure 12.2.2 calculation sketch forseismic-isolation structure

2 In generally, the calculation should adopt the time-history analyzing method; the response spectrum characteristics and amount of the input seismic wave shall comply with the provisions in Clause 5.1.2 of this code; the calculation result should choose the average value. When building assigned to Category A and B is located within 10km of the causative fault and the input seismic-wave is not the near-field accelerogram, the calculation results shall be multiplied by the following near-field affected factors: within 5km, multiplied by 1.5; beyond 5km, multiplied by 1.25.

3 Masonry structures and structures similar to its foundamental period may carry out simplified calculation provided according to Appendix L of this code.

12.2.3 The rubber isolation units consisting of overlapped rubber layers and thin steel plates in the isolation story shall comply with the following requirements:

1 The ultimate horizontal displacement of the rubber isolator unit under the compressive stress provided in Table 12.2.3 shall exceed 0.55 times of its effective diameter or 3 times of the total thickness of all rubber layers, whichever is greater.

Table 12.2.3 Limit values of the average compressive stress for rubber isolator unit

Building category	A	B	C
Limit values of the average compressive stress (MPa)	10	12	15

Notes: 1. The average compressive stress design value shall be calculated according to the combination of the permanent load and variable load. For structures that need to check overturning, the horizontal seismic effect combination shall also be taken into consideration; for structures that need to calculate the vertical seismic action, the vertical seismic effect combination shall also be taken into consideration.

2. When the secondary form-factor of the rubber isolator unit is less than 5.0, the limit value of average compressive stress shall be reduced as follows: for less than 5 but not less than 4, reduce by 20%; for less than 4 but not less than 3, reduce by 40%.

3. In case of rubber isolator unit with outer diameter less than 300mm, the limit value for its average compressive stress shall be 12MPa for buildings assigned to Category C.

2 After passing the durability tests of corresponding Design-reference-period, the changes of stiffness and damping characteristics of the isolator unit shall not exceed ±20% of the primary stage values, the creep shall not exceed 5% of the total thickness of all rubber layers.

3 The average compressive stress design values of all rubber isolator units shall not

exceed the provisions in Table 12.2.3.

12.2.4 The arrangement, vertical bearing capacity, lateral stiffness and damping of the seismic-isolation story shall comply with the following requirements:

1 The seismic-isolation story should be installed at locations under the first story of the structure. The rubber isolator units shall be placed at locations where the interior forces are greater, the spacing should not be too large; the size, amount and distribution shall be determined according to the requirements of the vertical bearing capacity, the lateral stiffness and the damping. The seismic-isolation story shall be stabilized under rare earthquake, and should not have non-restorable deformations. The rubber isolator units should not have tensile stress under the rare earthquakes.

2 The horizontal dynamic stiffness and equivalent viscous damping ratio can be calculated according to the following equations:

$$K_h = \Sigma K_j \tag{12.2.4-1}$$

$$\zeta_{eq} = \Sigma K_j \zeta_j / K_h \tag{12.2.4-2}$$

where: ζ_{eq}——equivalent viscous damping ratio of the seismic-isolation story.

K_h——horizontal dynamic stiffness of the seismic-isolation story.

ζ_j——equivalent viscous damping ratio of the j-th isolator unit determined by testing; when damper is set up independently, corresponding damping ratio of the damper shall also be taken into account.

K_j——horizontal dynamic stiffness of the j-th isolator unit determined by testing; if the dynamic stiffness of isolator units dependent on the rate of loading, the dynamic stiffness value corresponding to the vibration period of the seismic-isolated structure shall be selected.

3 When the design parameters of the isolator units are determined by testing, the vertical load shall keep the limit values of average compressive stress of provision in Table 12.2.3. For checking of frequent earthquakes, the horizontal stiffness and equivalent viscous damping ratio shall be determined by test where horizontal loading frequency is 0.3Hz and the shear deformation of the unit is 50%. For checking of rare earthquake and the isolator units with a diameter smaller than 600mm, the horizontal stiffness and equivalent viscous damping ratio should be determined by test where horizontal loading frequency is 0.1Hz and the shear deformation of the unit is 250%. For checking of rare earthquake and the isolator units with a diameter not smaller than 600mm, the horizontal stiffness and equivalent viscous damping ratio may be determined by test where horizontal loading frequency is 0.2Hz and the shear deformation of the unit is 100%.

12.2.5 The calculation of seismic action for structures above the seismic-isolation story shall comply with the following requirements:

1 The distribution of the horizontal seismic action over the height of the structure may adopt rectangular; the maximum value of horizontal seismic influence coefficient can adopt the multiplication of the maximum value of horizontal seismic influence coefficient provided in Clause 5.1.4 of this code and the horizontal seismic-reduced factor. The horizontal seismic-reduced factor shall be determined according to Table 12.2.5 dependent upon the maximum ratio, of isolated story shear force to non-isolated that, in of all stories of structure.

Table 12.2.5 Corresponding relation of the maximum ratio of story shear force and the horizontal seismic-reduced factor

Max. ratio of story shear force	0.53	0.35	0.26	0. 18
Horizontal seismic-reduced factor	0.75	0.50	0.38	0.25

2 The horizontal seismic-reduced factor shall not be smaller than 0.25, and the total horizontal seismic action of the seismic-isolated structure shall not be lower than that of non-isolated structures for Intensity 6. The story shear force of each story shall also comply with the requirements regarding minimum shear force factors in Clause 5.2.5 of this code.

3 The calculation of the vertical seismic action for structures above the seismic-isolation story shall be carried out for Intensity 9 or for Intensity 8 with the horizontal seismic-reduced factor 0.25. And the calculation for vertical seismic action should be carried out for Intensity 8 with the seismic-reduced factor not greater than 0.5.

When calculating the vertical seismic action characteristic values of the structure above seismic-isolation story, each story may be deemed as a mass, and the vertical seismic action characteristic values may be distributed over the height of structure in accordance with the Equation (5.3.1-2) of this code.

12.2.6 The shear force of the isolator unit shall be determined according to the horizontal stiffness of all isolator units from which is the horizontal shear force of the seismic-isolation story under the rare earthquake. When calculation considered torsion, the torsion stiffness of the isolator unit shall be taken into consideration.

The horizontal displacement of the isolator unit of seismic-isolation story under the rare earthquakes shall comply with the following requirements:

$$u_i \leqslant [u_i] \tag{12.2.6-1}$$

$$u_i = \beta_i u_c \tag{12.2.6-2}$$

where: u_i——horizontal displacement of i-th isolator unit when taking into account of the torsion under the rare earthquakes.

$[u_i]$——limit value of horizontal displacement of i-th isolator unit; in the case of rubber isolator unit, its displacement shall not exceed 0.55 times of the effective

diameter of the unit or 3 times of the total thickness of all rubber layers, whichever is smaller .

u_c——horizontal displacement in the center of the seismic-isolation story under rare earthquakes, or when torsion is not taken into consideration.

β_i——torsion factor of i-th seismic-isolator unit, that shall be taken as the ratio of both calculated displacements which torsion is taken into account and not taken into account. When the mass center of the structure above the seismic-isolation story and the rigidity center of the seismic-isolation story are not eccentric in both major axial directions, the torsion factor of the side unit shall not be less than 1.15.

12.2.7 The seismic-isolation details of the structure above the seismic-isolation story shall comply with the following provisions:

1 For structural systems above the seismic-isolation story, the following details, which will not disturb significant displacement of the seismic-isolation story under rare earthquake, shall be taken:

1) The seismic joints shall be installed along the perimeter of the structure above the seismic-isolation story, the width of the joint should not be less than 1.2 times of the maximum horizontal displacement of each seismic-isolator unit under the rare earthquake;

2) Between the structure (including connected members) above the seismic-isolation story and the ground surface (including foundation and structures below the isolation interface), the clear horizontal separations shall be installed; when it is difficult to install horizontal separations, a reliable horizontal slipped cushion shall be installed;

3) In location such as the corridors, the staircase, and the elevators etc., shall be no any obstruction for movement.

2 For the buildings assigned to Category C, the seismic-measures of structures above the seismic-isolation story shall not reduce the drequirements in non-isolated provision of this code when the horizontal seismic-reduced factor is 0.75. And the requirements in non-isolated provisions may be properly loosened when this factor is not greater than 0.5, but corresponding vertical seismic detail requirements shall not be loosened.

For masonry structures, seismic-measures may be taken according to Appendix L of this code.

For reinforced concrete structures, the axial force ratio control of the columns and the walls shall comply with relevant provisions for non-isolated. And other requirements of calculation and seismic details of structures shall comply with provisions for corresponding seismic-measure Grade, which may be determined according to Table 12.2.7, in Chapter 6 of this code.

Table 12.2.7 Grade of cast in-situ reinforced concrete seismic-isolated structures

Structural type		Intensity 7		Intensity 8		Intensity 9	
Frame structure	Height (m)	<20	>20	<20	>20	<20	>20
	Common frame	Grade 4	Grade3	Grade3	Grade2	Grade2	Grade1
wall structure	Height (m)	<25	>25	<25	>25	<25	>25
	Common wall	Grade 4	Grade3	Grade3	Grade2	Grade2	Grade1

12.2.8 The connection between the seismic-isolation story and the structure above the seismic-isolation story shall comply with the following requirements:

1 Beam-slab type floor shall be installed on the top of the seismic-isolation story, and shall comply with the following requirements:

1) Cast-in-place or fabricated monolithic concrete floor slab shall be adopted; the thickness of the in-situ slab shall not be less than 140mm, and the thickness of in-situ toppling of fabricated slabs shall not be less than 50mm. The longitudinal and transversal beams above the isolator units shall adopt the cast-in-place reinforced concrete beams;

2) The stiffness and the bearing capacity of the beams and slabs on the top of the seismic-isolation story shall be greater than the stiffness and bearing capacity of ordinary beams and slabs;

3) Beams and columns near the isolator units shall check the punching-shear and local-compression capacity, the hoops shall be densified and mesh reinforcements shall be installed if necessary.

2 The connected details of the isolator units and the damper shall comply with the following requirements:

1) The isolator units and the damper shall be installed in locations where the maintenance personnel can access easily;

2) The connected elements of the isolator unit with the floor of the top of the seismic-isolation story and with the foundation shall be able to transfer the maximum horizontal shear force of the isolator unit under rare earthquake;

3) The spacing of the isolator units under the wall shall not be greater than 2.0m;

4) Exposed embedded parts shall have reliable anti-rust treatment. The anchoring reinforcements of the embedded parts shall be connected to the steel plate firmly, the developed length of the anchoring reinforcement should be greater than 20 times of diameter of the anchoring bar or 250mm, whichever is greater.

12.2.9 The seismic action and check for structures under the seismic-isolation story shall adopt the vertical force, horizontal force and moment of force at the bottom of the isolator units under rare earthquake.

The seismic check for the foundation and the foundation treatment of the seismic-iso-

lated buildings shall be carried out according to the local Intensity. The anti-liquefaction measures for buildings assigned to Category A and B shall be determined according to one grade higher than the liquefaction grade, until all liquefaction settlements are eliminated.

12.3 Essentials in design of seismic energy-dissipated buildings

12.3.1 For the seismic energy-dissipated buildings, the proper energy-dissipating components shall be installed according to the expected displacement of the structure under rare earthquakes. The energy-dissipating components may consist of the energy-dissipating device and the diagonal bracings, the walls, the beams or joints. The energy-dissipating device may adopt the speed-related type, the displacement-related type or other types.

Notes: 1. The speed-related type energy-dissipating device refers to the viscous energy dissipating device and viscoelastic energy-dissipating device etc.;

2. The displacement-related type energy-dissipating device refers to metal yield energy dissipating device and friction energy dissipating device etc.

12.3.2 The energy-dissipating components may be installed along the two main axis of the structure respectively if necessary. The energy-dissipating components shall be installed at locations where the story drift is significant, its number and distribution shall be determined according to comprehensive analysis, and shall be favorable for increasing the energy-dissipating capacity and for composing a even and reasonable lateral-force-resistant system of the whole structure.

12.3.3 The calculating analysis of the energy-dissipated building structures shall comply with the following provisions:

1 In generally, static non-linear analysis method or non-linear time-history analysis method should be adopted.

2 When the main structure is basically in elastic working stage, the linear analysis method may be adopted for simplified estimation. The linear analysis method depending on the deformation characteristics and height of the structure, may adopt the base-shear method, mode analysis response spectrum method and time-history analysis method according to the provisions in Section 5.1 of this code. The seismic influence coefficient can be adopted according to the provisions in Clause 5.1.5 of this code, which depends on the total damping ratio of the energy-dissipated building structures.

3 The total stiffness of the energy-dissipated structure shall be the total sum of the structural stiffness and the effective stiffness of the energy-dissipating components.

4 The total damping ratio of the energy-dissipated structure shall be the total sum of the structural damping ratio and the effective damping ratio added to the structure by the energy dissipating components.

5 For the frame structure, the limit value of the elasto-plastic story-drift-angel of the energy-dissipated structure should adopt 1/80.

12.3.4 The effective damping ratio added to the structure by energy-dissipating components may be determined according to the following methods:

1 The effective damping ratio added by the energy dissipating components, may be estimated with the following equation:

$$\zeta_a = W_c/(4\pi W_s) \tag{12.3.4-1}$$

where: ζ_a——effective damping ratio added by the energy dissipating components.

W_c——energy dissipated by all energy-dissipating components in one cycle of expected displacement of the structure.

W_s——total strain energy of energy-dissipated structures under expected displacement.

2 When the torsion effect is not taken into consideration, the total strain energy of the energy-dissipated structure under horizontal seismic action can be estimated with the following equation:

$$W_s = (1/2)\Sigma F_i u_i \tag{12.3.4-2}$$

where: F_i——horizontal seismic action characteristic value of i-th mass.

u_i——displacement of i-th mass corresponding to horizontal seismic action characteristic value.

3 The energy dissipated by the speed-linear-relevant type energy-dissipating device under horizontal seismic action can be estimated with the following equation:

$$W_c = (2\pi^2/T_1)\Sigma C_j \cos^2\theta_j \Delta u_j^2 \tag{12.3.4-3}$$

where: T_1——foundamental vibration period of the energy-dissipated structure.

C_j——linear damping factor determined by testing for j-th energy-dissipating device.

θ——angel between the energy-dissipating direction and the horizon for j-th energy-dissipating device.

Δu_j——relative horizontal displacement at the two ends of j-th energy-dissipating device.

When the damping factor and effective stiffness of the energy-dissipating device relate to the structure vibration period, the value corresponding to the foundamental vibration period of the energy-dissipated structure may be selected.

4 The energy dissipated by displacement-related type, speed non-linear-related type and other types of energy-dissipating devices under hovizontal seismic action can be estimated with the following equation:

$$W_c = \Sigma A_j \tag{12.3.4-4}$$

where: A_j——the area of restoring-force characteristics of the j-th energy-dissipating device at the relative horizontal displacement Δu_j.

The effective stiffness of energy-dissipating devices can be the secant stiffness of restor-

ing-force characteristics of the j-th energy-dissipating device at the relative horizontal displacement Δu_j.

5 When the effective damping ratio added to the structure by the energy-dissipating components exceeds 20%, it should be counted according to 20%.

12.3.5 For non-linear time-history analysis method, it should adopt the restoring-force characteristic model of the energy-dissipating components for the calculation. For static non-linear analysis method, the effective damping ratio and stiffness added to the structure by the energy-dissipating device may be determined according to the method provided in Clause 12.3.4 of this chapter.

12.3.6 The design parameters of the effective stiffness, damping ratio and restoring-force characteristic model of the energy-dissipating components shall be determined by testing, and shall comply with the following provisions:

1 For the speed-related type energy-dissipating device, the design allowable displacement, ultimate displacement and the restoring force characteristic model, which is corresponding to design allowable displacement, different temperature conditions and the loading frequency 0.1～4Hz, that shall be determined by testing. For energy-dissipating components consisted of the speed-related energy-dissipating device with supporting members such as the diagonal, wall or beam, the stiffness of the energy-dissipating components in the direction of energy-dissipating device may be calculated with the following equation:

$$K_b = (6\pi/T_1)C_v \qquad (12.3.6\text{-}1)$$

where: K_b——stiffness of the supporting component in the direction of the energy dissipating device.

C_v——linear damping factor of the energy-dissipating device, which corresponds to the fundamental vibration period of the structure and is determined by testing.

T_1——fundamental vibration period of the energy-dissipated structure.

2 For the displacement-related energy-dissipating device, the design allowance displacement, ultimate displacement and restoring force characteristic model parameters shall be determined by repeated static loading. For energy-dissipating components consisted of the displacement-related energy-dissipating device with the supporting members such as the diagonal, wall or beam, the restoring force characteristic model parameters of this component shall comply with the following requirements:

$$\Delta u_{py}/\Delta u_{sy} \leqslant 2/3 \qquad (12.3.6\text{-}2)$$

$$(K_p/K_s)(\Delta u_{py}/\Delta u_{sy}) \geqslant 0.8 \qquad (12.3.6\text{-}3)$$

where: K_p——primary stiffness of the energy-dissipating component in horizontal direction.

Δu_{py}——yield displacement of energy-dissipating component.

K_s——lateral stiffness of the story of energy-dissipated structure.

Δu_{sy}——yield story drift of energy-dissipated structure.

3 Under the maximum allowable displacement value and repeated 60 cycles, the major performance decrease of the energy-dissipating device shall not exceed 10%, and shall not have obvious low-cycle fatigue.

12.3.7 The connection of the energy-dissipating device with the diagonal, wall, beam and joints shall comply with the requirements for the connection of steel members or steel and reinforced concrete members, and shall be resistant to the maximum action brought to the connected point by the energy-dissipating device.

12.3.8 For the structural members connected with the energy-dissipating components, the additional internal force due to the energy-dissipating components shall be taken into consideration, and shall be transferred it to the foundation.

12.3.9 The energy-dissipating device and its connected parts shall have good durability and shall be convenient for maintenance.

Chapter 13 Nonstructural components

13.1 General

13.1.1 This chapter is mainly applicable to the connection of nonstructural components with the building structure. The nonstructural components include architectural members as well as mechanical and electrical equipment, which are permanently attached to structures.

Notes: 1. Architectural members refer to the fixing members and parts except the load-bearing skeleton system, mainly including non-bearing wall, members subordinating to the floor and roof, decorative members and parts, and large-sized storage racks fixed on the floor etc.

2. The attached mechanical and electrical equipment refer to the mechanical and electrical members, parts and systems serving the functions of modern buildings, mainly including elevators, lighting, emergency power, communication facility, pipeline, heating and air-conditioning system, smoke and fire monitoring and protection system, and community antenna.

13.1.2 The different seismic-measures for nonstructural components shall be taken according to the Category of the building, the consequence and the range of the influence to the whole building structure due to nonstructural components earthquake damage. When there are detailed requirements in specifications, seismic analysis shall be carried out by adopting different functional factors and type factors etc.

13.1.3 When two nonstructural components, which the calculation and seismic-measures requirements are different, are connected with each other, the seismic design shall be carried out according to the higher requirements.

When the connections of nonstructural components are damaged, this damage shall not cause the failure of another adjacent nonstructural components with higher requirements.

13.2 Basic requirements for calculation

13.2.1 When seismic calculation is made for building structures, the influence due to nonstructural components shall be taken into consideration according to the following provisions:

1 When calculating the seismic action, the gravity of architectural members and the mechanical and electrical equipment attached to the structural members shall be taken into consideration.

2 In the calculation, for architectural members using flexible connection, the stiffness shall not be taken into consideration; for rigid nonstructural components filled in the lateral-force-resisting member plane, the stiffness influence can be taken into consideration by adopting simplified methods such as the period adjustment. In generally, the seismic bearing capacity shall not be taken into consideration; when specific measures have been taken,

the seismic bearing capacity may be taken into consideration according to relevant specifications.

3 For subordinating mechanical and electrical equipment of the building that needs to adopt the floor spectrum for calculation, the interaction of the equipment and the structure should be taken into consideration by adopting appropriate simplified computation model.

4 The structural members supporting nonstructural components shall take the seismic action of nonstructural components as additional action, and shall satisfy the anchoring requirements of connected part.

13.2.2 The calculating method of seismic action for the nonstructural components shall comply with the following requirements:

1 The seismic force of all members and components shall be applied at the center of gravity, and the horizontal seismic force shall be applied non-concurrently in any horizontal direction.

2 In generally, the seismic action produced by nonstructural components themselves can adopt the equivalent lateral-force method for the calculation. For nonstructural components supported in the different floors or the two sides of the seismic joints, besides considering the seismic action produced by the self gravity, the effect of the relative displacement between all supporting points shall be taken into consideration at the same time.

3 When the vibration period of the equipment is greater than 0.1s and its gravity exceeds 1% of the gravity of the floor it locates, or when the gravity of the equipment exceed 10% of the floor gravity, the floor-response-spectrum method should be adopted. Of them, for the equipment non-elastically connected with the floor of the building, the equipment and the floor can be deemed as a mass in calculation for whole structure, and such seismic action of equipment may be obtained.

13.2.3 When equivalent lateral-force method is adopted, the characteristic value of the horizontal seismic action should be calculated according to the following equation:

$$F = \gamma\eta\zeta_1\zeta_2\alpha_{max}G \tag{13.2.3}$$

where: F——the horizontal seismic action characteristic value applied at the center of gravity of the nonstructural components in the most unfavorable direction;

γ——functional factor of the nonstructural components, which is determined depending on the fortification Category and the use requirements according to relevant specification;

η——type factor of nonstructural components, which is determined depending on factors such as the material performance according to relevant specification;

ζ_1——factor of state, in cases such as precast members, cantilever members, any equipment braced to structural member lower than its center of mass, and flexible systems, the factor should be taken as 2.0; and in other cases may be taken as 1.0;

ζ_2——factor of location, for the top of structures, the factor should be taken as 2.0; for the bottom, the factor should be taken as 1.0, and is in linear distribution along the height of structures; for the structures that need adopt the time-history analyzing method to carry out calculations according to the provisions in Section 5.1 of this code, such factor shall be adjusted depending on this results of calculation;

α_{max}——maximum value of seismic influence coefficient, which may be adopted in accordance with the provision for frequent earthquakes in Clause 5.1.4 of this code;

G——gravity of nonstructural components, it shall include the gravity of relevant personnel, medium in the container and tube during operations, as well as the gravity of the storage cabinet.

13.2.4 The internal force, which produced by the relative horizontal displacement between the supporting points of the nonstructural component, shall be calculated. That may be determined by using the product of the stiffness of the component in the displacement direction and the nominal relative horizontal displacement between the supporting points.

The stiffness of the nonstructural component in the displacement direction shall be determined based on the practical connected state, which may adopt simplified mechanic models such as rigid connection, hinge connection, elastic connection or sliding connection.

The relative horizontal displacement between the adjacent floors may adopt limit values of story-drift of structure as provided in Section 5.5 of this code, and the relative horizontal displacement between two sides of seismic joint should be determined according to use requirements.

13.2.5 When the floor-response-spectrum method is to be adopted, the horizontal seismic action characteristic value of nonstructural components may be calculated with the following equation:

$$F = \gamma \eta \beta_s G \tag{13.2.5}$$

where β_s——the floor response spectrum value of nonstructural components, which is determined by the fortification Intensity and the site-condition of the structure, the period ratio, gravity ratio and damping between nonstructural and structural system, as well as the supporting location, number and connection state of nonstructural components in the structure. In generally, the nonstructural components are simplified as single-mass systems supported to the structure, but the nonstructural components, which have relative displacement between supporting points, shall adopt multi-mass system, and shall be calculated according to special methods.

13.2.6 The basic combination of the seismic effect and other loading effect of nonstruc-

tural components shall be determined according to the provision in Section 5.4 of this code. For the curtain walls, the wind loading effect shall also be combined; for the containers, the temperature effect of equipment operation and the working pressure shall also be taken into consideration.

For seismic checking of nonstructural components, the frictional resistance shall not be taken into consideration; for the seismic adjustment factor of the bearing capacity, the connected part shall be taken as 1.0, and others can be taken according to the provisions of relevant specifications.

13.3 Basic seismic-measures for architectural nonstructural members

13.3.1 In building structures, locations installing the embedded and anchoring parts which are conneced with the architectural members such as curtain, enclosure, separation wall, parapet, awning, signs, billboards, ceilings and large-sized storage cabinets, the strengthened measures shall be taken to resist the seismic action transferred by nonstructural components.

13.3.2 The material, type and arrangement of non-bearing walls shall be determined after comprehensive analysis dependent on the Intensify, the height of the building, the configuration of the building, the story drift of the structure, and the lateral-force-resisting performance of the wall itself.

1 The selection of wall materials shall comply with the following requirements:

1) For reinforced concrete and steel structure, the non-bearing wall shall first adopt lightweight wall materials;

2) For single-story reinforced concrete column factory buildings, the enclosures should adopt lightweight wall panels or large-sized reinforced concrete wall panels; when the spacing of the side-columns is 12m, lightweight wall panels or large-sized reinforced concrete wall panels shall be adopted. The high span enclosing wall of factory buildings with different heights and the suspending wall at the intersection of longitudinal and transversal factory units should adopt lightweight wall panels, but lightweight wall panels shall be adopted for Intensity 8 and 9;

3) For steel structure factory buildings, the enclosure walls should adopt lightweight wall panels or reinforced concrete wall panels and shall not adopt masonry walls for Intensity 7 and 8, but the lightweight wall panels should be adopted for Intensity 9.

2 The arrangement of rigid non-bearing walls shall try to avoid abrupt changes in the stiffness and strength of the structure. For single-story reinforced concrete column factory building, the rigid enclosure wall should be arranged evenly along the longitudinal direction.

3 The walls shall be tied reliably to the seismic-force-resisting system and shall be able

to suit the story drift of any direction of seismic-force-resisting system. For Intensity 8 and 9, the walls shall have the capability of deformation to satisfy the requirements of story drift; and the walls shall also have the capability to satisfy the requirements of vertical deflection due to joint-zone rotation of adjacent cantilever structural members.

4 The connected part of the exterior wall panels shall have sufficient ductility and proper rotating capability so that the requirements for story drift of the seismic-force-resisting system under the fortification earthquake can be satisfied.

13.3.3 For masonry walls, measures shall be taken to reduce unfavorable influence on the seismic-force-resisting system, and the steel tie-bars, the horizontal tie-bands, the ring-beams, and the tie-columns shall be installed to ensure reliable tying with the seismic-force-resisting system.

1 In multi-story masonry structures, the post-laying non-bearing partition walls shall have 2ϕ6 tie-bars in each 500mm along the height of the wall for tying with the load-bearing wall or the column, and shall be extended into each wall with length not less than 500mm. For Intensity 8 and 9, when the length of the post-laying partition walls is greater than 5m, the top of these walls shall be tied together with the slabs or beams.

2 The masonry filling wall in the reinforced concrete structures should be separated with the column or be adopted flexible connection with columns, and shall comply with the following requirements:

1) The arrangement of filling wall in plane and vertical directions should be symmetrical and even, weak story or short columns should be avoided.

2) The strength grade of the mortar used for the masonry shall not be lower than M5, and the top of the wall shall combine with the frame beam closely.

3) The 2ϕ6 tie-bars, which are set in each 500mm along the height of the filling wall, shall be installed. And that length extended into the wall, shall be not less than 1/5 of the filling wall length or 700mm for Intensity 6 and 7, and shall be overall length of the filling wall for Intensity 8 and 9.

4) When the length of the filling wall is greater than 5m, the top of the wall shall be tied with the beam; when the length of the wall exceeds two times of the story height, tie-columns shall be installed. When the height of the filling wall is more than 4m, the horizontal tie-band shall be installed at the half height of the wall and be connected to column on both ends of the wall.

3 Masonry partition wall and enclosure wall of single-story reinforced concrete column factory buildings shall comply with the following requirements:

1) The masonry partition wall shall be separated with the column or flexibly connected with the column, and measures shall be taken to ensure the stability of the wall, cast-in-situ reinforced concrete coping shall be installed on the top of the partition wall.

2) The masonry enclosure wall of the factory building shall adopt laid to close outside and tied reliably with the column. When masonry fascia walls above lower roof level of factory buildings with different heights and the masonry suspending walls at the intersection of longitudinal and transversal units have been adopted, the masonry shall not built on the lower roof directly.

3) Cast-in-situ reinforced concrete ring-beam of masonry enclosure walls shall be installed in the following locations:

——The ring-beam shall be installed at the levels of top chord of the trapezoid truss end and the top of the column respectively; however, when the height of the truss end post is not greater than 900mm, the ring-beam may be installed synthetically;

——For Intensity 8 and 9, the added ring-beam shall be installed at the top level of the window in each 4 m approximately, according to the principle that the vertical spacing of ring-beam in upper portion shall be denser than in lower portion. Meanwhile, for the fascia walls above lower roof level of factory buildings with different heights and the suspending walls at the intersection of longitudinal and transversal units, the vertical spacing of the ring-beam shall not be greater than 3m;

——A ring-beam shall be installed on the gable wall along the roof level, and shall be connected with the ring-beam at top chord level of roof truss end.

4) The details of the ring-beam shall comply with the following requirements:

——Ring-beam shall be enclosed. The beam width should be the same as the thickness of the wall, the beam depth shall not be less than 180mm, and the longitudinal reinforcement shall not be less than 4ϕ12 for Intensity 6 to 8 or 4ϕ14 for Intensity 9;

——For the column top ring-beam at the corner of the factory unit, the longitudinal reinforcements within the scope of the end bay shall be not less than 4ϕ14 for Intensity 6 to 8 or 4ϕ16 for Intensity 9. And the diameter of stirrups of such ring beam within 1m on the two sides of the corner shall not be less than ϕ8, the spacing shall not be greater than 100mm. Three horizontal diagonal bars shall be added to the corner of the ring beam, and the diameter of the bars should be the same as that of the longitudinal bars;

——Ring-beam shall be firmly connected with the column or the roof truss, the ring-beam of the gable shall be tied with the roof slabs. The anchoring bar connecting the top ring-beam with the column or roof truss shall not be less than 4ϕ12, and the developed length shall not be less than 35 times of the bar diameter. At the seismic joints, the tie between the ring-beam with the column or the roof truss should be intensified.

5) For Site class Ⅲ and Ⅳ of Intensity 8 or for Intensity 9, the precast foundation beam of the brick enclosure wall shall adopt cast-in-situ joints. When stripped footing of enclosure wall is established, continuous cast-in-situ reinforced concrete ring-beams shall be installed at the level of the top of column footing, and the reinforcement of ring-beam shall not be less than 4ϕ12.

6) The wall-beam should adopt cast-in-situ beam; if precast beams are adopted, then the bottom of the beam shall be firmly tied with the top of the brick wall, and shall also be tied with the columns. The two adjacent wall-beams at the corner of the factory building shall be firmly connected with one another.

4 The masonry enclosure wall of single-story steel structure factory building shall not adopt engaged built; for Intensity 8, the measures shall also be taken so that the wall does not hinder the horizontal displacement of the column rows along the longitudinal direction of the factory building.

5 The masonry parapet shall be anchored with the seismic-force-resisting structural system at the entrances and exits; the clear width of the seismic joints shall be sufficient, and the free ends of parapet at the two sides of the joints shall be intensified.

13.3.4 The connected parts of various ceilings with the floor shall be able to undertake the self-weight of the ceiling, the suspending heavy objects, and the relevant mechanic and electrical equipment, as well as the additional seismic action. The bearing capacity of the anchoring shall be greater than the bearing capacity of the connected parts.

13.3.5 Cantilever awnings or awnings with one end supported by the column shall be reliably connected with the seismic-force-resisting system.

13.3.6 The seismic-details of the glass curtain, precast wall panels, the cantilever members subordinated to the floor, and large-sized storage rack shall comply with the provisions of relevant specifications.

13.4 Basic seismic-measures for the supports of mechanical and electrical devices

13.4.1 The attached devices are those such as the elevator, lighting, emergency-power system, smoke-fire monitoring and protection system, heating and air-conditioning system, communication system, and community antenna. The seismic-details of connections between the attached devices and the building structure shall be determined according to the provisions of relevant specifications. In which the comprehensive analysis shall be made to consider the Fortification Intensity, the use function and the height of the building, the type and the deformation characteristics of structures, the supporting locations and the operational requirements for the equipment.

The seismic restraints are not required for the mechanical and electrical components for any of the following conditions:

——The equipment that has gravity not more than 1.8kN;

——The gas pipe that has less than 25mm inside diameter, and electrical conduit that has less than 60mm inside diameter;

——The rectangular air-pipes that has a cross-sectional area less than 0.38 m^2 and air-pipes that has less than 0.7m diameter;

——The conduits suspended by hangers less than 600mm in length form the top of the conduit to the supporting structure.

13.4.2 The mechanical and electrical equipment shall not be installed in locations that may cause its operation failure or other secondary harms. For equipment with isolation devices, it shall be paid attention to the strong vibration to influence on connected parts, and resonance of the equipment with the building structure shall also be avoided.

The braced frame of mechanical and electrical equipment shall have sufficient stiffness and strength; that must have reliable connection and anchoring with the building structure, so the equipment can be able to restore failure-free operation after the fortification earthquake occurrence.

13.4.3 The openings for pipelines, cables, air-pipes, and equipment shall try to reduce the weakening caused to the load-bearing structural members; strengthened measures shall be taken for perimeter of the openings.

The connections of pipelines and the equipment with the building structures shall be able to allow a certain relative deflection between each other.

13.4.4 The supports or the connected parts of the mechanical and electrical equipment shall be able to transfer all the seismic action of the equipment to the building structures. In building structures, locations installing the embedded and anchoring parts where are connected the mechanical and electrical equipment, the strengthened measures shall be taken to resist the seismic action transferred by mechanical and electrical components.

13.4.5 The water tank, which located at upper-position of the building, shall be connected with the structural members reliably. For high-rise buildings which need to adopt time-history analysis according to the provision in Clause 5.1.2 of this code, the additional seismic effect caused by interaction of the water and the structures should also be taken into consideration for Intensity 8 and 9.

13.4.6 The mechanical and electrical equipment, which need operating continuously within the fortification earthquake, should be installed at locations of the building structure where the seismic response is smaller, corresponding strengthened measures shall be taken for the structural members in relevant portion.

Appendix A
The fortification intensity, design basic accelerations of ground motion and design earthquake groups of main cities and towns in China

This appendix provide the fortification intensity, the design basic accelerations of ground motion and the design earthquake groups used in the seismic design of building construction situated at the central region of cities and towns that assigned to county level and above in the seismic fortification zones of China.

Note: In this appendix, "the design earthquake group 1, 2 and 3" is referred as "group 1, group 2 and group 3".

A.0.1 The capital city and municipalities directly under the central government

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Beijing(11 districts except Changping and Mentougou), Pinggu, Daxing, Yanqing, Ninghe and Hangu.

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Miyun, Huairou, Changping, Mentougou, Tianjin(12 districts except Hangu and Dagang), jixian, Baodi, and Jinghai.

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Dagang, Shanghai(15 districts except Jinshan), Nanhui, and Fengxian

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Chongming, Jinshan, Chongqing(14 districts), Wushan, Fengjie, Yunyang, Zhongxian, Fengdu, Changshou, Bishan, Hechuan, Tongliang, Dazu, Rongchang, Yongchuan, Jiangjin, Qijiang, Nanchuan, Qianjiang, Shizhu, and Wuxi*

Notes:

1. The design earthquake group of all the fortification cities and towns of the county level or above in the capital city and the municipalities directly under the central government is group 1;

2. The superscript * means that the center of the city situates on the dividing line of the such fortification zone and lower fortification zone, similarly hereinafter.

A.0.2 Hebei Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of

ground motion equal 0.20g:

Group 1: Langfang (2 districts), Tangshan (5 districts), Sanhe, Dachang, Xianghe, Fengnan, Fengrun, Huailai, and Zhuolu.

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Handan (4 districts), Handan county, Wenan, Renqiu, Hejian, Dacheng, Zhuozhou, Gaobeidian, Laishui, Gu'an, Yongqing, Yutian, Qianan, Lulong, Luanxian, Luannan, Tanghai, Leting, Xuanhua, Weixian, Yangyuan, Chengan, Cixian, Linzhang, Daming, Ningjin, and Xiahuayuan.

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Shijiazhuang (6 districts), Baoding (3 districts), Zhangjiakou (Qiaodong, Qiaoxi districts), Cangzhou(2 districts), Hengshui, Xingtai(2 districts), Bazhou, Xiongxian, Yixian, Cangxian, Zhangbei, Wanquan, Huai'an, Xinglong, Qianxi, Funing, Changli, Qingxian, Xianxian, Guangzong, Pingxiang, Jize, Longyao, Xinhe, Quzhou, Feixiang, Guantao, Guangping, Gaoyi, Neiqiu, Xingtai county, Zhaoxian, Wuan, Shexian, Chicheng, Laiyuan, Dingxing, Rongcheng, Xushui, Anxin, Gaoyang, Boye, Lixian, Suning, Shenze, Anping, Raoyang, Weixian, Gaocheng, Luancheng, Jinzhou, Shenzhou, Wuqiang, Xinji, Yizhou, Renxian, Baixiang, Julu, Nanhe, Shahe, Lincheng, Botou, Yongnian, Chongli, and Nangong*.

Group 2: Qinhuangdao(Haigang, Beidaihe), Qingyuan, Zunhua, and Anguo

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Zhengding, Weichang, Shangyi, Lingshou, Wuji, Pingshan, Luquan, Jingjing, Yuanshi, Nanpi, Wuqiao, Jingxian, and Dongguang.

Group 2: Chengde (2 districts except Yingshouyingzi), Longhua, Chengde county, Kuancheng, Qinglong, Fuping, Mancheng, Shunping, Tangxian, Wangdu, Quyang, Dingzhou, Xingtang, Zanhuang, Huangye, Haixing, Mengcun, Yanshan, Fucheng, Gucheng, Qinghe, Shanhaiguan, Guyuan, Xinle, Wuyi, Zaoqiang, and Weixian.

Group 3: Fengning, Luanping, Yingshouyingzi, Pingquan, Linxi, and Qiuxian.

A.0.3 Shanxi Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Taiyuan (6 districts), Linfen, Xizhou, Qixian, Pingyao, Guxian, Daixian, Yuanping, Dingxiang, Yangqu, Taigu, Jiexiu, Lingshi, Fenxi, Huozhou, Hongtong, Xiangfen, Jinzhong, Fushan, Yongji and Qingxu.

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Datong(4 districts), Suozhou(Suocheng District), Datongxian, Huairen, Hunyuan, Guangling, Yingxian, Shanyin, Lingqiu, Fanshi, Wutai, Gujiao, Jiaocheng, Wenshui, Fenyang, Quwo, Xiaoyi, Houma, Xinjiang, Jishan, Jiangxian, Hejin, Wenxi, Yicheng, Wanrong, Linqi, Xiaxian, Yuncheng, Ruicheng, Pinglu, Qinquan*, and Ningwu*.

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Changzhi(2 districts), Yangquan(3 districts), Changzhi county, Yanggao, Tianzhen, Zuoyun, Youyu, Shenchi, Shouyang, Xiyang, Anze, Xiangning, Hengqu, Qinshui, Pingding, Heshun, Licheng, Lucheng and Huguan.

Group 2: Pingshun, Yushe, Wuxiang, Loufan, Jiaokou, Xixian, Puxian, Jixian, Jingle, Mengxian, Qinxian, Lingchuan, and Pinggu.

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 2: Pianguan, Hequ, Baode, Xingxian, Linxian, Fangshan, and Liulin.

Group 3: Jincheng, Lishi, Zuoquan, Xiangheng, Tunliu, Changzi, Gaoping, Yangcheng, Zezhou, Wuzhai, Kelan, Lanxian, Zhongyang, Shilou, Yonghe, and Daning.

A.0.4 Inner Mongolia Autonomous Region

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.30g:

Group 1: Tumoteyouqi, Dalateqi*

2 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Baotou(5 districts except Baiyun Mining District), Huhehaote(4 districts), Tumotezuoqi, Wuhai(3 districts), Hangjinhouqi, Dengkou, Ningcheng, Tuoketuo*.

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Kalaqinqi, Wuyuan, Wulateqianqi, Linhe, Guyang, Wuchuan, Liangcheng, Helingeer, Chifeng(Hongshan* and Yuanbaoshan District)

Group 2: Alashanzuoqi

4 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Jining, Qingshuihe, Kailu, Aohanqi, Wutelahouqi, Zhuozi, Chayouqianqi, Fengzhen, Zhalantun, Wutelazhongqi, Chifeng(Songshan District), and Tongliao*.

Group 3: Dongsheng, Zhungeerqi

5 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Manzhouli, Xinbaerhuyouqi, Molidawaqi, Arongqi, Zhafenteqi, Wongniuteqi, Xinghe, Shangdu, Chayouhouqi, Kezuozhongqi, Kezuohouqi, Naimanqi, Kulunqi, Wushenqi,

and Suniteyouqi.

Group 2: Da′ erhanmaoming′ anlianheqi, Alashanyouqi, Etuokeqi, Etuokeqianqi, and Baiyun.

Group 3: Yijinhuoluoqi, Hangjinqi, Siwangziqi, and Chayouzhongqi.

A.0.5 Liaoning Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Pulandian, Donggang.

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Yingkou (4 districts), Dandong (3 districts), Haicheng, Dashiqiao, Wafangdian, Gaizhou, and Jinzhou.

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Shenyang (9 districts), Anshan (4 districts), Dalian (5 districts except Jinzhou), Chaoyang(2 districts), Liaoyang(5 districts), Fushun(3 districts except shun cheng), Tieling(2 districts), Panjin(2 districts), Panshan, Chaoyang County, Liaoyang County, Xiuyan, Tieling County, Lingyuan, Beipiao, Jianping, Kaiyuan, Fushun County, Dengta, Taian, Dawa, and Liaozhng.

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Benxi(4 districts), Fuxin (5 districts), Jinzhou (3 districts), Huludao (3 districts), Changtu, Xifeng, Faku, Zhangwu, Tiefa, Fuxin County, Kangping, Xinmin, Heishan, Beining, Yixian, Kalaqin, Linghai, Xingcheng, Suizhong, Jianchang, Kuandian, Fengcheng, Zhuanghe, Changhai, and Shuncheng

Note: the design seismic group of cities of the county level and above in all over the province belong to Group 1 except Xingcheng, Suizhong, Jianchang, and Nanpiao, which belong to Group 2.

A.0.6 Jilin Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Qianguoerluosi, Songyuan

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Da'an*

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Changchun(6 districts), Jilin(3 districts except Fengman), Baicheng, Qian'an, Shulan, Jiutai, and Yongji*

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Siping(2 districts), Liaoyuan(2 districts), Zhenlai, Zhaonan, Yanji, Wangqing, Tumen, Huichun, Longjing, Helong, Antu, Jiaohe, Huadian, Lishu, Panshi, Dongfeng, Huinan, Meihekou, Dongliao, Yushu, Jingyu, Fusong, Changling, Tongyu*, Dehui, Nongan, Yitong, Gongzhuling, Fuyu, and Fengman.

Note: the design seismic group of all cities of the county level and above in all over the provinces belong to Group 1.

A.0.7 Heilongjiang Province

1 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Suihua, Luobei and Tailai.

2 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Harbin(7 districts), Qiqihaer(7 districts), Daqing(5 districts), Hegang (6 districts), Mudanjiang(4 districts), Jixi(6 districts), Jiamusi(5 districts), Qitaihe(3 districts), Yichun (Yichun District, Wumahe District), Jidong, Wangkui, Muleng, Suifenhe, Dongning, Ningan, Wudalianchi, Jiayin, Tangyuan, Huanan, Huachuan, Yilan, Boli, Tonghe, Fangzheng, Mulan, Bayan, Yanshou, Shangzhi, Binxian, Anda, Mingshui, Suiling, Qingan, Lanxi, Zhaodong, Zhaozhou, Zhaoyuan, Hulan, Echeng, Shuangcheng, Wuchang, Nahe, Beian, Gannan, Fuyu, Longjiang, Heihe, Qinggang*, and Hailin*.

Note: the design seismic group of all cities of the county level and above in all over the provinces belong to Group 1.

A.0.8 Jiangsu Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.30g:

Group 1: Suqian, Suyu*

2 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Xinyi, Peizhou, Suining

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Yangzhou(3 districts), Zhenjiang (2 districts), Donghai, Shuyang, Sihong, Jiangdu, and Dafeng.

4 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Nanjing(11 districts), Huai'an(3 districts except Chuzhou), Xuzhou(5 districts), Tongshan, Peixian, Changzhou(4 districts), Taizhou(2 districts), Ganyu, Siyang,

Yutai, Sheyang, Jiangpu, Wujing, Yancheng, Yandu, Dongtai, Hai'an, Jiangyan, Rugao, Rudong, Yangzhong, Yizheng, Xinghua, Gaoyou, Liuhe, Jurong, Danyang, Jintan, Dantu, Liyang, Lishui, Kunshan, and Taichang.

Group 3: Lianyungang(4 districts), Guanyun

5 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Nantong(2 districts), Wuxi(6 districts), Suzhou(6 districts), Tongzhou, Yixing, Jiangyin, Hongze, Jianhu, Changshu, Wujiang, Jingjiang, Taixing, Zhangjiagang, Haimen, Qidong, Gaochun and Fengxian.

Group 2: Xiangshui, Binhai, Funing, Baoying, and Jinhu

Group 3: Guannan. Lianshui, and Chuzhou

A.0.9 Zhejiang Province

1 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Daishan, Chengsi, Zhoushan(2 districts), Zhen hai, and Beicang

2 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Hangzhou(6 districts), Ningbo (3 districts), Huzhou, Jiaxing (2 districts), Wenzhou(3 districts), Shaoxing, Shaoxing County, Changxing, Anji, Linan, Fenghua, Jinxian, Xiangshan, Deqing, Jiashan, Pinghu, Haiyan, Tongxiang, Yuhang, Haining, Xiaoshan, Shangyu, Cixi, Yuyao, Ruian, Fuyang, Pingyang, Cangnan, Leqing, Yongjia, Taishun, Jingning, Yunhe, Qingyuan, and Dongtou.

Note: the design seismic group of all cities of the county level and above in all over the provinces belong to Group 1.

A.0.10 Anhui Province

1 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Wuhe, Sixian

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Hefei(4 districts), Bangbu (4 districts), Fuyang(3 districts), Huainan (5 districts), Zongyang, Huaiyuan, Changfeng, Liu'an(2 districts), Lingbi, Guzhen, Fengyang, Mingguang, Dingyuan, Feidong, Feixi, Shucheng, Lujiang, Tongcheng, Huoshan, Woyang, Anqing(3 districts)*, and Tongling County*

3 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Tongling(3 districts), Wuhu(4 districts), Chaohu, Maanshan(4 districts), Chuzhou(2 districts), Wuhu County, Dangshan, Xiaoxian, Bozhou, Jieshou, Taihe, Linquan,

Funan, Lixin, Mengcheng, Fengtai, Shouxian, Yingshang, Huoqiu, Jinzhai, Tianchang, Laian, Quanjiao, Hanshan, Hexian, Dangtu, Wuwei, Fanchang, Chizhou, Yuexi, Qianshan, Taihu, Huaining, Wangjiang, Dongzhi, Susong, Nanling, Xuancheng, Langxi, Guangde, Jingxian, Qingyang, and Shitai.

Group 2: Suixi, Huaibei

Group 3: Suzhou

A.0.11 Fujian Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Jinmen*

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Xiamen (7 districts), Zhangzhou (2 districts), Jinjiang, Shishi, Longhai, Changtai, Zhangpu, Dongshan and Zhaoan.

Group 2: Quanzhou (4 districts)

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Fuzhou (4 districts except Mawei), Anxi, Nanjing, Huaan, Pinghe and Yunxiao

Group 2: Putian (2 districts), Changle, Fuqing, Putian County, Pingtan, Huian, Nan'an, and Mawei.

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Sanming (2 districts), Zhenghe, Bingnan, Xiapu, Fuding, Fuan, Tuorong, Shouning, Zhouning, Songxi, Ningde, Gutian, Luoyuan, Shaxian, Youxi, Minqing, Minhou, Nanping, Datian, Zhangping, Longyan, Yongding, Taining, Ninghua, Changding, Wuping, Jianning, Jiangle, Mingxi, Qingliu, Liancheng, Shanghang, Yongan, and Jian'ou

Group 2: Lianjiang, Yongtai, Dehua, Yongchun, Xianyou and Mazu.

A.0.12 Jiangxi Province

1 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Xunwu, Huichang

2 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Nanchang (5 districts), Jiujiang (2 districts), Nanchang County, Jinxian, Yugan, Jiujiang County, Pengze, Hukou, Xingzi, Ruichang, Dean, Duchang, Wuning, Xiushui, Jing'an, Tonggu, Yifeng, Ningdu, Shicheng, Ruijin, Anyuan, Dingnan, Longnan, Quannan, and Dayu

Note: the design seismic group of all cities of the county level and above in all over the provinces

belong to Group 1.

A.0.13 Shandong Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Tancheng, Linmu, Lvnan, Lvxian, Yishui, Anqiu, and Yanggu

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Linyi (3 districts), Weifang (4 districts), Heze, Dongming, Liaocheng, Cangshan, Yinan, Changyi, Changle, Qingzhou, Linju, Zhucheng, Wulian, Changdao, Penglai, Longkou, Shenxian, Zhencheng, Taierzhuang, Dongying (Hekou) and Shouguang*

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Yantai (4 districts), Weihai, Zaozhuang (4 districts except Taierzhuang), Zibo (4 districts except Boshan), Pingyuan, Gaotang, Renping, Dong'e, Pingyin, Liangshan, Yuncheng, Dingtao, Juye, Chengwu, Caoxian, Guangrao, Boxing, Gaoqing, Hengtai, Wendeng, Yiyuan, Mengyin, Feixian, Weishan, Yucheng, Guanxian, Laiwu (2 districts)*, Shanxian*, Xiajin*

Group 2: Dongying (Dongying district), Zhaoyuan, Xintai, Qixia, Laizhou, Rizhao, Pingdu, Gaomi, Kenli, Boshan, Bingzhou*, Pingyi*

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Dezhou, Ningyang, Lingxian, Qufu, Zhoucheng, Yutai, Rushan, Rongcheng, Yanzhou

Group 2: Jinan (5 districts), Qingdao (7 districts), Taian (2 districts), Jining (2 districts), Wucheng, Leling, Qingyun, Wuli, Yangxin, Ningjin, Zhanhua, Lijin, Huimin, Shanghe, Linyi, Jiyang, Qihe, Zhouping, Zhangqiu, Sihui, Laiyang, Haiyang, Jinxiang, Tengzhou, Laixi, and Jimo

Group 3: Jiaonan, Jiaozhou, Dongping, Wenshang, Jiaxiang, Linqing, Changqing, and Feicheng

A.0.14 Henan Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Xinxiang (4 districts), Xinxiang County, Anyang (4 districts), Anyang County, Hebi (3 districts), Yuanyang, Yanjin, Tangyin, Qixian, Weihui, Huojia, Fanxian, and Huixian

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Zhengzhou (5 districts except Shangjie), Puyang, Puyang County, Changyuan,

Fengqiu, Xiuwu, Wuzhi, Neihuang, Junxian, Huaxian, Taiqian, Nanle, Qingfeng, Lingbao, Sanmenxia, Shanxian and Linzhou*

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Luoyang(6 districts), Jiaozuo(4 districts), Kaifeng(5 districts), Nanyang(2 districts), Kaifeng County, Xuchang County, Qinyang, Boai, Mengzhou, Mengjin, Gongyi, Yanshi, Jiyuan, Xinmi, Xinzheng, Minquan, Lankao, Changge, Wenxian, Xingyang, Zhongmu, Jixian*, Xuchang*, and Shangjie.

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Shangqiu(2 districts), Xinyang(2 districts), Luohe, Pingdingshan(4 districts), Dengfeng, Yima, Yucheng, Xiayi, Tongxu, Weishi, Yeixian, Ningling, Zhecheng, Xin'an, Yiyang, Songxian, Ruyang, Yichuan, Yuzhou, Xiaxian, Baofeng, Xiangcheng, Yancheng, Yanling, Fugou, Taikang, Luyi, Dancheng, Shenqiu, Xiangcheng, Huaiyang, Zhoukou, Shangshui, Shangcai, Linying, Xihua, Xiping, Luanchuan, Neixiang, Zhenping, Tanghe, Dengzhou, Xinye, Sheqi, Pingyu, Xinxian, Zhumadian, Biyang, Ru'nan, Tongbai, Huaibin, Xixian, Zhengyang, Suiping, Guangshan, Luoshan, Huangchuan, Shangcheng, Gushi, Nanzhao, and Wuyang*

Group 2: Ruzhou, Suixian, and Yongcheng

Group 3: Lushi, Luoning, and Mianchi

A.0.15 Hubei Province

1 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Zhuxi, Zhushan, Fangxian

2 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Wuhan(13 districts), Jinzhou(2 districts), Jinmen, Xiangfan(2 districts), Xiangyang, Shiyan(2 districts), Yichang(4 districts), Yichang County, Huangshi(4 districts), Enshi, Xianning, Macheng, Tuanfeng, Luotian, Yingshan, Huanggang, E'zhou, Xishui, Xinchun, Huangmei, Wuxue, Yunxi, Yunxian, Danjiangkou, Gucheng, Laohekou, Yicheng, Nanzhang, Baokang, Shennongjia, Zhongxiang, Shayang, Yuan'an, Xingshan, Badong, Zigui, Dangyang, Jianshi, Lichuan, Gongan, Xuanen, Xianfeng, Changyang, Yidu, Zhijiang, Songzi, Jiangling, Shishou, Jianli, Honghu, Xiaogan, Yingcheng, Yunmeng, Tianmen, Xiantao, Hongan, Anlu, Qianjiang, Jiayu, Daye, Tongshan, Chibi, Chongyang, Tongcheng, Wufeng*, and Jingshan*.

Note: the design seismic group of all cities of the county level and above in all over the provinces belong to Group 1.

A.0.16 Hunan Province

1 The place-names assigned to Intensity 7 with the design basic acceleration of ground

motion equal 0.15g:

Changde (2 districts)

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Yueyang (2 districts), Yueyang County, Miluo, Xiangyin, Linli, Lixian, Jinshi, Taoyuan, Anxiang, and Hanshou

3 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Changsha(5 districts), Changsha County, Yiyang(2 districts), Zhangjiajie(2 districts), Chenzhou(2 districts), Shaoyang(3 districts), Shaoyang County, Luxi, Ruanling, Loudi, Yizhang, Zixing, Pingjiang, Ningxiang, Xinhua, Lengshijiang, Lianyuan, Shuangfeng, Xinshao, Shaodong, Longhui, Shimen, Cili, Huarong, Nanxian, Linxiang, Ruanjiang, Taojiang, Wangcheng, Xupu, Huitong, Jingzhou, Shaoshan, Jianghua, Ningyuan, Daoxian, Linwu, Xiangxiang*, Anhua*, Zhougfang*, Hongjiang*, and Yunxi.

Note: the design seismic group of all cities of the county level and above in all over the provinces belong to Group 1.

A.0.17 Guangdong Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Shantou(5 districts), Chenghai, Chaoan, Nan'ao, Xuwen, and Chaozhou*

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Jieyang, Jiedong, Chaoyang, and Raoping

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Guangzhou(9 districts except Huadu), Shenzhen(6 districts), Zhanjiang (4 districts), Shanwei, Haifeng, Puning, Huilai, Yangjiang, Yangdong, Yangxi, Maoming, Huazhou, Lianjiang, Suixi, Wuchuan, Fengshun, Nanhai, Shunde, Zhongshan, Zhuhai, Doumen, Dianbai, Leizhou, Foshan(2 districts)*, Jiangmen(2 districts)*, Xinhui*, and Lufeng*

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Shaoguan (3 districts), Zhaoqing (2 districts), Huadu, Heyuan, Jiexi, Dongyuan, Meizhou, Dongguan, Qingyuan, Qingxin, Nanxiong, Renhua, Shixing, Ruyuan, Qujiang, Yingde, Fogang, Longmen, Longchuan, Pingyuan, Dapu, Conghua, Meixian, Xingning, Wuhua, Zijin, Luhe, Zengcheng, Boluo, Huizhou, Huiyang, Huidong, Sanshui, Sihui, Yunfu, Yun'an, Gaoyao, Gaoming, Heshan, Fengkai, Yunan, Luoding, Xinyi, Xinxing, Kaiping, Enping, Taishan, Yangchun, Gaozhou, Wengyuan, Lianping, Heping, Jiaoling, and Xinfeng*

Note: the design seismic group of all cities of the county level and above in all over the provinces

belong to Group 1.

A.0.18 Guangxi Zhuangzu Autonomous Region

1 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Lingshan, Tiandong

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Yulin, Xingye, Hengxian, Beiliu, Baise, Tianyang, Pingguo, Longan, Pubei, Bobai, and Leye*

3 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Nanning(6 districts), Guilin(5 districts), Liuzhou(5 districts), Wuzhou(3 districts), Qinzhou(2 districts), Guigang(2 districts), Fangchenggang(2 districts), Beihai(2 districts), Xing'an, Lingchuan, Lingui, Yongfu, Luzhai, Tian'e, Donglan, Bama, Du'an, Dahua, Mashan, Rong'an, Xiangzhou, Wuxuan, Guiping, Pingnan, Shanglin, Binyang, Wuming, Da xin, Fusui, Huning, Dongxing, Hepu, Zhongshan, Hezhou, Tengxian, Cangwu, Rongxian, Cenxi, Luchuan, Fengshan, Lingyun, Tianlin, Longlin, Xilin, Debao, Jingxi, Napo, Tiandeng, Chongzuo, Shangsi, Longzhou, Ningming, Rongshui, Pingxiang, and Quanzhou

Note: the design seismic group of all cities of the county level and above in all over the provinces belong to Group 1.

A.0.19 Hainan Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.30g:

Haikou(3 districts), Qiongshan

2 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Wenchang, Ding'an

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Chengmai

4 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Lingao, Qionghai, Chanzhou, and Tunchang

5 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Sanya, Wanning, Qiongzhong, Changjiang, Baisha, Baoting, Lingshui, Dongfang, Ledong, and Tongshi

Note: the design seismic group of all cities of the county level and above in all over the provinces belong to Group 1.

A.0.20 Sichuan Province

1 The place-names assigned to Intensity not less than 9 with the design basic acceleration of ground motion not less than 0.40g:

Group 1: Kangding, Xichang

2 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.30g:

Group 1: Mianning*

3 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Songpan, Daofu, Luding, Ganzi, Luhuo, Shimian, Xide, Puge, Ningnan, Dechang, and Litang

Group 2: Jiuzhaigou

4 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Baoxing, Maoxian, Batang, Dege, Mabian, and Leibo.

Group 2: Yuexi, Yajiang, Jiulong, Pingwu, Muli, Yanyuan, Huidong, and Xinlong.

Group 3: Tianquan, Xingjing, Hanyuan, Zhaojue, Butuo, Danba, Lushan, and Ganluo.

5 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Chengdu(5 districts except Longquanyi, Qingbaijiang), Leshan(3 districts except Jinhekou), Zigong (4 districts), Yibin, Yibin County, Beichuan, Anxian, Mianzhu, Wenchuan, Dujiangyan, Shuangliu, Xinjin, Qingshen, E'bian, Muchuan, Bingshan, Lixian, Derong, and Xindu*

Group 2: Panzhihua(3 districts), Jiangyou, Shifang, Pangzhou, Bixian, Wenjiang, Dayi, Chongzhou, Gonglai, Pujiang, Pengshan, Danling, Meishan, Hongya, Jiajiang, E'meishan, Ruoergai, Seda, Yangtang, Maerkang, Shiqu, Baiyu, Jinchuan, Heishui, Yanbian, Miyi, Xiangcheng, Daocheng, Jinkouhe, and Chaotian district*

Group 3: Qingchuan, Ya'an, Mingshan, Meigu, Jinyang, Xiaojin, and Huili

6 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Luzhou(3 districts), Neijiang(2 districts), Deyang, Xuanhan, Dazhou, Daxian, Dazhu, Linshui, Quxian, Guangan, Huaying, Longchang, Fushun, Luxian, Nanxi, Jiang'an, Changning, Gaoxian, Hongxian, Xingwen, Xuyong, Gulin, Jintang, Guanghan, Jianyang, Ziyang, Renshou, Zizhong, Jianwei, Rongxian, Weiyuan, Nanjiang, Tongjiang, Wanyuan, Bazhong, Cangxi, Langzhong, Yilong, Xichong, Nanbu, Yanting, Santai, shehong, Daying, Lezhi, Wangcang, Longquanyi, and Qingbaijiang

Group 2: Mianyang(2 districts), Zitong, Zhongjiang, A'ba, Junlian, and Jingyan

Group 3: Guangyuan (2 districts except Chaotian district), Jiange, Luojiang, and Hongyuan

A.0.21 Guizhou Province

1 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Wangmo

Group 2: Weining

2 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Guiyang(5 districts except Baiyun), Kaili, Bijie, Anshun, Duyun, Liupanshui, Huangping, Fuquan, Guiding, Majiang, Qingzhen, Longli, Pingba, Nayong, Zhijin, Shuicheng, Puding, Liuzhi, Zhenning, Huishui, Changshun, Guanling, Ziyun, Luodian, Xingren, Zhenfeng, Anlong, Ceheng, Jinsha, Yinjiang, Chishui, Xishui, and Sinan*.

Group 2: Hezhang, Puan, Qinglong, and Xingyi

Group 3: Panxian

A.0.22 Yunnan Province

1 The place-names assigned to Intensity not less than 9 with the design basic acceleration of ground motion not less than 0.40g:

Group 1: Xundian, Dongchuan

Group 2: Lancang

2 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.30g:

Group 1: Jianchuan, Songming, Yiliang, Lijiang, Heqing, Yongsheng, Luxi, Longling, Shibing, and Jianshui

Group 2: Gengma, Shuangjiang, Cangyuan, Menghai, Ximeng, and Menglian

3 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Shilin, Yuxi, Dali, Qiaojia, Jiangchuan, Huaning, E'shan, Tonghai, Eryuan, Bingchuan, Midu, Xiangyun, Huize, and Nanjian

Group 2: Kunming (4 districts except Dongchuan), Simao, Baoshan, Malong, Chenggong, Chengjiang, Jinning, Yimen, Yangbi, Weishan, Yunxian, Tengchong, Shidian, Ruili, Lianghe, Anning, Fengqing*, and Longchuan*

Group 3: Jinghong, Yongde, Zhenkang, Lincang

4 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Zhongdian, Lushui, Daguan, Xinping*, and Yongshan.

Group 2: Zhanyi, Gejiu, Honghe, Yuanjiang, Lufeng, Shuangbai, Kaiyuan, Yingjiang,

Yongping, Changning, Ninglang, Nanhua, Chuxiong, Mengla, Huaping, and Jingdong*

Group 3: Qujing, Mile, Luliang, Fumin, Luquan, Wuding, Lanping, Yunlong, Jinggu, and Puer

5 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Yanjin, Suijiang, Deqin, Shuifu, and Gongshan.

Group 2: Zhaotong, Yiliang, Ludian, Fugong, Yongren, Dayao, Yuanmou, Yaoan, Muding, Mojiang, Lvchun, Zhenwan, Jiangcheng, and Jinping

Group 3: Fuyuan, Shizong, Luxi, Mengzi, Yuanyang, Weixi, and Xuanwei

6 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Weixin, Zhenxiong, Guangnan, Funing, Xichou, Malipo, and Maguan

Group 2: Qiubei, Yanshan, Bingbian, Hekou, and Wenshan

Group 3: Luoping

A.0.23 Tibet Autonomous Region

1 The place-names assigned to Intensity not less than 9 with the design basic acceleration of ground motion not less than 0.40g:

Group 2: Dangxiong, Motuo

2 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.30g:

Group 1: Shenzha

Group 2: Milin, Bomi

3 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Pulan, Nielamu, and Saga

Group 2: Lasa, Duilongdeqing, Nimu, Renbu, Nima, Luolong, Longzi, Cuona, and Qusong

Group 3: Naqu, Linzhi(Bayi Town), and Linzhou

4 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Zhada, Jilong, Lazi, Xietongmen, Yadong, Luozha, and Angren,

Group 2: Ritu, Jiangzi, Kangma, Bailang, Zhanang, Cuomei, Sangri, Jiacha, Bianba, Basuo, Dingqing, Leiwuqi, Naidong, Qiongjie, Gongga, Langxian, Dazi, Rikaze*, and Gaer*

Group 3: Nanmulin, Bage, Langkazi, Mozhugongka, Qushui, Anduo, and Nierong

5 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Gaize, Cuoqin, Zhongba, Dingjie, and Mangkang

Group 2: Changdu, Dingri, Sajia, Gangba, Baqing, Gongbujiangda, Suoxian, Biru, Jiali,

Chaya, Zuogong, Chayu, Jiangda, and Gongjue

6 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group1: Geji

A.0.24 Shanxi Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group1: Xian(8 districts), Weinan, Huaxian, Huayin, Tongguan, and Dali

Group2: Longxian

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Xianyang(2 districts and Yanling), Baoji(2 districts), Gaoling, Qianyang, Qishan, Fengxiang, Fufeng, Wugong, Xingping, Zhouzhi, Meixian, Baoji County, Sanyuan, Fuping, Chengcheng, Pucheng, Jingyang, Liquan, Chang'an, Huxian, Lantian, Hancheng, and Heyang

Group 2: Fengxian

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Ankang, Pingli, Qianxian, Luonan

Group 2: Baishui, Yaoxian, Chunhua, Linyou, Yongshou, Shangzhou, Tongchuan(2 districts)*, and Zhashui*

Group 3: Taibai, Liuba, Mianxian, and Lueyang

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Yanan, Qingjian, Shenmu, Jiaxian, Mizhi, Suide, Ansai, Yanchuan, Yanchang, Dingbian, Wuqi, Zhidan, Ganquan, Fuxian, Shangnan, Xunyang, Ziyang, Zhenba, Baihe, Langao, Zhenping, and Zichang*

Group 2: Fugu, Wubao, Luochuan, Huangling, Xunyi, Yangxian, Xixiang, Shiquan, Hanyin, Ningshan, Hanzhong, Nanzheng, and Chenggu

Group 3: Ningqiang, Yichuan, Huanglong, Yijun, Changwu, Binxian, Foping, Zhenan, Danfeng, and Shanyang

A.0.25 Gansu Province

1 The place-names assigned to Intensity not less than 9 with the design basic acceleration of ground motion not less than 0.40g:

Group 1: Gulang

2 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.30g:

Group 1: Tianshui(2 districts), Lixian, Xihe, and Pingchuan district

3 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Dang chang, Wenxian, Subei, and Wudu

Group 2: Lanzhou (4 districts), Chengxian, Zhouqu, Huixian, Kangxian, Wuwei, Yongdeng, Tianzhu, Jingtai, Jingyuan, Longxi, Wushan, Qinan, Qingshui, Gangu, Zhangxian, Huining, Jingning, Zhuanglang, Zhangjiachuan, Tongwei, and Huating

4 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Kangle, Jiayuguan, Yumen, Jiuquan, Gaotai, Linze, and Sunan

Group 2: Baiyin(Baiyin districts), Yongjing, Minxian, Dongxiang, Hezheng, Guanghe, Lintan, Zhuoni, Diebu, Linzhao, Weiyuan, Golan, Chongxin, Yuzhong, Dingxi, Jinchang, Liangdang, Akesai, Minle, Yongchang, and Honggu.

Group 3: Pingliang

5 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Zhangye, Hezuo, Maqu, Jinta, and Jishishan

Group 2: Dunhuang, Anxi, Shandan, Linxia, Linxia County, Xiahe, Luqu, Jingchuan, and Lingtai

Group 3: Minqin, Zhenyuan, and Huanxian

6 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 2: Huachi, Zhengning, Qingyang, Heshui, and Ningxian

Group 3: Xifeng

A.0.26 Qinghai Province

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Maqin

Group 2: Maduo, Dari

2 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Qilian, Yushu

Group 2: Gande, Menyuan

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Wulan, Zhiduo, Chengduo, Zaduo, and Nangqian

Group 2: Xining (4 districts), Tongren, Gonghe, Delingha, Haiyan, Huangyuan, Huangzhong, Ping'an, Minhe. Hualong, Guide, Jianzha, Xunhua, Ge'ermu, Guinan, Tongde, Henan, Qumalai, Jiuzhi, Banma, Tianjun, and Gangcha

Group 3: Datong, Huzhu, Ledu, Dulan, Xinghai

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 2: Zeku

A.0.27 Ningxia Autonomous Region

1 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.30g:

Group 1: Haiyuan

2 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Yinchuan (3 districts), Shizuishan (3 districts), Wuzhong, Huinong, Pingluo, Helan, Yongning, Qingtongxia, Jingyuan, Lingwu, Taole, and Guyuan

Group 2: Xiji, Zhongwei, Zhongning, Tongxin, and Longde

3 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 3: Pengyang

4 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 3: Yanchi

A.0.28 Xinjiang Autonomous Region

1 The place-names assigned to Intensity not be less than 9 with the design basic acceleration of ground motion not less than 0.40g:

Group 2: Wuqia, Tashiku'ergan

2 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.30g:

Group 2: Atushi, Kashi, Shufu

3 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 1: Wulumuqi (7 districts), Wulumuqi County, Wensu, Akesu, Keping, Miquan, Zhaosu, Tekesi, Kuche, Balishen, Qinghe, Fuyun, and Wushi*

Group 2: Nileke, Xinyuan, Gongliu, Jinghe, Kuitun, Shawan, Manasi, Shihezi, Dushanzi

Group 3: Shule, Jiashi, Aketao, and Yingjisha

4 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Ku'erle, Xinhe, Luntai, Hejing, Yangi, Bohu, Bachu, Changji, Baicheng, Fukang*, and Mulei*

Group 2: Yining, Yining County, Huocheng, Chabucha'er, and Hutubi

Group 3: Yuepuhu

5 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Tulufan, Hetian, Hetian County, Jimusa'er, Luopu, Qitai, Yiwu, Shanshan, Tuokexun, Heshuo, Weili, Moyu, Hami, and Cele

Group 2: Kelamayi(Kelamayi district), Bole, Wenquan, Aheqi, Awati, and Shaya

Group 3: Shache, Zepu, Yecheng, Maigaiti, and Pishan

6 The place-names assigned to Intensity 6 with the design basic acceleration of ground motion equal 0.05g:

Group 1: Yutian, Habahe, Tacheng, E'min, Fuhai, Hebukesai'er, and Wu'erhe

Group 2: Aletai, Tuoli, Minfeng, Ruoqiang, Bu'erjin, Jimunai, Yumin, and Baijiantan

Group 3: Qiemu

A.0.29 Hongkong, Macao and Taiwan

1 The place-names assigned to Intensity not less than 9 with the design basic acceleration of ground motion not less than0.40g:

Group 1: Taizhong

Group 2: Miaoli, Yunlin, Jiayi, and Hualian

2 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.30g:

Group 2: Taibei, Taoyuan, Tainan, Jilong, Yilan, Taidong, and Bingdong

3 The place-names assigned to Intensity 8 with the design basic acceleration of ground motion equal 0.20g:

Group 2: Gaoxiong, Penghu

4 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.15g:

Group 1: Hong kong

5 The place-names assigned to Intensity 7 with the design basic acceleration of ground motion equal 0.10g:

Group 1: Macao

Appendix B
Requirements for seismic design of high strength grades concrete structures

B.0.1 The concrete strength grade adopted in the high strength concrete structures shall conform to the provisions in Clause 3.9.3 of this code. And the seismic design of these structures, besides complying with the requirements for seismic design of common concrete structures, shall also comply with the provisions in this Appendix.

B.0.2 In formulae for cross-sectional shear stress design limit values of structural member, the terms that contain specified concrete compressive strength (f_c) shall be multiplied by the adjusting factor of concrete strength (β_c). The β_c shall be taken as 1.0 for the concrete strength grade C50, taken as 0.8 for the strength grade C80, and taken as the interpolation value from grade C50 through C80.

When calculating the compressive zone height or checking the bearing capacity of structural member, terms in the equation that contain the specified concrete compressive strength (f_c) shall be multiplied by corresponding adjusting factor of concrete strength . They are as per specified in relevant provisions in the national standard "Code for design of concrete structures GB50010".

B.0.3 The seismic details of high strength concrete frames shall conform to the following requirements:

1 The reinforcement ratio for the longitudinal tensile reinforcement at the end of the beam should not be greater than 3% (for HRB335 steel) or 2.6% (for HRB400 steel). The minimum diameter of the stirrups in the densified hoop ranges at the end of the beam shall increase 2mm than the minimum diameter of reinforcement used for common reinforced concrete beams.

2 The axial-force-ratio limit value of the column should be adopted according to the following provisions:

For the strength grade less than and equal to C60, the limit value may be taken as that of common concrete columns; for the strength grade C65 or C70, should reduce 0.05 than that; and for the strength grade C75 through C80, should reduce 0.1 than that.

3 When the strength grade of the concrete is greater than C60, the minimum total reinforcement ratio for the longitudinal reinforcement of the column shall increase by 0.1% than that of common reinforced concrete columns.

4 The minimum hoop characteristic values in the densified ranges of the column should be adopted according to the following provisions. Meanwhile, for the concrete strength is greater than C60, the stirrups shall adopt composite hoops, composite spiral hoops or continu-

ous composite rectangular spiral hoops.

1)When the axial-force-ratio is not greater than 0.6, this value shall be 0.02 greater than that of common concrete columns;

2)When the axial-force-ratio is greater than 0.6, this value shall be 0.03 greater than that of common concrete columns.

B.0.4 When the strength grade of the concrete is greater than C60, the hoop characteristic factor of the confine bounding elements in the seismic structural wall should be 0.02 greater than that of common concrete wall.

Appendix C
Seismic design requirements for prestressed concrete structures

C.1 General

C.1.1 This appendix is applicable to the seismic designs of pretensioning and posttensioning concrete structures with bonding prestress for Intensity 6 to 8; the special studies shall be carried out for Intensity 9. The seismic design of concrete structures without bonding prestress shall comply with special provisions.

C.1.2 For seismic design, the posttensioning prestressed members of the frame should adopt reinforcement with bonding prestress.

C.1.3 The anchoring device of the posttensioning prestress reinforcement should not be installed at core zone of the joint.

C.2 Prestressed frame structures

C.2.1 The prestressed frame beam shall conform to the following provisions:

1 The posttensioning prestressed concrete frame beam shall adopt the composite reinforcement arrangement mixing prestress reinforcement and non-prestress together. The prestress strength ratio calculated according to the following equation should not be greater than 0.60 for frames assigned to Grade 1, and should not be greater than 0.75 for frames assigned to Grade 2 or 3.

$$\lambda = \frac{A_p f_{py}}{A_p f_{py} + A_s f_y} \tag{C.2.1}$$

where: λ——prestressed strength ratio.

A_p, A_s——the section areas of the prestressed reinforcement and the non-prestressed reinforcement in the tensile zone respectively.

f_{py}——the specified tensile strength of the prestressed reinforcement.

f_y——the specified tensile strength of the non-prestressed reinforcement.

2 The reinforcement ratio, which calculated according to non-prestressed reinforcement strength design value, for the total longitudinal tensile reinforcement at the end of the prestressed concrete frame beam, shall not be greater than 2.5%. Meanwhile, the ratio of the neutral depth, where the compressive reinforcement may be taken into consideration, to the effective depth in section at the beam end shall not exceed 0.25 for Grade 1, and 0.35 for Grade 2 or 3.

3 The ratio of non-prestressed reinforcement arrangements at the bottom-face and the top-face of the beam end cross section, besides being determined through calculation, shall not be less than 1.0 for Grade 1, and not less than 0.8 for Grade 2 or 3. Meanwhile, the rein-

forcement ratio of non-prestressed reinforcement at the bottom-face shall not be less than 0.2% of the gross cross section.

C.2.2 Prestressed concrete cantilever beams shall conform to the following provisions:

1 The prestressed strength ratio of the cantilever beams may be adopted according to the provisions in point 1 of Clause C.2.1 of this Appendix. The ratio of the compressive zone depth to the effective depth of beam may be adopted according to the provisions in point 2 of Clause C.2.1 of this Appendix.

2 The ratio of non-prestressed reinforcement arrangements at the bottom-face and top-face of the cantilever beams, besides being determined through calculation, shall not be less than 1.0, and the steel ratio of non-prestressed reinforcement at the bottom-face shall not be less than 0.2% of the gross cross section.

C.2.3 Prestressed concrete frame columns shall conform to the following provisions:

1 The top story side column for large-span prestressed frames should adopt non-symmetrical reinforcement arrangements; mixing prestressed and non-prestressed together shall be adopted for one side, and non-prestressed reinforcements shall be adopted for the other side.

2 The adjustment for internal force design values of prestressed frame columns shall also comply with corresponding requirements for the frame column at Section 6.2 of this code.

3 The ratio of the compressive zone depth to the effective depth of the cross section of the prestressed concrete frame column shall not be greater than 0.25 for Grade 1, and shall not be greater than 0.35 for Grade 2 or 3.

4 The hoops of the prestressed concrete frame columns shall be densified along the overall height of the columns.

Appendix D
Seismic check for the core zone of the frame joints

D.1 Joints of common beam and column

D.1.1 For frames assigned to Grade 1 and 2, the combinating shear force design value for the core zone at the joint of beam and column shall be determined according to the following equation:

$$V_j = \frac{\eta_{jb}\Sigma M_b}{h_{b0} - a'_s}\left(1 - \frac{h_{b0} - a'_s}{H_c - h_b}\right) \qquad \text{(D.1.1-1)}$$

For Grade 1 framed structures or Intensity 9, it shall also conform to

$$V_j = \frac{1.15\Sigma M_{bua}}{h_{b0} - a'_s}\left(1 - \frac{h_{b0} - a'_s}{H_c - h_b}\right) \qquad \text{(D.1.1-2)}$$

where: V_j——the combinating shear force design value for the core zone at the joint of beam and column.

h_{b0}——the effective depth of the beam section, when the depths of the beam sections at the two sides of the joint are unequal, the average value may be adopted.

a'_s——the distance measured from the extreme compression fiber to the centroid of compressive reinforcement of the beam.

H_c——the calculated height of the column, which may adopt the distance of inflection points of the upper and lower columns on the joint.

h_b——the depth of beam section, when the depths of the beam sections at the two sides of the joint are unequal, the average value may be adopted.

η_{jb}——shear force amplification factor of joint, take as 1.35 for Grade 1, and 1.2 for Grade 2.

ΣM_b——the sum of the clockwise and counter-clockwise combined bending moments at the right and left ends of the joint. If both bending moments at the left and right end of the joint are negative together for Grade 1, the bending moment with smaller absolute value shall choose zero.

ΣM_{bua}——sum of moments in clockwise or counter-clockwise direction at the faces of the joint corresponding to actual seismic bending capacity of normal cross section of right and left beam ends of the joint, which may be determined by actual reinforcement area and characteristic value of the material strength.

D.1.2 Effective checking width of the cross section at the core zone shall be adopted according to the following provisions:

1 For the beam width at the checking direction is not less than 1/2 of the column width,

the effective checking width of the cross section at the core zone may be taken as the width of column. And for that is less than 1/2, the smaller value of the following two cases may be adopted:

$$b_j = b_b + 0.5h_c \tag{D.1.2-1}$$

$$b_j = b_c \tag{D.1.2-2}$$

where: b_j——the effective checking width of the cross section at the core zone of the joint.

b_b——the width of beam cross-section.

h_c——the depth of column cross-section at the checking direction.

b_c——the width of column cross-section at the checking direction.

2 When the eccentricity between the axes of beam and the column is not greater than 1/4 of the column width, such width may be taken as that, which is the smaller value between that provision in the above clause and that obtained from the following equation:

$$b_j = 0.5(b_b + b_c) + 0.25h_c - e \tag{D.1.2-3}$$

where: e——the eccentricity between both axes of the beam and column.

D.1.3 The combinating shear force design value of joint core zone shall comply with the following requirements:

$$V_j \leqslant \frac{1}{\gamma_{RE}}(0.30\eta_j f_c b_j h_j) \tag{D.1.3}$$

where: η_j——the confined effect factor of the orthogonal beams. In generally, this factor shall be taken as 1.0; and may be taken as 1.5 (1.25 for Intensity 9) when all following conditions are satisfied:

(1) The floor is of cast-in-situ ; (2) The axes of the beam and column are concord; (3) The beam width at the four sides is not less than 1/2 of the corresponding column width; and (4) The beam depth at the orthogonal direction is not less than 3/4 of that at checking direction.

h_j——cross-sectional height at the core zone of the joint may adopt the column cross-sectional height at the checking direction.

γ_{RE}——seismic adjusting factor for load-bearing capacity of the structural member.

D.1.4 The seismic shear bearing capacity of the cross section at the joint core zone shall comply with the following requirements:

$$V_j \leqslant \frac{1}{\gamma_{RE}}\left(1.1\eta_j f_t b_j h_j + 0.05\eta_j N\frac{b_j}{b_c} + f_{yv}A_{svj}\frac{h_{b0} - a'_s}{s}\right) \tag{D.1.4-1}$$

for Intensity 9

$$V_j \leqslant \frac{1}{\gamma_{RE}}\left(0.9\eta_j f_t b_j h_j + f_{yv}A_{svj}\frac{h_{b0} - a'_s}{s}\right) \tag{D.1.4-2}$$

where: N——the smaller axial compressive force of the upper column corresponding to the combinating shear design value. Such value shall not be greater than 50% of product of the column cross-sectional area and the specified concrete compressive strength design value; and when N is tensile force, take as zero.

f_{yv}——design value of specified tensile strength of hoop.

f_t——design value of specified axial tensile strength of concrete.

A_{svj}——total hoop cross-sectional area within the same cross section at the checking direction within the effective checking width of the core zone.

s——spacing of hoop.

D.2 Beam and column joint for flat beam frames

D.2.1 When the beam width is greater than the column width in flat beam frames, the seismic check for joint of the beam and the column shall conform to the provisions in this section.

D.2.2 The each joint core zone of flat beam and column shall be divided in two parts based on the ratio of the longitudinal reinforcement section area within and beyond the range of the column width. The checks on the shear bearing capacity of joint core zone shall be made for within and beyond the range of column width respectively.

D.2.3 The seismic check for the core zone, besides satisfying the requirements for common frame beam-column joint, shall also satisfy the following requirements:

1 When checking the shear force limit value of the core zone according to equation (D.1.3) in this Appendix, the effective width of the core zone may adopt the average value of column width and beam width.

2 The confined effect factor of orthogonal beam, for within the range of column width may be taken as 1.5, and for beyond the range of the column width should be taken as 1.0.

3 When checking the shear bearing capacity of the core zone, the value of the axial force, within the range of the column width, may be the taken as same as the common joint; but that beyond the range of the column width, may not be taken into consideration.

4 The total reinforcements developed to the range of column width should be greater than 60% of total reinforcements in the top-face of the flat beam.

D.3 Beam and column joint for cylinder column frame

D.3.1 When the axes of the beam and the column are concord, the combinating shear force design value of joint core zone for the cylinder column frame shall comply with the following requirements:

$$V_j \leqslant \frac{1}{\gamma_{RE}}(0.30\eta_j f_c A_j) \qquad (D.3.1)$$

where: η_j——the confined effect factor of orthogonal beam shall be determined according to Clause D.1.3 of this Appendix, the cross-sectional width of column may be taken as the diameter of the column:

A_j——the effective cross-sectional area of the joint core zone. When the beam width (b_b) is not less than 50% of the column diameter (D), then $A_j=0.8D^2$; when the beam width (b_b) is less than 50% of the column diameter (D) but not less than $0.4D$, then $A_j=0.8\ D\ (b_b+D/2)$.

D.3.2 When the axes of beam and column are concord, the seismic shear bearing capacity of joint core zone of the cylinder column frame shall be checked according to the following equation:

$$V_j \leqslant \frac{1}{\gamma_{RE}}\left(1.5\eta_j f_t A_j + 0.05\eta_j \frac{N}{D^2}A_j + 1.57 f_{yv} A_{sh} \frac{h_{b0} - a'_s}{s} + f_{yv} A_{svj} \frac{h_{b0} - a'_s}{s}\right) \quad \text{(D.3.2-1)}$$

For Intensity 9

$$V_j \leqslant \frac{1}{\gamma_{RE}}\left(1.2\eta_j f_t A_j + 1.57 f_{yv} A_{sh} \frac{b_{b0} - a'_s}{s} + f_{yv} A_{svj} \frac{h_{b0} - a'_s}{s}\right) \quad \text{(D.3.2-2)}$$

where: A_{sh}——cross-sectional area of single circular hoop.

A_{svj}——total cross-sectional area of tie steel and non-circular hoops in the checking direction.

D——column diameter.

N——axial force design value, that shall comply with provisions for common joint in this Appendix.

Appendix E
Seismic design requirements for the transition-stories

E.1 Frame-supporting floor design requirement for rectangular plane wall structure

E.1.1 The frame-supporting floor shall comply with the following provisions:

The floor shall adopt cast-in-situ, the thickness shall not be less than 180mm, the concrete strength grade shall not be less than C30, two-way and double layer reinforcement shall be arranged, besides, the reinforcement ratio of each layer in each direction shall not be less than 0.25%.

E.1.2 The shear force design value of frame-supporting floor slab for frame-support-wall structure shall satisfy the following requirements:

$$V_f \leqslant \frac{1}{\gamma_{RE}}(0.1 f_c b_f t_f) \tag{E.1.2}$$

where: V_f——the combinating shear force design value in the frame-supporting floor slab transmitted from discontinuous wall to continuous wall. That is calculated by rigid floor assumption and shall be multiplied by the amplification factor 2 for Intensity 8, and 1.5 for Intensity 7; for checking on the continuous wall, the amplification factor need not be considered.

b_f, t_f——the width and thickness of the frame-supporting floor slab respectively.

γ_{RE}——seismic adjusting factor for bearing capacity, may be taken as 0.85.

E.1.3 The seismic shear capacity at the inter section of the frame-supporting slab and the continuous to ground wall in frame-support wall structures shall be checked in accordance with the following equation:

$$V_f \leqslant \frac{1}{\gamma_{RE}}(f_y A_s) \tag{E.1.3}$$

where: A_s——the total reinforcement section areas of the frame-supporting floor that traverses the continuous wall.

E.1.4 Boundary beams shall be installed along the perimeters of the edge and larger openings in the frame-supporting floor, its width shall not be less than 2 times of the slab thickness. The reinforcement ratio of longitudinal reinforcement shall not be less than 1%, and the adapters of reinforcement in the beam should adopt mechanical connection or welded. The reinforcement in the slab shall be developed to the boundary beams.

E.1.5 For frame-supporting story with longer-size plane or irregular in configuration or the

internal force of the seismic structural wall have big differences, the in-plane flexural and shear bearing capacity of the slab may also be checked, that may adopt simplification methods.

E.2 Seismic design requirements for transition-stories of tube structures

E.2.1 The structural gravity centers above and below the transition-story shall be coincidence (the appartment works is not included), and the lateral-stiffness-ratio of first upper-story to lower-story shall not be greater than 2.

E.2.2 The discontinuous vertical lateral-force-resisting members (walls and/or columns) above the transition-story should directly be connected to the major structural members of the transition-floor.

E.2.3 The thick-slab-transition-story structure should not be used in tall buildings for Intensity 7 or above.

E.2.4 The transition floor shall not have larger openings, and should be close to infinite-rigidity diaphragm.

E.2.5 Reliable connections shall be made between the transition-floor and the tube or wall, the seismic check and the design details of the transition-floor should conform to relevant provisions in Section E.1 of this appendix.

E.2.6 Vertical seismic action for the transition-story structures shall be considered for Intensity 8.

E.2.7 The transition-story structure shall not be adopted for Intensity 9.

Appendix F
Seismic design for reinforced concrete small-sized hollow block wall buildings

F.1 General

F.1.1 The maximum height of the reinforced concrete small-sized hollow block wall buildings (hereinafter refer to reinforced small-block buildings) applicable to this Appendix shall conform to the provisions in Table F. 1. 1-1. And the maximum ratio of the total height to total width of the building shall not exceed the provisions in Table F. 1. 1-2. For buildings with rather less transversal walls or sited on Site-class IV, the applicable maximum height shall be reduced accordingly.

Table F.1.1-1 Applicable maximum height of reinforced small-block buildings (m)

Min. Wall thickness (mm)	Intensity 6	Intensity 7	Intensity 8
190	54	45	30

Note: when the building height exceeds the height as provisions in the Table, the effective strengthened measures shall be taken based on special studies.

Table F.1.1-2 Maximum height-width ratio of reinforced small-block buildings

Intensity	6	7	8
Maximum height-width ratio	5	4	3

F.1.2 For reinforced small-block buildings, different measure grades shall be adopted based on Fortification Category, Intensity and the height of buildings, and shall also comply with the requirements of corresponding calculations and design details. The measure grades of buildings assigned to Category C shall be determined in accordance with the provisions in Table F.1.2.

Table F1.2 Seismic measure grade of reinforced small-block buildings

Intensity	6		7		8	
Height (m)	≤24	>24	≤24	>24	≤24	>24
Seismic-measure-grade	Grade 4	Grade 3	Grade 3	Grade 2	Grade 2	Grade 1

Note: When the building height is close or equal to the height boundary, the seismic measure grade may be adjusted in consideration of the irregular degree of the building, the site and the foundation condition.

F.1.3 The building structural design shall avoid the irregular structures as provision in Section 3.4 of this code, and shall also comply with the following requirements:

1 The plane configuration should be simple, regular, and re-entrant corners should not be too significant; the vertical configuration shall be regular, even, and the setback or overhang should not be too significant.

2 Each of longitudinal or transversal seismic structural walls should be aligned in-plane. The length of each wall should not be too long, the height-width ratio of each wall portion shall not be less than 2; and the door openings should be in a line and should be arranged in rows.

3 The maximum spacing of the transversal walls in the building shall satisfy the requirements in Table F.1.3.

Table F.1.3 Maximum spacing of the transversal seismic structural well

Intensity	6	7	8
Maximum spacing (m)	15	15	11

F.1.4 The building should select regular and reasonable structure systems, so that not to install seismic joint. In case seismic joint is needed, the minimum clear width of the joint shall conform to the following requirements:

When the building height does not exceed 20m, the width of the joint may be taken as 70mm. And when the building height is more than 20 m, then for the Intensity 6, 7 and 8, the width should be added by 20 mm for each 6m, 5m and 4m increases in height respectively.

F.2 Essentials in calculation

F.2.1 When seismic analysis is made for reinforced small-block buildings, the seismic effect shall be adjusted according to the provisions of this section. The seismic checking may not be carried out for Intensity 6.

F.2.2 For checking the seismic bearing capacity of reinforced small-block wall, the combinating shear force design value of the cross section at the bottom-strengthened location shall be determined in accordance with the following equation:

$$V = \eta_{vw} V_w \qquad \text{(F.2.2)}$$

where: V——the combinating shear force design value of the cross section at the bottom-strengthened location.

V_w——the combinating shear force calculating value of the cross section at bottom-strengthened location.

η_{vw}——the shear force amplification factor, taken as 1.6 for Grade 1, as 1.4 for Grade 2, as 1.2 for Grade 3, and as 1.0 for Grade 4.

F.2.3 The combinating shear force design value of cross section for the reinforced small-

block wall shall comply with the following requirements:

When the shear span-to-depth ratio is greater than 2

$$V \leqslant \frac{1}{\gamma_{RE}} (0.2 f_{gc} b_w h_w) \qquad \text{(F.2.3-1)}$$

When the shear span-to-depth ratio is not greater than 2

$$V \leqslant \frac{1}{\gamma_{RE}} (0.15 f_{gc} b_w h_w) \qquad \text{(F.2.3-2)}$$

where: f_{gc}——the specified compressive strength of core-grouting small-block masonry, when fully grouted, the value may be taken as 2 times of the specified compressive strength of small-block masonry.

b_w——width of seismic-wall cross-section.

h_w——depth of seismic-wall cross-section.

γ_{RE}——seismic adjusting factor for bearing capacity, may be taken as 0.85.

Note: the shear span-to-depth ratio shall be calculated according to equation (6.2.9-3) of this code.

F.2.4 The shear bearing capacity of cross section for eccentric compressive reinforced small-block wall shall be checked in accordance with the following equations:

$$V \leqslant \frac{1}{\gamma_{RE}} \left[\frac{1}{\lambda - 0.5} (0.48 f_{gv} b_w h_w + 0.1N) + 0.72 f_{yh} \frac{A_{sb}}{s} h_{w0} \right] \qquad \text{(F.2.4-1)}$$

$$0.5V \leqslant \frac{1}{\gamma_{RE}} \left(0.72 f_{yh} \frac{A_{sh}}{s} h_{w0} \right) \qquad \text{(F.2.4-2)}$$

where: N——axial force design value of the wall, shall not be greater than $0.2 f_{gc} b_w h_w$.

λ——shear span-to-depth ratio of the checking cross section, take as $\lambda = M/Vh_w$, as 1.5 for the ratio less than 1.5, and as 2.2 for the ratio greater than 2.2.

f_{gv}——the shear strength design value of core-grouting small-block masonry, take $f_{gv} = 0.2 f_{gc}^{0.55}$.

A_{sh}——cross-sectional area of horizontal reinforcement with the same cross section.

s——spacing of horizontally distributed reinforcement.

f_{yh}——specified tensile strength design value of horizontally distributed reinforcement.

h_{w0}——effective depth of cross section of the seismic structural wall.

γ_{RE}——seismic adjusting factor for bearing capacity, may be taken as 0.85.

F.2.5 For the reinforced small-block wall, the coupling beams with span-depth ratio greater than 2.5 should adopt reinforced concrete coupling beam, which combinating shear

force design value and the seismic shear bearing capacity shall conform to relevant provisions for coupling beam in the current national standard"Code for design of concrete structures" GB 50010.

F.3 Details of seismic design

F.3.1 The core-grouted concrete of reinforced small-block buildings shall adopt concrete with good slump, flowing and workable properties, as well as good bonding characters with blocks. The strength grade of core-grouted concrete shall not be lower than C20.

F.3.2 For the wall bottom of reinforced small-block buildings, that height is not less than 1/6 of the total height of wall and not less than 2 stories, the longitudinal and horizontal reinforcements shall be arranged subject to the requirements for bottom-strengthening location.

F.3.3 The arrangement of the longitudinal and horizontal reinforcement in reinforced small-block wall shall conform to the following requirements:

1 The vertical reinforcement may adopt single-row arrangement, the minimum diameter shall be 12mm; the maximum spacing shall be 600mm, and the spacing at the top and bottom stories shall be reduced accordingly.

2 The horizontal reinforcement should adopt double-row arrangement, the minimum diameter shall be 8mm, the maximum spacing shall be 600mm, and the spacing at the top and the bottom stories shall not be greater than 400mm.

3 The minimum reinforcement ratio of the vertical and horizontal reinforcement shall not be less than 0.13% for Grade 1, than 0.10% for general portion of Grade 2, than 0.10% for Grade 3 or Grade 4; and should not be less than 0.13% for bottom-strengthened portion of Grade 2.

F.3.4 The splicing length of the vertical and horizontal reinforcements in the reinforced small-block wall shall not be less than 48 times of the reinforcement diameter, the developed length shall not be less than 42 time of the reinforcement diameter.

F.3.5 The axial-force-ratio of the reinforced small-block wall under gravity load representative value shall not be greater than 0.5 for Grade 1, and than 0.6 for Grade 2 or 3.

F.3.6 When the compressive stress of the reinforced small-block walls is greater than 0.5 times of the specified core-grouted small-block compressive strength(f_{gc}), boundary elements shall be installed at the end of the wall. The length of boundary elements, which is measured from extreme compression fiber toward to neutral axis of web, is not less than 3 times of the wall thickness and the reinforcement of the element shall conform to the requirements of Table F.3.6:

F.3.7 The seismic details of the coupling beams for reinforced small-block wall shall comply with the following requirements:

1 The length extended in wall for the longitudinal bars of the coupling-beam shall not

be less than 1.15 times of the developed length for Grade 1 or 2, than 1.05 times of the developed length for Grade 3, and than the developed length and 600mm for Grade 4.

Table F.3.6 Requirements for reinforcement in the boundary elements of reinforced small-block wall

Grade	Minimum longitudinal bar amount in strengthened portion	Minimum longitudinal bar amount in general portion	Min. diameter of stirrup	Max. spacing of stirrup
Grade 1	3φ20	3φ18	φ8	200mm
Grade 2	3φ18	3φ16	φ8	200mm
Grade 3	3φ16	3φ14	φ8	200mm
Grade 4	3φ14	3φ12	φ8	20mm

2 The hoop arrangement along the overall length of the coupling beam shall satisfy the requirements for hoop densified range of the frame beam end.

3 Within the developed length range of longitudinal bars of the top story coupling beam, hoops with spacing not greater than 200mm shall be installed, and the diameter of the hoops shall be the same as the hoops of the coupling beam.

4 For coupling beam with span-depth ratio not greater than 2.5, horizontally distributed reinforcement shall be increased from 200mm below the top-face to 200mm above the bottom-face of the beam. The spacing of this reinforcement shall not be greater than 200mm; the reinforcements of each layer shall not be less than 2φ12 for Grade 1 (2φ10 for Grade 2 to 4); and the length extended into the wall shall not be less than 30 times of the bar diameter and 300mm.

5 Openings should not be made on the coupling beam of the reinforced small-block wall, when openings necessary, that shall conform to the following requirements:

1) The steel sleeves with outer diameter not greater than 200mm shall be embedded in 1/3 location of the beam depth in the span middle;

2) The effective depth above and below the opening shall not be less than 1/3 of the beam depth and 200mm;

3) The strengthened bars shall be arranged near the openings, the shear bearing capacity check shall be made for cross sections weakened by the opening.

F.3.8 The details of the floor shall comply with the following requirements:

1 The floors and roof of reinforced small-block buildings should adopt the cast-in-situ concrete slabs; for Grade 4, may also be adopted monolithic-precast reinforcd concrete floor.

2 Cast-in-situ concrete ring-beams shall be installed for each story. The concrete strength grade of ring-beams shall reach 2 times of that of the small-block. The depth of

ring-beams should not be less than 200mm for the cast-in-situ slab, and than 120mm for monolithic-precast reinforced concrete floor. The diameter of the longitudinal bars shall not be less than that of the horizontally distributed bars of masonry wall, the diameter of the stirrups shall not be less than 8mm, and the spacing of the stirrups shall not be greater than 200mm.

Appendix G
Seismic design for multi-story steel structure factory buildings

G.0.1 The structural arrangement of the multi-story steel structure factory building shall conform to relevant requirements in Clauses 8.1.4 to 8.1.7 of this code, as well as the following provisions:

1 When the plane having greater irregularity, the height differences of all framework or loads of different stories vary significantly, the seismic joints shall be installed or other measures shall be taken.

2 When equipment, such as the feed hopper, through the floor is supported in such floor, the gravity center of total equipment (contents included) should be close to the supporting point on such floor. When an equipment through two or more floors, one of floor shall be selected as the supporting, another one floor may be installed as the horizontal support point when necessary.

3 When the equipment assigned to itself-bearing structure, the floors of the building shall be separated with the equipment.

4 The brace systems in the factory building shall comply with the following requirements:

1) The column bracing should be arranged at locations where the load of the column bays is greater, and shall be continuously arranged from top to footing in the same bay. In case it is not continuous in the same bay, it shall be arranged at stagger adjacent bay continuously, and horizontal bracing at adjacent roof or floor shall be added to ensure that the horizontal seismic action of the bracing can be transferred to the foundation.

2) In case of structures with discontinuity column, horizontal bracing should be added accordingly for the roof and floor close to the column, and vertical bracing shall be installed at adjacent column bay.

3) The whole piece material shall be adopted for the column-braces, when exceed the maximum normal length of the material, the equal-strong-splicing such as butt-welding joints may be adopted. The capacity of connection of column-braces and the members shall not be less than 1.2 times of the plastic capacity of the brace.

5 The floor system of the factory building should adopt composite floor with cast-in-situ concrete and fluted plate, and steel floor plate may also be used.

6 When the lateral stiffness of each frame varies significantly and the arrangement of

column bracing is irregular, the inter-floor horizontal bracing shall be installed; in other cases, the installation of inter-floor horizontal bracing shall be determined according to Table G.0.1.

Table G.0.1 Requirements for the installation of horizontal bracing in the floor

Item	Type of the floor		Floor load characteristic value≤10kN/m²	Floor load characteristic value >10kN/m² or greater concentrating load
1	Steel-concrete composite floor, R.C. slab connected reliable with steel beam	Only small opening	No need installed	No need installed
		Larger opening	Shall be installed in the column-grid around the opening	Shall be installed in the column-grid around the opening
2	Metallic plate (connected reliable with main beam)		Should be installed	Shall be installed
3	Movable lattice plate		Shall be installed	Shall be installed

Notes: 1. The floor load refers to the live load, pipes and cables etc. except the dead weight of structure.

2. The openings on the floor are different for every industry, so the classification of big or small openings shall be determined based on practical case of each project.

3. For Intensity 6 and 7, if the metallic plate connected to the main beam reliably, then horizontal bracing may not be installed.

G.0.2 The seismic calculation for factory buildings, besides complying with relevant requirements in Section 8.2 of this code, shall also conform to the following provisions:

1 The gravity load representative value and the variable load combinating coefficient shall complying with the provisions in Chapter 5 of this code. And shall also adopt the corresponding combinating value coefficients for the flooring inspection load, finished or raw material piled floor load, as well as the materials inside the equipment, the feed hopper and the pipes, based on the characteristics of different industries.

2 For the members and the connection of members that directly support equipment and the feed hopper, the seismic action caused by the equipment etc. shall take into consideration:

1) The horizontal seismic action caused by the equipment and the feed hopper to the support members and the member connection may be determined according to the following equation:

$$F_s = \alpha_{max} \lambda G_{eq} \qquad \text{(G.0.2-1)}$$

$$\lambda = 1.0 + H_x/H_n \qquad \text{(G.0.2-2)}$$

where: F_S——the horizontal seismic action characteristic value at the gravity centers of the equipment or the feed hopper.

α_{max}——maximum value of horizontal seismic influence coefficient.

G_{eq}——gravity load representative value of the equipment or feed hopper.

λ——amplifying factor.

H_x——distance from the base of the building to the gravity centers of the equipment or the feed hopper.

H_n——distance from the base of the building to the top of the building.

2) The bending and torsion moments caused by horizontal seismic action to the supporting members shall be calculated by the distance from the gravity center of equipment or feed hopper to the center of the supporting members.

3 In the case of composite floor with cast-in-situ concrete and fluted plate, when the opening on the plate is small and is connected with the steel beam with shear studs etc. connection parts, the floor may be deemed as rigid diaphragm.

G.0.3 The seismic-details for multi-story steel factory buildings, besides complying with the requirements in Sections 8.3 and 8.4 of this code, shall also conform to the following requirements:

1 The connection of multi-story steel frame and the bracing may adopt connection by welding or by high strength bolts, the arrangement of longitudinal column-braces and the horizontal bracing of the floor shall satisfy the following requirements:

1) Longitudinal column-braces should be installed close to the middle of the column row;

2) The horizontal bracing of the roof and the column-braces of the top story should be installed at the same bay at the end of the factory-building unit. When the length of factory building unit is too long, a row of horizontal bracing or column-braces shall be installed in each 3-5 bay.

2 The details for the floor horizontal bracing shall comply with the following requirements:

1) Horizontal bracing may be installed at the bottom of the secondary beam, but the end of the bracing bar shall be connected with both the web plate and the lower flange of the frame beams on the floor;

2) The arrangement of horizontal bracing of the floor shall be compatible with the location of the column bracing;

3) The frame beam on the floor may be used as the chord bar of horizontal bracing system, the angel between the chord bar and the diagonal should be taken as 30°to 60°;

4) When the secondary beam within the column grid have greater equipment load, the rigid tie rod shall be added; so that the seismic action of the equipment gravity can be transferred to the bracing chord bar (the frame beam on the axes) or the joint.

Table H.2.3-2 (continued)

Roof Type	Distance between gable or load-bearing transverse walls (m)										
	≤12	18	24	30	36	42	48	54	60	66	72
R. C. roof with purlin or wooden roof with full sheathing	0.65	0.70	0.75	0.80	0.90	0.95	0.95	1.00	1.05	1.05	1.10

H.2.4 The seismic shear and bending moment determined by the base shear method for all the sections of the reinforced concrete column above the bracket, supported the lower span roof at the intersection of the higher and lower span of the factory, shall be multiplied by the amplifying factor. Such values may be adopted according to the following equation:

$$\eta = \zeta\left(1 + 1.7\frac{n_h}{n_0} \cdot \frac{G_{EL}}{G_{Eh}}\right) \tag{H.2.4}$$

where: η——the seismic shear and the bending moment amplifying factor.

ζ——the effect factor for space interaction at the intersection of higher and lower span in factory buildings with unequal height, that may be taken in accordance with Table H.2.4.

n_h——number of higher span.

n_0——number of span for computation; if only one side has lower span, that shall be taken as the total number of span; if there are lower spans on the two sides, shall be taken as the sum of the total number of span and the number of higher span.

G_{EL}——total gravity load representative value, which is concentrated at the roof level of all the lower spans on one side of the intersection.

G_{Eh}——total gravity load representative value, which is concentrated at the level of column top of the entire higher spans.

Table H.2.4 Effect factor for space interaction of reinforced concrete upper column at the intersection of the higher and lower spans

Roof Type	Gable	Roof length (m)										
		≤36	42	48	54	60	66	72	78	84	90	96
Without purlin	Both gables		0.70	0.76	0.82	0.88	0.94	1.00	1.06	1.06	1.06	1.06
	Only one end gable	1.25										
With purlin	Both gables		0.90	1.00	1.05	1.10	1.10	1.15	1.15	1.15	1.20	1.20
	Only one end gable	1.05										

H.3 Seismic effect amplifying factor caused by the bridge crane

H.3.1 For the upper column section at the level of top of the crane girder in reinforced

concrete single-story factory building, the seismic shear and the bending moment caused by the bridge crane shall be multiplied by the amplifying factor. For that determined by the base shear method, such values may be adopted according to Table H.3.1.

Table H.3.1 Amplifying factor of seismic shear and bending moment caused by bridge crane

Type of roof	Gable	Side column	Column at intersection of higher and lower spans	Other mid-columns
Without purlin	Both gables	2.0	2.5	3.0
	Only one end gable	1.5	2.0	2.5
With purlin	Both gables	1.5	2.0	2.5
	Only one end gable	1.2	2.0	2.0

Appendix J
Longitudinal seismic check for single-story factory with reinforced concrete columns

J.1 Modifying stiffness method for longitudinal seismic analysis of factory buildings

J.1.1 The calculation of longitudinal fundamental natural period:

The longitudinal seismic action of the single-span and multi-span reinforced concrete column factory buildings shall be computed by its longitudinal fundamental natural period in this Appendix. And the longitudinal fundamental natural period of the single-span and multi-span reinforced concrete column factory buildings, when the level of column top is not greater than 15m and the average span is not greater than 30m, may be determined according to the following equations:

1 The fundamental natural period of such factory with brick enclosure wall may be obtained according to the following equation:

$$T_1 = 0.23 + 0.00025\psi_1 l\sqrt{H^3} \qquad \text{(J.1.1-1)}$$

where: ψ_1——the roof type factor; for reinforced concrete truss with large-sized roof slab, that may be taken as 1.0; and for steel truss, taken as 0.85.

l——span of the factory building (m); for multi-span factory building, the average value for all spans may be adopted.

H——column height, which is the distance computing from the surface of the footing to the top of the column (m).

2 The fundamental natural period of factory buildings, which has no enclosure wall, half-open wall or that the wall panels are flexibly connected with columns, may be calculated according to equation (J.1.1-1) of point 1 in this clause, and then multiply by the enclosure wall influence factor below:

$$\psi_2 = 2.6 - 0.002l\sqrt{H^3} \qquad \text{(J.1.1-2)}$$

where: ψ_2——the influence factor of the enclosure wall ; if it is less than 1.0, shall equal 1.0.

J.1.2 The calculation for the seismic action of column-row:

1 For factory buildings with equal height and multi-span reinforced concrete floor, the seismic action characteristic value at the top level of all longitudinal column-rows may be determined according to the following equations:

$$F_i = \alpha_1 G_{eq}\frac{K_{ai}}{\Sigma K_{ai}} \qquad \text{(J.1.2-1)}$$

$$K_{ai} = \psi_3\psi_4 K_i \qquad \text{(J.1.2-2)}$$

where: F_i——the longitudinal seismic action characteristic value at the top level of the i-th column-row.

α_1——the horizontal seismic influence coefficient, corresponding to the longitudinal fundamental natural period of the factory building, shall be determined according to Clause 5.1.5 of this code.

G_{eq}——the representative value of total equivalent gravity load of all column-rows for factory building unit. It shall include the roof gravity load representative value determined according to Clause 5.1.3 of this code, 70% of the weight of the longitudinal wall, 50% of the weight of the transversal wall and the gable, and the conversed weight of the column(The conversed weight of column adopts 10% of the column weight when the factory there is crane, and 50% of the column weight when there is no crane).

K_i——total lateral stiffness of the i-th column-row. It shall include the lateral stiffness of the inner columns of the i-th row, the column bracing between the upper and lower column, and the total sum of the deducted lateral stiffness of the longitudinal wall. The deducting factor for the lateral stiffness of the brick enclosure wall may adopt 0.2～0.6 according to the lateral displacement value of the column-row.

K_{ai}——adjusting lateral stiffness at the top of the i-th column-row.

ψ_3——the enclosure wall influence factor of the lateral stiffness of column-row, which may be adopted according to Table J.1.2-1; for four-span and five-span factory buildings with longitudinal brick enclosure wall, the factor of 3rd column-row counted from the side column-row may be adopted according to 1.15 times of the corresponding values in this Table.

ψ_4——the column bracing influence factor for lateral stiffness of column-row; when the longitudinal wall is of brick enclosure wall, the factor of side column-row may adopt 1.0, and the mid-column-row may adopt the values as per set in Table J.1.2-2.

Table J.1.2-1 Enclosure wall influence factors

Type of enclosure wall and Intensity		Type of column-row and roof				
		Side-column-row	Mid-column-row			
240mm brick wall	370mm brick wall		Roof without purlin		Roof with purlin	
			Side span without skylight	Side span with skylight	Side span without skylight	Side span with skylight
	Intensity 7	0.85	1.7	1.8	1.8	1.9
Intensity 7	Intensity 8	0.85	1.5	1.6	1.6	1.7
Intensity 8	Intensity 9	0.85	1.3	1.4	1.4	1.5
Intensity 9		0.85	1.2	1.3	1.3	1.4
No wall, asbestos sheet, or wall panel		0.90	1.1	1.1	1.2	1.2

Table J.1.2-2 Bracing influence factor of mid-column-row for the longitudinal brick enclosure wall

Number of bay with lower column bracing	Slenderness ratio of lower column bracing					Mid-column-row without bracing
	≤40	41~80	81~120	121~150	>150	
One bay	0.90	0.95	1.00	1.10	1.25	1.40
Two bays	/	/	0.90	0.95	1.10	1.40

2 For factory buildings with equal height and multi-span reinforced concrete floor, the longitudinal seismic action characteristic value at the level of top of the crane beam of column-rows may be determined according to the following equation:

$$F_{ci} = \alpha_1 G_{ci} \frac{H_{ci}}{H_i} \tag{J.1.2-3}$$

where: F_{ci}——the longitudinal seismic action characteristic value at the level of top of crane beam of i-th column-row.

G_{ci}——the equivalent gravity load representative value concentrated at the level of top of crane beam of i-th column-row. It shall include the gravity load representative values of the crane beam and the suspended object, which is determined according to Clause 5.1.3 of this code, and 40% of the weight of the column.

H_{ci}——height of the top of crane beam for the i-th column-row.

H_i——height of the top for the i-th column-row.

J.2 Seismic action effect and check for column bracing

J.2.1 The horizontal displacement of column bracing under the unit lateral force, that slenderness ratio is not greater than 200, may be determined according to the following equation:

$$u = \Sigma \frac{1}{1 + \varphi_i} u_{ti} \tag{J.2.1}$$

where: u——displacement under unit lateral force.

φ_i——diagonal axial compression stability coefficient of the i-th portion of column bracing, that shall be determined according to the provisions of current national standard "Code for design of steel structures" GB 50017.

u_{ti}——relative displacement of the i-th portion of column bracing under the unit lateral force when only the tensile diagonal tie is taken into consideration.

J.2.2 Diagonal cross section with slenderness ratio not greater than 200 may only be checked the tensile capacity, but the unloading effect of the compressive diagonal bar shall be taken into consideration, the tensile force may be determined according to the following equation:

$$N_t = \frac{l_i}{(1 + \psi_c \varphi_i) s_c} V_{bi} \tag{J.2.2}$$

where: N_t——axial tensile force design value of the diagonal tie of i-th portion of column bracing for tensile capacity check.

l_i——the total length of the diagonal tie of i-th portion of column bracing.

ψ_c——unloading factor of the compressive diagonal bar; when the slenderness ratios of the compressive bar are separately 60, 100 and 200, such factor may be taken as 0.7, 0.6 and 0.5 separately.

V_{bi}——the seismic shear design value of i-th portion of column bracing.

s_c——net bay of the column bracing location.

J.2.3 For the longitudinal column-row without the masonry walls nestling closely to the outside face of column, the bracing of the upper column and lower column should be designed with uniform strength.

J.3 Seismic check for embedded parts of end joint of column bracing

J.3.1 When the anchoring bars are taken as the anchor of embedded parts connecting the column bracing with the column, its cross section should be determined according to the following equation:

$$N \leqslant \frac{0.8 f_y A_s}{\gamma_{RE}\left(\frac{\cos\theta}{0.8\zeta_m\psi} + \frac{\sin\theta}{\zeta_r\zeta_v}\right)} \tag{J.3.1-1}$$

$$\psi = \frac{1}{1 + \frac{0.6e_0}{\zeta_r s}} \tag{J.3.1-2}$$

$$\zeta_m = 0.6 + 0.25t/d \tag{J.3.1-3}$$

$$\zeta_v = (4 - 0.08d)\sqrt{f_c/f_y} \tag{J.3.1-4}$$

where: A_s——the total cross-sectional area of the anchoring bars.

γ_{RE}——the seismic adjusting factor of the bearing capacity, and it may be taken as 1.0.

N——inclining tensile force of the embedded part, and it may adopt 1.05 times of the diagonal bar axial force, which calculated according to the yielding strength of the whole cross section of diagonal bar.

e_0——eccentricity of the inclining tensile force to the application line of the resultant force of anchoring bar (mm); and it shall be less than 20% of the distance between the both of anchoring bars of the outer-row.

θ——the angel between the inclining tensile force and its horizontal projection.

ψ——the eccentricity effect factor.

s——distance between both of outer-row anchoring bars (mm).

ζ_m——flexural deformation effect factor of embedded part.

t——thickness of embedded part (mm).

d——diameter of anchoring bar (mm).

ζ_r——effect factor of number of anchoring bar rows along the checking direction; for row 2, 3 and 4, the factor may be taken as 1.0, 0.9 and 0.85 separately.

ζ_v——shear effect factor of the anchoring bar, it shall be taken as 0.7 when greater than 0.7.

J.3.2 When the angle bar added end plate is taken as the anchor of embedded part of connection for the column bracing and column, its cross section should be determined according to the following equations:

$$N \leqslant \frac{0.7}{\gamma_{RE}\left(\frac{\sin\theta}{V_{u0}} + \frac{\cos\theta}{\psi N_{u0}}\right)} \tag{J.3.2-1}$$

$$V_{u0} = 3n\zeta_r \sqrt{W_{min} b f_a f_c} \tag{J.3.2-2}$$

$$N_{u0} = 0.8 n f_a A_s \tag{J.3.2-3}$$

where: n——number of angle bar.

b——with of angle bar leg.

W_{min}——angle bar minimum cross-sectional modulus orthogonal to the shear direction.

A_s——cross-sectional area of one angle bar.

f_a——specified tensile strength of angle bar.

Appendix K
Modifying stiffness method of longitudinal seismic analysis for single-story factory with brick columns

K.0.1 This appendix is applicable to the longitudinal seismic check for single-story brick column factory with the reinforced concrete roof (with or without purlin) and equal height and multi-spans.

K.0.2 Longitudinal fundamental natural period of structures of single-story brick column factory may be determined according to the following equation:

$$T_1 = 2\psi_T\sqrt{\frac{\Sigma G_s}{\Sigma K_s}} \qquad \text{(K.0.2)}$$

where: ψ_T——period modifying factor, and it may be adopted according to Table K.0.2.

G_s——concentrated gravity load of the s-th column-row. It shall include the gravity load of half of span roofing and gable in the left and right of the column-row, and the conversed gravity load concentrated at the top level of the column or the wall. In which, the gravity of wall and column is conversed according to the principle of dynamic energy equivalence.

K_s——lateral stiffness of the s-th column-row.

Table K.0.2 Modifying factor of longitudinal fundamental natural period of factory buildings

Type of roof	R.C. roof without purlin		R.C. roof with purlin	
	Side span without skylight	Side span with skylight	Side span without skylight	Side span with skylight
Factors	1.3	1.35	1.4	1.45

K.0.3 The total horizontal seismic action characteristic value in the longitudinal direction of single-story brick-column factory shall be determined according to the following equation:

$$F_{Ek} = \alpha_1 \Sigma G_s \qquad \text{(K.0.3)}$$

where: α_1——seismic influence coefficient corresponding to the longitudinal fundamental natural period T_1 of single-story brick column factory building.

G_s——the conversed gravity load representative value concentrated at the top level of the wall of the s-th column-row; that conversed according to the principle of base shear force equality.

K.0.4 Horizontal seismic action of the top of the s-th longitudinal column-row of the factory building may be determined according to the following equation:

$$F_s = \frac{\psi_s K_s}{\Sigma \psi_s K_s} F_{Ek} \qquad \text{(K.0.4)}$$

where: ψ_s——stiffness adjusting factor of column-row considering the horizontal deformation of the roof, and it shall be adopted according to the values in Table K.0.4 depend on the type of the roof and the arrangement of longitudinal wall in all of column-rows.

Table K.0.4 Stiffness adjusting factor of column-rows

Type of longitudinal walls		Type of roof			
		R.C. roof without purlin		R.C. roof with purlin	
		Side-column-row	Mid-column-row	Side-column-row	Mid-column-row
All of row without wall		0.95	1.1	0.9	1.6
All of row with wall		0.95	1.1	0.9	1.2
Side-row with wall	Wall of mid-row is not less 4 bays	0.7	1.4	0.75	1.5
	Wall of mid-row is less 4 bays	0.6	1.8	0.65	1.9

Appendix L
Simplified calculation for seismic-isolation design and seismic-isolation measures of masonry structures

L.1 Simplified calculation for seismic-isolation design

L.1.1 When the multi-story masonry structures and structures with similar foundamental natural period adopt seismic-isolation designs, the total horizontal seismic action of the structures above the seismic-isolation story may be determined simply according to Equation (5.2.1-1) of this code, but the following provisions shall be observed:

1 For the multi-story masonry structure, the horizontal seismic-reduced factor should be determined according to the following equation, that dependent upon the fundamental natural period of the seismic-isolation structural system:

$$\psi = \sqrt{2}\eta_2(T_{gm}/T_1)^{\gamma} \qquad \text{(L.1.1-1)}$$

where: ψ——horizontal seismic-reduced factor.

η_2——damping adjustment factor of seismic influence coefficient, it shall be determined according to Clause 5.1.5 of this code dependent on the equivalent damping of the seismic-isolation story.

γ——the damped exponential index of the curvilinear decrease portion of the seismic influence coefficient, it shall be determined according to Clause 5.1.5 of this code dependent on the equivalent damping of the seismic-isolation story.

T_{gm}——the design characteristic period of seismic-isolation design for the masonry structure, it shall be determined according to Clause 5.1.4 of this code dependent on the design earthquake group, but shall be taken as 0.4s when that is less than 0.4s.

T_1——the fundamental natural period of the seismic-isolation system, it shall not be more than that is larger between the 2.0s and five times of the design characteristic period.

2 For structures that have similar fundamental period with masonry structure, the horizontal seismic-reduced factor should be determined according to the following equation, that dependent upon the fundamental natural period of the seismic-isolation structural system:

$$\psi = \sqrt{2}\eta_2(T_g/T_1)^{\gamma}(T_0/T_g)^{0.9} \qquad \text{(L.1.1-2)}$$

where: T_0——computed period of non-isolation structure, when it is less than the period characteristic value, shall be taken as the characteristic period value.

T_1——the fundamental natural period of the seismic-isolation system, it shall not be more than five times of the characteristic period value.

T_g——characteristic period; other signs are the same as described above.

3 The fundamental natural period of the seismic-isolation masonry structures and structures with similar period may be determined according to the following equation:

$$T_1 = 2\pi\sqrt{G/K_h g} \qquad \text{(L.1.1-3)}$$

where: T_1——the fundamental natural period of seismic-isolation structural system.

G——gravity load representative value of structures above the seismic-isolation story.

K_h——horizontal dynamic stiffness of the seismic-isolation story, it may be determined according to the provisions in Clause 12.2.4 of this code.

g——acceleration of gravity.

L.1.2 For the masonry structures and structures with similar period under rarely earthquakes, the horizontal shear force of the seismic-isolation story may be determined according to the following equation:

$$V_c = \lambda_s \alpha_1(\zeta_{eq})G \qquad \text{(L.1.2)}$$

where: V_c——horizontal shear force of the isolation story under rarely earthquake.

L.1.3 For the masonry structures and structures with similar period under rarely earthquakes, the horizontal displacement of centroid of the isolation story may be determined according to the following equation:

$$u_e = \lambda_s \alpha_1(\zeta_{eq})G/K_h \qquad \text{(L.1.3)}$$

where: λ_s——near-field effect factors; For building assigned to Category A and B is located within 5km of the causative fault, it shall be taken as 1.5; within 5～10km, taken as 1.25; and more than 10km, taken as 1.0; for building assigned to Category C, it may be taken as 1.0.

$\alpha_1(\zeta_{eq})$——the seismic influence coefficient value under rarely earthquake, it may be determined according to Clause 5.1.5 of this code dependent on the seismic-isolation story design parameters.

K_h——the horizontal dynamic stiffness of the seismic-isolation story under rarely earthquake, it shall be determined according to provisions in Clause 12.2.4 of this code.

L.1.4 For the plan arrangement of the isolator-unit is rectangular or nearly rectangular, which the mass center of the structure above the seismic-isolation story and the rigidity center of the seismic-isolation story are eccentric, the torsion-effect factor of the isolator-unit may be determined as follows:

1 When the torsion of only one-way seismic action is considered, the torsion-effect factor of the isolator-unit may be estimated according to the following equation:

$$\beta_i = 1 + 12es_i/(a^2 + b^2) \qquad \text{(L.1.4-1)}$$

where: e——eccentricity of the mass center of the structure above the isolation story and the rigidity center of the isolation story, that orthogonal to the direction of seismic action.

s_i——the distance from the i-th isolator-unit to the rigidity center of the isolation story orthogonal to the direction of seismic action.

a, b——the length of two sides in the plane of the seismic-isolation story.

For side isolator-unit, its torsion-effect factor shall not be less than 1.15; when effective anti-torsion measures are taken for the seismic-isolation story and the structures above it, or when the torsion period is less than 70% of the fundamental period, the torsion-effect factor may be taken as 1.15.

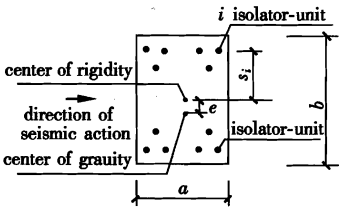

Figure L.1.4 sketch for torsion calculation

2 When the torsion of both way seismic actions are considered, the torsion-effect factor of the isolator-unit may be determined according to Equation (L.1.4-1), but the eccentricity value (e) in this equation shall be replaced by the greater value estimated according to the following equations:

$$e = \sqrt{e_x^2 + (0.85e_y)^2} \qquad \text{(L.1.4-2)}$$

$$e = \sqrt{e_y^2 + (0.85e_x)^2} \qquad \text{(L.1.4-3)}$$

where: e_x——eccentricity of seismic action along the y-axial direction.

e_y——eccentricity of seismic action along the x-axial direction.

For side isolator-unit, its torsion-effect factor shall not be less than 1.2.

L.1.5 When vertical seismic check for masonry structures is made according to the provisions in Clause 12.2.5 of this code, the normal-stress effect factors for the seismic shear strength of masonry should be determined according to the mean compressive stress deducted of the vertical seismic effect.

L.1.6 For the longitudinal or transversal beams at the top of the seismic-isolation story of the masonry structure, the design calculation may adopt the single-span simple support or the multi-span continuous beam under the uniformly distributed load. The uniformly distributed load values may be determined according to the provisions for reinforced concrete spandrel girder in Clause 7.2.5 of this code. When the mid-span bending moment computed by continuous beam is less than 0.8 times of that computed by the single-span simple support beam, the reinforcement shall be arranged according to 0.8 times of that computed by the single-span simple support beam.

L.2 The details requirement for seismic-isolation masonry structure

L.2.1 When the horizontal seismic-reduced factor of masonry structure assigned to Cate-

gory C is not more than 0.50, the limit value of number of stories, the total height and the height-width ratio may be adopted according to the provisions, which is corresponding to one grade lower, Section 7.1 of this code.

L.2.2 Details of seismic-isolation story of masonry structure shall conform to the following provisions:

1 When the seismic-isolation story of multi-story masonry building locates at the top of the basement, the isolator-unit should not be placed on the masonry wall directly, and the local compressive capacity of the masonry wall shall be checked too.

2 The detailing of the longitudinal and transversal beams at the top of the seismic-isolation story shall comply with the requirements for reinforced concrete spandrel girder of brick buildings with bottom-frame, which as per set in Clause 7.5.4 of this code.

L.2.3 For building assigned to Category C, the seismic design details for the masonry structure above the seismic-isolation story shall comply with the following requirements:

1 The minimum distance from a bearing exterior wall end to the side of the door or window opening, as well as the cross section and reinforcement detailing of the ring-beam, shall comply with relevant provisions in Sections 7.1 and 7.3 of this code.

2 The reinforced concrete tie-columns for multi-story clay brick and perforated brick buildings shall be installed as follows:

When the horizontal seismic-reduced factor is 0.75, it shall comply with the provisions in Table 7.3.1 of this code. For Intensity 7 to 9, when the horizontal seismic-reduced factor are 0.5 and 0.38, it shall comply with the provisions in Table L.2.3-1; and when such factor is 0.25, it should comply with provisions, which corresponding to one grade lower, in Table 7.3.1 of this code.

Table L.2.3-1 Requirements for tie-column installation of seismic-isolation brick building

<table>
<tr><td colspan="3">Number of stories in building</td><td colspan="2" rowspan="2">Location of installation</td></tr>
<tr><td>Intensity 7</td><td>Intensity 8</td><td>Intensity 9</td></tr>
<tr><td>3, 4</td><td>2, 3</td><td></td><td rowspan="4">Four corners of the exterior wall, staircase and elevator shaft; intersections of the transversal wall in the slit-level part and the exterior longitudinal wall; both sides of larger openings; inter-sections of interior wall and exterior longitudinal walls of large rooms</td><td>Intersections of each 15m or the unit transversal wall and exterior longitudinal wall</td></tr>
<tr><td>5</td><td>4</td><td>2</td><td>Intersection of each 3 bay transversal wall and exterior longitudinal wall</td></tr>
<tr><td>6,7</td><td>5</td><td>3,4</td><td>Intersections of every other transversal wall (axis) and exterior wall; intersections of gable and interior walls; intersections of exterior and interior walls for four stories and Intensity 9</td></tr>
<tr><td>8</td><td>6,7</td><td>5</td><td>Intersections of interior wall and exterior wall, smaller piers of the interior wall, Intersections of every other transversal wall (axis) and interior longitudinal wall for seven stories and Intensity 8;
Intersections of interior longitudinal and transversal wall (axis) for Intensity 9</td></tr>
</table>

3 The reinforced concrete core-columns for multi-story small-block buildings shall be installed as follows:

When the horizontal seismic-reduced factor is 0.75, it shall comply with the provisions in Table 7.4.1 of this code. For Intensity 7 to 9, when the horizontal seismic-reduced factor are 0.5 and 0.38, it shall comply with the provisions in Table L.2.3-2; and when such factor is 0.25, it should comply with provisions, which is corresponding to one grade lower, in Table 7.4.1 of this code.

Table L.2.3-2 Requirements for arrangement of core column of seismic-isolation small-block buildings

Number of stories			Location of core-columns	Number of core-columns (filled holes)
Intensity 7	Intensity 8	Intensity 9		
3, 4	2, 3		Corner of exterior wall, four corners of staircase, intersection of interior and exterior walls in large rooms, intersections of each 15m or the unit transversal wall and exterior longitudinal wall	Corners of the exterior wall, 3 holes shall be filled; intersection of interior and exterior walls, 4 holes
5	4	2	Corner of exterior wall, four corners of staircase, intersection of interior and exterior walls in large rooms, intersection of the interior wall and the gable, intersection of each 3 bay transversal wall (axis) and exterior longitudinal wall	
6	5	3	Corner of exterior wall, four corners of staircase, intersection of interior and exterior walls in large rooms, intersection of the interior wall and the gable, intersection of other bay transversal wall (axis) and exterior longitudinal wall; For Intensity 8 or 9, intersection of exterior longitudinal wall and transversal wall (axis), both sides of bigger openings	Corners of the exterior wall, 5 holes shall be filled; intersection of interior and exterior walls, 4 holes; intersection of interior walls, 4 or 5 holes; both sides of opening, 1 holes
7	6	4	Corner of exterior wall, four corners of staircase, intersection of all interior and exterior walls; For Intensity 8 or 9, intersection of interior longitudinal wall and transversal wall (axis), both sides of bigger openings	Corners of the exterior wall, 7 holes shall be filled; intersection of interior and exterior walls, 5 holes; intersection of interior walls, 4 or 5 holes; both sides of opening, 1 hole

4 Other seismic detailing requirements for structures above the seismic-isolation story shall conform to follows:

When the horizontal seismic-reduced factor is 0.75, it shall comply with the provisions in Chapter 7 of this code;

When the horizontal seismic-reduced factor is 0.5 or 0.38, it can comply with the provisions, which is corresponding to one grade lower for Intensity 7 to 9, in Chapter 7 of this code; and

When the horizontal seismic-reduced factor is 0.25, it may be two grades lower but not less than Intensity 6.

Explanation of words in this code

1 In order to treat different situations according to their individual conditions during the implementation of this code, words denoting the different degrees of strictness of demands are explained as follows:

1) Words denoting a very strict or mandatory requirement:

"must" is used for affirmation;

"must not" is used for negation.

2) Words denoting a strict requirement under normal conditions:

"shall" is used for affirmation;

"shall not" is used for negation.

3) Words denoting a permission of slight choice or an indication of the most suitable choice when conditions allow:

"should" is used for affirmation;

"should not" is used for negation.

And "may" denoting a permission of choice or an indication of the suitable choice when conditions allow.

2 "Be in accordance with" or "be compliance with" are used to indicate that it is necessary to implement items in this code according to other relative standards and stipulations.

中华人民共和国国家标准

建筑抗震设计规范

Code for seismic design of buildings

GB 50011—2001

主编部门：中华人民共和国建设部
批准部门：中华人民共和国建设部
施行日期：2002年1月1日

2001 北 京

关于发布国家标准
《建筑抗震设计规范》的通知

建标［2001］156号

根据我部《关于印发1997年工程建设标准制订、修订计划的通知》（建标［1997］108号）的要求，由建设部会同有关部门共同修订的《建筑抗震设计规范》，经有关部门会审，批准为国家标准，编号为GB50011—2001，自2002年1月1日起施行。其中，1.0.2、1.0.4、3.1.1、3.1.3、3.3.1、3.3.2、3.4.1、3.5.2、3.7.1、3.8.1、3.9.1、3.9.2、4.1.6、4.1.9、4.2.2、4.3.2、4.4.5、5.1.1、5.1.3、5.1.4、5.1.6、5.2.5、5.4.1、5.4.2、6.1.2、6.3.3、6.3.8、6.4.3、7.1.2、7.1.5、7.1.8、7.2.4、7.2.7、7.3.1、7.3.3、7.3.5、7.4.1、7.4.4、7.5.3、7.5.4、8.1.3、8.3.1、8.3.6、8.4.2、8.5.1、10.1.3、10.2.5、10.3.3、12.1.2、12.1.5、12.2.1、12.2.9为强制性条文，必须严格执行。原《建筑抗震设计规范》GBJ 11—89以及《工程建设国家标准局部修订公告》（第1号）于2002年12月31日废止。

本标准由建设部负责管理，中国建筑科学研究院负责具体解释工作，建设部标准定额研究所组织中国建筑工业出版社出版发行。

中华人民共和国建设部

2001年7月20日

前　言

本规范是根据建设部［1997］建标第108号文的要求，由中国建筑科学研究院会同有关的设计、勘察、研究和教学单位对《建筑抗震设计规范》GBJ 11—89进行修订而成。

修订过程中，开展了专题研究和部分试验研究，调查总结了近年来国内外大地震的经验教训，采纳了地震工程的新科研成果，考虑了我国的经济条件和工程实践，并在全国范围内广泛征求了有关设计、勘察、科研、教学单位及抗震管理部门的意见，经反复讨论、修改、充实和试设计，最后经审查定稿。

本次修订后共有13章11个附录，主要修订内容是：调整了建筑的抗震设防分类，提出了按设计基本地震加速度进行抗震设计的要求，将原规范的设计近、远震改为设计地震分组；修改了建筑场地划分、液化判别、地震影响系数和扭转效应计算的规定；增补了不规则建筑结构的概念设计、结构抗震分析、楼层地震剪力控制和抗震变形验算的要求；改进了砌体结构、混凝土结构、底部框架房屋的抗震措施；增加了有关发震断裂、桩基、混凝土筒体结构、钢结构房屋、配筋砌块房屋、非结构等抗震设计的内容以及房屋隔震、消能减震设计的规定。还取消了有关单排柱内框架房屋、中型砌块房屋及烟囱、水塔等构筑物的抗震设计规定。

本规范将来可能需要进行局部修订，有关局部修订的信息和条文内容将刊登在《工程建设标准化》杂志上。

本规范以黑体字标志的条文为强制性条文，必须严格执行。

本规范的具体解释由中国建筑科学研究院工程抗震研究所负责。在执行过程中，请各单位结合工程实践，认真总结经验，并将意见和建议寄交北京市北三环东路30号中国建筑科学研究院国家标准《建筑抗震设计规范》管理组（邮编：100013，E-mail：ieecabr @ public3.bta.net.cn）。

本规范的主编单位：中国建筑科学研究院

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1 总　　则

1.0.1 为贯彻执行《中华人民共和国建筑法》和《中华人民共和国防震减灾法》并实行以预防为主的方针，使建筑经抗震设防后，减轻建筑的地震破坏，避免人员伤亡，减少经济损失，制定本规范。

按本规范进行抗震设计的建筑，其抗震设防目标是：当遭受低于本地区抗震设防烈度的多遇地震影响时，一般不受损坏或不需修理可继续使用，当遭受相当于本地区抗震设防烈度的地震影响时，可能损坏，经一般修理或不需修理仍可继续使用，当遭受高于本地区抗震设防烈度预估的罕遇地震影响时，不致倒塌或发生危及生命的严重破坏。

1.0.2　抗震设防烈度为 6 度及以上地区的建筑，必须进行抗震设计。

1.0.3 本规范适用于抗震设防烈度为 6、7、8 和 9 度地区建筑工程的抗震设计及隔震、消能减震设计。抗震设防烈度大于 9 度地区的建筑和行业有特殊要求的工业建筑，其抗震设计应按有关专门规定执行。

注：本规范一般略去“抗震设防烈度”字样，如“抗震设防烈度为 6 度、7 度、8 度、9 度”，简称为“6 度、7 度、8 度、9 度”。

1.0.4　抗震设防烈度必须按国家规定的权限审批、颁发的文件（图件）确定。

1.0.5 一般情况下，抗震设防烈度可采用中国地震动参数区划图的地震基本烈度（或与本规范设计基本地震加速度值对应的烈度值）。对已编制抗震设防区划的城市，可按批准的抗震设防烈度或设计地震动参数进行抗震设防。

1.0.6 建筑的抗震设计，除应符合本规范要求外，尚应符合国家现行的有关强制性标准的规定。

2 术语和符号

2.1 术　语

2.1.1 抗震设防烈度　seismic fortification intensity

按国家规定的权限批准作为一个地区抗震设防依据的地震烈度。

2.1.2 抗震设防标准　seismic fortification criterion

衡量抗震设防要求的尺度，由抗震设防烈度和建筑使用功能的重要性确定。

2.1.3 地震作用　earthquake action

由地震动引起的结构动态作用，包括水平地震作用和竖向地震作用。

2.1.4 设计地震动参数　design parameters of ground motion

抗震设计用的地震加速度（速度、位移）时程曲线、加速度反应谱和峰值加速度。

2.1.5 设计基本地震加速度　design basic acceleration of ground motion

50年设计基准期超越概率10%的地震加速度的设计取值。

2.1.6 设计特征周期　design characteristic period of ground motion

抗震设计用的地震影响系数曲线中，反映地震震级、震中距和场地类别等因素的下降段起始点对应的周期值。

2.1.7 场地　site

工程群体所在地，具有相似的反应谱特征。其范围相当于厂区、居民小区和自然村或不小于1.0km^{2}的平面面积。

2.1.8 建筑抗震概念设计　seismic concept design of buildings

根据地震灾害和工程经验等所形成的基本设计原则和设计思想，进行建筑和结构总体布置并确定细部构造的过程。

2.1.9 抗震措施　seismic fortification measures

除地震作用计算和抗力计算以外的抗震设计内容，包括抗震构造措施。

2.1.10 抗震构造措施　details of seismic design

根据抗震概念设计原则，一般不需计算而对结构和非结构各部分必须采取的各种细部要求。

2.2 主 要 符 号

2.2.1 作用和作用效应

F_{Ek}、F_{Evk}——结构总水平、竖向地震作用标准值；

G_E、G_{eq}——地震时结构（构件）的重力荷载代表值、等效总重力荷载代表值；

w_k——风荷载标准值；

S_E——地震作用效应（弯矩、轴向力、剪力、应力和变形）；

S——地震作用效应与其他荷载效应的基本组合；

S_k——作用、荷载标准值的效应；

M——弯矩；

N——轴向压力；

V——剪力；

p——基础底面压力；

u——侧移；

θ——楼层位移角。

2.2.2 材料性能和抗力

K——结构（构件）的刚度；

R——结构构件承载力；

f、f_k、f_E——各种材料强度（含地基承载力）设计值、标准值和抗震设计值；

$[\theta]$——楼层位移角限值。

2.2.3 几何参数

A——构件截面面积；

A_s——钢筋截面面积；

B——结构总宽度；

H——结构总高度、柱高度；

L——结构（单元）总长度；

a——距离；

a_s、a'_s——纵向受拉钢筋合力点至截面边缘的最小距离；

b——构件截面宽度；

d——土层深度或厚度，钢筋直径；

h——计算楼层层高，构件截面高度；

l——构件长度或跨度；

t——抗震墙厚度、楼板厚度。

2.2.4 计算系数

α——水平地震影响系数；

α_{max}——水平地震影响系数最大值；

α_{vmax}——竖向地震影响系数最大值；

γ_G、γ_E、γ_w——作用分项系数；

γ_{RE}——承载力抗震调整系数；

ζ——计算系数；

η——地震作用效应（内力和变形）的增大或调整系数；

λ——构件长细比，比例系数；

ξ_y——结构（构件）屈服强度系数；

ρ——配筋率，比率；

φ——构件受压稳定系数；

ψ——组合值系数，影响系数。

2.2.5 其他

T——结构自振周期；

N——贯入锤击数；

I_{lE}——地震时地基的液化指数；

X_{ji}——位移振型坐标（j 振型 i 质点的 x 方向相对位移）；

Y_{ji}——位移振型坐标（j 振型 i 质点的 y 方向相对位移）；

n——总数，如楼层数、质点数、钢筋根数、跨数等；

v_{se}——土层等效剪切波速；

Φ_{ji}——转角振型坐标（j 振型 i 质点的转角方向相对位移）。

3 抗震设计的基本要求

3.1 建筑抗震设防分类和设防标准

3.1.1 **建筑应根据其使用功能的重要性分为甲类、乙类、丙类、丁类四个抗震设防类别。甲类建筑应属于重大建筑工程和地震时可能发生严重次生灾害的建筑，乙类建筑应属于地震时使用功能不能中断或需尽快恢复的建筑，丙类建筑应属于除甲、乙、丁类以外的一般建筑，丁类建筑应属于抗震次要建筑。**

3.1.2 建筑抗震设防类别的划分，应符合国家标准《建筑抗震设防分类标准》GB50223的规定。

3.1.3 **各抗震设防类别建筑的抗震设防标准，应符合下列要求：**

1 **甲类建筑，地震作用应高于本地区抗震设防烈度的要求，其值应按批准的地震安全性评价结果确定；抗震措施，当抗震设防烈度为6～8度时，应符合本地区抗震设防烈度提高一度的要求，当为9度时，应符合比9度抗震设防更高的要求。**

2 **乙类建筑，地震作用应符合本地区抗震设防烈度的要求；抗震措施，一般情况下，当抗震设防烈度为6～8度时，应符合本地区抗震设防烈度提高一度的要求，当为9度时，应符合比9度抗震设防更高的要求；地基基础的抗震措施，应符合有关规定。**

对较小的乙类建筑，当其结构改用抗震性能较好的结构类型时，应允许仍按本地区抗震设防烈度的要求采取抗震措施。

3 **丙类建筑，地震作用和抗震措施均应符合本地区抗震设防烈度的要求。**

4 **丁类建筑，一般情况下，地震作用仍应符合本地区抗震设防烈度的要求；抗震措施应允许比本地区抗震设防烈度的要求适当降低，但抗震设防烈度为6度时不应降低。**

3.1.4 抗震设防烈度为6度时，除本规范有具体规定外，对乙、丙、丁类建筑可不进行地震作用计算。

3.2 地 震 影 响

3.2.1 建筑所在地区遭受的地震影响，应采用相应于抗震设防烈度的设计基本地震加速度和设计特征周期或本规范第1.0.5条规定的设计地震动参数来表征。

3.2.2 抗震设防烈度和设计基本地震加速度取值的对应关系，应符合表3.2.2的规定。设计基本地震加速度为0.15g和0.30g地区内的建筑，除本规范另有规定外，应分别按抗震设防烈度7度和8度的要求进行抗震设计。

表3.2.2 抗震设防烈度和设计基本地震加速度值的对应关系

抗震设防烈度	6	7	8	9
设计基本地震加速度值	0.05g	0.10（0.15）g	0.20（0.30）g	0.40g

注：g为重力加速度。

3.2.3 建筑的设计特征周期应根据其所在地的设计地震分组和场地类别确定。本规范的设计地震共分为三组。对Ⅱ类场地，第一组、第二组和第三组的设计特征周期，应分别按0.35s、0.40s和0.45s采用。

注：本规范一般把“设计特征周期”简称为“特征周期”。

3.2.4 我国主要城镇（县级及县级以上城镇）中心地区的抗震设防烈度、设计基本地震加速度值和所属的设计地震分组，可按本规范附录A采用。

3.3 场地和地基

3.3.1 选择建筑场地时，应根据工程需要，掌握地震活动情况、工程地质和地震地质的有关资料，对抗震有利、不利和危险地段作出综合评价。对不利地段，应提出避开要求；当无法避开时应采取有效措施；不应在危险地段建造甲、乙、丙类建筑。

3.3.2 建筑场地为Ⅰ类时，甲、乙类建筑应允许仍按本地区抗震设防烈度的要求采取抗震构造措施；丙类建筑应允许按本地区抗震设防烈度降低一度的要求采取抗震构造措施，但抗震设防烈度为6度时仍应按本地区抗震设防烈度的要求采取抗震构造措施。

3.3.3 建筑场地为Ⅲ、Ⅳ类时，对设计基本地震加速度为0.15g和0.30g的地区，除本规范另有规定外，宜分别按抗震设防烈度8度（0.20g）和9度（0.40g）时各类建筑的要求采取抗震构造措施。

3.3.4 地基和基础设计应符合下列要求：

1 同一结构单元的基础不宜设置在性质截然不同的地基上；

2 同一结构单元不宜部分采用天然地基部分采用桩基；

3 地基为软弱粘性土、液化土、新近填土或严重不均匀土时，应估计地震时地基不均匀沉降或其他不利影响，并采取相应的措施。

3.4 建筑设计和建筑结构的规则性

3.4.1 建筑设计应符合抗震概念设计的要求，不应采用严重不规则的设计方案。

3.4.2 建筑及其抗侧力结构的平面布置宜规则、对称，并应具有良好的整体性；建筑的立面和竖向剖面宜规则，结构的侧向刚度宜均匀变化，竖向抗侧力构件的截面尺寸和材料强度宜自下而上逐渐减小，避免抗侧力结构的侧向刚度和承载力突变。

当存在表3.4.2-1所列举的平面不规则类型或表3.4.2-2所列举的竖向不规则类型时，应符合本章第3.4.3条的有关规定。

表3.4.2-1 平面不规则的类型

不规则类型	定　义
扭转不规则	楼层的最大弹性水平位移（或层间位移），大于该楼层两端弹性水平位移（或层间位移）平均值的1.2倍
凹凸不规则	结构平面凹进的一侧尺寸，大于相应投影方向总尺寸的30%
楼板局部不连续	楼板的尺寸和平面刚度急剧变化，例如，有效楼板宽度小于该层楼板典型宽度的50%，或开洞面积大于该层楼面面积的30%，或较大的楼层错层

表 3.4.2-2　竖向不规则的类型

不规则类型	定　　义
侧向刚度不规则	该层的侧向刚度小于相邻上一层的 70%，或小于其上相邻三个楼层侧向刚度平均值的 80%；除顶层外，局部收进的水平向尺寸大于相邻下一层的 25%
竖向抗侧力构件不连续	竖向抗侧力构件（柱、抗震墙、抗震支撑）的内力由水平转换构件（梁、桁架等）向下传递
楼层承载力突变	抗侧力结构的层间受剪承载力小于相邻上一楼层的 80%

3.4.3　不规则的建筑结构，应按下列要求进行水平地震作用计算和内力调整，并应对薄弱部位采取有效的抗震构造措施：

1　平面不规则而竖向规则的建筑结构，应采用空间结构计算模型，并应符合下列要求：

1）扭转不规则时，应计及扭转影响，且楼层竖向构件最大的弹性水平位移和层间位移分别不宜大于楼层两端弹性水平位移和层间位移平均值的 1.5 倍；

2）凹凸不规则或楼板局部不连续时，应采用符合楼板平面内实际刚度变化的计算模型，当平面不对称时尚应计及扭转影响。

2　平面规则而竖向不规则的建筑结构，应采用空间结构计算模型，其薄弱层的地震剪力应乘以 1.15 的增大系数，应按本规范有关规定进行弹塑性变形分析，并应符合下列要求：

1）竖向抗侧力构件不连续时，该构件传递给水平转换构件的地震内力应乘以 1.25～1.5 的增大系数；

2）楼层承载力突变时，薄弱层抗侧力结构的受剪承载力不应小于相邻上一楼层的 65%。

3　平面不规则且竖向不规则的建筑结构，应同时符合本条 1、2 款的要求。

3.4.4　砌体结构和单层工业厂房的平面不规则性和竖向不规则性，应分别符合本规范有关章节的规定。

3.4.5　体型复杂、平立面特别不规则的建筑结构，可按实际需要在适当部位设置防震缝，形成多个较规则的抗侧力结构单元。

3.4.6　防震缝应根据抗震设防烈度、结构材料种类、结构类型、结构单元的高度和高差情况，留有足够的宽度，其两侧的上部结构应完全分开。

当设置伸缩缝和沉降缝时，其宽度应符合防震缝的要求。

3.5　结　构　体　系

3.5.1　结构体系应根据建筑的抗震设防类别、抗震设防烈度、建筑高度、场地条件、地基、结构材料和施工等因素，经技术、经济和使用条件综合比较确定。

3.5.2　结构体系应符合下列各项要求：

1　应具有明确的计算简图和合理的地震作用传递途径。

2　应避免因部分结构或构件破坏而导致整个结构丧失抗震能力或对重力荷载的承载能力。

3 应具备必要的抗震承载力、良好的变形能力和消耗地震能量的能力。

4 对可能出现的薄弱部位，应采取措施提高抗震能力。

3.5.3 结构体系尚宜符合下列各项要求：

1 宜有多道抗震防线。

2 宜具有合理的刚度和承载力分布，避免因局部削弱或突变形成薄弱部位，产生过大的应力集中或塑性变形集中。

3 结构在两个主轴方向的动力特性宜相近。

3.5.4 结构构件应符合下列要求：

1 砌体结构应按规定设置钢筋混凝土圈梁和构造柱、芯柱，或采用配筋砌体等。

2 混凝土结构构件应合理地选择尺寸、配置纵向受力钢筋和箍筋，避免剪切破坏先于弯曲破坏、混凝土的压溃先于钢筋的屈服、钢筋的锚固粘结破坏先于构件破坏。

3 预应力混凝土的抗侧力构件，应配有足够的非预应力钢筋。

4 钢结构构件应合理控制尺寸，避免局部失稳或整个构件失稳。

3.5.5 结构各构件之间的连接，应符合下列要求：

1 构件节点的破坏，不应先于其连接的构件。

2 预埋件的锚固破坏，不应先于连接件。

3 装配式结构构件的连接，应能保证结构的整体性。

4 预应力混凝土构件的预应力钢筋，宜在节点核心区以外锚固。

3.5.6 装配式单层厂房的各种抗震支撑系统，应保证地震时结构的稳定性。

3.6 结 构 分 析

3.6.1 除本规范特别规定者外，建筑结构应进行多遇地震作用下的内力和变形分析，此时，可假定结构与构件处于弹性工作状态，内力和变形分析可采用线性静力方法或线性动力方法。

3.6.2 不规则且具有明显薄弱部位可能导致地震时严重破坏的建筑结构，应按本规范有关规定进行罕遇地震作用下的弹塑性变形分析。此时，可根据结构特点采用静力弹塑性分析或弹塑性时程分析方法。

当本规范有具体规定时，尚可采用简化方法计算结构的弹塑性变形。

3.6.3 当结构在地震作用下的重力附加弯矩大于初始弯矩的10%时，应计入重力二阶效应的影响。

注：重力附加弯矩指任一楼层以上全部重力荷载与该楼层地震层间位移的乘积；初始弯矩指该楼层地震剪力与楼层层高的乘积。

3.6.4 结构抗震分析时，应按照楼、屋盖在平面内变形情况确定为刚性、半刚性和柔性的横隔板，再按抗侧力系统的布置确定抗侧力构件间的共同工作并进行各构件间的地震内力分析。

3.6.5 质量和侧向刚度分布接近对称且楼、屋盖可视为刚性横隔板的结构，以及本规范有关章节有具体规定的结构，可采用平面结构模型进行抗震分析。其他情况，应采用空间结构模型进行抗震分析。

3.6.6 利用计算机进行结构抗震分析，应符合下列要求：

1 计算模型的建立、必要的简化计算与处理，应符合结构的实际工作状况。

2 计算软件的技术条件应符合本规范及有关标准的规定，并应阐明其特殊处理的内容和依据。

3 复杂结构进行多遇地震作用下的内力和变形分析时，应采用不少于两个不同的力学模型，并对其计算结果进行分析比较。

4 所有计算机计算结果，应经分析判断确认其合理、有效后方可用于工程设计。

3.7 非结构构件

3.7.1 非结构构件，包括建筑非结构构件和建筑附属机电设备，自身及其与结构主体的连接，应进行抗震设计。

3.7.2 非结构构件的抗震设计，应由相关专业人员分别负责进行。

3.7.3 附着于楼、屋面结构上的非结构构件，应与主体结构有可靠的连接或锚固，避免地震时倒塌伤人或砸坏重要设备。

3.7.4 围护墙和隔墙应考虑对结构抗震的不利影响，避免不合理设置而导致主体结构的破坏。

3.7.5 幕墙、装饰贴面与主体结构应有可靠连接，避免地震时脱落伤人。

3.7.6 安装在建筑上的附属机械、电气设备系统的支座和连接，应符合地震时使用功能的要求，且不应导致相关部件的损坏。

3.8 隔震和消能减震设计

3.8.1 隔震和消能减震设计，应主要应用于使用功能有特殊要求的建筑及抗震设防烈度为 8、9 度的建筑。

3.8.2 采用隔震或消能减震设计的建筑，当遭遇到本地区的多遇地震影响、抗震设防烈度地震影响和罕遇地震影响时，其抗震设防目标应高于本规范第 1.0.1 条的规定。

3.9 结构材料与施工

3.9.1 抗震结构对材料和施工质量的特别要求，应在设计文件上注明。

3.9.2 结构材料性能指标，应符合下列最低要求：

1 砌体结构材料应符合下列规定：

1）烧结普通粘土砖和烧结多孔粘土砖的强度等级不应低于 MU10，其砌筑砂浆强度等级不应低于 M5；

2）混凝土小型空心砌块的强度等级不应低于 MU7.5，其砌筑砂浆强度等级不应低于 M7.5。

2 混凝土结构材料应符合下列规定：

1）混凝土的强度等级，框支梁、框支柱及抗震等级为一级的框架梁、柱、节点核芯区，不应低于 C30；构造柱、芯柱、圈梁及其他各类构件不应低于 C20；

2）抗震等级为一、二级的框架结构，其纵向受力钢筋采用普通钢筋时，钢筋的抗拉强度实测值与屈服强度实测值的比值不应小于 1.25；且钢筋的屈服强度实测值与强度标准值的比值不应大于 1.3。

3 钢结构的钢材应符合下列规定：

1）钢材的抗拉强度实测值与屈服强度实测值的比值不应小于 1.2；

2）钢材应有明显的屈服台阶，且伸长率应大于 20%；

3）钢材应有良好的可焊性和合格的冲击韧性。

3.9.3 结构材料性能指标，尚宜符合下列要求：

1 普通钢筋宜优先采用延性、韧性和可焊性较好的钢筋；普通钢筋的强度等级，纵向受力钢筋宜选用 HRB400 级和 HRB335 级热轧钢筋，箍筋宜选用 HRB335、HRB400 和 HPB235 级热轧钢筋。

注：钢筋的检验方法应符合现行国家标准《混凝土结构工程施工及验收规范》GB50204 的规定。

2 混凝土结构的混凝土强度等级，9 度时不宜超过 C60，8 度时不宜超过 C70。

3 钢结构的钢材宜采用 Q235 等级 B、C、D 的碳素结构钢及 Q345 等级 B、C、D、E 的低合金高强度结构钢；当有可靠依据时，尚可采用其他钢种和钢号。

3.9.4 在施工中，当需要以强度等级较高的钢筋替代原设计中的纵向受力钢筋时，应按照钢筋受拉承载力设计值相等的原则换算，并应满足正常使用极限状态和抗震构造措施的要求。

3.9.5 采用焊接连接的钢结构，当钢板厚不小于 40mm 且承受沿板厚方向的拉力时，受拉试件板厚方向截面收缩率，不应小于国家标准《厚度方向性能钢板》GB50313 关于 Z15 级规定的容许值。

3.9.6 钢筋混凝土构造柱、芯柱和底部框架-抗震墙砖房中砖抗震墙的施工，应先砌墙后浇构造柱、芯柱和框架梁柱。

3.10 建筑的地震反应观测系统

3.10.1 抗震设防烈度为 7、8、9 度时，高度分别超过 160m，120m，80m 的高层建筑，应设置建筑结构的地震反应观测系统，建筑设计应留有观测仪器和线路的位置。

4 场地、地基和基础

4.1 场　　地

4.1.1 选择建筑场地时，应按表 4.1.1 划分对建筑抗震有利、不利和危险的地段。

表 4.1.1 有利、不利和危险地段的划分

地段类别	地质、地形、地貌
有利地段	稳定基岩，坚硬土，开阔、平坦、密实、均匀的中硬土等
不利地段	软弱土，液化土，条状突出的山嘴，高耸孤立的山丘，非岩质的陡坡，河岸和边坡的边缘，平面分布上成因、岩性、状态明显不均匀的土层（如故河道、疏松的断层破碎带、暗埋的塘浜沟谷和半填半挖地基）等
危险地段	地震时可能发生滑坡、崩塌、地陷、地裂、泥石流等及发震断裂带上可能发生地表位错的部位

4.1.2 建筑场地的类别划分，应以土层等效剪切波速和场地覆盖层厚度为准。

4.1.3 土层剪切波速的测量，应符合下列要求：

1 在场地初步勘察阶段，对大面积的同一地质单元，测量土层剪切波速的钻孔数量，应为控制性钻孔数量的 1/3～1/5，山间河谷地区可适量减少，但不宜少于 3 个。

2 在场地详细勘察阶段，对单幢建筑，测量土层剪切波速的钻孔数量不宜少于 2 个，数据变化较大时，可适量增加；对小区中处于同一地质单元的密集高层建筑群，测量土层剪切波速的钻孔数量可适量减少，但每幢高层建筑下不得少于一个。

3 对丁类建筑及层数不超过 10 层且高度不超过 30m 的丙类建筑，当无实测剪切波速时，可根据岩土名称和性状，按表 4.1.3 划分土的类型，再利用当地经验在表 4.1.3 的剪切波速范围内估计各土层的剪切波速。

表 4.1.3 土的类型划分和剪切波速范围

土的类型	岩土名称和性状	土层剪切波速范围（m/s）
坚硬土或岩石	稳定岩石，密实的碎石土	$v_s>500$
中硬土	中密、稍密的碎石土，密实、中密的砾、粗、中砂，$f_{ak}>200$ 的粘性土和粉土，坚硬黄土	$500\geqslant v_s>250$
中软土	稍密的砾、粗、中砂，除松散外的细、粉砂，$f_{ak}\leqslant 200$ 的粘性土和粉土，$f_{ak}>130$ 的填土，可塑黄土	$250\geqslant v_s>140$
软弱土	淤泥和淤泥质土，松散的砂，新近沉积的粘性土和粉土，$f_{ak}\leqslant 130$ 的填土，流塑黄土	$v_s\leqslant 140$

注：f_{ak}为由载荷试验等方法得到的地基承载力特征值（kPa）；v_s为岩土剪切波速。

4.1.4 建筑场地覆盖层厚度的确定，应符合下列要求：

1 一般情况下，应按地面至剪切波速大于 500m/s 的土层顶面的距离确定。

2 当地面 5m 以下存在剪切波速大于相邻上层土剪切波速 2.5 倍的土层，且其下卧岩土的剪切波速均不小于 400m/s 时，可按地面至该土层顶面的距离确定。

3 剪切波速大于 500m/s 的孤石、透镜体，应视同周围土层。

4 土层中的火山岩硬夹层，应视为刚体，其厚度应从覆盖土层中扣除。

4.1.5 土层的等效剪切波速，应按下列公式计算：

$$v_{se}=d_0/t \tag{4.1.5-1}$$

$$t=\sum_{i=1}^{n}(d_i/v_{si}) \tag{4.1.5-2}$$

式中 v_{se}——土层等效剪切波速（m/s）；

d_0——计算深度(m)，取覆盖层厚度和20m二者的较小值；

t——剪切波在地面至计算深度之间的传播时间；

d_i——计算深度范围内第 i 土层的厚度（m）；

v_{si}——计算深度范围内第 i 土层的剪切波速（m/s）；

n——计算深度范围内土层的分层数。

4.1.6 建筑的场地类别，应根据土层等效剪切波速和场地覆盖层厚度按表4.1.6划分为四类。当有可靠的剪切波速和覆盖层厚度且其值处于表4.1.6所列场地类别的分界线附近时，应允许按插值方法确定地震作用计算所用的设计特征周期。

表4.1.6 各类建筑场地的覆盖层厚度（m）

等效剪切波速 (m/s)	场地类别			
	Ⅰ	Ⅱ	Ⅲ	Ⅳ
$v_{se}>500$	0			
$500\geqslant v_{se}>250$	<5	≥5		
$250\geqslant v_{se}>140$	<3	3~50	>50	
$v_{se}\leqslant 140$	<3	3~15	>15~80	>80

4.1.7 场地内存在发震断裂时，应对断裂的工程影响进行评价，并应符合下列要求：

1 对符合下列规定之一的情况，可忽略发震断裂错动对地面建筑的影响：

1）抗震设防烈度小于8度；

2）非全新世活动断裂；

3）抗震设防烈度为8度和9度时，前第四纪基岩隐伏断裂的土层覆盖厚度分别大于60m和90m。

2 对不符合本条1款规定的情况，应避开主断裂带。其避让距离不宜小于表4.1.7对发震断裂最小避让距离的规定。

表4.1.7 发震断裂的最小避让距离（m）

烈度	建筑抗震设防类别			
	甲	乙	丙	丁
8	专门研究	300m	200m	—
9	专门研究	500m	300m	—

4.1.8 当需要在条状突出的山嘴、高耸孤立的山丘、非岩石的陡坡、河岸和边坡边缘等不利地段建造丙类及丙类以上建筑时，除保证其在地震作用下的稳定性外，尚应估计不利地段对设计地震动参数可能产生的放大作用，其地震影响系数最大值应乘以增大系数。其

值可根据不利地段的具体情况确定，但不宜大于1.6。

4.1.9 场地岩土工程勘察，应根据实际需要划分对建筑有利、不利和危险的地段，提供建筑的场地类别和岩土地震稳定性（如滑坡、崩塌、液化和震陷特性等）评价，对需要采用时程分析法补充计算的建筑，尚应根据设计要求提供土层剖面、场地覆盖层厚度和有关的动力参数。

4.2 天然地基和基础

4.2.1 下列建筑可不进行天然地基及基础的抗震承载力验算：

1 砌体房屋。

2 地基主要受力层范围内不存在软弱粘性土层的下列建筑：

1）一般的单层厂房和单层空旷房屋；

2）不超过8层且高度在25m以下的一般民用框架房屋；

3）基础荷载与2）项相当的多层框架厂房。

3 本规范规定可不进行上部结构抗震验算的建筑。

注：软弱粘性土层指7度、8度和9度时，地基承载力特征值分别小于80、100和120kPa的土层。

4.2.2 天然地基基础抗震验算时，应采用地震作用效应标准组合，且地基抗震承载力应取地基承载力特征值乘以地基抗震承载力调整系数计算。

4.2.3 地基抗震承载力应按下式计算：

$$f_{aE}=\zeta_a f_a \tag{4.2.3}$$

式中 f_{aE}——调整后的地基抗震承载力；

ζ_a——地基抗震承载力调整系数，应按表4.2.3采用；

f_a——深宽修正后的地基承载力特征值，应按现行国家标准《建筑地基基础设计规范》GB50007采用。

表4.2.3 地基土抗震承载力调整系数

岩土名称和性状	ζ_a
岩石，密实的碎石土，密实的砾、粗、中砂，$f_{ak}\geqslant 300$的粘性土和粉土	1.5
中密、稍密的碎石土，中密和稍密的砾、粗、中砂，密实和中密的细、粉砂，$150\leqslant f_{ak}<300$的粘性土和粉土，坚硬黄土	1.3
稍密的细、粉砂，$100\leqslant f_{ak}<150$的粘性土和粉土，可塑黄土	1.1
淤泥，淤泥质土，松散的砂，杂填土，新近堆积黄土及流塑黄土	1.0

4.2.4 验算天然地基地震作用下的竖向承载力时，按地震作用效应标准组合的基础底面平均压力和边缘最大压力应符合下列各式要求：

$$p\leqslant f_{aE} \tag{4.2.4-1}$$

$$p_{max}\leqslant 1.2 f_{aE} \tag{4.2.4-2}$$

式中 p——地震作用效应标准组合的基础底面平均压力；

p_{max}——地震作用效应标准组合的基础边缘的最大压力。

高宽比大于4的高层建筑，在地震作用下基础底面不宜出现拉应力；其他建筑，基础底面与地基土之间零应力区面积不应超过基础底面面积的15%。

4.3 液化土和软土地基

4.3.1 饱和砂土和饱和粉土（不含黄土）的液化判别和地基处理，6度时，一般情况下可不进行判别和处理，但对液化沉陷敏感的乙类建筑可按7度的要求进行判别和处理，7～9度时，乙类建筑可按本地区抗震设防烈度的要求进行判别和处理。

4.3.2 存在饱和砂土和饱和粉土（不含黄土）的地基，除6度设防外，应进行液化判别；存在液化土层的地基，应根据建筑的抗震设防类别、地基的液化等级，结合具体情况采取相应的措施。

4.3.3 饱和的砂土或粉土（不含黄土），当符合下列条件之一时，可初步判别为不液化或可不考虑液化影响：

1 地质年代为第四纪晚更新世（Q_3）及其以前时，7、8度时可判为不液化。

2 粉土的粘粒（粒径小于0.005mm的颗粒）含量百分率，7度、8度和9度分别不小于10、13和16时，可判为不液化土。

注：用于液化判别的粘粒含量系采用六偏磷酸钠作分散剂测定，采用其他方法时应按有关规定换算。

3 天然地基的建筑，当上覆非液化土层厚度和地下水位深度符合下列条件之一时，可不考虑液化影响：

$$d_u > d_0 + d_b - 2 \tag{4.3.3-1}$$

$$d_w > d_0 + d_b - 3 \tag{4.3.3-2}$$

$$d_u + d_w > 1.5d_0 + 2d_b - 4.5 \tag{4.3.3-3}$$

式中 d_w——地下水位深度（m），宜按设计基准期内年平均最高水位采用，也可按近期内年最高水位采用；

d_u——上覆盖非液化土层厚度（m），计算时宜将淤泥和淤泥质土层扣除；

d_b——基础埋置深度（m），不超过2m时应采用2m；

d_0——液化土特征深度（m），可按表4.3.3采用。

表4.3.3 液化土特征深度（m）

饱和土类别	7度	8度	9度
粉　土	6	7	8
砂　土	7	8	9

4.3.4 当初步判别认为需进一步进行液化判别时，应采用标准贯入试验判别法判别地面下15m深度范围内的液化；当采用桩基或埋深大于5m的深基础时，尚应判别15～20m范围内土的液化。当饱和土标准贯入锤击数（未经杆长修正）小于液化判别标准贯入锤击数临界值时，应判为液化土。当有成熟经验时，尚可采用其他判别方法。

在地面下15m深度范围内，液化判别标准贯入锤击数临界值可按下式计算：

$$N_{cr} = N_0 \left[0.9 + 0.1 (d_s - d_w)\right] \sqrt{3/\rho_c} \quad (d_s \leqslant 15) \tag{4.3.4-1}$$

在地面下15～20m范围内，液化判别标准贯入锤击数临界值可按下式计算：

$$N_{cr} = N_0 (2.4 - 0.1d_w) \sqrt{3/\rho_c} \quad (15 \leqslant d_s \leqslant 20) \tag{4.3.4-2}$$

式中 N_{cr}——液化判别标准贯入锤击数临界值；

N_0——液化判别标准贯入锤击数基准值，应按表 4.3.4 采用；

d_s——饱和土标准贯入点深度（m）；

ρ_c——粘粒含量百分率，当小于 3 或为砂土时，应采用 3。

表 4.3.4　标准贯入锤击数基准值

设计地震分组	7 度	8 度	9 度
第一组	6 (8)	10 (13)	16
第二、三组	8 (10)	12 (15)	18

注：括号内数值用于设计基本地震加速度为 0.15g 和 0.30g 的地区。

4.3.5　对存在液化土层的地基，应探明各液化土层的深度和厚度，按下式计算每个钻孔的液化指数，并按表 4.3.5 综合划分地基的液化等级：

$$I_{lE} = \sum_{i=1}^{n} \left(1 - \frac{N_i}{N_{cri}}\right) d_i W_i \tag{4.3.5}$$

式中　I_{lE}——液化指数；

n——在判别深度范围内每一个钻孔标准贯入试验点的总数；

N_i、N_{cri}——分别为 i 点标准贯入锤击数的实测值和临界值，当实测值大于临界值时应取临界值的数值；

d_i——i 点所代表的土层厚度（m），可采用与该标准贯入试验点相邻的上、下两标准贯入试验点深度差的一半，但上界不高于地下水位深度，下界不深于液化深度；

W_i——i 土层单位土层厚度的层位影响权函数值（单位为 m^{-1}）。若判别深度为 15m，当该层中点深度不大于 5m 时应采用 10，等于 15m 时应采用零值，5～15m 时应按线性内插法取值；若判别深度为 20m，当该层中点深度不大于 5m 时应采用 10，等于 20m 时应采用零值，5～20m 时应按线性内插法取值。

表 4.3.5　液　化　等　级

液化等级	轻　微	中　等	严　重
判别深度为 15m 时的液化指数	$0<I_{lE}\leqslant 5$	$5<I_{lE}\leqslant 15$	$I_{lE}>15$
判别深度为 20m 时的液化指数	$0<I_{lE}\leqslant 6$	$6<I_{lE}\leqslant 18$	$I_{lE}>18$

4.3.6　当液化土层较平坦且均匀时，宜按表 4.3.6 选用地基抗液化措施；尚可计入上部

表 4.3.6　抗 液 化 措 施

建筑抗震设防类别	地基的液化等级		
	轻　微	中　等	严　重
乙　类	部分消除液化沉陷，或对基础和上部结构处理	全部消除液化沉陷，或部分消除液化沉陷且对基础和上部结构处理	全部消除液化沉陷
丙　类	基础和上部结构处理，亦可不采取措施	基础和上部结构处理，或更高要求的措施	全部消除液化沉陷，或部分消除液化沉陷且对基础和上部结构处理
丁　类	可不采取措施	可不采取措施	基础和上部结构处理，或其他经济的措施

结构重力荷载对液化危害的影响，根据液化震陷量的估计适当调整抗液化措施。

不宜将未经处理的液化土层作为天然地基持力层。

4.3.7 全部消除地基液化沉陷的措施，应符合下列要求：

1 采用桩基时，桩端伸入液化深度以下稳定土层中的长度（不包括桩尖部分），应按计算确定，且对碎石土，砾、粗、中砂，坚硬粘性土和密实粉土尚不应小于0.5m，对其他非岩石土尚不宜小于1.5m。

2 采用深基础时，基础底面应埋入液化深度以下的稳定土层中，其深度不应小于0.5m。

3 采用加密法（如振冲、振动加密、挤密碎石桩、强夯等）加固时，应处理至液化深度下界；振冲或挤密碎石桩加固后，桩间土的标准贯入锤击数不宜小于本节第4.3.4条规定的液化判别标准贯入锤击数临界值。

4 用非液化土替换全部液化土层。

5 采用加密法或换土法处理时，在基础边缘以外的处理宽度，应超过基础底面下处理深度的1/2且不小于基础宽度的1/5。

4.3.8 部分消除地基液化沉陷的措施，应符合下列要求：

1 处理深度应使处理后的地基液化指数减少，当判别深度为15m时，其值不宜大于4，当判别深度为20m时，其值不宜大于5；对独立基础和条形基础，尚不应小于基础底面下液化土特征深度和基础宽度的较大值。

2 采用振冲或挤密碎石桩加固后，桩间土的标准贯入锤击数不宜小于按本节第4.3.4条规定的液化判别标准贯入锤击数临界值。

3 基础边缘以外的处理宽度，应符合本节第4.3.7条5款的要求。

4.3.9 减轻液化影响的基础和上部结构处理，可综合采用下列各项措施：

1 选择合适的基础埋置深度。

2 调整基础底面积，减少基础偏心。

3 加强基础的整体性和刚度，如采用箱基、筏基或钢筋混凝土交叉条形基础，加设基础圈梁等。

4 减轻荷载，增强上部结构的整体刚度和均匀对称性，合理设置沉降缝，避免采用对不均匀沉降敏感的结构形式等。

5 管道穿过建筑处应预留足够尺寸或采用柔性接头等。

4.3.10 液化等级为中等液化和严重液化的故河道、现代河滨、海滨，当有液化侧向扩展或流滑可能时，在距常时水线约100m以内不宜修建永久性建筑，否则应进行抗滑动验算，采取防土体滑动措施或结构抗裂措施。

注：常时水线宜按设计基准期内年平均最高水位采用，也可按近期年最高水位采用。

4.3.11 地基主要受力层范围内存在软弱粘性土层与湿陷性黄土时，应结合具体情况综合考虑，采用桩基、地基加固处理或本节第4.3.9条的各项措施，也可根据软土震陷量的估计，采取相应措施。

4.4 桩　　基

4.4.1 承受竖向荷载为主的低承台桩基，当地面下无液化土层，且桩承台周围无淤泥、

淤泥质土和地基承载力特征值不大于100kPa的填土时，下列建筑可不进行桩基抗震承载力验算：

1 本章第4.2.1条之1、3款规定的建筑；

2 7度和8度时的下列建筑：

1）一般的单层厂房和单层空旷房屋；

2）不超过8层且高度在25m以下的一般民用框架房屋；

3）基础荷载与2）项相当的多层框架厂房。

4.4.2 非液化土中低承台桩基的抗震验算，应符合下列规定：

1 单桩的竖向和水平向抗震承载力特征值，可均比非抗震设计时提高25%。

2 当承台周围的回填土夯实至干密度不小于《建筑地基基础设计规范》对填土的要求时，可由承台正面填土与桩共同承担水平地震作用；但不应计入承台底面与地基土间的摩擦力。

4.4.3 存在液化土层的低承台桩基抗震验算，应符合下列规定：

1 对一般浅基础，不宜计入承台周围土的抗力或刚性地坪对水平地震作用的分担作用。

2 当桩承台底面上、下分别有厚度不小于1.5m、1.0m的非液化土层或非软弱土层时，可按下列二种情况进行桩的抗震验算，并按不利情况设计：

1）桩承受全部地震作用，桩承载力按本节第4.4.2条取用，液化土的桩周摩阻力及桩水平抗力均应乘以表4.4.3的折减系数。

表4.4.3 土层液化影响折减系数

实际标贯锤击数/临界标贯锤击数	深度 d_s（m）	折减系数
≤0.6	$d_s \leqslant 10$	0
	$10 < d_s \leqslant 20$	1/3
>0.6~0.8	$d_s \leqslant 10$	1/3
	$10 < d_s \leqslant 20$	2/3
>0.8~1.0	$d_s \leqslant 10$	2/3
	$10 < d_s \leqslant 20$	1

2）地震作用按水平地震影响系数最大值的10%采用，桩承载力仍按本节第4.4.2条1款取用，但应扣除液化土层的全部摩阻力及桩承台下2m深度范围内非液化土的桩周摩阻力。

3 打入式预制桩及其他挤土桩，当平均桩距为2.5~4倍桩径且桩数不少于5×5时，可计入打桩对土的加密作用及桩身对液化土变形限制的有利影响。当打桩后桩间土的标准贯入锤击数值达到不液化的要求时，单桩承载力可不折减，但对桩尖持力层作强度校核时，桩群外侧的应力扩散角应取为零。打桩后桩间土的标准贯入锤击数宜由试验确定，也可按下式计算：

$$N_1 = N_p + 100\rho\ (1 - e^{-0.3N_p}) \tag{4.4.3}$$

式中 N_1——打桩后的标准贯入锤击数；

ρ——打入式预制桩的面积置换率；

N_p——打桩前的标准贯入锤击数。

4.4.4 处于液化土中的桩基承台周围，宜用非液化土填筑夯实，若用砂土或粉土则应使

土层的标准贯入锤击数不小于本章第 4.3.4 条规定的液化判别标准贯入锤击数临界值。

4.4.5　液化土中桩的配筋范围，应自桩顶至液化深度以下符合全部消除液化沉陷所要求的深度，其纵向钢筋应与桩顶部相同，箍筋应加密。

4.4.6　在有液化侧向扩展的地段，距常时水线 100m 范围内的桩基除应满足本节中的其他规定外，尚应考虑土流动时的侧向作用力，且承受侧向推力的面积应按边桩外缘间的宽度计算。

5 地震作用和结构抗震验算

5.1 一 般 规 定

5.1.1 各类建筑结构的地震作用，应符合下列规定：

1 一般情况下，应允许在建筑结构的两个主轴方向分别计算水平地震作用并进行抗震验算，各方向的水平地震作用应由该方向抗侧力构件承担。

2 有斜交抗侧力构件的结构，当相交角度大于15°时，应分别计算各抗侧力构件方向的水平地震作用。

3 质量和刚度分布明显不对称的结构，应计入双向水平地震作用下的扭转影响；其他情况，应允许采用调整地震作用效应的方法计入扭转影响。

4 8、9度时的大跨度和长悬臂结构及9度时的高层建筑，应计算竖向地震作用。

注：8、9度时采用隔震设计的建筑结构，应按有关规定计算竖向地震作用。

5.1.2 各类建筑结构的抗震计算，应采用下列方法：

1 高度不超过40m、以剪切变形为主且质量和刚度沿高度分布比较均匀的结构，以及近似于单质点体系的结构，可采用底部剪力法等简化方法。

2 除1款外的建筑结构，宜采用振型分解反应谱法。

3 特别不规则的建筑、甲类建筑和表5.1.2-1所列高度范围的高层建筑，应采用时程分析法进行多遇地震下的补充计算，可取多条时程曲线计算结果的平均值与振型分解反应谱法计算结果的较大值。

采用时程分析法时，应按建筑场地类别和设计地震分组选用不少于二组的实际强震记录和一组人工模拟的加速度时程曲线，其平均地震影响系数曲线应与振型分解反应谱法所采用的地震影响系数曲线在统计意义上相符，其加速度时程的最大值可按表5.1.2-2采用。弹性时程分析时，每条时程曲线计算所得结构底部剪力不应小于振型分解反应谱法计算结果的65%，多条时程曲线计算所得结构底部剪力的平均值不应小于振型分解反应谱法计算结果的80%。

表5.1.2-1 采用时程分析的房屋高度范围

烈度、场地类别	房屋高度范围（m）
8度Ⅰ、Ⅱ类场地和7度	>100
8度Ⅲ、Ⅳ类场地	>80
9 度	>60

表5.1.2-2 时程分析所用地震加速度时程曲线的最大值（cm/s^2）

地震影响	6度	7度	8度	9度
多遇地震	18	35（55）	70（110）	140
罕遇地震	—	220（310）	400（510）	620

注：括号内数值分别用于设计基本地震加速度为0.15g和0.30g的地区。

4 计算罕遇地震下结构的变形，应按本章第5.5节规定，采用简化的弹塑性分析方法或弹塑性时程分析法。

注：建筑结构的隔震和消能减震设计，应采用本规范第12章规定的计算方法。

5.1.3 计算地震作用时，建筑的重力荷载代表值应取结构和构配件自重标准值和各可变荷载组合值之和。各可变荷载的组合值系数，应按表5.1.3采用。

表5.1.3 组合值系数

可变荷载种类		组合值系数
雪荷载		0.5
屋面积灰荷载		0.5
屋面活荷载		不计入
按实际情况计算的楼面活荷载		1.0
按等效均布荷载计算的楼面活荷载	藏书库、档案库	0.8
	其他民用建筑	0.5
吊车悬吊物重力	硬钩吊车	0.3
	软钩吊车	不计入

注：硬钩吊车的吊重较大时，组合值系数应按实际情况采用。

5.1.4 建筑结构的地震影响系数应根据烈度、场地类别、设计地震分组和结构自振周期以及阻尼比确定。其水平地震影响系数最大值应按表5.1.4-1采用；特征周期应根据场地类别和设计地震分组按表5.1.4-2采用，计算8、9度罕遇地震作用时，特征周期应增加0.05s。

注：1 周期大于6.0s的建筑结构所采用的地震影响系数应专门研究；

2 已编制抗震设防区划的城市，应允许按批准的设计地震动参数采用相应的地震影响系数。

表5.1.4-1 水平地震影响系数最大值

地震影响	6度	7度	8度	9度
多遇地震	0.04	0.08（0.12）	0.16（0.24）	0.32
罕遇地震	—	0.50（0.72）	0.90（1.20）	1.40

注：括号中数值分别用于设计基本地震加速度为0.15g和0.30g的地区。

表5.1.4-2 特征周期值（s）

设计地震分组	场地类别			
	Ⅰ	Ⅱ	Ⅲ	Ⅳ
第一组	0.25	0.35	0.45	0.65
第二组	0.30	0.40	0.55	0.75
第三组	0.35	0.45	0.65	0.90

5.1.5 建筑结构地震影响系数曲线（图5.1.5）的阻尼调整和形状参数应符合下列要求：

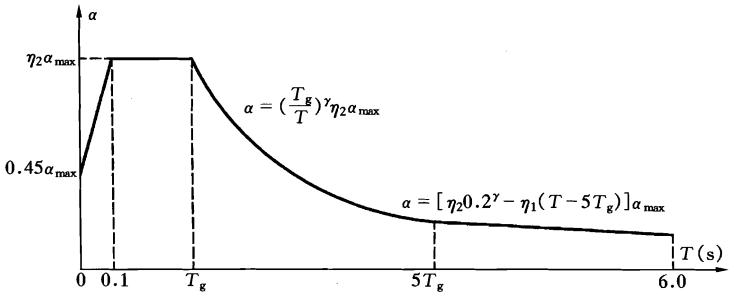
1 除有专门规定外，建筑结构的阻尼比应取0.05，地震影响系数曲线的阻尼调整系数应按1.0采用，形状参数应符合下列规定：

1）直线上升段，周期小于0.1s的区段。

2）水平段，自0.1s至特征周期区段，应取最大值（α_{max}）。

3）曲线下降段，自特征周期至5倍特征周期区段，衰减指数应取0.9。

4）直线下降段，自5倍特征周期至6s区段，下降斜率调整系数应取0.02。



α—地震影响系数；α_{max}—地震影响系数最大值；η_1—直线下降段的下降斜率调整系数；γ—衰减指数；T_g—特征周期；η_2—阻尼调整系数；T—结构自振周期

图5.1.5　地震影响系数曲线

2　当建筑结构的阻尼比按有关规定不等于0.05时，地震影响系数曲线的阻尼调整系数和形状参数应符合下列规定：

1）曲线下降段的衰减指数应按下式确定：

$$\gamma=0.9+\frac{0.05-\zeta}{0.5+5\zeta} \tag{5.1.5-1}$$

式中　γ——曲线下降段的衰减指数；

ζ——阻尼比。

2）直线下降段的下降斜率调整系数应按下式确定：

$$\eta_1=0.02+(0.05-\zeta)/8 \tag{5.1.5-2}$$

式中　η_1——直线下降段的下降斜率调整系数，小于0时取0。

3）阻尼调整系数应按下式确定：

$$\eta_2=1+\frac{0.05-\zeta}{0.06+1.7\zeta} \tag{5.1.5-3}$$

式中　η_2——阻尼调整系数，当小于0.55时，应取0.55。

5.1.6　结构抗震验算，应符合下列规定：

1　6度时的建筑（建造于Ⅳ类场地上较高的高层建筑除外），以及生土房屋和木结构房屋等，应允许不进行截面抗震验算，但应符合有关的抗震措施要求。

2　6度时建造于Ⅳ类场地上较高的高层建筑，7度和7度以上的建筑结构（生土房屋和木结构房屋等除外），应进行多遇地震作用下的截面抗震验算。

注：采用隔震设计的建筑结构，其抗震验算应符合有关规定。

5.1.7　符合本章第5.5节规定的结构，除按规定进行多遇地震作用下的截面抗震验算外，尚应进行相应的变形验算。

5.2　水平地震作用计算

5.2.1　采用底部剪力法时，各楼层可仅取一个自由度，结构的水平地震作用标准值，应按下列公式确定（图5.2.1）：

$$F_{Ek}=\alpha_1 G_{eq} \tag{5.2.1-1}$$

$$F_i = \frac{G_i H_i}{\sum_{j=1}^{n} G_j H_j} F_{Ek}(1-\delta_n) \quad (i = 1,2\cdots n) \qquad (5.2.1\text{-}2)$$

$$\Delta F_n = \delta_n F_{Ek} \qquad (5.2.1\text{-}3)$$

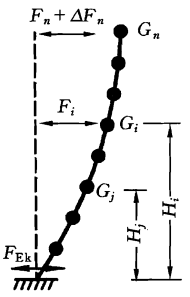


图 5.2.1 结构水平地震作用计算简图

式中 F_{Ek}——结构总水平地震作用标准值；

α_1——相应于结构基本自振周期的水平地震影响系数值，应按本章第 5.1.4 条确定，多层砌体房屋、底部框架和多层内框架砖房，宜取水平地震影响系数最大值；

G_{eq}——结构等效总重力荷载，单质点应取总重力荷载代表值，多质点可取总重力荷载代表值的 85%；

F_i——质点 i 的水平地震作用标准值；

G_i，G_j——分别为集中于质点 i、j 的重力荷载代表值，应按本章第 5.1.3 条确定；

H_i，H_j——分别为质点 i、j 的计算高度；

δ_n——顶部附加地震作用系数，多层钢筋混凝土和钢结构房屋可按表 5.2.1 采用，多层内框架砖房可采用 0.2，其他房屋可采用 0.0；

ΔF_n——顶部附加水平地震作用。

表 5.2.1 顶部附加地震作用系数

T_g (s)	$T_1>1.4T_g$	$T_1\leqslant 1.4T_g$
$\leqslant 0.35$	$0.08T_1+0.07$	0.0
$>0.35\sim 0.55$	$0.08T_1+0.01$	
>0.55	$0.08T_1-0.02$	

注：T_1 为结构基本自振周期。

5.2.2 采用振型分解反应谱法时，不进行扭转耦联计算的结构，应按下列规定计算其地震作用和作用效应：

1 结构 j 振型 i 质点的水平地震作用标准值，应按下列公式确定：

$$F_{ji} = \alpha_j \gamma_j X_{ji} G_i \quad (i = 1,2,\cdots n,\ j = 1,2,\cdots m) \qquad (5.2.2\text{-}1)$$

$$\gamma_j = \sum_{i=1}^{n} X_{ji} G_i \Big/ \sum_{i=1}^{n} X_{ji}^2 G_i \qquad (5.2.2\text{-}2)$$

式中 F_{ji}——j 振型 i 质点的水平地震作用标准值；

α_j——相应于 j 振型自振周期的地震影响系数，应按本章第 5.1.4 条确定；

X_{ji}——j 振型 i 质点的水平相对位移；

γ_j——j 振型的参与系数。

2 水平地震作用效应（弯矩、剪力、轴向力和变形），应按下式确定：

$$S_{Ek} = \sqrt{\Sigma S_j^2} \qquad (5.2.2\text{-}3)$$

式中 S_{Ek}——水平地震作用标准值的效应；

S_j——j 振型水平地震作用标准值的效应，可只取前 2～3 个振型，当基本自振周期大于 1.5s 或房屋高宽比大于 5 时，振型个数应适当增加。

5.2.3 建筑结构估计水平地震作用扭转影响时，应按下列规定计算其地震作用和作用效

应：

1 规则结构不进行扭转耦联计算时，平行于地震作用方向的两个边榀，其地震作用效应应乘以增大系数。一般情况下，短边可按 1.15 采用，长边可按 1.05 采用；当扭转刚度较小时，宜按不小于 1.3 采用。

2 按扭转耦联振型分解法计算时，各楼层可取两个正交的水平位移和一个转角共三个自由度，并应按下列公式计算结构的地震作用和作用效应。确有依据时，尚可采用简化计算方法确定地震作用效应。

1） j 振型 i 层的水平地震作用标准值，应按下列公式确定：

$$\begin{aligned} F_{xji} &= \alpha_j \gamma_{tj} X_{ji} G_i \\ F_{yji} &= \alpha_j \gamma_{tj} Y_{ji} G_i \quad (i=1,\ 2,\ \cdots n,\ j=1,\ 2,\ \cdots m) \\ F_{tji} &= \alpha_j \gamma_{tj} r_i^2 \varphi_{ji} G_i \end{aligned} \tag{5.2.3-1}$$

式中 F_{xji}、F_{yji}、F_{tji}——分别为 j 振型 i 层的 x 方向、y 方向和转角方向的地震作用标准值；

X_{ji}、Y_{ji}——分别为 j 振型 i 层质心在 x、y 方向的水平相对位移；

φ_{ji}——j 振型 i 层的相对扭转角；

r_i——i 层转动半径，可取 i 层绕质心的转动惯量除以该层质量的商的正二次方根；

γ_{tj}——计入扭转的 j 振型的参与系数，可按下列公式确定：

当仅取 x 方向地震作用时

$$\gamma_{tj} = \sum_{i=1}^{n} X_{ji} G_i \Big/ \sum_{i=1}^{n} (X_{ji}^2 + Y_{ji}^2 + \varphi_{ji}^2 r_i^2) G_i \tag{5.2.3-2}$$

当仅取 y 方向地震作用时

$$\gamma_{tj} = \sum_{i=1}^{n} Y_{ji} G_i \Big/ \sum_{i=1}^{n} (X_{ji}^2 + Y_{ji}^2 + \varphi_{ji}^2 r_i^2) G_i \tag{5.2.3-3}$$

当取与 x 方向斜交的地震作用时，

$$\gamma_{tj} = \gamma_{xj} \cos\theta + \gamma_{yj} \sin\theta \tag{5.2.3-4}$$

式中 γ_{xj}、γ_{yj}——分别由式（5.2.3-2）、（5.2.3-3）求得的参与系数；

θ——地震作用方向与 x 方向的夹角。

2） 单向水平地震作用的扭转效应，可按下列公式确定：

$$S_{Ek} = \sqrt{\sum_{j=1}^{m} \sum_{k=1}^{m} \rho_{jk} S_j S_k} \tag{5.2.3-5}$$

$$\rho_{jk} = \frac{8\zeta_j \zeta_k (1+\lambda_T) \lambda_T^{1.5}}{(1-\lambda_T^2)^2 + 4\zeta_j \zeta_k (1+\lambda_T)^2 \lambda_T} \tag{5.2.3-6}$$

式中 S_{Ek}——地震作用标准值的扭转效应；

S_j、S_k——分别为 j、k 振型地震作用标准值的效应，可取前 9～15 个振型；

ζ_j、ζ_k——分别为 j、k 振型的阻尼比；

ρ_{jk}——j 振型与 k 振型的耦联系数；

λ_T——k 振型与 j 振型的自振周期比。

3）双向水平地震作用的扭转效应，可按下列公式中的较大值确定：

$$S_{Ek}=\sqrt{S_x^2+(0.85S_y)^2} \tag{5.2.3-7}$$

或

$$S_{Ek}=\sqrt{S_y^2+(0.85S_x)^2} \tag{5.2.3-8}$$

式中 S_x、S_y 分别为 x 向、y 向单向水平地震作用按式（5.2.3-5）计算的扭转效应。

5.2.4 采用底部剪力法时，突出屋面的屋顶间、女儿墙、烟囱等的地震作用效应，宜乘以增大系数 3，此增大部分不应往下传递，但与该突出部分相连的构件应予计入；采用振型分解法时，突出屋面部分可作为一个质点；单层厂房突出屋面天窗架的地震作用效应的增大系数，应按本规范 9 章的有关规定采用。

5.2.5 **抗震验算时，结构任一楼层的水平地震剪力应符合下式要求：**

$$V_{Eki}>\lambda\sum_{j=i}^{n}G_j \tag{5.2.5}$$

式中 V_{Eki}——**第 i 层对应于水平地震作用标准值的楼层剪力；**

λ——**剪力系数，不应小于表 5.2.5 规定的楼层最小地震剪力系数值，对竖向不规则结构的薄弱层，尚应乘以 1.15 的增大系数；**

G_j——**第 j 层的重力荷载代表值。**

表 5.2.5 楼层最小地震剪力系数值

类 别	7 度	8 度	9 度
扭转效应明显或基本周期小于 3.5s 的结构	0.016（0.024）	0.032（0.048）	0.064
基本周期大于 5.0s 的结构	0.012（0.018）	0.024（0.032）	0.040

注：1 基本周期介于 3.5s 和 5s 之间的结构，可插入取值；

2 括号内数值分别用于设计基本地震加速度为 0.15g 和 0.30g 的地区。

5.2.6 结构的楼层水平地震剪力，应按下列原则分配：

1 现浇和装配整体式混凝土楼、屋盖等刚性楼盖建筑，宜按抗侧力构件等效刚度的比例分配。

2 木楼盖、木屋盖等柔性楼盖建筑，宜按抗侧力构件从属面积上重力荷载代表值的比例分配。

3 普通的预制装配式混凝土楼、屋盖等半刚性楼、屋盖的建筑，可取上述两种分配结果的平均值。

4 计入空间作用、楼盖变形、墙体弹塑性变形和扭转的影响时，可按本规范各有关规定对上述分配结果作适当调整。

5.2.7 结构抗震计算，一般情况下可不计入地基与结构相互作用的影响；8 度和 9 度时建造于Ⅲ、Ⅳ类场地，采用箱基、刚性较好的筏基和桩箱联合基础的钢筋混凝土高层建筑，当结构基本自振周期处于特征周期的 1.2 倍至 5 倍范围时，若计入地基与结构动力相互作用的影响，对刚性地基假定计算的水平地震剪力可按下列规定折减，其层间变形可按折减后的楼层剪力计算。

1 高宽比小于 3 的结构，各楼层水平地震剪力的折减系数，可按下式计算：

$$\psi=\left(\frac{T_1}{T_1+\Delta T}\right)^{0.9} \tag{5.2.7}$$

式中 ψ——计入地基与结构动力相互作用后的地震剪力折减系数；

T_1——按刚性地基假定确定的结构基本自振周期（s）；

ΔT——计入地基与结构动力相互作用的附加周期（s），可按表 5.2.7 采用。

表 5.2.7 附加周期（s）

烈　度	场地类别	
	Ⅲ　类	Ⅳ　类
8	0.08	0.20
9	0.10	0.25

2 高宽比不小于 3 的结构，底部的地震剪力按 1 款规定折减，顶部不折减，中间各层按线性插入值折减。

3 折减后各楼层的水平地震剪力，应符合本章第 5.2.5 条的规定。

5.3 竖向地震作用计算

5.3.1 9 度时的高层建筑，其竖向地震作用标准值应按下列公式确定（图 5.3.1）；楼层的竖向地震作用效应可按各构件承受的重力荷载代表值的比例分配，并宜乘以增大系数 1.5。

$$F_{Evk} = \alpha_{vmax} G_{eq} \qquad (5.3.1\text{-}1)$$

$$F_{vi} = \frac{G_i H_i}{\Sigma G_j H_j} F_{Evk} \qquad (5.3.1\text{-}2)$$

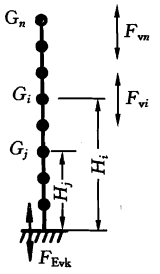


图 5.3.1 结构竖向地震作用计算简图

式中 F_{Evk}——结构总竖向地震作用标准值；

F_{vi}——质点 i 的竖向地震作用标准值；

α_{vmax}——竖向地震影响系数的最大值，可取水平地震影响系数最大值的 65%；

G_{eq}——结构等效总重力荷载，可取其重力荷载代表值的 75%。

5.3.2 平板型网架屋盖和跨度大于 24m 屋架的竖向地震作用标准值，宜取其重力荷载代表值和竖向地震作用系数的乘积；竖向地震作用系数可按表 5.3.2 采用。

表 5.3.2 竖向地震作用系数

结构类型	烈度	场地类别		
		Ⅰ	Ⅱ	Ⅲ、Ⅳ
平板型网架、钢屋架	8	可不计算（0.10）	0.08（0.12）	0.10（0.15）
	9	0.15	0.15	0.20
钢筋混凝土屋架	8	0.10（0.15）	0.13（0.19）	0.13（0.19）
	9	0.20	0.25	0.25

注：括号中数值用于设计基本地震加速度为 0.30g 的地区。

5.3.3 长悬臂和其他大跨度结构的竖向地震作用标准值，8 度和 9 度可分别取该结构、构件重力荷载代表值的 10%和 20%，设计基本地震加速度为 0.30g 时，可取该结构、构件重力荷载代表值的 15%。

5.4 截面抗震验算

5.4.1 结构构件的地震作用效应和其他荷载效应的基本组合，应按下式计算：

$$S = \gamma_G S_{GE} + \gamma_{Eh} S_{Ehk} + \gamma_{Ev} S_{Evk} + \psi_w \gamma_w S_{wk} \tag{5.4.1}$$

式中 S——结构构件内力组合的设计值，包括组合的弯矩、轴向力和剪力设计值；

γ_G——重力荷载分项系数，一般情况应采用 1.2，当重力荷载效应对构件承载能力有利时，不应大于 1.0；

γ_{Eh}、γ_{Ev}——分别为水平、竖向地震作用分项系数，应按表 5.4.1 采用；

γ_w——风荷载分项系数，应采用 1.4；

S_{GE}——重力荷载代表值的效应，有吊车时，尚应包括悬吊物重力标准值的效应；

S_{Ehk}——水平地震作用标准值的效应，尚应乘以相应的增大系数或调整系数；

S_{Evk}——竖向地震作用标准值的效应，尚应乘以相应的增大系数或调整系数；

S_{wk}——风荷载标准值的效应；

ψ_w——风荷载组合值系数，一般结构取 0.0，风荷载起控制作用的高层建筑应采用 0.2。

注：本规范一般略去表示水平方向的下标。

表 5.4.1 地震作用分项系数

地震作用	γ_{Eh}	γ_{Ev}
仅计算水平地震作用	1.3	0.0
仅计算竖向地震作用	0.0	1.3
同时计算水平与竖向地震作用	1.3	0.5

5.4.2 结构构件的截面抗震验算，应采用下列设计表达式：

$$S \leqslant R / \gamma_{RE} \tag{5.4.2}$$

式中 γ_{RE}——承载力抗震调整系数，除另有规定外，应按表 5.4.2 采用；

R——结构构件承载力设计值。

表 5.4.2 承载力抗震调整系数

材料	结构构件	受力状态	γ_{RE}
钢	柱，梁		0.75
	支撑		0.80
	节点板件，连接螺栓		0.85
	连接焊缝		0.90
砌体	两端均有构造柱、芯柱的抗震墙	受剪	0.9
	其他抗震墙	受剪	1.0
混凝土	梁	受弯	0.75
	轴压比小于 0.15 的柱	偏压	0.75
	轴压比不小于 0.15 的柱	偏压	0.80
	抗震墙	偏压	0.85
	各类构件	受剪、偏拉	0.85

5.4.3 当仅计算竖向地震作用时，各类结构构件承载力抗震调整系数均宜采用1.0。

5.5 抗震变形验算

5.5.1 表5.5.1所列各类结构应进行多遇地震作用下的抗震变形验算，其楼层内最大的弹性层间位移应符合下式要求：

$$\Delta u_e \leqslant [\theta_e] h \tag{5.5.1}$$

式中 Δu_e——多遇地震作用标准值产生的楼层内最大的弹性层间位移；计算时，除以弯曲变形为主的高层建筑外，可不扣除结构整体弯曲变形；应计入扭转变形，各作用分项系数均应采用1.0；钢筋混凝土结构构件的截面刚度可采用弹性刚度；

$[\theta_e]$——弹性层间位移角限值，宜按表5.5.1采用；

h——计算楼层层高。

表5.5.1 弹性层间位移角限值

结构类型	$[\theta_e]$
钢筋混凝土框架	1/550
钢筋混凝土框架-抗震墙、板柱-抗震墙、框架-核心筒	1/800
钢筋混凝土抗震墙、筒中筒	1/1000
钢筋混凝土框支层	1/1000
多、高层钢结构	1/300

5.5.2 结构在罕遇地震作用下薄弱层的弹塑性变形验算，应符合下列要求：

1 下列结构应进行弹塑性变形验算：

1） 8度Ⅲ、Ⅳ类场地和9度时，高大的单层钢筋混凝土柱厂房的横向排架；

2） 7～9度时楼层屈服强度系数小于0.5的钢筋混凝土框架结构；

3） 高度大于150m的钢结构；

4） 甲类建筑和9度时乙类建筑中的钢筋混凝土结构和钢结构；

5） 采用隔震和消能减震设计的结构。

2 下列结构宜进行弹塑性变形验算：

1） 表5.1.2-1所列高度范围且属于表3.4.2-2所列竖向不规则类型的高层建筑结构；

2） 7度Ⅲ、Ⅳ类场地和8度时乙类建筑中的钢筋混凝土结构和钢结构；

3） 板柱-抗震墙结构和底部框架砖房；

4） 高度不大于150m的高层钢结构。

注：楼层屈服强度系数为按构件实际配筋和材料强度标准值计算的楼层受剪承载力和按罕遇地震作用标准值计算的楼层弹性地震剪力的比值；对排架柱，指按实际配筋面积、材料强度标准值和轴向力计算的正截面受弯承载力与按罕遇地震作用标准值计算的弹性地震弯矩的比值。

5.5.3 结构在罕遇地震作用下薄弱层（部位）弹塑性变形计算，可采用下列方法：

1 不超过12层且层刚度无突变的钢筋混凝土框架结构、单层钢筋混凝土柱厂房可采用本节第5.5.4条的简化计算法；

2　除1款以外的建筑结构，可采用静力弹塑性分析方法或弹塑性时程分析法等；

3　规则结构可采用弯剪层模型或平面杆系模型，属于本规范第3.4节规定的不规则结构应采用空间结构模型。

5.5.4　结构薄弱层（部位）弹塑性层间位移的简化计算，宜符合下列要求：

1　结构薄弱层（部位）的位置可按下列情况确定：

1）楼层屈服强度系数沿高度分布均匀的结构，可取底层；

2）楼层屈服强度系数沿高度分布不均匀的结构，可取该系数最小的楼层（部位）和相对较小的楼层，一般不超过2～3处；

3）单层厂房，可取上柱。

2　弹塑性层间位移可按下列公式计算：

$$\Delta u_p = \eta_p \Delta u_e \tag{5.5.4-1}$$

或

$$\Delta u_p = \mu \Delta u_y = \frac{\eta_p}{\xi_y} \Delta u_y \tag{5.5.4-2}$$

式中　Δu_p——弹塑性层间位移；

Δu_y——层间屈服位移；

μ——楼层延性系数；

Δu_e——罕遇地震作用下按弹性分析的层间位移；

η_p——弹塑性层间位移增大系数，当薄弱层（部位）的屈服强度系数不小于相邻层（部位）该系数平均值的0.8时，可按表5.5.4采用。当不大于该平均值的0.5时，可按表内相应数值的1.5倍采用；其他情况可采用内插法取值；

ξ_y——楼层屈服强度系数。

表5.5.4　弹塑性层间位移增大系数

结构类型	总层数 n 或部位	ξ_y		
		0.5	0.4	0.3
多层均匀框架结构	2～4	1.30	1.40	1.60
	5～7	1.50	1.65	1.80
	8～12	1.80	2.00	2.20
单层厂房	上　柱	1.30	1.60	2.00

5.5.5　结构薄弱层（部位）弹塑性层间位移应符合下式要求：

$$\Delta u_p \leqslant [\theta_p] h \tag{5.5.5}$$

式中　$[\theta_p]$——弹塑性层间位移角限值，可按表5.5.5采用；对钢筋混凝土框架结构，当轴压比小于0.40时，可提高10%；当柱子全高的箍筋构造比本规范表6.3.12条规定的最小配箍特征值大30%时，可提高20%，但累计不超过25%。

h——薄弱层楼层高度或单层厂房上柱高度。

表 5.5.5　弹塑性层间位移角限值

结　构　类　型	$[\theta_p]$
单层钢筋混凝土柱排架	1/30
钢筋混凝土框架	1/50
底部框架砖房中的框架-抗震墙	1/100
钢筋混凝土框架-抗震墙、板柱-抗震墙、框架-核心筒	1/100
钢筋混凝土抗震墙、筒中筒	1/120
多、高层钢结构	1/50

6 多层和高层钢筋混凝土房屋

6.1 一 般 规 定

6.1.1 本章适用的现浇钢筋混凝土房屋的结构类型和最大高度应符合表 6.1.1 的要求。平面和竖向均不规则的结构或建造于Ⅳ类场地的结构，适用的最大高度应适当降低。

注：本章的“抗震墙”即国家标准《混凝土结构设计规范》GB50010 中的剪力墙。

表 6.1.1 现浇钢筋混凝土房屋适用的最大高度（m）

结构类型	烈度			
	6	7	8	9
框 架	60	55	45	25
框架-抗震墙	130	120	100	50
抗震墙	140	120	100	60
部分框支抗震墙	120	100	80	不应采用
框架-核心筒	150	130	100	70
筒中筒	180	150	120	80
板柱-抗震墙	40	35	30	不应采用

注：1 房屋高度指室外地面到主要屋面板板顶的高度（不包括局部突出屋顶部分）；

2 框架-核心筒结构指周边稀柱框架与核心筒组成的结构；

3 部分框支抗震墙结构指首层或底部两层框支抗震墙结构；

4 乙类建筑可按本地区抗震设防烈度确定适用的最大高度；

5 超过表内高度的房屋，应进行专门研究和论证，采取有效的加强措施。

6.1.2 钢筋混凝土房屋应根据烈度、结构类型和房屋高度采用不同的抗震等级，并应符合相应的计算和构造措施要求。丙类建筑的抗震等级应按表 6.1.2 确定。

表 6.1.2 现浇钢筋混凝土房屋的抗震等级

结构类型		烈度						
		6		7		8		9
框架结构	高 度（m）	≤30	>30	≤30	>30	≤30	>30	≤25
	框 架	四	三	三	二	二	一	一
	剧场、体育馆等大跨度公共建筑	三		二		一		一
框架-抗震墙结构	高 度（m）	≤60	>60	≤60	>60	≤60	>60	≤50
	框 架	四	三	三	二	二	一	一
	抗 震 墙	三		二		一	一	一
抗震墙结构	高 度（m）	≤80	>80	≤80	>80	≤80	>80	≤60
	抗 震 墙	四	三	三	二	二	一	一
部分框支抗震墙结构	抗 震 墙	三	二	二		一		
	框支层框架	二		二	一	一		

续表 6.1.2

结构类型			烈度			
			6	7	8	9
筒体结构	框架-核心筒	框架	三	二	一	一
		核心筒	二	二	一	一
	筒中筒	外筒	三	二	一	一
		内筒	三	二	一	一
板柱-抗震墙结构	板柱的柱		三	二	一	
	抗震墙		二	二	二	

注：1 建筑场地为Ⅰ类时，除6度外可按表内降低一度所对应的抗震等级采取抗震构造措施，但相应的计算要求不应降低；

2 接近或等于高度分界时，应允许结合房屋不规则程度及场地、地基条件确定抗震等级；

3 部分框支抗震墙结构中，抗震墙加强部位以上的一般部位，应允许按抗震墙结构确定其抗震等级。

6.1.3 钢筋混凝土房屋抗震等级的确定，尚应符合下列要求：

1 框架-抗震墙结构，在基本振型地震作用下，若框架部分承受的地震倾覆力矩大于结构总地震倾覆力矩的50%，其框架部分的抗震等级应按框架结构确定，最大适用高度可比框架结构适当增加。

2 裙房与主楼相连，除应按裙房本身确定外，不应低于主楼的抗震等级；主楼结构在裙房顶层及相邻上下各一层应适当加强抗震构造措施。裙房与主楼分离时，应按裙房本身确定抗震等级。

3 当地下室顶板作为上部结构的嵌固部位时，地下一层的抗震等级应与上部结构相同，地下一层以下的抗震等级可根据具体情况采用三级或更低等级。地下室中无上部结构的部分，可根据具体情况采用三级或更低等级。

4 抗震设防类别为甲、乙、丁类的建筑，应按本规范第3.1.3条规定和表6.1.2确定抗震等级；其中，8度乙类建筑高度超过表6.1.2规定的范围时，应经专门研究采取比一级更有效的抗震措施。

注：本章“一、二、三、四级”即“抗震等级为一、二、三、四级”的简称。

6.1.4 高层钢筋混凝土房屋宜避免采用本规范第3.4节规定的不规则建筑结构方案，不设防震缝；当需要设置防震缝时，应符合下列规定：

1 防震缝最小宽度应符合下列要求：

1）框架结构房屋的防震缝宽度，当高度不超过15m时可采用70mm；超过15m时，6度、7度、8度和9度相应每增加高度5m、4m、3m和2m，宜加宽20mm。

2）框架-抗震墙结构房屋的防震缝宽度可采用1）项规定数值的70%，抗震墙结构房屋的防震缝宽度可采用1）项规定数值的50%；且均不宜小于70mm。

3）防震缝两侧结构类型不同时，宜按需要较宽防震缝的结构类型和较低房屋高度确定缝宽。

2 8、9度框架结构房屋防震缝两侧结构高度、刚度或层高相差较大时，可在缝两侧房屋的尽端沿全高设置垂直于防震缝的抗撞墙，每一侧抗撞墙的数量不应少于两道，宜分别对称布置，墙肢长度可不大于一个柱距，框架和抗撞墙的内力应按设置和不设置抗撞墙两种情况分别进行分析，并按不利情况取值。防震缝两侧抗撞墙的端柱和框架的边柱，箍筋应沿房屋全高加密。

6.1.5 框架结构和框架-抗震墙结构中，框架和抗震墙均应双向设置，柱中线与抗震墙中线、梁中线与柱中线之间偏心距不宜大于柱宽的1/4。

6.1.6 框架-抗震墙和板柱-抗震墙结构中，抗震墙之间无大洞口的楼、屋盖的长宽比，不宜超过表6.1.6的规定；超过时，应计入楼盖平面内变形的影响。

表6.1.6 抗震墙之间楼、屋盖的长宽比

楼、屋盖类型	烈度			
	6	7	8	9
现浇、叠合梁板	4	4	3	2
装配式楼盖	3	3	2.5	不宜采用
框支层和板柱-抗震墙的现浇梁板	2.5	2.5	2	不应采用

6.1.7 采用装配式楼、屋盖时，应采取措施保证楼、屋盖的整体性及其与抗震墙的可靠连接。采用配筋现浇面层加强时，厚度不宜小于50mm。

6.1.8 框架-抗震墙结构中的抗震墙设置，宜符合下列要求：

1 抗震墙宜贯通房屋全高，且横向与纵向的抗震墙宜相连。

2 抗震墙宜设置在墙面不需要开大洞口的位置。

3 房屋较长时，刚度较大的纵向抗震墙不宜设置在房屋的端开间。

4 抗震墙洞口宜上下对齐；洞边距端柱不宜小于300mm。

5 一、二级抗震墙的洞口连梁，跨高比不宜大于5，且梁截面高度不宜小于400mm。

6.1.9 抗震墙结构和部分框支抗震墙结构中的抗震墙设置，应符合下列要求：

1 较长的抗震墙宜开设洞口，将一道抗震墙分成长度较均匀的若干墙段，洞口连梁的跨高比宜大于6，各墙段的高宽比不应小于2。

2 墙肢的长度沿结构全高不宜有突变；抗震墙有较大洞口时，以及一、二级抗震墙的底部加强部位，洞口宜上下对齐。

3 矩形平面的部分框支抗震墙结构，其框支层的楼层侧向刚度不应小于相邻非框支层楼层侧向刚度的50%；框支层落地抗震墙间距不宜大于24m，框支层的平面布置尚宜对称，且宜设抗震筒体。

6.1.10 部分框支抗震墙结构的抗震墙，其底部加强部位的高度，可取框支层加框支层以上二层的高度及落地抗震墙总高度的1/8二者的较大值，且不大于15m；其他结构的抗震墙，其底部加强部位的高度可取墙肢总高度的1/8和底部二层二者的较大值，且不大于15m。

6.1.11 框架单独柱基有下列情况之一时，宜沿两个主轴方向设置基础系梁：

1 一级框架和Ⅳ类场地的二级框架；

2 各柱基承受的重力荷载代表值差别较大；

3 基础埋置较深，或各基础埋置深度差别较大；

4 地基主要受力层范围内存在软弱粘性土层、液化土层和严重不均匀土层；

5 桩基承台之间。

6.1.12 框架-抗震墙结构中的抗震墙基础和部分框支抗震墙结构的落地抗震墙基础，应有良好的整体性和抗转动的能力。

6.1.13 主楼与裙房相连且采用天然地基，除应符合本规范第4.2.4条的规定外，在地震作用下主楼基础底面不宜出现零应力区。

6.1.14 地下室顶板作为上部结构的嵌固部位时，应避免在地下室顶板开设大洞口，并应采用现浇梁板结构，其楼板厚度不宜小于180mm，混凝土强度等级不宜小于C30，应采用双层双向配筋，且每层每个方向的配筋率不宜小于0.25%；地下室结构的楼层侧向刚度不宜小于相邻上部楼层侧向刚度的2倍，地下室柱截面每侧的纵向钢筋面积，除应满足计算要求外，不应少于地上一层对应柱每侧纵筋面积的1.1倍；地上一层的框架结构柱和抗震墙墙底截面的弯矩设计值应符合本章第6.2.3、6.2.6、6.2.7条的规定，位于地下室顶板的梁柱节点左右梁端截面实际受弯承载力之和不宜小于上柱下端实际受弯承载力。

6.1.15 框架的填充墙应符合本规范第13章的规定。

6.1.16 高强混凝土结构抗震设计应符合本规范附录B的规定。

6.1.17 预应力混凝土结构抗震设计应符合本规范附录C的规定。

6.2 计 算 要 点

6.2.1 钢筋混凝土结构应按本节规定调整构件的组合内力设计值，其层间变形应符合本规范第5.5节有关规定；构件截面抗震验算时，凡本章和有关附录未作规定者，应符合现行有关结构设计规范的要求，但其非抗震的构件承载力设计值应除以本规范规定的承载力抗震调整系数。

6.2.2 一、二、三级框架的梁柱节点处，除框架顶层和柱轴压比小于0.15者及框支梁与框支柱的节点外，柱端组合的弯矩设计值应符合下式要求：

$$\Sigma M_c = \eta_c \Sigma M_b \tag{6.2.2-1}$$

一级框架结构及9度时尚应符合

$$\Sigma M_c = 1.2 \Sigma M_{bua} \tag{6.2.2-2}$$

式中 ΣM_c——节点上下柱端截面顺时针或反时针方向组合的弯矩设计值之和，上下柱端的弯矩设计值，可按弹性分析分配；

ΣM_b——节点左右梁端截面反时针或顺时针方向组合的弯矩设计值之和，一级框架节点左右梁端均为负弯矩时，绝对值较小的弯矩应取零；

ΣM_{bua}——节点左右梁端截面反时针或顺时针方向实配的正截面抗震受弯承载力所对应的弯矩值之和，根据实配钢筋面积(计入受压筋)和材料强度标准值确定；

η_c——柱端弯矩增大系数，一级取1.4，二级取1.2，三级取1.1。

当反弯点不在柱的层高范围内时，柱端截面组合的弯矩设计值可乘以上述柱端弯矩增大系数。

6.2.3 一、二、三级框架结构的底层，柱下端截面组合的弯矩设计值，应分别乘以增大系数1.5、1.25和1.15。底层柱纵向钢筋宜按上下端的不利情况配置。

注：底层指无地下室的基础以上或地下室以上的首层。

6.2.4 一、二、三级的框架梁和抗震墙中跨高比大于2.5的连梁，其梁端截面组合的剪力设计值应按下式调整：

$$V = \eta_{vb} (M_b^l + M_b^r) / l_n + V_{Gb} \tag{6.2.4-1}$$

一级框架结构及9度时尚应符合

$$V = 1.1 (M_{bua}^l + M_{bua}^r) / l_n + V_{Gb} \tag{6.2.4-2}$$

式中 V——梁端截面组合的剪力设计值；

l_n——梁的净跨；

V_{Gb}——梁在重力荷载代表值（9度时高层建筑还应包括竖向地震作用标准值）作用下，按简支梁分析的梁端截面剪力设计值；

M_b^l、M_b^r——分别为梁左右端截面反时针或顺时针方向组合的弯矩设计值，一级框架两端弯矩均为负弯矩时，绝对值较小的弯矩应取零；

M_{bua}^l、M_{bua}^r——分别为梁左右端截面反时针或顺时针方向实配的正截面抗震受弯承载力所对应的弯矩值，根据实配钢筋面积（计入受压筋）和材料强度标准值确定；

η_{vb}——梁端剪力增大系数，一级取1.3，二级取1.2，三级取1.1。

6.2.5 一、二、三级的框架柱和框支柱组合的剪力设计值应按下式调整：

$$V=\eta_{vc}(M_c^b+M_c^t)/H_n \tag{6.2.5-1}$$

一级框架结构及9度时尚应符合

$$V=1.2(M_{cua}^b+M_{cua}^t)/H_n \tag{6.2.5-2}$$

式中 V——柱端截面组合的剪力设计值；框支柱的剪力设计值尚应符合本节第6.2.10条的规定；

H_n——柱的净高；

M_c^t、M_c^b——分别为柱的上下端顺时针或反时针方向截面组合的弯矩设计值，应符合本节第6.2.2、6.2.3条的规定；框支柱的弯矩设计值尚应符合本节第6.2.10条的规定；

M_{cua}^t、M_{cua}^b——分别为偏心受压柱的上下端顺时针或反时针方向实配的正截面抗震受弯承载力所对应的弯矩值，根据实配钢筋面积、材料强度标准值和轴压力等确定；

η_{vc}——柱剪力增大系数，一级取1.4，二级取1.2，三级取1.1。

6.2.6 一、二、三级框架的角柱，经本节第6.2.2、6.2.3、6.2.5、6.2.10条调整后的组合弯矩设计值、剪力设计值尚应乘以不小于1.10的增大系数。

6.2.7 抗震墙各墙肢截面组合的弯矩设计值，应按下列规定采用：

1 一级抗震墙的底部加强部位及以上一层，应按墙肢底部截面组合弯矩设计值采用；其他部位，墙肢截面的组合弯矩设计值应乘以增大系数，其值可采用1.2。

2 部分框支抗震墙结构的落地抗震墙墙肢不宜出现小偏心受拉。

3 双肢抗震墙中，墙肢不宜出现小偏心受拉；当任一墙肢为大偏心受拉时，另一墙肢的剪力设计值、弯矩设计值应乘以增大系数1.25。

6.2.8 一、二、三级的抗震墙底部加强部位，其截面组合的剪力设计值应按下式调整：

$$V=\eta_{vw}V_w \tag{6.2.8-1}$$

9度时尚应符合

$$V=1.1\frac{M_{wua}}{M_w}V_w \tag{6.2.8-2}$$

式中 V——抗震墙底部加强部位截面组合的剪力设计值；

V_w——抗震墙底部加强部位截面组合的剪力计算值；

M_{wua}——抗震墙底部截面实配的抗震受弯承载力所对应的弯矩值，根据实配纵向钢筋

面积、材料强度标准值和轴力等计算；有翼墙时应计入墙两侧各一倍翼墙厚度范围内的纵向钢筋；

M_w——抗震墙底部截面组合的弯矩设计值；

η_{vw}——抗震墙剪力增大系数，一级为1.6，二级为1.4，三级为1.2。

6.2.9 钢筋混凝土结构的梁、柱、抗震墙和连梁，其截面组合的剪力设计值应符合下列要求：

跨高比大于2.5的梁和连梁及剪跨比大于2的柱和抗震墙：

$$V \leqslant \frac{1}{\gamma_{RE}}(0.20 f_c b h_0) \tag{6.2.9-1}$$

跨高比不大于2.5的连梁、剪跨比不大于2的柱和抗震墙、部分框支抗震墙结构的框支柱和框支梁、以及落地抗震墙的底部加强部位：

$$V \leqslant \frac{1}{\gamma_{RE}}(0.15 f_c b h_0) \tag{6.2.9-2}$$

剪跨比应按下式计算：

$$\lambda = M^c / (V^c h_0) \tag{6.2.9-3}$$

式中 λ——剪跨比，应按柱端或墙端截面组合的弯矩计算值M^c、对应的截面组合剪力计算值V^c及截面有效高度h_0确定，并取上下端计算结果的较大值；反弯点位于柱高中部的框架柱可按柱净高与2倍柱截面高度之比计算；

V——按本节第6.2.4、6.2.5、6.2.6、6.2.8、6.2.10条等规定调整后的梁端、柱端或墙端截面组合的剪力设计值；

f_c——混凝土轴心抗压强度设计值；

b——梁、柱截面宽度或抗震墙墙肢截面宽度，圆形截面柱可按面积相等的方形截面计算；

h_0——截面有效高度，抗震墙可取墙肢长度。

6.2.10 部分框支抗震墙结构的框支柱尚应满足下列要求：

1 框支柱承受的最小地震剪力，当框支柱的数目多于10根时，柱承受地震剪力之和不应小于该楼层地震剪力的20%；当少于10根时，每根柱承受的地震剪力不应小于该楼层地震剪力的2%。

2 一、二级框支柱由地震作用引起的附加轴力应分别乘以增大系数1.5、1.2；计算轴压比时，该附加轴力可不乘以增大系数。

3 一、二级框支柱的顶层柱上端和底层柱下端，其组合的弯矩设计值应分别乘以增大系数1.5和1.25，框支柱的中间节点应满足本节第6.2.2条的要求。

4 框支梁中线宜与框支柱中线重合。

6.2.11 部分框支抗震墙结构的一级落地抗震墙底部加强部位尚应满足下列要求：

1 验算抗震墙受剪承载力时不宜计入混凝土的受剪作用，若需计入混凝土的受剪作用，则墙肢在边缘构件以外的部位在两排钢筋间应设置直径不小于8mm的拉结筋，且水平和竖向间距分别不大于该方向分布筋间距两倍和400mm的较小值。

2 无地下室且墙肢底部截面出现偏心受拉时，宜在墙肢与基础交接面另设交叉防滑斜筋，防滑斜筋承担的拉力可按交接面处剪力设计值的30%采用。

6.2.12 部分框支抗震墙结构的框支层楼板应符合本规范附录E.1的规定。

6.2.13 钢筋混凝土结构抗震计算时，尚应符合下列要求：

1 侧向刚度沿竖向分布基本均匀的框架-抗震墙结构，任一层框架部分的地震剪力，不应小于结构底部总地震剪力的20%和按框架-抗震墙结构分析的框架部分各楼层地震剪力中最大值1.5倍二者的较小值。

2 抗震墙连梁的刚度可折减，折减系数不宜小于0.50。

3 抗震墙结构、部分框支抗震墙结构、框架-抗震墙结构、筒体结构、板柱-抗震墙结构计算内力和变形时，其抗震墙应计入端部翼墙的共同工作。翼墙的有效长度，每侧由墙面算起可取相邻抗震墙净间距的一半、至门窗洞口的墙长度及抗震墙总高度的15%三者的最小值。

6.2.14 一级抗震墙的施工缝截面受剪承载力，应采用下式验算：

$$V_{wj} \leqslant \frac{1}{\gamma_{RE}} (0.6 f_y A_s + 0.8N) \tag{6.2.14}$$

式中 V_{wj}——抗震墙施工缝处组合的剪力设计值；

f_y——竖向钢筋抗拉强度设计值；

A_s——施工缝处抗震墙的竖向分布钢筋、竖向插筋和边缘构件（不包括边缘构件以外的两侧翼墙）纵向钢筋的总截面面积；

N——施工缝处不利组合的轴向力设计值，压力取正值，拉力取负值。

6.2.15 框架节点核芯区的抗震验算应符合下列要求：

1 一、二级框架的节点核芯区，应进行抗震验算；三、四级框架节点核芯区，可不进行抗震验算，但应符合抗震构造措施的要求。

2 核芯区截面抗震验算方法应符合本规范附录D的规定。

6.3 框架结构抗震构造措施

6.3.1 梁的截面尺寸，宜符合下列各项要求：

1 截面宽度不宜小于200mm；

2 截面高宽比不宜大于4；

3 净跨与截面高度之比不宜小于4。

6.3.2 采用梁宽大于柱宽的扁梁时，楼板应现浇，梁中线宜与柱中线重合，扁梁应双向布置，且不宜用于一级框架结构。扁梁的截面尺寸应符合下列要求，并应满足现行有关规范对挠度和裂缝宽度的规定：

$$b_b \leqslant 2b_c \tag{6.3.2-1}$$

$$b_b \leqslant b_c + h_b \tag{6.3.2-2}$$

$$h_b \geqslant 16d \tag{6.3.2-3}$$

式中 b_c——柱截面宽度，圆形截面取柱直径的0.8倍；

b_b、h_b——分别为梁截面宽度和高度；

d——柱纵筋直径。

6.3.3 梁的钢筋配置，应符合下列各项要求：

1 梁端纵向受拉钢筋的配筋率不应大于2.5%，且计入受压钢筋的梁端混凝土受压

区高度和有效高度之比，一级不应大于0.25，二、三级不应大于0.35。

2 梁端截面的底面和顶面纵向钢筋配筋量的比值，除按计算确定外，一级不应小于0.5，二、三级不应小于0.3。

3 梁端箍筋加密区的长度、箍筋最大间距和最小直径应按表6.3.3采用，当梁端纵向受拉钢筋配筋率大于2%时，表中箍筋最小直径数值应增大2mm。

表6.3.3 梁端箍筋加密区的长度、箍筋的最大间距和最小直径

抗震等级	加密区长度（采用较大值）（mm）	箍筋最大间距（采用最小值）（mm）	箍筋最小直径（mm）
一	$2h_b$，500	$h_b/4$，$6d$，100	10
二	$1.5h_b$，500	$h_b/4$，$8d$，100	8
三	$1.5h_b$，500	$h_b/4$，$8d$，150	8
四	$1.5h_b$，500	$h_b/4$，$8d$，150	6

注：d为纵向钢筋直径，h_b为梁截面高度。

6.3.4 梁的纵向钢筋配置，尚应符合下列各项要求：

1 沿梁全长顶面和底面的配筋，一、二级不应少于2ϕ14，且分别不应少于梁两端顶面和底面纵向配筋中较大截面面积的1/4，三、四级不应少于2ϕ12；

2 一、二级框架梁内贯通中柱的每根纵向钢筋直径，对矩形截面柱，不宜大于柱在该方向截面尺寸的1/20；对圆形截面柱，不宜大于纵向钢筋所在位置柱截面弦长的1/20。

6.3.5 梁端加密区的箍筋肢距，一级不宜大于200mm和20倍箍筋直径的较大值，二、三级不宜大于250mm和20倍箍筋直径的较大值，四级不宜大于300mm。

6.3.6 柱的截面尺寸，宜符合下列各项要求：

1 截面的宽度和高度均不宜小于300mm；圆柱直径不宜小于350mm。

2 剪跨比宜大于2。

3 截面长边与短边的边长比不宜大于3。

6.3.7 柱轴压比不宜超过表6.3.7的规定；建造于Ⅳ类场地且较高的高层建筑，柱轴压比限值应适当减小。

表6.3.7 柱轴压比限值

结构类型	抗震等级		
	一	二	三
框架结构	0.7	0.8	0.9
框架-抗震墙，板柱-抗震墙及筒体	0.75	0.85	0.95
部分框支抗震墙	0.6	0.7	—

注：1 轴压比指柱组合的轴压力设计值与柱的全截面面积和混凝土轴心抗压强度设计值乘积之比值；可不进行地震作用计算的结构，取无地震作用组合的轴力设计值；

2 表内限值适用于剪跨比大于2、混凝土强度等级不高于C60的柱；剪跨比不大于2的柱轴压比限值应降低0.05；剪跨比小于1.5的柱，轴压比限值应专门研究并采取特殊构造措施；

3 沿柱全高采用井字复合箍且箍筋肢距不大于200mm、间距不大于100mm、直径不小于12mm，或沿柱全高采用复合螺旋箍、螺旋间距不大于100mm、箍筋肢距不大于200 mm、直径不小于12mm，或沿柱全高采用连续复合矩形螺旋箍、螺旋净距不大于80mm、箍筋肢距不大于200 mm、直径不小于10mm，轴压比限值均可增加0.10；上述三种箍筋的配箍特征值均应按增大的轴压比由本节表6.3.12确定；

4 在柱的截面中部附加芯柱，其中另加的纵向钢筋的总面积不少于柱截面面积的0.8%，轴压比限值可增加0.05；此项措施与注3的措施共同采用时，轴压比限值可增加0.15，但箍筋的配箍特征值仍可按轴压比增加0.10的要求确定；

5 柱轴压比不应大于1.05。

6.3.8 柱的钢筋配置，应符合下列各项要求：

1 柱纵向钢筋的最小总配筋率应按表 6.3.8-1 采用，同时每一侧配筋率不应小于 0.2%；对建造于Ⅳ类场地且较高的高层建筑，表中的数值应增加 0.1。

表 6.3.8-1 柱截面纵向钢筋的最小总配筋率（百分率）

类 别	抗震等级			
	一	二	三	四
中柱和边柱	1.0	0.8	0.7	0.6
角柱、框支柱	1.2	1.0	0.9	0.8

注：采用 HRB400 级热轧钢筋时应允许减少 0.1，混凝土强度等级高于 C60 时应增加 0.1。

2 柱箍筋在规定的范围内应加密，加密区的箍筋间距和直径，应符合下列要求：

1）一般情况下，箍筋的最大间距和最小直径，应按表 6.3.8-2 采用；

表 6.3.8-2 柱箍筋加密区的箍筋最大间距和最小直径

抗震等级	箍筋最大间距（采用较小值，mm）	箍筋最小直径（mm）
一	$6d$，100	10
二	$8d$，100	8
三	$8d$，150（柱根 100）	8
四	$8d$，150（柱根 100）	6（柱根 8）

注：d 为柱纵筋最小直径；柱根指框架底层柱的嵌固部位。

2）二级框架柱的箍筋直径不小于 10mm 且箍筋肢距不大于 200mm 时，除柱根外最大间距应允许采用 150mm；三级框架柱的截面尺寸不大于 400mm 时，箍筋最小直径应允许采用 6mm；四级框架柱剪跨比不大于 2 时，箍筋直径不应小于 8mm。

3）框支柱和剪跨比不大于 2 的柱，箍筋间距不应大于 100mm。

6.3.9 柱的纵向钢筋配置，尚应符合下列各项要求：

1 宜对称配置。

2 截面尺寸大于 400mm 的柱，纵向钢筋间距不宜大于 200mm。

3 柱总配筋率不应大于 5%。

4 一级且剪跨比不大于 2 的柱，每侧纵向钢筋配筋率不宜大于 1.2%。

5 边柱、角柱及抗震墙端柱在地震作用组合产生小偏心受拉时，柱内纵筋总截面面积应比计算值增加 25%。

6 柱纵向钢筋的绑扎接头应避开柱端的箍筋加密区。

6.3.10 柱的箍筋加密范围，应按下列规定采用：

1 柱端，取截面高度（圆柱直径），柱净高的 1/6 和 500mm 三者的最大值。

2 底层柱，柱根不小于柱净高的 1/3；当有刚性地面时，除柱端外尚应取刚性地面上下各 500mm。

3 剪跨比不大于 2 的柱和因设置填充墙等形成的柱净高与柱截面高度之比不大于 4 的柱，取全高。

4 框支柱，取全高。

5 一级及二级框架的角柱，取全高。

6.3.11 柱箍筋加密区箍筋肢距，一级不宜大于 200mm，二、三级不宜大于 250mm 和 20

倍箍筋直径的较大值，四级不宜大于300mm。至少每隔一根纵向钢筋宜在两个方向有箍筋或拉筋约束；采用拉筋复合箍时，拉筋宜紧靠纵向钢筋并钩住箍筋。

6.3.12 柱箍筋加密区的体积配箍率，应符合下列要求：

$$\rho_v \geqslant \lambda_v f_c / f_{yv} \tag{6.3.12}$$

式中 ρ_v——柱箍筋加密区的体积配箍率，一级不应小于0.8%，二级不应小于0.6%，三、四级不应小于0.4%；计算复合箍的体积配箍率时，应扣除重叠部分的箍筋体积；

f_c——混凝土轴心抗压强度设计值；强度等级低于C35时，应按C35计算；

f_{yv}——箍筋或拉筋抗拉强度设计值，超过360N/mm^{2}时，应取360N/mm^{2}计算；

λ_v——最小配箍特征值，宜按表6.3.12采用。

表6.3.12 柱箍筋加密区的箍筋最小配箍特征值

抗震等级	箍筋形式	柱轴压比								
		≤0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.05
一	普通箍、复合箍	0.10	0.11	0.13	0.15	0.17	0.20	0.23		
	螺旋箍、复合或连续复合矩形螺旋箍	0.08	0.09	0.11	0.13	0.15	0.18	0.21		
二	普通箍、复合箍	0.08	0.09	0.11	0.13	0.15	0.17	0.19	0.22	0.24
	螺旋箍、复合或连续复合矩形螺旋箍	0.06	0.07	0.09	0.11	0.13	0.15	0.17	0.20	0.22
三	普通箍、复合箍	0.06	0.07	0.09	0.11	0.13	0.15	0.17	0.20	0.22
	螺旋箍、复合或连续复合矩形螺旋箍	0.05	0.06	0.07	0.09	0.11	0.13	0.15	0.18	0.20

注：1 普通箍指单个矩形箍和单个圆形箍；复合箍指由矩形、多边形、圆形箍或拉筋组成的箍筋；复合螺旋箍指由螺旋箍与矩形、多边形、圆形箍或拉筋组成的箍筋；连续复合矩形螺旋箍指全部螺旋箍为同一根钢筋加工而成的箍筋；

2 框支柱宜采用复合螺旋箍或井字复合箍，其最小配箍特征值应比表内数值增加0.02，且体积配箍率不应小于1.5%；

3 剪跨比不大于2的柱宜采用复合螺旋箍或井字复合箍，其体积配箍率不应小于1.2%，9度时不应小于1.5%；

4 计算复合螺旋箍的体积配箍率时，其非螺旋箍的箍筋体积应乘以换算系数0.8。

6.3.13 柱箍筋非加密区的体积配箍率不宜小于加密区的50%；箍筋间距，一、二级框架柱不应大于10倍纵向钢筋直径，三、四级框架柱不应大于15倍纵向钢筋直径。

6.3.14 框架节点核芯区箍筋的最大间距和最小直径宜按本章6.3.8条采用，一、二、三级框架节点核芯区配箍特征值分别不宜小于0.12、0.10和0.08且体积配箍率分别不宜小于0.6%、0.5%和0.4%。柱剪跨比不大于2的框架节点核芯区配箍特征值不宜小于核芯区上、下柱端的较大配箍特征值。

6.4 抗震墙结构抗震构造措施

6.4.1 抗震墙的厚度，一、二级不应小于160mm且不应小于层高的1/20，三、四级不应小于140mm且不应小于层高的1/25。底部加强部位的墙厚，一、二级不宜小于200mm且不宜小于层高的1/16；无端柱或翼墙时不应小于层高的1/12。

6.4.2 抗震墙厚度大于 140 mm 时，竖向和横向分布钢筋应双排布置；双排分布钢筋间拉筋的间距不应大于 600mm，直径不应小于 6mm；在底部加强部位，边缘构件以外的拉筋间距应适当加密。

6.4.3 **抗震墙竖向、横向分布钢筋的配筋，应符合下列要求：**

1 一、二、三级抗震墙的竖向和横向分布钢筋最小配筋率均不应小于 0.25%；四级抗震墙不应小于 0.20%；钢筋最大间距不应大于 300mm，最小直径不应小于 8mm。

2 部分框支抗震墙结构的抗震墙底部加强部位，竖向和横向分布钢筋配筋率均不应小于 0.3%，钢筋间距不应大于 200mm。

6.4.4 抗震墙竖向、横向分布钢筋的钢筋直径不宜大于墙厚的 1/10。

6.4.5 一级和二级抗震墙，底部加强部位在重力荷载代表值作用下墙肢的轴压比，一级（9 度）时不宜超过 0.4，一级（8 度）时不宜超过 0.5，二级不宜超过 0.6。

6.4.6 抗震墙两端和洞口两侧应设置边缘构件，并应符合下列要求：

1 抗震墙结构，一、二级抗震墙底部加强部位及相邻的上一层应按本章第 6.4.7 条设置约束边缘构件，但墙肢底截面在重力荷载代表值作用下的轴压比小于表 6.4.6 的 规定值时可按本章第 6.4.8 条设置构造边缘构件。

表 6.4.6 抗震墙设置构造边缘构件的最大轴压比

等级或烈度	一级（9 度）	一级（8 度）	二级
轴压比	0.1	0.2	0.3

2 部分框支抗震墙结构，一、二级落地抗震墙底部加强部位及相邻的上一层的两端应设置符合约束边缘构件要求的翼墙或端柱，洞口两侧应设置约束边缘构件；不落地抗震墙应在底部加强部位及相邻的上一层的墙肢两端设置约束边缘构件。

3 一、二级抗震墙的其他部位和三、四级抗震墙，均应按本章 6.4.8 条设置构造边缘构件。

6.4.7 抗震墙的约束边缘构件包括暗柱、端柱和翼墙（图 6.4.7）。约束边缘构件沿墙肢的长度和配箍特征值宜符合表 6.4.7 的要求，一、二级抗震墙约束边缘构件在设置箍筋范围内（即图 6.4.7 中阴影部分）的纵向钢筋配筋率，分别不应小于 1.2%和 1.0%。

表 6.4.7 约束边缘构件范围 l_c 及其配箍特征值 λ_v

项　目	一级（9 度）	一级（8 度）	二　级
λ_v	0.2	0.2	0.2
l_c（暗柱）	$0.25h_w$	$0.20h_w$	$0.20h_w$
l_c（有翼墙或端柱）	$0.20h_w$	$0.15h_w$	$0.15h_w$

注：1 抗震墙的翼墙长度小于其 3 倍厚度或端柱截面边长小于 2 倍墙厚时，视为无翼墙、无端柱；

2 l_c 为约束边缘构件沿墙肢长度，不应小于表内数值、$1.5b_w$ 和 450 mm 三者的最大值；有翼墙或端柱时尚不应小于翼墙厚度或端柱沿墙肢方向截面高度加 300 mm；

3 λ_v 为约束边缘构件的配箍特征值，计算配箍率时，箍筋或拉筋抗拉强度设计值超过 360N/mm^2，应按 360N/mm^2 计算；箍筋或拉筋沿竖向间距，一级不宜大于 100mm，二级不宜大于 150mm；

4 h_w 为抗震墙墙肢长度。

6.4.8 抗震墙的构造边缘构件的范围，宜按图 6.4.8 采用；构造边缘构件的配筋应满足

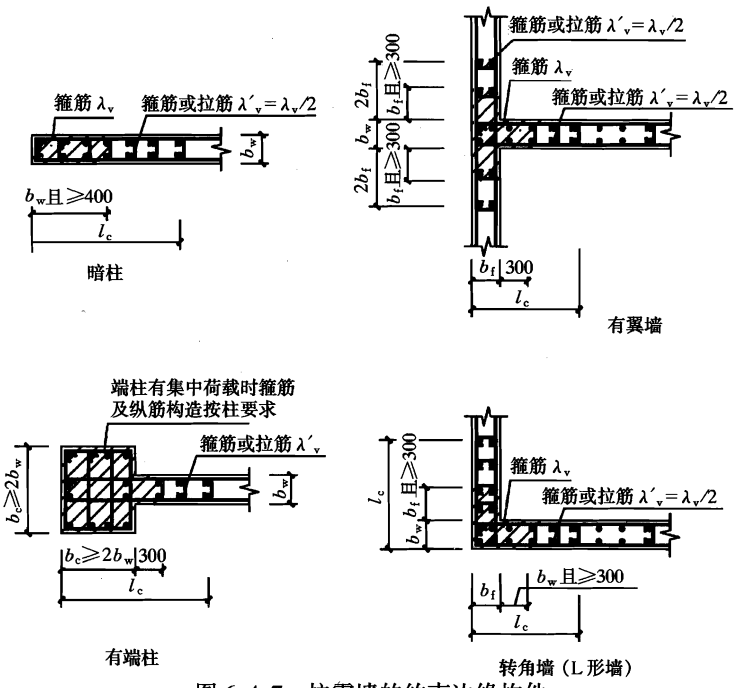


图 6.4.7　抗震墙的约束边缘构件

受弯承载力要求，并宜符合表 6.4.8 的要求。

表 6.4.8　抗震墙构造边缘构件的配筋要求

抗震等级	底部加强部位			其他部位		
	纵向钢筋最小量（取较大值）	箍筋		纵向钢筋最小量	拉筋	
		最小直径（mm）	沿竖向最大间距（mm）		最小直径（mm）	沿竖向最大间距（mm）
一	0.010A_c，6ϕ16	8	100	6ϕ14	8	150
二	0.008A_c，6ϕ14	8	150	6ϕ12	8	200
三	0.005A_c，4ϕ12	6	150	4ϕ12	6	200
四	0.005A_c，4ϕ12	6	200	4ϕ12	6	250

注：1　A_c为计算边缘构件纵向构造钢筋的暗柱或端柱面积，即图 6.4.8 抗震墙截面的阴影部分；

2　对其他部位，拉筋的水平间距不应大于纵筋间距的 2 倍，转角处宜用箍筋；

3　当端柱承受集中荷载时，其纵向钢筋、箍筋直径和间距应满足柱的相应要求。

6.4.9　抗震墙的墙肢长度不大于墙厚的 3 倍时，应按柱的要求进行设计，箍筋应沿全高加密。

6.4.10　一、二级抗震墙跨高比不大于 2 且墙厚不小于 200mm 的连梁，除普通箍筋外宜

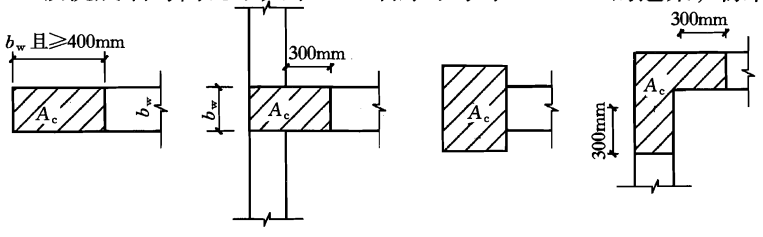


图 6.4.8　抗震墙的构造边缘构件范围

另设斜向交叉构造钢筋。

6.4.11 顶层连梁的纵向钢筋锚固长度范围内，应设置箍筋。

6.5 框架-抗震墙结构抗震构造措施

6.5.1 抗震墙的厚度不应小于160mm且不应小于层高的1/20，底部加强部位的抗震墙厚度不应小于200mm且不应小于层高的1/16，抗震墙的周边应设置梁（或暗梁）和端柱组成的边框；端柱截面宜与同层框架柱相同，并应满足本章第6.3节对框架柱的要求；抗震墙底部加强部位的端柱和紧靠抗震墙洞口的端柱宜按柱箍筋加密区的要求沿全高加密箍筋。

6.5.2 抗震墙的竖向和横向分布钢筋，配筋率均不应小于0.25%，并应双排布置，拉筋间距不应大于600mm，直径不应小于6mm。

6.5.3 框架-抗震墙结构的其他抗震构造措施，应符合本章第6.3节、6.4节对框架和抗震墙的有关要求。

6.6 板柱-抗震墙结构抗震设计要求

6.6.1 板柱-抗震墙结构的抗震墙，其抗震构造措施应符合本章第6.4节的有关规定，且底部加强部位及相邻上一层应按本章第6.4.7条设置约束边缘构件，其他部位应按第6.4.8条设置构造边缘构件；柱（包括抗震墙端柱）的抗震构造措施应符合本章第6.3节对框架柱的有关规定。

6.6.2 房屋的周边和楼、电梯洞口周边应采用有梁框架。

6.6.3 8度时宜采用有托板或柱帽的板柱节点，托板或柱帽根部的厚度（包括板厚）不宜小于柱纵筋直径的16倍。托板或柱帽的边长不宜小于4倍板厚及柱截面相应边长之和。

6.6.4 房屋的屋盖和地下一层顶板，宜采用梁板结构。

6.6.5 板柱-抗震墙结构的抗震墙，应承担结构的全部地震作用，各层板柱部分应满足计算要求，并应能承担不少于各层全部地震作用的20%。

6.6.6 板柱结构在地震作用下按等代平面框架分析时，其等代梁的宽度宜采用垂直于等代平面框架方向柱距的50%。

6.6.7 无柱帽平板宜在柱上板带中设构造暗梁，暗梁宽度可取柱宽及柱两侧各不大于1.5倍板厚。暗梁支座上部钢筋面积应不小于柱上板带钢筋面积的50%，暗梁下部钢筋不宜少于上部钢筋的1/2。

6.6.8 无柱帽柱上板带的板底钢筋，宜在距柱面为2倍纵筋锚固长度以外搭接，钢筋端部宜有垂直于板面的弯钩。

6.6.9 沿两个主轴方向通过柱截面的板底连续钢筋的总截面面积，应符合下式要求：

$$A_s \geqslant N_G / f_y \tag{6.6.9}$$

式中 A_s——板底连续钢筋总截面面积；

N_G——在该层楼板重力荷载代表值作用下的柱轴压力（设计值）；

f_y——楼板钢筋的抗拉强度设计值。

6.7 筒体结构抗震设计要求

6.7.1 框架-核心筒结构应符合下列要求：

1 核心筒与框架之间的楼盖宜采用梁板体系。

2 低于9度采用加强层时，加强层的大梁或桁架应与核心筒内的墙肢贯通；大梁或桁架与周边框架柱的连接宜采用铰接或半刚性连接。

3 结构整体分析应计入加强层变形的影响。

4 9度时不应采用加强层。

5 在施工程序及连接构造上，应采取措施减小结构竖向温度变形及轴向压缩对加强层的影响。

6.7.2 框架-核心筒结构的核心筒、筒中筒结构的内筒，其抗震墙应符合本章第6.4节的有关规定，且抗震墙的厚度、竖向和横向分布钢筋应符合本章第6.5节的规定；筒体底部加强部位及相邻上一层不应改变墙体厚度。一、二级筒体角部的边缘构件应按下列要求加强：底部加强部位，约束边缘构件沿墙肢的长度应取墙肢截面高度的1/4，且约束边缘构件范围内应全部采用箍筋；底部加强部位以上的全高范围内宜按本章图6.4.7的转角墙设置约束边缘构件，约束边缘构件沿墙肢的长度仍取墙肢截面高度的1/4。

6.7.3 内筒的门洞不宜靠近转角。

6.7.4 楼层梁不宜集中支承在内筒或核心筒的转角处，也不宜支承在洞口连梁上；内筒或核心筒支承楼层梁的位置宜设暗柱。

6.7.5 一、二级核心筒和内筒中跨高比不大于2的连梁，当梁截面宽度不小于400mm时，宜采用交叉暗柱配筋，全部剪力应由暗柱的配筋承担，并按框架梁构造要求设置普通箍筋；当梁截面宽度小于400mm且不小于200mm时，除普通箍筋外，宜另加设交叉的构造钢筋。

6.7.6 筒体结构转换层的抗震设计应符合本规范附录E.2的规定。

7 多层砌体房屋和底部框架、内框架房屋

7.1 一 般 规 定

7.1.1 本章适用于烧结普通粘土砖、烧结多孔粘土砖、混凝土小型空心砌块等砌体承重的多层房屋，底层或底部两层框架-抗震墙和多层的多排柱内框架砖砌体房屋。

配筋混凝土小型空心砌块抗震墙房屋的抗震设计，应符合本规范附录F的规定。

注：1 本章中“普通砖、多孔砖、小砌块”即“烧结普通粘土砖、烧结多孔粘土砖、混凝土小型空心砌块”的简称。采用其他烧结砖、蒸压砖的砌体房屋，块体的材料性能应有可靠的试验数据；当砌体抗剪强度不低于粘土砖砌体时，可按本章粘土砖房屋的相应规定执行；

2 6、7度时采用蒸压灰砂砖和蒸压粉煤灰砖砌体的房屋，当砌体的抗剪强度不低于粘土砖砌体的70%时，房屋的层数应比粘土砖房屋减少一层，高度应减少3m，且钢筋混凝土构造柱应按增加一层的层数所对应的粘土砖房屋设置，其他要求可按粘土砖房屋的相应规定执行。

7.1.2 多层房屋的层数和高度应符合下列要求：

1 一般情况下，房屋的层数和总高度不应超过表7.1.2的规定。

2 对医院、教学楼等及横墙较少的多层砌体房屋，总高度应比表7.1.2的规定降低3m，层数相应减少一层；各层横墙很少的多层砌体房屋，还应根据具体情况再适当降低总高度和减少层数。

注：横墙较少指同一楼层内开间大于4.20m的房间占该层总面积的40%以上。

3 横墙较少的多层砖砌体住宅楼，当按规定采取加强措施并满足抗震承载力要求时，其高度和层数应允许仍按表7.1.2的规定采用。

表7.1.2 房屋的层数和总高度限值（m）

房屋类别		最小墙厚度（mm）	烈度							
			6		7		8		9	
			高度	层数	高度	层数	高度	层数	高度	层数
多层砌体	普通砖	240	24	8	21	7	18	6	12	4
	多孔砖	240	21	7	21	7	18	6	12	4
	多孔砖	190	21	7	18	6	15	5	—	—
	小砌块	190	21	7	21	7	18	6	—	—
底部框架-抗震墙		240	22	7	22	7	19	6	—	—
多排柱内框架		240	16	5	16	5	13	4	—	—

注：1 房屋的总高度指室外地面到主要屋面板板顶或檐口的高度，半地下室从地下室室内地面算起，全地下室和嵌固条件好的半地下室应允许从室外地面算起；对带阁楼的坡屋面应算到山尖墙的1/2高度处；

2 室内外高差大于0.6m时，房屋总高度应允许比表中数据适当增加，但不应多于1m；

3 本表小砌块砌体房屋不包括配筋混凝土小型空心砌块砌体房屋。

7.1.3 普通砖、多孔砖和小砌块砌体承重房屋的层高，不应超过3.6m；底部框架-抗震墙房屋的底部和内框架房屋的层高，不应超过4.5m。

7.1.4 多层砌体房屋总高度与总宽度的最大比值，宜符合表 7.1.4 的要求。

表 7.1.4 房屋最大高宽比

烈　　度	6	7	8	9
最大高宽比	2.5	2.5	2.0	1.5

注：1 单面走廊房屋的总宽度不包括走廊宽度；

2 建筑平面接近正方形时，其高宽比宜适当减小。

7.1.5 房屋抗震横墙的间距，不应超过表 7.1.5 的要求：

表 7.1.5 房屋抗震横墙最大间距（m）

房屋类别			烈度			
			6	7	8	9
多层砌体	现浇或装配整体式钢筋混凝土楼、屋盖		18	18	15	11
	装配式钢筋混凝土楼、屋盖		15	15	11	7
	木楼、屋盖		11	11	7	4
底部框架-抗震墙		上部各层	同多层砌体房屋			—
		底层或底部两层	21	18	15	—
多排柱内框架			25	21	18	—

注：1 多层砌体房屋的顶层，最大横墙间距应允许适当放宽；

2 表中木楼、屋盖的规定，不适用于小砌块砌体房屋。

7.1.6 房屋中砌体墙段的局部尺寸限值，宜符合表 7.1.6 的要求：

表 7.1.6 房屋的局部尺寸限值（m）

部　　位	6 度	7 度	8 度	9 度
承重窗间墙最小宽度	1.0	1.0	1.2	1.5
承重外墙尽端至门窗洞边的最小距离	1.0	1.0	1.2	1.5
非承重外墙尽端至门窗洞边的最小距离	1.0	1.0	1.0	1.0
内墙阳角至门窗洞边的最小距离	1.0	1.0	1.5	2.0
无锚固女儿墙（非出入口处）的最大高度	0.5	0.5	0.5	0.0

注：1 局部尺寸不足时应采取局部加强措施弥补；

2 出入口处的女儿墙应有锚固；

3 多层多排柱内框架房屋的纵向窗间墙宽度，不应小于 1.5m。

7.1.7 多层砌体房屋的结构体系，应符合下列要求：

1 应优先采用横墙承重或纵横墙共同承重的结构体系。

2 纵横墙的布置宜均匀对称，沿平面内宜对齐，沿竖向应上下连续；同一轴线上的窗间墙宽度宜均匀。

3 房屋有下列情况之一时宜设置防震缝，缝两侧均应设置墙体，缝宽应根据烈度和房屋高度确定，可采用 50～100mm：

1）房屋立面高差在 6m 以上；

2）房屋有错层，且楼板高差较大；

3）各部分结构刚度、质量截然不同。

4 楼梯间不宜设置在房屋的尽端和转角处。

5 烟道、风道、垃圾道等不应削弱墙体；当墙体被削弱时，应对墙体采取加强措施；不宜采用无竖向配筋的附墙烟囱及出屋面的烟囱。

6 不应采用无锚固的钢筋混凝土预制挑檐。

7.1.8 底部框架-抗震墙房屋的结构布置，应符合下列要求：

1 上部的砌体抗震墙与底部的框架梁或抗震墙应对齐或基本对齐。

2 房屋的底部，应沿纵横两方向设置一定数量的抗震墙，并应均匀对称布置或基本均匀对称布置。6、7度且总层数不超过五层的底层框架-抗震墙房屋，应允许采用嵌砌于框架之间的砌体抗震墙，但应计入砌体墙对框架的附加轴力和附加剪力；其余情况应采用钢筋混凝土抗震墙。

3 底层框架-抗震墙房屋的纵横两个方向，第二层与底层侧向刚度的比值，6、7度时不应大于2.5，8度时不应大于2.0，且均不应小于1.0。

4 底部两层框架-抗震墙房屋的纵横两个方向，底层与底部第二层侧向刚度应接近，第三层与底部第二层侧向刚度的比值，6、7度时不应大于2.0，8度时不应大于1.5，且均不应小于1.0。

5 底部框架-抗震墙房屋的抗震墙应设置条形基础、筏式基础或桩基。

7.1.9 多层多排柱内框架房屋的结构布置，应符合下列要求：

1 房屋宜采用矩形平面，且立面宜规则；楼梯间横墙宜贯通房屋全宽。

2 7度时横墙间距大于18m或8度时横墙间距大于15m，外纵墙的窗间墙宜设置组合柱。

3 多排柱内框架房屋的抗震墙应设置条形基础、筏式基础或桩基。

7.1.10 底部框架-抗震墙房屋和多层多排柱内框架房屋的钢筋混凝土结构部分，除应符合本章规定外，尚应符合本规范第6章的有关要求；此时，底部框架-抗震墙房屋的框架和抗震墙的抗震等级，6、7、8度可分别按三、二、一级采用；多排柱内框架的抗震等级，6、7、8度可分别按四、三、二级采用。

7.2 计 算 要 点

7.2.1 多层砌体房屋、底部框架房屋和多层多排柱内框架房屋的抗震计算，可采用底部剪力法，并应按本节规定调整地震作用效应。

7.2.2 对砌体房屋，可只选择从属面积较大或竖向应力较小的墙段进行截面抗震承载力验算。

7.2.3 进行地震剪力分配和截面验算时，砌体墙段的层间等效侧向刚度应按下列原则确定：

1 刚度的计算应计及高宽比的影响。高宽比小于1时，可只计算剪切变形；高宽比不大于4且不小于1时，应同时计算弯曲和剪切变形；高宽比大于4时，等效侧向刚度可取0.0。

注：墙段的高宽比指层高与墙长之比，对门窗洞边的小墙段指洞净高与洞侧墙宽之比。

2 墙段宜按门窗洞口划分；对小开口墙段按毛墙面计算的刚度，可根据开洞率乘以表7.2.3的洞口影响系数：

表7.2.3 墙段洞口影响系数

开洞率	0.10	0.20	0.30
影响系数	0.98	0.94	0.88

注：开洞率为洞口面积与墙段毛面积之比；窗洞高度大于层高50%时，按门洞对待。

7.2.4 底部框架-抗震墙房屋的地震作用效应，应按下列规定调整：

1　对底层框架-抗震墙房屋，底层的纵向和横向地震剪力设计值均应乘以增大系数，其值应允许根据第二层与底层侧向刚度比值的大小在1.2～1.5范围内选用。

2　对底部两层框架-抗震墙房屋，底层和第二层的纵向和横向地震剪力设计值亦均应乘以增大系数，其值应允许根据侧向刚度比在1.2～1.5范围内选用。

3　底层或底部两层的纵向和横向地震剪力设计值应全部由该方向的抗震墙承担，并按各抗震墙侧向刚度比例分配。

7.2.5　底部框架-抗震墙房屋中，底部框架的地震作用效应宜采用下列方法确定：

1　底部框架柱的地震剪力和轴向力，宜按下列规定调整：

1）框架柱承担的地震剪力设计值，可按各抗侧力构件有效侧向刚度比例分配确定；有效侧向刚度的取值，框架不折减，混凝土墙可乘以折减系数0.30，砖墙可乘以折减系数0.20。

2）框架柱的轴力应计入地震倾覆力矩引起的附加轴力，上部砖房可视为刚体，底部各轴线承受的地震倾覆力矩，可近似按底部抗震墙和框架的侧向刚度的比例分配确定。

2　底部框架-抗震墙房屋的钢筋混凝土托墙梁计算地震组合内力时，应采用合适的计算简图。若考虑上部墙体与托墙梁的组合作用，应计入地震时墙体开裂对组合作用的不利影响，可调整有关的弯矩系数、轴力系数等计算参数。

7.2.6　多层多排柱内框架房屋各柱的地震剪力设计值，宜按下式确定：

$$V_c = \frac{\psi_c}{n_b \cdot n_s}(\zeta_1 + \zeta_2\lambda)V \tag{7.2.6}$$

式中　V_c——各柱地震剪力设计值；

V——抗震横墙间的楼层地震剪力设计值；

ψ_c——柱类型系数，钢筋混凝土内柱可采用0.012，外墙组合砖柱可采用0.0075；

n_b——抗震横墙间的开间数；

n_s——内框架的跨数；

λ——抗震横墙间距与房屋总宽度的比值，当小于0.75时，按0.75采用；

ζ_1、ζ_2——分别为计算系数，可按表7.2.6采用：

表7.2.6　计　算　系　数

房屋总层数	2	3	4	5
ζ_1	2.0	3.0	5.0	7.5
ζ_2	7.5	7.0	6.5	6.0

7.2.7　各类砌体沿阶梯形截面破坏的抗震抗剪强度设计值，应按下式确定：

$$f_{vE} = \zeta_N f_v \tag{7.2.7}$$

式中　f_{vE}——砌体沿阶梯形截面破坏的抗震抗剪强度设计值；

f_v——非抗震设计的砌体抗剪强度设计值；

ζ_N——砌体抗震抗剪强度的正应力影响系数，应按表7.2.7采用。

表 7.2.7　砌体强度的正应力影响系数

砌体类别	σ_0/f_v							
	0.0	1.0	3.0	5.0	7.0	10.0	15.0	20.0
普通砖，多孔砖	0.80	1.00	1.28	1.50	1.70	1.95	2.32	
小砌块		1.25	1.75	2.25	2.60	3.10	3.95	4.80

注：σ_0 为对应于重力荷载代表值的砌体截面平均压应力。

7.2.8　普通砖、多孔砖墙体的截面抗震受剪承载力，应按下列规定验算：

1　一般情况下，应按下式验算：

$$V \leqslant f_{vE}A/\gamma_{RE} \tag{7.2.8-1}$$

式中　V——墙体剪力设计值；

f_{vE}——砖砌体沿阶梯形截面破坏的抗震抗剪强度设计值；

A——墙体横截面面积，多孔砖取毛截面面积；

γ_{RE}——承载力抗震调整系数，承重墙按本规范表 5.4.2 采用，自承重墙按 0.75 采用。

2　当按式（7.2.8-1）验算不满足要求时，可计入设置于墙段中部、截面不小于 240mm×240mm 且间距不大于 4m 的构造柱对受剪承载力的提高作用，按下列简化方法验算：

$$V \leqslant \frac{1}{\gamma_{RE}}[\eta_c f_{vE}(A - A_c) + \zeta f_t A_c + 0.08 f_y A_s] \tag{7.2.8-2}$$

式中　A_c——中部构造柱的横截面总面积（对横墙和内纵墙，$A_c > 0.15A$ 时，取 $0.15A$；对外纵墙，$A_c > 0.25A$ 时，取 $0.25A$）；

f_t——中部构造柱的混凝土轴心抗拉强度设计值；

A_s——中部构造柱的纵向钢筋截面总面积（配筋率不小于 0.6%，大于 1.4%时取 1.4%）；

f_y——钢筋抗拉强度设计值；

ζ——中部构造柱参与工作系数；居中设一根时取 0.5，多于一根时取 0.4；

η_c——墙体约束修正系数；一般情况取 1.0，构造柱间距不大于 2.8m 时取 1.1。

7.2.9　水平配筋普通砖、多孔砖墙体的截面抗震受剪承载力，应按下式验算：

$$V \leqslant \frac{1}{\gamma_{RE}}(f_{vE}A + \zeta_s f_y A_s) \tag{7.2.9}$$

式中　A——墙体横截面面积，多孔砖取毛截面面积；

f_y——钢筋抗拉强度设计值；

A_s——层间墙体竖向截面的钢筋总截面面积，其配筋率应不小于 0.07%且不大于 0.17%；

ζ_s——钢筋参与工作系数，可按表 7.2.9 采用。

表 7.2.9　钢筋参与工作系数

墙体高宽比	0.4	0.6	0.8	1.0	1.2
ζ_s	0.10	0.12	0.14	0.15	0.12

7.2.10　小砌块墙体的截面抗震受剪承载力，应按下式验算：

$$V \leqslant \frac{1}{\gamma_{RE}}[f_{vE}A + (0.3f_tA_c + 0.05f_yA_s)\zeta_c] \tag{7.2.10}$$

式中 f_t——芯柱混凝土轴心抗拉强度设计值；

A_c——芯柱截面总面积；

A_s——芯柱钢筋截面总面积；

ζ_c——芯柱参与工作系数，可按表 7.2.10 采用。

注：当同时设置芯柱和构造柱时，构造柱截面可作为芯柱截面，构造柱钢筋可作为芯柱钢筋。

表 7.2.10 芯柱参与工作系数

填孔率 ρ	$\rho<0.15$	$0.15\leqslant\rho<0.25$	$0.25\leqslant\rho<0.5$	$\rho\geqslant0.5$
ζ_c	0.0	1.0	1.10	1.15

注：填孔率指芯柱根数（含构造柱和填实孔洞数量）与孔洞总数之比。

7.2.11 底层框架-抗震墙房屋中嵌砌于框架之间的普通砖抗震墙，当符合本章第 7.5.6 条的构造要求时，其抗震验算应符合下列规定：

1 底层框架柱的轴向力和剪力，应计入砖抗震墙引起的附加轴向力和附加剪力，其值可按下列公式确定：

$$N_f = V_wH_f/l \tag{7.2.11-1}$$

$$V_f = V_w \tag{7.2.11-2}$$

式中 V_w——墙体承担的剪力设计值，柱两侧有墙时可取二者的较大值；

N_f——框架柱的附加轴压力设计值；

V_f——框架柱的附加剪力设计值；

H_f、l——分别为框架的层高和跨度。

2 嵌砌于框架之间的普通砖抗震墙及两端框架柱，其抗震受剪承载力应按下式验算：

$$V \leqslant \frac{1}{\gamma_{REc}}\Sigma(M_{yc}^u + M_{yc}^l)/H_0 + \frac{1}{\gamma_{REw}}\Sigma f_{vE}A_{w0} \tag{7.2.11-3}$$

式中 V——嵌砌普通砖抗震墙及两端框架柱剪力设计值；

A_{w0}——砖墙水平截面的计算面积，无洞口时取实际截面的 1.25 倍，有洞口时取截面净面积，但不计入宽度小于洞口高度 1/4 的墙肢截面面积；

M_{yc}^u、M_{yc}^l——分别为底层框架柱上下端的正截面受弯承载力设计值，可按现行国家标准《混凝土结构设计规范》GB 50010 非抗震设计的有关公式取等号计算；

H_0——底层框架柱的计算高度，两侧均有砖墙时取柱净高的 2/3，其余情况取柱净高；

γ_{REc}——底层框架柱承载力抗震调整系数，可采用 0.8；

γ_{REw}——嵌砌普通砖抗震墙承载力抗震调整系数，可采用 0.9。

7.2.12 多层内框架房屋的外墙组合砖柱，其抗震验算可按本规范第 9.3.9 条的规定执行。

7.3 多层粘土砖房抗震构造措施

7.3.1 多层普通砖、多孔砖房，应按下列要求设置现浇钢筋混凝土构造柱（以下简称构

造柱)：

1 构造柱设置部位，一般情况下应符合表 7.3.1 的要求。

2 外廊式和单面走廊式的多层房屋，应根据房屋增加一层后的层数，按表 7.3.1 的要求设置构造柱，且单面走廊两侧的纵墙均应按外墙处理。

3 教学楼、医院等横墙较少的房屋，应根据房屋增加一层后的层数，按表 7.3.1 的要求设置构造柱；当教学楼、医院等横墙较少的房屋为外廊式或单面走廊式时，应按 2 款要求设置构造柱，但 6 度不超过四层、7 度不超过三层和 8 度不超过二层时，应按增加二层后的层数对待。

表 7.3.1 砖房构造柱设置要求

房屋层数				设置部位	
6 度	7 度	8 度	9 度		
四、五	三、四	二、三		外墙四角， 错层部位横墙与外纵墙交接处， 大房间内外墙交接处， 较大洞口两侧	7、8 度时，楼、电梯间的四角；隔 15m 或单元横墙与外纵墙交接处
六、七	五	四	二		隔开间横墙(轴线)与外墙交接处，山墙与内纵墙交接处；7～9 度时，楼、电梯间的四角
八	六、七	五、六	三、四		内墙(轴线)与外墙交接处，内墙的局部较小墙垛处；7～9 度时，楼、电梯间的四角；9 度时内纵墙与横墙(轴线)交接处

7.3.2 多层普通砖、多孔砖房屋的构造柱应符合下列要求：

1 构造柱最小截面可采用 240mm×180mm，纵向钢筋宜采用 4ϕ12，箍筋间距不宜大于 250mm，且在柱上下端宜适当加密；7 度时超过六层、8 度时超过五层和 9 度时，构造柱纵向钢筋宜采用 4ϕ14，箍筋间距不应大于 200mm；房屋四角的构造柱可适当加大截面及配筋。

2 构造柱与墙连接处应砌成马牙槎，并应沿墙高每隔 500mm 设 2ϕ6 拉结钢筋，每边伸入墙内不宜小于 1m。

3 构造柱与圈梁连接处，构造柱的纵筋应穿过圈梁，保证构造柱纵筋上下贯通。

4 构造柱可不单独设置基础，但应伸入室外地面下 500mm，或与埋深小于 500mm 的基础圈梁相连。

5 房屋高度和层数接近本章表 7.1.2 的限值时，纵、横墙内构造柱间距尚应符合下列要求：

1) 横墙内的构造柱间距不宜大于层高的二倍；下部 1/3 楼层的构造柱间距适当减小；

2) 当外纵墙开间大于 3.9m 时，应另设加强措施。内纵墙的构造柱间距不宜大于 4.2m。

7.3.3 多层普通砖、多孔砖房屋的现浇钢筋混凝土圈梁设置应符合下列要求：

1 装配式钢筋混凝土楼、屋盖或木楼、屋盖的砖房，横墙承重时应按表 7.3.3 的要求设置圈梁；纵墙承重时每层均应设置圈梁，且抗震横墙上的圈梁间距应比表内要求适当加密。

2 现浇或装配整体式钢筋混凝土楼、屋盖与墙体有可靠连接的房屋，应允许不另设

圈梁，但楼板沿墙体周边应加强配筋并应与相应的构造柱钢筋可靠连接。

表 7.3.3 砖房现浇钢筋混凝土圈梁设置要求

墙 类	烈 度		
	6、7	8	9
外墙和内纵墙	屋盖处及每层楼盖处	屋盖处及每层楼盖处	屋盖处及每层楼盖处
内横墙	同上；屋盖处间距不应大于 7m；楼盖处间距不应大于 15m；构造柱对应部位	同上；屋盖处沿所有横墙，且间距不应大于 7m；楼盖处间距不应大于 7m；构造柱对应部位	同上；各层所有横墙

7.3.4 多层普通砖、多孔砖房屋的现浇钢筋混凝土圈梁构造应符合下列要求：

1 圈梁应闭合，遇有洞口圈梁应上下搭接。圈梁宜与预制板设在同一标高处或紧靠板底；

2 圈梁在本节第 7.3.3 条要求的间距内无横墙时，应利用梁或板缝中配筋替代圈梁；

3 圈梁的截面高度不应小于 120mm，配筋应符合表 7.3.4 的要求；按本规范第 3.3.4 条 3 款要求增设的基础圈梁，截面高度不应小于 180mm，配筋不应少于 4ϕ12。

表 7.3.4 砖房圈梁配筋要求

配 筋	烈 度		
	6、7	8	9
最小纵筋	4ϕ10	4ϕ12	4ϕ14
最大箍筋间距（mm）	250	200	150

7.3.5 多层普通砖、多孔砖房屋的楼、屋盖应符合下列要求：

1 现浇钢筋混凝土楼板或屋面板伸进纵、横墙内的长度，均不应小于 120mm。

2 装配式钢筋混凝土楼板或屋面板，当圈梁未设在板的同一标高时，板端伸进外墙的长度不应小于 120mm，伸进内墙的长度不应小于 100mm，在梁上不应小于 80mm。

3 当板的跨度大于 4.8m 并与外墙平行时，靠外墙的预制板侧边应与墙或圈梁拉结。

4 房屋端部大房间的楼盖，8 度时房屋的屋盖和 9 度时房屋的楼、屋盖，当圈梁设在板底时，钢筋混凝土预制板应相互拉结，并应与梁、墙或圈梁拉结。

7.3.6 楼、屋盖的钢筋混凝土梁或屋架应与墙、柱（包括构造柱）或圈梁可靠连接，梁与砖柱的连接不应削弱柱截面，各层独立砖柱顶部应在两个方向均有可靠连接。

7.3.7 7 度时长度大于 7.2m 的大房间，及 8 度和 9 度时，外墙转角及内外墙交接处，应沿墙高每隔 500mm 配置 2ϕ6 拉结钢筋，并每边伸入墙内不宜小于 1m。

7.3.8 楼梯间应符合下列要求：

1 8 度和 9 度时，顶层楼梯间横墙和外墙应沿墙高每隔 500mm 设 2ϕ6 通长钢筋；9 度时其他各层楼梯间墙体应在休息平台或楼层半高处设置 60mm 厚的钢筋混凝土带或配筋砖带，其砂浆强度等级不应低于 M7.5，纵向钢筋不应少于 2ϕ10。

2 8 度和 9 度时，楼梯间及门厅内墙阳角处的大梁支承长度不应小于 500mm，并应与圈梁连接。

3 装配式楼梯段应与平台板的梁可靠连接；不应采用墙中悬挑式踏步或踏步竖肋插入墙

体的楼梯，不应采用无筋砖砌栏板。

4 突出屋顶的楼、电梯间，构造柱应伸到顶部，并与顶部圈梁连接，内外墙交接处应沿墙高每隔500mm设2ϕ6拉结钢筋，且每边伸入墙内不应小于1m。

7.3.9 坡屋顶房屋的屋架应与顶层圈梁可靠连接，檩条或屋面板应与墙及屋架可靠连接，房屋出入口处的檐口瓦应与屋面构件锚固；8度和9度时，顶层内纵墙顶宜增砌支承山墙的踏步式墙垛。

7.3.10 门窗洞处不应采用无筋砖过梁；过梁支承长度，6～8度时不应小于240mm，9度时不应小于360mm。

7.3.11 预制阳台应与圈梁和楼板的现浇板带可靠连接。

7.3.12 后砌的非承重砌体隔墙应符合本规范第13.3节的有关规定。

7.3.13 同一结构单元的基础（或桩承台），宜采用同一类型的基础，底面宜埋置在同一标高上，否则应增设基础圈梁并应按1:2的台阶逐步放坡。

7.3.14 横墙较少的多层普通砖、多孔砖住宅楼的总高度和层数接近或达到表7.1.2规定限值，应采取下列加强措施：

1 房屋的最大开间尺寸不宜大于6.6m。

2 同一结构单元内横墙错位数量不宜超过横墙总数的1/3，且连续错位不宜多于两道；错位的墙体交接处均应增设构造柱，且楼、屋面板应采用现浇钢筋混凝土板。

3 横墙和内纵墙上洞口的宽度不宜大于1.5m；外纵墙上洞口的宽度不宜大于2.1m或开间尺寸的一半；且内外墙上洞口位置不应影响内外纵墙与横墙的整体连接。

4 所有纵横墙均应在楼、屋盖标高处设置加强的现浇钢筋混凝土圈梁：圈梁的截面高度不宜小于150mm，上下纵筋各不应少于3ϕ10，箍筋不小于ϕ6，间距不大于300mm。

5 所有纵横墙交接处及横墙的中部，均应增设满足下列要求的构造柱：在横墙内的柱距不宜大于层高，在纵墙内的柱距不宜大于4.2m，最小截面尺寸不宜小于240mm×240mm，配筋宜符合表7.3.14的要求。

表7.3.14 增设构造柱的纵筋和箍筋设置要求

位置	纵向钢筋			箍筋		
	最大配筋率（%）	最小配筋率（%）	最小直径（mm）	加密区范围（mm）	加密区间距（mm）	最小直径（mm）
角柱	1.8	0.8	14	全高	100	6
边柱			14	上端700 下端500		
中柱	1.4	0.6	12			

6 同一结构单元的楼、屋面板应设置在同一标高处。

7 房屋底层和顶层的窗台标高处，宜设置沿纵横墙通长的水平现浇钢筋混凝土带；其截面高度不小于60mm，宽度不小于240mm，纵向钢筋不少于3ϕ6。

7.4 多层砌块房屋抗震构造措施

7.4.1 小砌块房屋应按表7.4.1的要求设置钢筋混凝土芯柱，对医院、教学楼等横墙较少的房屋，应根据房屋增加一层后的层数，按表7.4.1的要求设置芯柱。

7.4.2 小砌块房屋的芯柱，应符合下列构造要求：

1 小砌块房屋芯柱截面不宜小于120mm×120mm。

2　芯柱混凝土强度等级，不应低于C20。

3　芯柱的竖向插筋应贯通墙身且与圈梁连接；插筋不应小于1ϕ12，7度时超过五层、8度时超过四层，插筋不应小于1ϕ14。

表7.4.1　小砌块房屋芯柱设置要求

房屋层数			设置部位	设置数量
6度	7度	8度		
四、五	三、四	二、三	外墙转角，楼梯间四角；大房间内外墙交接处；隔15m或单元横墙与外纵墙交接处	外墙转角，灌实3个孔；内外墙交接处，灌实4个孔
六	五	四	外墙转角，楼梯间四角，大房间内外墙交接处，山墙与内纵墙交接处，隔开间横墙（轴线）与外纵墙交接处	
七	六	五	外墙转角，楼梯间四角；各内墙（轴线）与外纵墙交接处；8、9度时，内纵墙与横墙（轴线）交接处和洞口两侧	外墙转角，灌实5个孔；内外墙交接处，灌实4个孔；内墙交接处，灌实4～5个孔；洞口两侧各灌实1个孔
	七	六	同上； 横墙内芯柱间距不宜大于2m	外墙转角，灌实7个孔；内外墙交接处，灌实5个孔；内墙交接处，灌实4～5个孔；洞口两侧各灌实1个孔

注：外墙转角、内外墙交接处、楼电梯间四角等部位，应允许采用钢筋混凝土构造柱替代部分芯柱。

4　芯柱应伸入室外地面下500mm或与埋深小于500mm的基础圈梁相连。

5　为提高墙体抗震受剪承载力而设置的芯柱，宜在墙体内均匀布置，最大净距不宜大于2.0m。

7.4.3　小砌块房屋中替代芯柱的钢筋混凝土构造柱，应符合下列构造要求：

1　构造柱最小截面可采用190mm×190mm，纵向钢筋宜采用4ϕ12，箍筋间距不宜大于250mm，且在柱上下端宜适当加密；7度时超过五层、8度时超过四层，构造柱纵向钢筋宜采用4ϕ14，箍筋间距不应大于200mm；外墙转角的构造柱可适当加大截面及配筋。

2　构造柱与砌块墙连接处应砌成马牙槎，与构造柱相邻的砌块孔洞，6度时宜填实，7度时应填实，8度时应填实并插筋；沿墙高每隔600mm应设拉结钢筋网片，每边伸入墙内不宜小于1m。

3　构造柱与圈梁连接处，构造柱的纵筋应穿过圈梁，保证构造柱纵筋上下贯通。

4　构造柱可不单独设置基础，但应伸入室外地面下500mm，或与埋深小于500mm的基础圈梁相连。

7.4.4　小砌块房屋的现浇钢筋混凝土圈梁应按表7.4.4的要求设置，圈梁宽度不应小于190mm，配筋不应少于4ϕ12，箍筋间距不应大于200mm。

表7.4.4　小砌块房屋现浇钢筋混凝土圈梁设置要求

墙　类	烈　度	
	6、7	8
外墙和内纵墙	屋盖处及每层楼盖处	屋盖处及每层楼盖处
内横墙	同上；屋盖处沿所有横墙； 楼盖处间距不应大于7m； 构造柱对应部位	同上；各层所有横墙

7.4.5 小砌块房屋墙体交接处或芯柱与墙体连接处应设置拉结钢筋网片，网片可采用直径 4mm 的钢筋点焊而成，沿墙高每隔 600mm 设置，每边伸入墙内不宜小于 1m。

7.4.6 小砌块房屋的层数，6 度时七层、7 度时超过五层、8 度时超过四层，在底层和顶层的窗台标高处，沿纵横墙应设置通长的水平现浇钢筋混凝土带；其截面高度不小于 60mm，纵筋不少于 2ϕ10，并应有分布拉结钢筋；其混凝土强度等级不应低于 C20。

7.4.7 小砌块房屋的其他抗震构造措施，应符合本章第 7.3.5 条至 7.3.13 条有关要求。

7.5 底部框架-抗震墙房屋抗震构造措施

7.5.1 底部框架-抗震墙房屋的上部应设置钢筋混凝土构造柱，并应符合下列要求：

1 钢筋混凝土构造柱的设置部位，应根据房屋的总层数按本章第 7.3.1 条的规定设置。过渡层尚应在底部框架柱对应位置处设置构造柱。

2 构造柱的截面，不宜小于 240mm×240mm。

3 构造柱的纵向钢筋不宜少于 4ϕ14，箍筋间距不宜大于 200mm。

4 过渡层构造柱的纵向钢筋，7 度时不宜少于 4ϕ16，8 度时不宜少于 6ϕ16。一般情况下，纵向钢筋应锚入下部的框架柱内；当纵向钢筋锚固在框架梁内时，框架梁的相应位置应加强。

5 构造柱应与每层圈梁连接，或与现浇楼板可靠拉结。

7.5.2 上部抗震墙的中心线宜同底部的框架梁、抗震墙的轴线相重合；构造柱宜与框架柱上下贯通。

7.5.3 底部框架-抗震墙房屋的楼盖应符合下列要求：

1 过渡层的底板应采用现浇钢筋混凝土板，板厚不应小于 120mm；并应少开洞、开小洞，当洞口尺寸大于 800mm 时，洞口周边应设置边梁。

2 其他楼层，采用装配式钢筋混凝土楼板时均应设现浇圈梁，采用现浇钢筋混凝土楼板时应允许不另设圈梁，但楼板沿墙体周边应加强配筋并应与相应的构造柱可靠连接。

7.5.4 底部框架-抗震墙房屋的钢筋混凝土托墙梁，其截面和构造应符合下列要求：

1 梁的截面宽度不应小于 300mm，梁的截面高度不应小于跨度的 1/10。

2 箍筋的直径不应小于 8mm，间距不应大于 200mm；梁端在 1.5 倍梁高且不小于 1/5梁净跨范围内，以及上部墙体的洞口处和洞口两侧各 500mm 且不小于梁高的范围内，箍筋间距不应大于 100mm。

3 沿梁高应设腰筋，数量不应少于 2ϕ14，间距不应大于 200mm。

4 梁的主筋和腰筋应按受拉钢筋的要求锚固在柱内，且支座上部的纵向钢筋在柱内的锚固长度应符合钢筋混凝土框支梁的有关要求。

7.5.5 底部的钢筋混凝土抗震墙，其截面和构造应符合下列要求：

1 抗震墙周边应设置梁（或暗梁）和边框柱（或框架柱）组成的边框；边框梁的截面宽度不宜小于墙板厚度的 1.5 倍，截面高度不宜小于墙板厚度的 2.5 倍；边框柱的截面高度不宜小于墙板厚度的 2 倍。

2 抗震墙墙板的厚度不宜小于 160mm，且不应小于墙板净高的 1/20；抗震墙宜开设洞口形成若干墙段，各墙段的高宽比不宜小于 2。

3 抗震墙的竖向和横向分布钢筋配筋率均不应小于 0.25%，并应采用双排布置；双

排分布钢筋间拉筋的间距不应大于600mm，直径不应小于6mm。

4 抗震墙的边缘构件可按本规范第6.4节关于一般部位的规定设置。

7.5.6 底层框架-抗震墙房屋的底层采用普通砖抗震墙时，其构造应符合下列要求：

1 墙厚不应小于240mm，砌筑砂浆强度等级不应低于M10，应先砌墙后浇框架。

2 沿框架柱每隔500mm配置2ϕ6拉结钢筋，并沿砖墙全长设置；在墙体半高处尚应设置与框架柱相连的钢筋混凝土水平系梁。

3 墙长大于5m时，应在墙内增设钢筋混凝土构造柱。

7.5.7 底部框架-抗震墙房屋的材料强度等级，应符合下列要求：

1 框架柱、抗震墙和托墙梁的混凝土强度等级，不应低于C30。

2 过渡层墙体的砌筑砂浆强度等级，不应低于M7.5。

7.5.8 底部框架-抗震墙房屋的其他抗震构造措施，应符合本章第7.3.5条至7.3.14条有关要求。

7.6 多排柱内框架房屋抗震构造措施

7.6.1 多层多排柱内框架房屋的钢筋混凝土构造柱设置，应符合下列要求：

1 下列部位应设置钢筋混凝土构造柱：

1）外墙四角和楼、电梯间四角；楼梯休息平台梁的支承部位；

2）抗震墙两端及未设置组合柱的外纵墙、外横墙上对应于中间柱列轴线的部位。

2 构造柱的截面，不宜小于240mm×240mm。

3 构造柱的纵向钢筋不宜少于4ϕ14，箍筋间距不宜大于200mm。

4 构造柱应与每层圈梁连接，或与现浇楼板可靠拉结。

7.6.2 多层多排柱内框架房屋的楼、屋盖，应采用现浇或装配整体式钢筋混凝土板。采用现浇钢筋混凝土楼板时应允许不设圈梁，但楼板沿墙体周边应加强配筋并应与相应的构造柱可靠连接。

7.6.3 多排柱内框架梁在外纵墙、外横墙上的搁置长度不应小于300mm，且梁端应与圈梁或组合柱、构造柱连接。

7.6.4 多排柱内框架房屋的其他抗震构造措施应符合本章第7.3.5条至7.3.13条有关要求。

8 多层和高层钢结构房屋

8.1 一 般 规 定

8.1.1 本章适用的钢结构民用房屋的结构类型和最大高度应符合表 8.1.1 的规定。平面和竖向均不规则或建造于Ⅳ类场地的钢结构，适用的最大高度应适当降低。

注：多层钢结构厂房的抗震设计，应符合本规范附录 G 的规定。

表 8.1.1 钢结构房屋适用的最大高度（m）

结 构 类 型	6、7 度	8 度	9 度
框架	110	90	50
框架-支撑（抗震墙板）	220	200	140
筒体(框筒,筒中筒,桁架筒,束筒)和巨型框架	300	260	180

注：1 房屋高度指室外地面到主要屋面板板顶的高度（不包括局部突出屋顶部分）；

2 超过表内高度的房屋，应进行专门研究和论证，采取有效的加强措施。

8.1.2 本章适用的钢结构民用房屋的最大高宽比不宜超过表 8.1.2 的规定，

表 8.1.2 钢结构民用房屋适用的最大高宽比

烈 度	6、7	8	9
最大高宽比	6.5	6.0	5.5

注：计算高宽比的高度从室外地面算起。

8.1.3 钢结构房屋应根据烈度、结构类型和房屋高度，采用不同的地震作用效应调整系数，并采取不同的抗震构造措施。

8.1.4 钢结构房屋宜避免采用本规范第 3.4 节规定的不规则建筑结构方案，不设防震缝；需要设置防震缝时，缝宽应不小于相应钢筋混凝土结构房屋的 1.5 倍。

8.1.5 不超过 12 层的钢结构房屋可采用框架结构、框架-支撑结构或其他结构类型；超过 12 层的钢结构房屋，8、9 度时，宜采用偏心支撑、带竖缝钢筋混凝土抗震墙板、内藏钢支撑钢筋混凝土墙板或其他消能支撑及筒体结构。

8.1.6 采用框架-支撑结构时，应符合下列规定：

1 支撑框架在两个方向的布置均宜基本对称，支撑框架之间楼盖的长宽比不宜大于 3。

2 不超过 12 层的钢结构宜采用中心支撑，有条件时也可采用偏心支撑等消能支撑。超过 12 层的钢结构采用偏心支撑框架时，顶层可采用中心支撑。

3 中心支撑框架宜采用交叉支撑，也可采用人字支撑或单斜杆支撑，不宜采用 K 形支撑；支撑的轴线应交汇于梁柱构件轴线的交点，确有困难时偏离中心不应超过支撑杆件宽度，并应计入由此产生的附加弯矩。

4 偏心支撑框架的每根支撑应至少有一端与框架梁连接，并在支撑与梁交点和柱之间或同一跨内另一支撑与梁交点之间形成消能梁段。

8.1.7 钢结构的楼盖宜采用压型钢板现浇钢筋混凝土组合楼板或非组合楼板。对不超过 12 层的钢结构尚可采用装配整体式钢筋混凝土楼板，亦可采用装配式楼板或其他轻型楼

盖；对超过 12 层的钢结构，必要时可设置水平支撑。

采用压型钢板钢筋混凝土组合楼板和现浇钢筋混凝土楼板时，应与钢梁有可靠连接。采用装配式、装配整体式钢筋混凝土楼板或轻型楼板时，应将楼板预埋件与钢梁焊接，或采取其他保证楼盖整体性的措施。

8.1.8 超过 12 层的钢框架-筒体结构，在必要时可设置由筒体外伸臂或外伸臂和周边桁架组成的加强层。

8.1.9 钢结构房屋设置地下室时，框架-支撑（抗震墙板）结构中竖向连续布置的支撑（抗震墙板）应延伸至基础；框架柱应至少延伸至地下一层。

8.1.10 超过 12 层的钢结构应设置地下室。其基础埋置深度，当采用天然地基时不宜小于房屋总高度的 1/15；当采用桩基时，桩承台埋深不宜小于房屋总高度的 1/20。

8.2 计 算 要 点

8.2.1 钢结构应按本节规定调整地震作用效应，其层间变形应符合本规范第 5.5 节的有关规定；构件截面和连接的抗震验算时，凡本章未作规定者，应符合现行有关结构设计规范的要求，但其非抗震的构件、连接的承载力设计值应除以本规范规定的承载力抗震调整系数。

8.2.2 钢结构在多遇地震下的阻尼比，对不超过 12 层的钢结构可采用 0.035，对超过 12 层的钢结构可采用 0.02；在罕遇地震下的分析，阻尼比可采用 0.05。

8.2.3 钢结构在地震作用下的内力和变形分析，应符合下列规定：

1 钢结构应按本规范第 3.6.3 条规定计入重力二阶效应。对框架梁，可不按柱轴线处的内力而按梁端内力设计。对工字形截面柱，宜计入梁柱节点域剪切变形对结构侧移的影响；中心支撑框架和不超过 12 层的钢结构，其层间位移计算可不计入梁柱节点域剪切变形的影响。

2 钢框架-支撑结构的斜杆可按端部铰接杆计算；框架部分按计算得到的地震剪力应乘以调整系数，达到不小于结构底部总地震剪力的 25%和框架部分地震剪力最大值 1.8 倍二者的较小者。

3 中心支撑框架的斜杆轴线偏离梁柱轴线交点不超过支撑杆件的宽度时，仍可按中心支撑框架分析，但应计及由此产生的附加弯矩；人字形和 V 形支撑组合的内力设计值应乘以增大系数，其值可采用 1.5。

4 偏心支撑框架构件的内力设计值，应按下列要求调整：

1）支撑斜杆的轴力设计值，应取与支撑斜杆相连接的消能梁段达到受剪承载力时支撑斜杆轴力与增大系数的乘积，其值在 8 度及以下时不应小于 1.4，9 度时不应小于 1.5；

2）位于消能梁段同一跨的框架梁内力设计值，应取消能梁段达到受剪承载力时框架梁内力与增大系数的乘积，其值在 8 度及以下时不应小于 1.5，9 度时不应小于 1.6；

3）框架柱的内力设计值，应取消能梁段达到受剪承载力时柱内力与增大系数的乘积，其值在 8 度及以下时不应小于 1.5，9 度时不应小于 1.6。

5 内藏钢支撑钢筋混凝土墙板和带竖缝钢筋混凝土墙板应按有关规定计算，带竖缝

钢筋混凝土墙板可仅承受水平荷载产生的剪力，不承受竖向荷载产生的压力。

6 钢结构转换层下的钢框架柱，地震内力应乘以增大系数，其值可采用1.5。

8.2.4 钢框架梁的上翼缘采用抗剪连接件与组合楼板连接时，可不验算地震作用下的整体稳定。

8.2.5 钢框架构件及节点的抗震承载力验算，应符合下列规定：

1 节点左右梁端和上下柱端的全塑性承载力应符合式（8.2.5-1）要求。当柱所在楼层的受剪承载力比上一层的受剪承载力高出25%，或柱轴向力设计值与柱全截面面积和钢材抗拉强度设计值乘积的比值不超过0.4，或作为轴心受压构件在2倍地震力下稳定性得到保证时，可不按该式验算。

$$\Sigma W_{pc}(f_{yc} - N/A_c) \geqslant \eta \Sigma W_{pb} f_{yb} \tag{8.2.5-1}$$

式中 W_{pc}、W_{pb}——分别为柱和梁的塑性截面模量；

N——柱轴向压力设计值；

A_c——柱截面面积；

f_{yc}、f_{yb}——分别为柱和梁的钢材屈服强度；

η——强柱系数，超过6层的钢框架，6度Ⅳ类场地和7度时可取1.0，8度时可取1.05，9度时可取1.15。

2 节点域的屈服承载力应符合下式要求：

$$\psi(M_{pb1} + M_{pb2})/V_p \leqslant (4/3) f_v \tag{8.2.5-2}$$

工字形截面柱

$$V_p = h_b h_c t_w \tag{8.2.5-3}$$

箱形截面柱

$$V_p = 1.8 h_b h_c t_w \tag{8.2.5-4}$$

3 工字形截面柱和箱形截面柱的节点域应按下列公式验算：

$$t_w \geqslant (h_b + h_c)/90 \tag{8.2.5-5}$$

$$(M_{b1} + M_{b2})/V_p \leqslant (4/3) f_v/\gamma_{RE} \tag{8.2.5-6}$$

式中 M_{pb1}、M_{pb2}——分别为节点域两侧梁的全塑性受弯承载力；

V_p——节点域的体积；

f_v——钢材的抗剪强度设计值；

ψ——折减系数，6度Ⅳ类场地和7度时可取0.6，8、9度时可取0.7；

h_b、h_c——分别为梁腹板高度和柱腹板高度；

t_w——柱在节点域的腹板厚度；

M_{b1}、M_{b2}——分别为节点域两侧梁的弯矩设计值；

γ_{RE}——节点域承载力抗震调整系数，取0.85。

注：当柱节点域腹板厚度不小于梁、柱截面高度之和的1/70时，可不验算节点域的稳定性。

8.2.6 中心支撑框架构件的抗震承载力验算，应符合下列规定：

1 支撑斜杆的受压承载力应按下式验算：

$$N/(\varphi A_{br}) \leqslant \psi f/\gamma_{RE} \tag{8.2.6-1}$$

$$\psi = 1/(1 + 0.35\lambda_n) \tag{8.2.6-2}$$

$$\lambda_n = (\lambda/\pi)\sqrt{f_{ay}/E} \tag{8.2.6-3}$$

式中 N——支撑斜杆的轴向力设计值；

A_{br}——支撑斜杆的截面面积；

φ——轴心受压构件的稳定系数；

ψ——受循环荷载时的强度降低系数；

λ_n——支撑斜杆的正则化长细比；

E——支撑斜杆材料的弹性模量；

f_{ay}——钢材屈服强度；

γ_{RE}——支撑承载力抗震调整系数。

2 人字支撑和V形支撑的横梁在支撑连接处应保持连续，该横梁应承受支撑斜杆传来的内力，并宜计入受压支撑屈服后产生的不平衡力，尚应按不计入支撑支点作用的简支梁验算重力荷载下的承载力。

注：顶层和塔屋的梁可不执行本款规定。

8.2.7 偏心支撑框架构件的抗震承载力验算，应符合下列规定：

1 偏心支撑框架消能梁段的受剪承载力应按下列公式验算：

当 $N \leqslant 0.15Af$ 时

$$V \leqslant \varphi V_l / \gamma_{RE} \tag{8.2.7-1}$$

$$V_l = 0.58A_w f_{ay} \quad 或 \quad V_l = 2M_{lp}/a,取较小值$$

$$A_w = (h - 2t_f)t_w$$

$$M_{lp} = W_p f$$

当 $N > 0.15Af$ 时

$$V \leqslant \varphi V_{lc} / \gamma_{RE} \tag{8.2.7-2}$$

$$V_{lc} = 0.58A_w f_{ay}\sqrt{1 - [N/(Af)]^2}$$

$$或 \quad V_{lc} = 2.4M_{lp}[1 - N/(Af)]/a,取较小值$$

式中 φ——系数，可取0.9；

V、N——分别为消能梁段的剪力设计值和轴力设计值；

V_l、V_{lc}——分别为消能梁段的受剪承载力和计入轴力影响的受剪承载力；

M_{lp}——消能梁段的全塑性受弯承载力；

a、h、t_w、t_f——分别为消能梁段的长度、截面高度、腹板厚度和翼缘厚度；

A、A_w——分别为消能梁段的截面面积和腹板截面面积；

W_p——消能梁段的塑性截面模量；

f、f_{ay}——分别为消能梁段钢材的抗拉强度设计值和屈服强度；

γ_{RE}——消能梁段承载力抗震调整系数，取0.85。

注：消能梁段指偏心支撑框架中斜杆与梁交点和柱之间的区段或同一跨内相邻两个斜杆与梁交点之间的区段，地震时消能梁段屈服而使其余区段仍处于弹性受力状态。

2 支撑斜杆与消能梁段连接的承载力不得小于支撑的承载力。若支撑需抵抗弯矩，

支撑与梁的连接应按抗压弯连接设计。

8.2.8 钢结构构件连接应按地震组合内力进行弹性设计，并应进行极限承载力验算：

1 梁与柱连接弹性设计时，梁上下翼缘的端截面应满足连接的弹性设计要求，梁腹板应计入剪力和弯矩。梁与柱连接的极限受弯、受剪承载力，应符合下列要求：

$$M_u \geqslant 1.2M_p \tag{8.2.8-1}$$

$$V_u \geqslant 1.3(2M_p/l_n) \text{ 且 } V_u \geqslant 0.58h_w t_w f_{ay} \tag{8.2.8-2}$$

式中 M_u——梁上下翼缘全熔透坡口焊缝的极限受弯承载力；

V_u——梁腹板连接的极限受剪承载力；垂直于角焊缝受剪时，可提高 1.22 倍；

M_p——梁（梁贯通时为柱）的全塑性受弯承载力；

l_n——梁的净跨（梁贯通时取该楼层柱的净高）；

h_w、t_w——梁腹板的高度和厚度；

f_{ay}——钢材屈服强度。

2 支撑与框架的连接及支撑拼接的极限承载力，应符合下式要求：

$$N_{ubr} \geqslant 1.2\ A_n f_{ay} \tag{8.2.8-3}$$

式中 N_{ubr}——螺栓连接和节点板连接在支撑轴线方向的极限承载力；

A_n——支撑的截面净面积；

f_{ay}——支撑钢材的屈服强度。

3 梁、柱构件拼接的弹性设计时，腹板应计入弯矩，且受剪承载力不应小于构件截面受剪承载力的 50%；拼接的极限承载力，应符合下列要求：

$$V_u \geqslant 0.58h_w t_w f_{ay} \tag{8.2.8-4}$$

无轴向力时

$$M_u \geqslant 1.2\ M_p \tag{8.2.8-5}$$

有轴向力时

$$M_u \geqslant 1.2\ M_{pc} \tag{8.2.8-6}$$

式中 M_u、V_u——分别为构件拼接的极限受弯、受剪承载力；

M_{pc}——构件有轴向力时的全截面受弯承载力；

h_w、t_w——拼接构件截面腹板的高度和厚度；

f_{ay}——被拼接构件的钢材屈服强度。

拼接采用螺栓连接时，尚应符合下列要求：

翼缘

$$nN_{cu}^b \geqslant 1.2A_f f_{ay}$$

$$\text{且}\quad nN_{vu}^b \geqslant 1.2A_f f_{ay} \tag{8.2.8-7}$$

腹板

$$N_{cu}^b \geqslant \sqrt{(V_u/n)^2 + (N_M^b)^2}$$

$$\text{且}\quad N_{vu}^b \geqslant \sqrt{(V_u/n)^2 + (N_M^b)^2} \tag{8.2.8-8}$$

式中 N_{vu}^b、N_{cu}^b——一个螺栓的极限受剪承载力和对应的板件极限承压力；

A_f——翼缘的有效截面面积；

N_M^b——腹板拼接中弯矩引起的一个螺栓的最大剪力；

n——翼缘拼接或腹板拼接一侧的螺栓数。

4 梁、柱构件有轴力时的全截面受弯承载力，应按下列公式计算：

工字形截面（绕强轴）和箱形截面

当 $N/N_y \leqslant 0.13$ 时　　$M_{pc}=M_p$　　(8.2.8-9)

当 $N/N_y > 0.13$ 时　　$M_{pc}=1.15(1-N/N_y)M_p$　　(8.2.8-10)

工字形截面（绕弱轴）

当 $N/N_y \leqslant A_w/A$ 时　　$M_{pc}=M_p$　　(8.2.8-11)

当 $N/N_y > A_w/A$ 时

$$M_{pc}=\{1-[(N-A_w f_{ay})/(N_y-A_w f_{ay})]^2\}M_p \qquad (8.2.8\text{-}12)$$

式中　N_y——构件轴向屈服承载力，取 $N_y=A_n f_{ay}$。

5 焊缝的极限承载力应按下列公式计算：

对接焊缝受拉　　$N_u = A_f^w f_u$　　(8.2.8-13)

角焊缝受剪　　$V_u = 0.58A_f^w f_u$　　(8.2.8-14)

式中　A_f^w——焊缝的有效受力面积；

f_u——构件母材的抗拉强度最小值。

6 高强度螺栓连接的极限受剪承载力，应取下列二式计算的较小者：

$$N_{vu}^b = 0.58n_f A_e^b f_u^b \qquad (8.2.8\text{-}15)$$

$$N_{cu}^b = d\Sigma t f_{cu}^b \qquad (8.2.8\text{-}16)$$

式中　N_{vu}^b、N_{cu}^b——分别为一个高强度螺栓的极限受剪承载力和对应的板件极限承压力；

n_f——螺栓连接的剪切面数量；

A_e^b——螺栓螺纹处的有效截面面积；

f_u^b——螺栓钢材的抗拉强度最小值；

d——螺栓杆直径；

Σt——同一受力方向的钢板厚度之和；

f_{cu}^b——螺栓连接板的极限承压强度，取 $1.5f_u$。

8.3　钢框架结构抗震构造措施

8.3.1　框架柱的长细比，应符合下列规定：

1　不超过 12 层的钢框架柱的长细比，6～8 度时不应大于 $120\sqrt{235/f_{ay}}$，9 度时不应大于 $100\sqrt{235/f_{ay}}$。

2　超过 12 层的钢框架柱的长细比，应符合表 8.3.1 的规定：

表 8.3.1　超过 12 层框架的柱长细比限值

烈　度	6 度	7 度	8 度	9 度
长 细 比	120	80	60	60

注：表列数值适用于 Q235 钢，采用其他牌号钢材时，应乘以 $\sqrt{235/f_{ay}}$。

8.3.2 框架梁、柱板件宽厚比应符合下列规定：

1 不超过 12 层框架的梁、柱板件宽厚比应符合表 8.3.2-1 的要求：

表 8.3.2-1 不超过 12 层框架的梁柱板件宽厚比限值

板件名称		7 度	8 度	9 度
柱	工字形截面翼缘外伸部分	13	12	11
	箱形截面壁板	40	36	36
	工字形截面腹板	52	48	44
梁	工字形截面和箱形截面翼缘外伸部分	11	10	9
	箱形截面翼缘在两腹板间的部分	36	32	30
	工字形截面和箱形截面腹板			
	$(N_b/Af<0.37)$	$85-120N_b/Af$	$80-110N_b/Af$	$72-100N_b/Af$
	$(N_b/Af\geqslant 0.37)$	40	39	35

注：表列数值适用于 Q235，当材料为其他牌号钢材时，应乘以 $\sqrt{235/f_{ay}}$；N_b 为梁轴向力。

2 超过 12 层框架梁、柱板件宽厚比应符合表 8.3.2-2 的规定：

表 8.3.2-2 超过 12 层框架的梁柱板件宽厚比限值

板件名称		6 度	7 度	8 度	9 度
柱	工字形截面翼缘外伸部分	13	11	10	9
	工字形截面腹板	43	43	43	43
	箱形截面壁板	39	37	35	33
梁	工字形截面和箱形截面翼缘外伸分	11	10	9	9
	箱形截面翼缘在两腹板间的部分	36	32	30	30
	工字形截面和箱形截面腹板	$85-120N_b/Af$	$80-110N_b/Af$	$72-100N_b/Af$	$72-100N_b/Af$

注：表列数值适用于 Q235 钢，采用其他牌号钢材时，应乘以 $\sqrt{235/f_{ay}}$。

8.3.3 梁柱构件的侧向支承应符合下列要求：

1 梁柱构件在出现塑性铰的截面处，其上下翼缘均应设置侧向支承。

2 相邻两支承点间的构件长细比，应符合国家标准《钢结构设计规范》GB50017 关于塑性设计的有关规定。

8.3.4 梁与柱的连接构造，应符合下列要求：

1 梁与柱的连接宜采用柱贯通型。

2 柱在两个互相垂直的方向都与梁刚接时，宜采用箱形截面。当仅在一个方向刚接时，宜采用工字形截面，并将柱腹板置于刚接框架平面内。

3 工字形截面柱（翼缘）和箱形截面柱与梁刚接时，应符合下列要求（图 8.3.4-1），有充分依据时也可采用其他构造形式。

1）梁翼缘与柱翼缘间应采用全熔透坡口焊缝；8 度乙类建筑和 9 度时，应检验 V 形切口的冲击韧性，其恰帕冲击韧性在 −20℃时不低于 27J；

2）柱在梁翼缘对应位置设置横向加劲肋，且加劲肋厚度不应小于梁翼缘厚度；

3）梁腹板宜采用摩擦型高强度螺栓通过连接板与柱连接；腹板角部宜设置扇形切角，其端部与梁翼缘的全熔透焊缝应隔开；

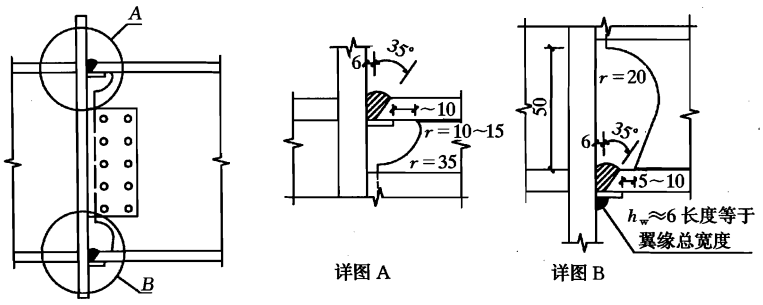


图 8.3.4-1　框架梁与柱的现场连接

4）当梁翼缘的塑性截面模量小于梁全截面塑性截面模量的 70%时，梁腹板与柱的连接螺栓不得少于二列；当计算仅需一列时，仍应布置二列，且此时螺栓总数不得少于计算值的 1.5 倍；

5）8 度Ⅲ、Ⅳ场地和 9 度时，宜采用能将塑性铰自梁端外移的骨形连接。

4　框架梁采用悬臂梁段与柱刚性连接时（图 8.3.4-2），悬臂梁段与柱应预先采用全焊接连接，梁的现场拼接可采用翼缘焊接腹板螺栓连接（*a*）或全部螺栓连接（*b*）。

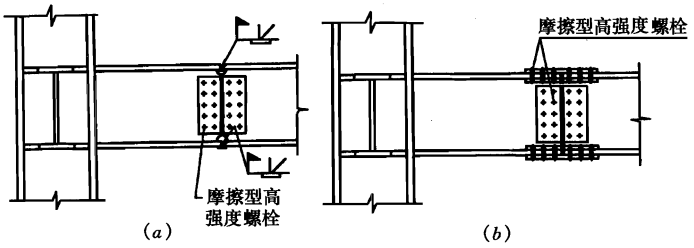


图 8.3.4-2　框架梁与柱通过梁悬臂段的连接

5　箱形截面柱在与梁翼缘对应位置设置的隔板应采用全熔透对接焊缝与壁板相连。工字形截面柱的横向加劲肋与柱翼缘应采用全熔透对接焊缝连接，与腹板可采用角焊缝连接。

8.3.5　当节点域的体积不满足本章第 8.2.5 条 3 款的规定时，应采取加厚节点域或贴焊补强板的措施。补强板的厚度及其焊缝应按传递补强板所分担剪力的要求设计。

8.3.6　梁与柱刚性连接时，柱在梁翼缘上下各 500mm 的节点范围内，柱翼缘与柱腹板间或箱形柱壁板间的连接焊缝，应采用坡口全熔透焊缝。

8.3.7　框架柱接头宜位于框架梁上方 1.3m 附近。

上下柱的对接接头应采用全熔透焊缝，柱拼接接头上下各 100mm 范围内，工字形截面柱翼缘与腹板间及箱形截面柱角部壁板间的焊缝，应采用全熔透焊缝。

8.3.8　超过 12 层钢结构的刚接柱脚宜采用埋入式，6、7 度时也可采用外包式。

8.4　钢框架-中心支撑结构抗震构造措施

8.4.1　当中心支撑采用只能受拉的单斜杆体系时，应同时设置不同倾斜方向的两组斜杆，且每组中不同方向单斜杆的截面面积在水平方向的投影面积之差不得大于 10%。

8.4.2　中心支撑杆件的长细比和板件宽厚比应符合下列规定：

1　支撑杆件的长细比，不宜大于表 8.4.2-1 的限值。

表 8.4.2-1　钢结构中心支撑杆件长细比限值

类　　型		6、7 度	8 度	9 度
不超过 12 层	按压杆设计	150	120	120
	按拉杆设计	200	150	150
超过 12 层		120	90	60

注：表列数值适用于 Q235 钢，采用其他牌号钢材应乘以 $\sqrt{235/f_{ay}}$。

2　支撑杆件的板件宽厚比，不应大于表 8.4.2-2 规定的限值。采用节点板连接时，应注意节点板的强度和稳定。

表 8.4.2-2　钢结构中心支撑板件宽厚比限值

板件名称	不超过 12 层			超过 12 层			
	7 度	8 度	9 度	6 度	7 度	8 度	9 度
翼缘外伸部分	13	11	9	9	8	8	7
工字形截面腹板	33	30	27	25	23	23	21
箱形截面腹板	31	28	25	23	21	21	19
圆管外径与壁厚比				42	40	40	38

注：表列数值适用于 Q235 钢，采用其他牌号钢材应乘以 $\sqrt{235/f_{ay}}$，但径厚比应乘以 $235/f_y$。

8.4.3　中心支撑节点的构造应符合下列要求：

1　超过 12 层时，支撑宜采用轧制 H 型钢制作，两端与框架可采用刚接构造，梁柱与支撑连接处应设置加劲肋；8、9 度采用焊接工字形截面的支撑时，其翼缘与腹板的连接宜采用全熔透连续焊缝；

2　支撑与框架连接处，支撑杆端宜做成圆弧；

3　梁在其与 V 形支撑或人字支撑相交处，应设置侧向支承；该支承点与梁端支承点间的侧向长细比（λ_y）以及支承力，应符合国家标准《钢结构设计规范》GB 50017 关于塑性设计的规定。

4　不超过 12 层时，若支撑与框架采用节点板连接，应符合国家标准《钢结构设计规范》GB 50017 关于节点板在连接杆件每侧有不小于 30°夹角的规定；支撑端部至节点板嵌固点在沿支撑杆件方向的距离（由节点板与框架构件焊缝的起点垂直于支撑杆轴线的直线至支撑端部的距离），不应小于节点板厚度的 2 倍。

8.4.4　框架-中心支撑结构的框架部分，当房屋高度不高于 100m 且框架部分承担的地震作用不大于结构底部总地震剪力的 25%时，8、9 度的抗震构造措施可按框架结构降低一度的相应要求采用；其他抗震构造措施，应符合本章第 8.3 节对框架结构抗震构造措施的规定。

8.5　钢框架-偏心支撑结构抗震构造措施

8.5.1　偏心支撑框架消能梁段的钢材屈服强度不应大于 345MPa。消能梁段及与消能梁段同一跨内的非消能梁段，其板件的宽厚比不应大于表 8.5.1 规定的限值。

表 8.5.1 偏心支撑框架梁板件宽厚比限值

板件名称		宽厚比限值
翼缘外伸部分		8
腹板	当 $N/Af \leqslant 0.14$ 时	$90\ [1-1.65N/(Af)]$
	当 $N/Af > 0.14$ 时	$33\ [2.3-N/(Af)]$

注：表列数值适用于 Q235 钢，当材料为其他钢号时，应乘以 $\sqrt{235/f_{ay}}$。

8.5.2 偏心支撑框架的支撑杆件的长细比不应大于 $120\sqrt{235/f_{ay}}$，支撑杆件的板件宽厚比不应超过国家标准《钢结构设计规范》GB 50017 规定的轴心受压构件在弹性设计时的宽厚比限值。

8.5.3 消能梁段的构造应符合下列要求：

1 当 $N>0.16Af$ 时，消能梁段的长度应符合下列规定：

当 $\rho(A_w/A)<0.3$ 时，

$$a < 1.6M_{lp}/V_l \tag{8.5.3-1}$$

当 $\rho(A_w/A)\geqslant 0.3$ 时，

$$a \leqslant [1.15-0.5\rho(A_w/A)]1.6\ M_{lp}/V_l \tag{8.5.3-2}$$

$$\rho = N/V \tag{8.5.3-3}$$

式中 a——消能梁段的长度；

ρ——消能梁段轴向力设计值与剪力设计值之比。

2 消能梁段的腹板不得贴焊补强板，也不得开洞。

3 消能梁段与支撑连接处，应在其腹板两侧配置加劲肋，加劲肋的高度应为梁腹板高度，一侧的加劲肋宽度不应小于（$b_f/2-t_w$），厚度不应小于 $0.75t_w$ 和 10mm 的较大值。

4 消能梁段应按下列要求在其腹板上设置中间加劲肋：

1） 当 $a\leqslant 1.6\ M_{lp}/V_l$ 时，加劲肋间距不大于（$30t_w-h/5$）；

2） 当 $2.6\ M_{lp}/V_l<a\leqslant 5M_{lp}/V_l$ 时，应在距消能梁段端部 $1.5b_f$ 处配置中间加劲肋，且中间加劲肋间距不应大于（$52t_w-h/5$）；

3） 当 $1.6M_{lp}/V_l<a\leqslant 2.6\ M_{lp}/V_l$ 时，中间加劲肋的间距宜在上述二者间线性插入；

4） 当 $a>5M_{lp}/V_l$ 时，可不配置中间加劲肋；

5） 中间加劲肋应与消能梁段的腹板等高，当消能梁段截面高度不大于 640mm 时，可配置单侧加劲肋，消能梁段截面高度大于 640mm 时，应在两侧配置加劲肋，一侧加劲肋的宽度不应小于（$b_f/2-t_w$），厚度不应小于 t_w 和 10mm。

8.5.4 消能梁段与柱的连接应符合下列要求：

1 消能梁段与柱连接时，其长度不得大于 $1.6M_{lp}/V_l$，且应满足第 8.2.7 条的规定。

2 消能梁段翼缘与柱翼缘之间应采用坡口全熔透对接焊缝连接，消能梁段腹板与柱之间应采用角焊缝连接；角焊缝的承载力不得小于消能梁段腹板的轴向承载力、受剪承载力和受弯承载力。

3 消能梁段与柱腹板连接时，消能梁段翼缘与连接板间应采用坡口全熔透焊缝，消能梁段腹板与柱间应采用角焊缝；角焊缝的承载力不得小于消能梁段腹板的轴向承载力、受剪承载力和受弯承载力。

8.5.5 消能梁段两端上下翼缘应设置侧向支撑，支撑的轴力设计值不得小于消能梁段翼

缘轴向承载力设计值（翼缘宽度、厚度和钢材抗压强度设计值三者的乘积）的6%，即$0.06b_{\mathrm{f}}t_{\mathrm{f}}f$。

8.5.6 偏心支撑框架梁的非消能梁段上下翼缘，应设置侧向支撑，支撑的轴力设计值不得小于梁翼缘轴向承载力设计值的2%，即$0.02b_{\mathrm{f}}t_{\mathrm{f}}f$。

8.5.7 框架-偏心支撑结构的框架部分，当房屋高度不高于100m且框架部分承担的地震作用不大于结构底部总地震剪力的25%时，8、9度的抗震构造措施可按框架结构降低一度的相应要求采用；其他抗震构造措施，应符合本章第8.3节对框架结构抗震构造措施的规定。

9 单层工业厂房

9.1 单层钢筋混凝土柱厂房

（Ⅰ）一 般 规 定

9.1.1 厂房的结构布置，应符合下列要求：

1 多跨厂房宜等高和等长。

2 厂房的贴建房屋和构筑物，不宜布置在厂房角部和紧邻防震缝处。

3 厂房体型复杂或有贴建的房屋和构筑物时，宜设防震缝；在厂房纵横跨交接处、大柱网厂房或不设柱间支撑的厂房，防震缝宽度可采用100～150mm，其他情况可采用50～90mm。

4 两个主厂房之间的过渡跨至少应有一侧采用防震缝与主厂房脱开。

5 厂房内上吊车的铁梯不应靠近防震缝设置；多跨厂房各跨上吊车的铁梯不宜设置在同一横向轴线附近。

6 工作平台宜与厂房主体结构脱开。

7 厂房的同一结构单元内，不应采用不同的结构型式；厂房端部应设屋架，不应采用山墙承重；厂房单元内不应采用横墙和排架混合承重。

8 厂房各柱列的侧移刚度宜均匀。

9.1.2 厂房天窗架的设置，应符合下列要求：

1 天窗宜采用突出屋面较小的避风型天窗，有条件或9度时宜采用下沉式天窗。

2 突出屋面的天窗宜采用钢天窗架；6～8度时，可采用矩形截面杆件的钢筋混凝土天窗架。

3 8度和9度时，天窗架宜从厂房单元端部第三柱间开始设置。

4 天窗屋盖、端壁板和侧板，宜采用轻型板材。

9.1.3 厂房屋架的设置，应符合下列要求：

1 厂房宜采用钢屋架或重心较低的预应力混凝土、钢筋混凝土屋架。

2 跨度不大于15m时，可采用钢筋混凝土屋面梁。

3 跨度大于24m，或8度Ⅲ、Ⅳ类场地和9度时，应优先采用钢屋架。

4 柱距为12m时，可采用预应力混凝土托架（梁）；当采用钢屋架时，亦可采用钢托架（梁）。

5 有突出屋面天窗架的屋盖不宜采用预应力混凝土或钢筋混凝土空腹屋架。

9.1.4 厂房柱的设置，应符合下列要求：

1 8度和9度时，宜采用矩形、工字形截面柱或斜腹杆双肢柱，不宜采用薄壁工字形柱、腹板开孔工字形柱、预制腹板的工字形柱和管柱。

2 柱底至室内地坪以上500mm范围内和阶形柱的上柱宜采用矩形截面。

9.1.5 厂房围护墙、女儿墙的布置和抗震构造措施，应符合本规范第13.3节对非结构构件的有关规定。

（Ⅱ）计　算　要　点

9.1.6　7度Ⅰ、Ⅱ类场地，柱高不超过10m且结构单元两端均有山墙的单跨及等高多跨厂房（锯齿形厂房除外），当按本规范的规定采取抗震构造措施时，可不进行横向及纵向的截面抗震验算。

9.1.7　厂房的横向抗震计算，应采用下列方法：

1　混凝土无檩和有檩屋盖厂房，一般情况下，宜计及屋盖的横向弹性变形，按多质点空间结构分析；当符合本规范附录H的条件时，可按平面排架计算，并按附录H的规定对排架柱的地震剪力和弯矩进行调整。

2　轻型屋盖厂房，柱距相等时，可按平面排架计算。

注：本节轻型屋盖指屋面为压型钢板、瓦楞铁、石棉瓦等有檩屋盖。

9.1.8　厂房的纵向抗震计算，应采用下列方法：

1　混凝土无檩和有檩屋盖及有较完整支撑系统的轻型屋盖厂房，可采用下列方法：

1）一般情况下，宜计及屋盖的纵向弹性变形，围护墙与隔墙的有效刚度，不对称时尚宜计及扭转的影响，按多质点进行空间结构分析；

2）柱顶标高不大于15m且平均跨度不大于30m的单跨或等高多跨的钢筋混凝土柱厂房，宜采用本规范附录J规定的修正刚度法计算。

2　纵墙对称布置的单跨厂房和轻型屋盖的多跨厂房，可按柱列分片独立计算。

9.1.9　突出屋面天窗架的横向抗震计算，可采用下列方法：

1　有斜撑杆的三铰拱式钢筋混凝土和钢天窗架的横向抗震计算可采用底部剪力法；跨度大于9m或9度时，混凝土天窗架的地震作用效应应乘以增大系数，增大系数可采用1.5。

2　其他情况下天窗架的横向水平地震作用可采用振型分解反应谱法。

9.1.10　突出屋面天窗架的纵向抗震计算，可采用下列方法：

1　天窗架的纵向抗震计算，可采用空间结构分析法，并计及屋盖平面弹性变形和纵墙的有效刚度。

2　柱高不超过15m的单跨和等高多跨混凝土无檩屋盖厂房的天窗架纵向地震作用计算，可采用底部剪力法，但天窗架的地震作用效应应乘以效应增大系数，其值可按下列规定采用：

1）单跨、边跨屋盖或有纵向内隔墙的中跨屋盖：

$$\eta = 1 + 0.5n \tag{9.1.10-1}$$

2）其他中跨屋盖：

$$\eta = 0.5n \tag{9.1.10-2}$$

式中　η——效应增大系数；

n——厂房跨数，超过四跨时取四跨。

9.1.11　两个主轴方向柱距均不小于12m、无桥式吊车且无柱间支撑的大柱网厂房，柱截面抗震验算应同时计算两个主轴方向的水平地震作用，并应计入位移引起的附加弯矩。

9.1.12　不等高厂房中，支承低跨屋盖的柱牛腿（柱肩）的纵向受拉钢筋截面面积，应按下式确定：

$$A_s \geqslant \left(\frac{N_G a}{0.85 h_0 f_y} + 1.2\frac{N_E}{f_y}\right)\gamma_{RE} \tag{9.1.12}$$

式中 A_s——纵向水平受拉钢筋的截面面积；

N_G——柱牛腿面上重力荷载代表值产生的压力设计值；

a——重力作用点至下柱近侧边缘的距离，当小于 $0.3h_0$ 时采用 $0.3h_0$；

h_0——牛腿最大竖向截面的有效高度；

N_E—— 柱牛腿面上地震组合的水平拉力设计值；

γ_{RE}——承载力抗震调整系数，可采用 1.0。

9.1.13 柱间交叉支撑斜杆的地震作用效应及其与柱连接节点的抗震验算，可按本规范附录 J 的规定进行。

9.1.14 8 度和 9 度时，高大山墙的抗风柱应进行平面外的截面抗震验算。

9.1.15 当抗风柱与屋架下弦相连接时，连接点应设在下弦横向支撑节点处，下弦横向支撑杆件的截面和连接节点应进行抗震承载力验算。

9.1.16 当工作平台和刚性内隔墙与厂房主体结构连接时，应采用与厂房实际受力相适应的计算简图，计入工作平台和刚性内隔墙对厂房的附加地震作用影响，变位受约束且剪跨比不大于 2 的排架柱，其斜截面受剪承载力应按国家标准《混凝土结构设计规范》GB 50010 的规定计算，并采取相应的抗震措施。

9.1.17 8 度Ⅲ、Ⅳ类场地和 9 度时，带有小立柱的拱形和折线型屋架或上弦节间较长且矢高较大的屋架，屋架上弦宜进行抗扭验算。

（Ⅲ） 抗震构造措施

9.1.18 有檩屋盖构件的连接及支撑布置，应符合下列要求：

1 檩条应与混凝土屋架（屋面梁）焊牢，并应有足够的支承长度。

2 双脊檩应在跨度 1/3 处相互拉结。

3 压型钢板应与檩条可靠连接，瓦楞铁、石棉瓦等应与檩条拉结。

4 支撑布置宜符合表 9.1.18 的要求。

表 9.1.18 有檩屋盖的支撑布置

<table>
<tr><th colspan="2" rowspan="2">支撑名称</th><th colspan="3">烈　　度</th></tr>
<tr><th>6、7</th><th>8</th><th>9</th></tr>
<tr><td rowspan="4">屋架支撑</td><td>上弦横向支撑</td><td>厂房单元端开间各设一道</td><td>厂房单元端开间及厂房单元长度大于 66m 的柱间支撑开间各设一道；
天窗开洞范围的两端各增设局部的支撑一道</td><td rowspan="3">厂房单元端开间及厂房单元长度大于 42m 的柱间支撑开间各设一道；
天窗开洞范围的两端各增设局部的上弦横向支撑一道</td></tr>
<tr><td>下弦横向支撑</td><td colspan="2" rowspan="2">同非抗震设计</td></tr>
<tr><td>跨中竖向支撑</td></tr>
<tr><td>端部竖向支撑</td><td colspan="3">屋架端部高度大于 900mm 时，厂房单元端开间及柱间支撑开间各设一道</td></tr>
<tr><td rowspan="2">天窗架支撑</td><td>上弦横向支撑</td><td>厂房单元天窗端开间各设一道</td><td rowspan="2">厂房单元天窗端开间及每隔 30m 各设一道</td><td rowspan="2">厂房单元天窗端开间及每隔 18m 各设一道</td></tr>
<tr><td>两侧竖向支撑</td><td>厂房单元天窗端开间及每隔 36m 各设一道</td></tr>
</table>

9.1.19 无檩屋盖构件的连接及支撑布置，应符合下列要求：

1 大型屋面板应与屋架（屋面梁）焊牢，靠柱列的屋面板与屋架（屋面梁）的连接焊缝长度不宜小于80mm。

2 6度和7度时，有天窗厂房单元的端开间，或8度和9度时各开间，宜将垂直屋架方向两侧相邻的大型屋面板的顶面彼此焊牢。

3 8度和9度时，大型屋面板端头底面的预埋件宜采用角钢并与主筋焊牢。

4 非标准屋面板宜采用装配整体式接头，或将板四角切掉后与屋架（屋面梁）焊牢。

5 屋架（屋面梁）端部顶面预埋件的锚筋，8度时不宜少于4ϕ10，9度时不宜少于4ϕ12。

6 支撑的布置宜符合表9.1.19-1的要求，有中间井式天窗时宜符合表9.1.19-2的要求；8度和9度跨度不大于15m的屋面梁屋盖，可仅在厂房单元两端各设竖向支撑一道。

表9.1.19-1 无檩屋盖的支撑布置

支撑名称			烈度		
			6、7	8	9
屋架支撑	上弦横向支撑		屋架跨度小于18m时同非抗震设计，跨度不小于18m时在厂房单元端开间各设一道	厂房单元端开间及柱间支撑开间各设一道，天窗开洞范围的两端各增设局部的支撑一道	
	上弦通长水平系杆		同非抗震设计	沿屋架跨度不大于15m设一道，但装配整体式屋面可不设； 围护墙在屋架上弦高度有现浇圈梁时，其端部处可不另设	沿屋架跨度不大于12m设一道，但装配整体式屋面可不设； 围护墙在屋架上弦高度有现浇圈梁时，其端部处可不另设
	下弦横向支撑			同非抗震设计	同上弦横向支撑
	跨中竖向支撑				
	两端竖向支撑	屋架端部高度≤900mm		厂房单元端开间各设一道	厂房单元端开间及每隔48m各设一道
		屋架端部高度>900mm	厂房单元端开间各设一道	厂房单元端开间及柱间支撑开间各设一道	厂房单元端开间、柱间支撑开间及每隔30m各设一道
天窗架支撑	两侧竖向支撑		厂房单元天窗端开间及每隔30m各设一道	厂房单元天窗端开间及每隔24m各设一道	厂房单元天窗端开间及每隔18m各设一道
	上弦横向支撑		同非抗震设计	天窗跨度≥9m时，厂房单元天窗端开间及柱间支撑开间各设一道	厂房单元端开间及柱间支撑开间各设一道

表9.1.19-2 中间井式天窗无檩屋盖支撑布置

支撑名称	6、7度	8度	9度
上弦横向支撑 下弦横向支撑	厂房单元端开间各设一道	厂房单元端开间及柱间支撑开间各设一道	
上弦通长水平系杆	天窗范围内屋架跨中上弦节点处设置		

续表 9.1.19-2

支撑名称		6、7度	8度	9度
下弦通长水平系杆		天窗两侧及天窗范围内屋架下弦节点处设置		
跨中竖向支撑		有上弦横向支撑开间设置，位置与下弦通长系杆相对应		
两端竖向支撑	屋架端部高度≤900mm	同非抗震设计		有上弦横向支撑开间，且间距不大于48m
	屋架端部高度>900mm	厂房单元端开间各设一道	有上弦横向支撑开间，且间距不大于48m	有上弦横向支撑开间，且间距不大于30m

9.1.20 屋盖支撑尚应符合下列要求：

1 天窗开洞范围内，在屋架脊点处应设上弦通长水平压杆。

2 屋架跨中竖向支撑在跨度方向的间距，6~8度时不大于15m，9度时不大于12m；当仅在跨中设一道时，应设在跨中屋架屋脊处；当设二道时，应在跨度方向均匀布置。

3 屋架上、下弦通长水平系杆与竖向支撑宜配合设置。

4 柱距不小于12m且屋架间距6m的厂房，托架（梁）区段及其相邻开间应设下弦纵向水平支撑。

5 屋盖支撑杆件宜用型钢。

9.1.21 突出屋面的混凝土天窗架，其两侧墙板与天窗立柱宜采用螺栓连接。

9.1.22 混凝土屋架的截面和配筋，应符合下列要求：

1 屋架上弦第一节间和梯形屋架端竖杆的配筋，6度和7度时不宜少于4ϕ12，8度和9度时不宜少于4ϕ14。

2 梯形屋架的端竖杆截面宽度宜与上弦宽度相同。

3 拱形和折线形屋架上弦端部支撑屋面板的小立柱，截面不宜小于200mm×200mm，高度不宜大于500mm，主筋宜采用Π形，6度和7度时不宜少于4ϕ12，8度和9度时不宜少于4ϕ14，箍筋可采用ϕ6，间距宜为100mm。

9.1.23 厂房柱子的箍筋，应符合下列要求：

1 下列范围内柱的箍筋应加密：

1） 柱头，取柱顶以下500mm并不小于柱截面长边尺寸；

2） 上柱，取阶形柱自牛腿面至吊车梁顶面以上300mm高度范围内；

3） 牛腿（柱肩），取全高；

4） 柱根，取下柱柱底至室内地坪以上500mm；

5） 柱间支撑与柱连接节点和柱变位受平台等约束的部位，取节点上、下各300mm。

2 加密区箍筋间距不应大于100mm，箍筋肢距和最小直径应符合表9.1.23的规定：

表9.1.23 柱加密区箍筋最大肢距和最小箍筋直径

烈度和场地类别		6度和7度Ⅰ、Ⅱ类场地	7度Ⅲ、Ⅳ类场地和8度Ⅰ、Ⅱ类场地	8度Ⅲ、Ⅳ类场地和9度
箍筋最大肢距（mm）		300	250	200
箍筋最小直径	一般柱头和柱根	ϕ6	ϕ8	ϕ8（ϕ10）
	角柱柱头	ϕ8	ϕ10	ϕ10
	上柱牛腿和有支撑的柱根	ϕ8	ϕ8	ϕ10
	有支撑的柱头和柱变位受约束部位	ϕ8	ϕ10	ϕ10

注：括号内数值用于柱根。

9.1.24 山墙抗风柱的配筋，应符合下列要求：

1 抗风柱柱顶以下300mm和牛腿（柱肩）面以上300mm范围内的箍筋，直径不宜小于6mm，间距不应大于100mm，肢距不宜大于250mm。

2 抗风柱的变截面牛腿（柱肩）处，宜设置纵向受拉钢筋。

9.1.25 大柱网厂房柱的截面和配筋构造，应符合下列要求：

1 柱截面宜采用正方形或接近正方形的矩形，边长不宜小于柱全高的1/18～1/16。

2 重屋盖厂房地震组合的柱轴压比，6、7度时不宜大于0.8，8度时不宜大于0.7，9度时不应大于0.6。

3 纵向钢筋宜沿柱截面周边对称配置，间距不宜大于200mm，角部宜配置直径较大的钢筋。

4 柱头和柱根的箍筋应加密，并应符合下列要求：

1）加密范围，柱根取基础顶面至室内地坪以上1m，且不小于柱全高的1/6；柱头取柱顶以下500mm，且不小于柱截面长边尺寸；

2）箍筋直径、间距和肢距，应符合本章第9.1.23条的规定。

9.1.26 厂房柱间支撑的设置和构造，应符合下列要求：

1 厂房柱间支撑的布置，应符合下列规定：

1）一般情况下，应在厂房单元中部设置上、下柱间支撑，且下柱支撑应与上柱支撑配套设置；

2）有吊车或8度和9度时，宜在厂房单元两端增设上柱支撑；

3）厂房单元较长或8度Ⅲ、Ⅳ类场地和9度时，可在厂房单元中部1/3区段内设置两道柱间支撑。

2 柱间支撑应采用型钢，支撑形式宜采用交叉式，其斜杆与水平面的交角不宜大于55°。

3 支撑杆件的长细比，不宜超过表9.1.26的规定。

4 下柱支撑的下节点位置和构造措施，应保证将地震作用直接传给基础；当6度和7度不能直接传给基础时，应计及支撑对柱和基础的不利影响。

5 交叉支撑在交叉点应设置节点板，其厚度不应小于10mm，斜杆与交叉节点板应焊接，与端节点板宜焊接。

表9.1.26 交叉支撑斜杆的最大长细比

位置	烈度			
	6度和7度Ⅰ、Ⅱ类场地	7度Ⅲ、Ⅳ类场地和8度Ⅰ、Ⅱ类场地	8度Ⅲ、Ⅳ类场地和9度Ⅰ、Ⅱ类场地	9度Ⅲ、Ⅳ类场地
上柱支撑	250	250	200	150
下柱支撑	200	200	150	150

9.1.27 8度时跨度不小于18m的多跨厂房中柱和9度时多跨厂房各柱，柱顶宜设置通长水平压杆，此压杆可与梯形屋架支座处通长水平系杆合并设置，钢筋混凝土系杆端头与屋架间的空隙应采用混凝土填实。

9.1.28 厂房结构构件的连接节点，应符合下列要求：

1 屋架（屋面梁）与柱顶的连接，8度时宜采用螺栓，9度时宜采用钢板铰，亦可

采用螺栓；屋架（屋面梁）端部支承垫板的厚度不宜小于16mm。

2 柱顶预埋件的锚筋，8度时不宜少于4ϕ14，9度时不宜少于4ϕ16；有柱间支撑的柱子，柱顶预埋件尚应增设抗剪钢板。

3 山墙抗风柱的柱顶，应设置预埋板，使柱顶与端屋架的上弦（屋面梁上翼缘）可靠连接。连接部位应位于上弦横向支撑与屋架的连接点处，不符合时可在支撑中增设次腹杆或设置型钢横梁，将水平地震作用传至节点部位。

4 支承低跨屋盖的中柱牛腿（柱肩）的预埋件，应与牛腿（柱肩）中按计算承受水平拉力部分的纵向钢筋焊接，且焊接的钢筋，6度和7度时不应少于2ϕ12，8度时不应少于2ϕ14，9度时不应少于2ϕ16。

5 柱间支撑与柱连接节点预埋件的锚件，8度Ⅲ、Ⅳ类场地和9度时，宜采用角钢加端板，其他情况可采用HRB335级或HRB400级热轧钢筋，但锚固长度不应小于30倍锚筋直径或增设端板。

6 厂房中的吊车走道板、端屋架与山墙间的填充小屋面板、天沟板、天窗端壁板和天窗侧板下的填充砌体等构件应与支承结构有可靠的连接。

9.2 单层钢结构厂房

（Ⅰ）一 般 规 定

9.2.1 本节主要适用于钢柱、钢屋架或实腹梁承重的单跨和多跨的单层厂房。不适用于单层轻型钢结构厂房。

9.2.2 厂房平面布置和钢筋混凝土屋面板的设置构造要求等，可参照本规范第9.1节单层钢筋混凝土柱厂房的有关规定。

9.2.3 厂房的结构体系应符合下列要求：

1 厂房的横向抗侧力体系，可采用屋盖横梁与柱顶刚接或铰接的框架、门式刚架、悬臂柱或其他结构体系。厂房纵向抗侧力体系宜采用柱间支撑，条件限制时也可采用刚架结构。

2 构件在可能产生塑性铰的最大应力区内，应避免焊接接头；对于厚度较大无法采用螺栓连接的构件，可采用对接焊缝等强度连接。

3 屋盖横梁与柱顶铰接时，宜采用螺栓连接。刚接框架的屋架上弦与柱相连的连接板，不应出现塑性变形。当横梁为实腹梁时，梁与柱的连接以及梁与梁拼接的受弯、受剪极限承载力，应能分别承受梁全截面屈服时受弯、受剪承载力的1.2倍。

4 柱间支撑杆件应采用整根材料，超过材料最大长度规格时可采用对接焊缝等强拼接；柱间支撑与构件的连接，不应小于支撑杆件塑性承载力的1.2倍。

（Ⅱ）计 算 要 点

9.2.4 厂房抗震计算时，应根据屋盖高差和吊车设置情况，分别采用单质点、双质点或多质点模型计算地震作用。

9.2.5 厂房地震作用计算时，围护墙的自重与刚度应符合下列规定：

1 轻质墙板或与柱柔性连接的预制钢筋混凝土墙板，应计入墙体的全部自重，但不应计入刚度。

2 与柱贴砌且与柱拉结的砌体围护墙，应计入全部自重，在平行于墙体方向计算时可计

入等效刚度，其等效系数可采用0.4。

9.2.6 厂房横向抗震计算可采用下列方法：

1 一般情况下，宜计入屋盖变形进行空间分析。

2 采用轻型屋盖时，可按平面排架或框架计算。

9.2.7 厂房纵向抗震计算，可采用下列方法：

1 采用轻质墙板或与柱柔性连接的大型墙板的厂房，可按单质点计算，各柱列的地震作用应按以下原则分配：

1）钢筋混凝土无檩屋盖可按柱列刚度比例分配；

2）轻型屋盖可按柱列承受的重力荷载代表值的比例分配；

3）钢筋混凝土有檩屋盖可取上述两种分配结果的平均值。

2 采用与柱贴砌的烧结普通粘土砖围护墙厂房，可参照本规范第9.1.8条的规定。

9.2.8 屋盖竖向支撑桁架的腹杆应能承受和传递屋盖的水平地震作用，其连接的承载力应大于腹杆的内力，并满足构造要求。

9.2.9 柱间交叉支撑的地震作用及验算可按本规范附录J.2的规定按拉杆计算，并计及相交受压杆的影响。交叉支撑端部的连接，对单角钢支撑应计入强度折减，8、9度时不得采用单面偏心连接；交叉支撑有一杆中断时，交叉节点板应予以加强，其承载力不小于1.1倍杆件承载力。

（Ⅲ） 抗震构造措施

9.2.10 屋盖的支撑布置，宜符合本规范第9.1节的有关要求。

9.2.11 柱的长细比不应大于$120\sqrt{235/f_{ay}}$。

9.2.12 单层框架柱、梁截面板件的宽厚比限值，除应符合现行《钢结构设计规范》GB50017对钢结构弹性阶段设计的有关规定外，尚应符合表9.2.12的规定：

表9.2.12 单层钢结构厂房板件宽厚比限值

构件	板件名称	7度	8度	9度
柱	工形截面翼缘外伸部分	13	11	10
	箱形截面两腹板间翼缘	38	36	36
	工形、箱形截面腹板（$N_c/Af<0.25$）	70	65	60
	（$N_c/Af\geqslant0.25$）	58	52	48
	圆管外径与壁厚比	60	55	50
梁	工形截面翼缘外伸部分	11	10	9
	箱形截面两腹板间翼缘	36	32	30
	工形、箱形截面腹板（$N_b/Af<0.37$）	$85-120\rho$	$80-110\rho$	$72-100\rho$
	（$N_b/Af\geqslant0.37$）	40	39	35

注：1 表列数值适用于Q235钢，当材料为其他钢号时，应乘以$\sqrt{235/f_{ay}}$；

2 N_c、N_b分别为柱、梁轴向力；A为相应构件截面面积；f为钢材抗拉强度设计值；

3 ρ指N_b/Af；

4 构件腹板宽厚比，可通过设置纵向加劲肋减小。

9.2.13 柱脚应采取保证能传递柱身承载力的插入式或埋入式柱脚。6、7度时亦可采用外露式刚性柱脚，但柱脚螺栓的组合弯矩设计值应乘以增大系数1.2。

实腹式钢柱采用插入式柱脚的埋入深度，不得小于钢柱截面高度的2倍；同时应满足下式要求：

$$d \geqslant \sqrt{6M/b_f f_c} \tag{9.2.13}$$

式中 d——柱脚埋深；

M——柱脚全截面屈服时的极限弯矩；

b_f——柱在受弯方向截面的翼缘宽度；

f_c——基础混凝土轴心抗压强度设计值。

9.2.14 柱间交叉支撑应符合下列要求：

1 有吊车时，应在厂房单元中部设置上下柱间支撑，并应在厂房单元两端增设上柱支撑；7度时结构单元长度大于120m，8、9度时结构单元长度大于90m，宜在单元中部1/3区段内设置两道上下柱间支撑。

2 柱间交叉支撑的长细比、支撑斜杆与水平面的夹角、支撑斜杆交叉点的节点板厚度，应符合本规范第9.1.26条的有关规定。

3 有条件时，可采用消能支撑。

9.3 单层砖柱厂房

（Ⅰ）一 般 规 定

9.3.1 本节适用于下列范围内的烧结普通粘土砖柱（墙垛）承重的中小型厂房：

1 单跨和等高多跨且无桥式吊车的车间、仓库等。

2 6～8度，跨度不大于15m且柱顶标高不大于6.6m。

3 9度，跨度不大于12m且柱顶标高不大于4.5m。

9.3.2 厂房的平立面布置，宜符合本章第9.1节的有关规定，但防震缝的设置，应符合下列要求：

1 轻型屋盖厂房，可不设防震缝。

2 钢筋混凝土屋盖厂房与贴建的建（构）筑物间宜设防震缝，其宽度可采用50～70mm。

3 防震缝处应设置双柱或双墙。

注：本节轻型屋盖指木屋盖和轻钢屋架、压型钢板、瓦楞铁、石棉瓦屋面的屋盖。

9.3.3 厂房两端均应设置承重山墙；天窗不应通至厂房单元的端开间，天窗不应采用端砖壁承重。

9.3.4 厂房的结构体系，尚应符合下列要求：

1 6～8度时，宜采用轻型屋盖，9度时，应采用轻型屋盖。

2 6度和7度时，可采用十字形截面的无筋砖柱；8度和9度时应采用组合砖柱，且中柱在8度Ⅲ、Ⅳ类场地和9度时宜采用钢筋混凝土柱。

3 厂房纵向的独立砖柱柱列，可在柱间设置与柱等高的抗震墙承受纵向地震作用，砖抗震墙应与柱同时咬槎砌筑，并应设置基础；无砖抗震墙的柱顶，应设通长水平压杆。

4 纵、横向内隔墙宜做成抗震墙，非承重横隔墙和非整体砌筑且不到顶的纵向隔墙宜采用轻质墙，当采用非轻质墙时，应计及隔墙对柱及其与屋架（梁）连接节点的附加地震剪力。独立的纵、横内隔墙应采取措施保证其平面外的稳定性，且顶部应设置现浇钢筋混凝土压顶梁。

（Ⅱ）计 算 要 点

9.3.5 按本节规定采取抗震构造措施的单层砖柱厂房，当符合下列条件时，可不进行横向或纵向截面抗震验算：

1 7度Ⅰ、Ⅱ类场地，柱顶标高不超过4.5m，且结构单元两端均有山墙的单跨及等高多跨砖柱厂房，可不进行横向和纵向抗震验算。

2 7度Ⅰ、Ⅱ类场地，柱顶标高不超过6.6m，两侧设有厚度不小于240mm且开洞截面面积不超过50%的外纵墙，结构单元两端均有山墙的单跨厂房，可不进行纵向抗震验算。

9.3.6 厂房的横向抗震计算，可采用下列方法：

1 轻型屋盖厂房可按平面排架进行计算。

2 钢筋混凝土屋盖厂房和密铺望板的瓦木屋盖厂房可按平面排架进行计算并计及空间作用，按本规范附录H调整地震作用效应。

9.3.7 厂房的纵向抗震计算，可采用下列方法：

1 钢筋混凝土屋盖厂房宜采用振型分解反应谱法进行计算。

2 钢筋混凝土屋盖的等高多跨砖柱厂房可按本规范附录K规定的修正刚度法进行计算。

3 纵墙对称布置的单跨厂房和轻型屋盖的多跨厂房，可采用柱列分片独立进行计算。

9.3.8 突出屋面天窗架的横向和纵向抗震计算应符合本章第9.1.9条和第9.1.10条的规定。

9.3.9 偏心受压砖柱的抗震验算，应符合下列要求：

1 无筋砖柱地震组合轴向力设计值的偏心距，不宜超过0.9倍截面形心到轴向力所在方向截面边缘的距离；承载力抗震调整系数可采用0.9。

2 组合砖柱的配筋应按计算确定，承载力抗震调整系数可采用0.85。

（Ⅲ）抗震构造措施

9.3.10 木屋盖的支撑布置，宜符合表9.3.10的要求，钢屋架、瓦楞铁、石棉瓦等屋面的支撑，可按表中无望板屋盖的规定设置，不应在端开间设置下弦水平系杆与山墙连接；支撑与屋架或天窗架应采用螺栓连接；木天窗架的边柱，宜采用通长木夹板或铁板并通过螺栓加强边柱与屋架上弦的连接。

表9.3.10 木屋盖的支撑布置

支撑名称		烈度					
		6、7	8			9	
		各类屋盖	满铺望板		稀铺望板或无望板	满铺望板	稀铺望板或无望板
			无天窗	有天窗			
屋架支撑	上弦横向支撑	同非抗震设计		房屋单元两端天窗开洞范围内各设一道	屋架跨度大于6m时，房屋单元两端第二开间及每隔20m设一道	屋架跨度大于6m时，房屋单元两端第二开间各设一道	屋架跨度大于6m时，房屋单元两端第二开间及每隔20m设一道
	下弦横向支撑	同非抗震设计					屋架跨度大于6m时，房屋单元两端第二开间及每隔20m设一道
	跨中竖向支撑	同非抗震设计					隔间设置并加下弦通长水平系杆

续表 9.3.10

支撑名称		烈度					
		6、7	8			9	
		各类屋盖	满铺望板		稀铺望板或无望板	满铺望板	稀铺望板或无望板
			无天窗	有天窗			
天窗架支撑	天窗两侧竖向支撑	天窗两端第一开间各设一道			天窗两端第一开间及每隔 20m 左右设一道		
	上弦横向支撑	跨度较大的天窗，参照无天窗屋架的支撑布置					

9.3.11 檩条与山墙卧梁应可靠连接，有条件时可采用檩条伸出山墙的屋面结构。

9.3.12 钢筋混凝土屋盖的构造措施，应符合本章第 9.1 节的有关规定。

9.3.13 厂房柱顶标高处应沿房屋外墙及承重内墙设置现浇闭合圈梁，8 度和 9 度时还应沿墙高每隔 3～4m 增设一道圈梁，圈梁的截面高度不应小于 180mm，配筋不应少于 4ϕ12；当地基为软弱粘性土、液化土、新近填土或严重不均匀土层时，尚应设置基础圈梁。当圈梁兼作门窗过梁或抵抗不均匀沉降影响时，其截面和配筋除满足抗震要求外，尚应根据实际受力计算确定。

9.3.14 山墙应沿屋面设置现浇钢筋混凝土卧梁，并应与屋盖构件锚拉；山墙壁柱的截面与配筋，不宜小于排架柱，壁柱应通到墙顶并与卧梁或屋盖构件连接。

9.3.15 屋架（屋面梁）与墙顶圈梁或柱顶垫块，应采用螺栓或焊接连接；柱顶垫块应现浇，其厚度不应小于 240mm，并应配置两层直径不小于 8mm 间距不大于 100mm 的钢筋网；墙顶圈梁应与柱顶垫块整浇，9 度时，在垫块两侧各 500mm 范围内，圈梁的箍筋间距不应大于 100mm。

9.3.16 砖柱的构造应符合下列要求：

1 砖的强度等级不应低于 MU10，砂浆的强度等级不应低于 M5；组合砖柱中的混凝土强度等级应采用 C20。

2 砖柱的防潮层应采用防水砂浆。

9.3.17 钢筋混凝土屋盖的砖柱厂房，山墙开洞的水平截面面积不宜超过总截面面积的 50%；8 度时，应在山、横墙两端设置钢筋混凝土构造柱；9 度时，应在山、横墙两端及高大的门洞两侧设置钢筋混凝土构造柱。

钢筋混凝土构造柱的截面尺寸，可采用 240mm×240mm；当为 9 度且山、横墙的厚度为 370mm 时，其截面宽度宜取 370mm；构造柱的竖向钢筋，8 度时不应少于 4ϕ12，9 度时不应少于 4ϕ14；箍筋可采用 ϕ6，间距宜为 250～300mm。

9.3.18 砖砌体墙的构造应符合下列要求：

1 8 度和 9 度时，钢筋混凝土无檩屋盖砖柱厂房，砖围护墙顶部宜沿墙长每隔 1m 埋入 1ϕ8 竖向钢筋，并插入顶部圈梁内。

2 7 度且墙顶高度大于 4.8m 或 8 度和 9 度时，外墙转角及承重内横墙与外纵墙交接处，当不设置构造柱时，应沿墙高每 500mm 配置 2ϕ6 钢筋，每边伸入墙内不小于 1m。

3 出屋面女儿墙的抗震构造措施，应符合本规范第 13.3 节的有关规定。

10 单层空旷房屋

10.1 一 般 规 定

10.1.1 本章适用于较空旷的单层大厅和附属房屋组成的公共建筑。

10.1.2 大厅、前厅、舞台之间，不宜设防震缝分开；大厅与两侧附属房屋之间可不设防震缝。但不设缝时应加强连接。

10.1.3 单层空旷房屋大厅，支承屋盖的承重结构，在下列情况下不应采用砖柱：

1 9度时与8度Ⅲ、Ⅳ类场地的建筑。

2 大厅内设有挑台。

3 8度Ⅰ、Ⅱ类场地和7度Ⅲ、Ⅳ类场地，大厅跨度大于15m或柱顶高度大于6m。

4 7度Ⅰ、Ⅱ类场地和6度Ⅲ、Ⅳ类场地，大厅跨度大于18m或柱顶高度大于8m。

10.1.4 单层空旷房屋大厅，支承屋盖的承重结构除第10.1.3条规定者外，可在大厅纵墙屋架支点下，增设钢筋混凝土-砖组合壁柱，不得采用无筋砖壁柱。

10.1.5 前厅结构布置应加强横向的侧向刚度，大门处壁柱，及前厅内独立柱应设计成钢筋混凝土柱。

10.1.6 前厅与大厅、大厅与舞台连接处的横墙，应加强侧向刚度，设置一定数量的钢筋混凝土抗震墙。

10.1.7 大厅部分其他要求可参照本规范第9章，附属房屋应符合本规范的有关规定。

10.2 计 算 要 点

10.2.1 单层空旷房屋的抗震计算，可将房屋划分为前厅、舞台、大厅和附属房屋等若干独立结构，按本规范有关规定执行，但应计及相互影响。

10.2.2 单层空旷房屋的抗震计算，可采用底部剪力法，地震影响系数可取最大值。

10.2.3 大厅的纵向水平地震作用标准值，可按下式计算：

$$F_{Ek} = \alpha_{max} G_{eq} \tag{10.2.3}$$

式中 F_{Ek}——大厅一侧纵墙或柱列的纵向水平地震作用标准值；

G_{eq}——等效重力荷载代表值。包括大厅屋盖和毗连附属房屋屋盖各一半的自重和50%雪荷载标准值，及一侧纵墙或柱列的折算自重。

10.2.4 大厅的横向抗震计算，宜符合下列原则：

1 两侧无附属房屋的大厅，有挑台部分和无挑台部分可各取一个典型开间计算；符合本规范第9章规定时，尚可计及空间工作。

2 两侧有附属房屋时，应根据附属房屋的结构类型，选择适当的计算方法。

10.2.5 8度和9度时，高大山墙的壁柱应进行平面外的截面抗震验算。

10.3 抗震构造措施

10.3.1 大厅的屋盖构造，应符合本规范第9章的规定。

10.3.2 大厅的钢筋混凝土柱和组合砖柱应符合下列要求：

1 组合砖柱纵向钢筋的上端应锚入屋架底部的钢筋混凝土圈梁内。组合柱的纵向钢筋，除按计算确定外，且6度Ⅲ、Ⅳ类场地和7度Ⅰ、Ⅱ类场地每侧不应少于4ϕ14；7度Ⅲ、Ⅳ类场地和8度Ⅰ、Ⅱ类场地每侧不应少于4ϕ16。

2 钢筋混凝土柱应按抗震等级为二级框架柱设计，其配筋量应按计算确定。

10.3.3 **前厅与大厅，大厅与舞台间轴线上横墙，应符合下列要求：**

1 应在横墙两端，纵向梁支点及大洞口两侧设置钢筋混凝土框架柱或构造柱。

2 嵌砌在框架柱间的横墙应有部分设计成抗震等级为二级的钢筋混凝土抗震墙。

3 舞台口的柱和梁应采用钢筋混凝土结构，舞台口大梁上承重砌体墙应设置间距不大于4m的立柱和间距不大于3m的圈梁，立柱、圈梁的截面尺寸、配筋及与周围砌体的拉结应符合多层砌体房屋要求。

4 9度时，舞台口大梁上的砖墙不应承重。

10.3.4 大厅柱（墙）顶标高处应设置现浇圈梁，并宜沿墙高每隔3m左右增设一道圈梁。梯形屋架端部高度大于900mm时还应在上弦标高处增设一道圈梁。圈梁的截面高度不宜小于180mm，宽度宜与墙厚相同，纵筋不应少于4ϕ12，箍筋间距不宜大于200mm。

10.3.5 大厅与两侧附属房屋间不设防震缝时，应在同一标高处设置封闭圈梁并在交接处拉通，墙体交接处应沿墙高每隔500mm设置2ϕ6拉结钢筋，且每边伸入墙内不宜小于1m。

10.3.6 悬挑式挑台应有可靠的锚固和防止倾覆的措施。

10.3.7 山墙应沿屋面设置钢筋混凝土卧梁，并应与屋盖构件锚拉；山墙应设置钢筋混凝土柱或组合柱，其截面和配筋分别不宜小于排架柱或纵墙组合柱，并应通到山墙的顶端与卧梁连接。

10.3.8 舞台后墙，大厅与前厅交接处的高大山墙，应利用工作平台或楼层作为水平支撑。

11 土、木、石结构房屋

11.1 村镇生土房屋

11.1.1 本节适用于6～8度未经焙烧的土坯、灰土和夯土承重墙体的房屋及土窑洞、土拱房。

注：1 灰土墙指掺石灰（或其他粘结材料）的土筑墙和掺石灰土坯墙；

2 土窑洞包括在未经扰动的原土中开挖而成的崖窑和由土坯砌筑拱顶的坑窑。

11.1.2 生土房屋宜建单层，6度和7度的灰土墙房屋可建二层，但总高度不应超过6m；单层生土房屋的檐口高度不宜大于2.5m，开间不宜大于3.2m；窑洞净跨不宜大于2.5m。

11.1.3 生土房屋开间均应有横墙，不宜采用土搁梁结构，同一房屋不宜采用不同材料的承重墙体。

11.1.4 应采用轻屋面材料；硬山搁檩的房屋宜采用双坡屋面或弧形屋面，檩条支撑处应设垫木；檐口标高处（墙顶）应有木圈梁（或木垫板），端檩应出檐，内墙上檩条应满搭或采用夹板对接和燕尾接。木屋盖各构件应采用圆钉、扒钉、铅丝等相互连接。

11.1.5 生土房屋内外墙体应同时分层交错夯筑或咬砌，外墙四角和内外墙交接处，宜沿墙高每隔300mm左右放一层竹筋、木条、荆条等拉结材料。

11.1.6 各类生土房屋的地基应夯实，应做砖或石基础；宜作外墙裙防潮处理（墙脚宜设防潮层）。

11.1.7 土坯宜采用粘性土湿法成型并宜掺入草苇等拉结材料；土坯应卧砌并宜采用粘土浆或粘土石灰浆砌筑。

11.1.8 灰土墙房屋应每层设置圈梁，并在横墙上拉通；内纵墙顶面宜在山尖墙两侧增砌踏步式墙垛。

11.1.9 土拱房应多跨连接布置，各拱脚均应支承在稳固的崖体上或支承在人工土墙上；拱圈厚度宜为300～400mm，应支模砌筑，不应后倾贴砌；外侧支承墙和拱圈上不应布置门窗。

11.1.10 土窑洞应避开易产生滑坡、山崩的地段；开挖窑洞的崖体应土质密实、土体稳定、坡度较平缓、无明显的竖向节理；崖窑前不宜接砌土坯或其他材料的前脸；不宜开挖层窑，否则应保持足够的间距，且上、下不宜对齐。

11.2 木结构房屋

11.2.1 本节适用于穿斗木构架、木柱木屋架和木柱木梁等房屋。

11.2.2 木结构房屋的平面布置应避免拐角或突出；同一房屋不应采用木柱与砖柱或砖墙等混合承重。

11.2.3 木柱木屋架和穿斗木构架房屋不宜超过二层，总高度不宜超过6m。木柱木梁房屋宜建单层，高度不宜超过3m。

11.2.4 礼堂、剧院、粮仓等较大跨度的空旷房屋，宜采用四柱落地的三跨木排架。

11.2.5 木屋架屋盖的支撑布置，应符合本规范第 9.3 节的有关规定的要求，但房屋两端的屋架支撑，应设置在端开间。

11.2.6 柱顶应有暗榫插入屋架下弦，并用 U 形铁件连接；8 度和 9 度时，柱脚应采用铁件或其他措施与基础锚固。

11.2.7 空旷房屋应在木柱与屋架（或梁）间设置斜撑；横隔墙较多的居住房屋应在非抗震隔墙内设斜撑，穿斗木构架房屋可不设斜撑；斜撑宜采用木夹板，并应通到屋架的上弦。

11.2.8 穿斗木构架房屋的横向和纵向均应在木柱的上、下柱端和楼层下部设置穿枋，并应在每一纵向柱列间设置 1～2 道剪刀撑或斜撑。

11.2.9 斜撑和屋盖支撑结构，均应采用螺栓与主体构件相连接；除穿斗木构件外，其他木构件宜采用螺栓连接。

11.2.10 椽与檩的搭接处应满钉，以增强屋盖的整体性。木构架中，宜在柱檐口以上沿房屋纵向设置竖向剪刀撑等措施，以增强纵向稳定性。

11.2.11 木构件应符合下列要求：

1 木柱的梢径不宜小于 150mm；应避免在柱的同一高度处纵横向同时开槽，且在柱的同一截面开槽面积不应超过截面总面积的 1/2。

2 柱子不能有接头。

3 穿枋应贯通木构架各柱。

11.2.12 围护墙应与木结构可靠拉结；土坯、砖等砌筑的围护墙不应将木柱完全包裹，宜贴砌在木柱外侧。

11.3 石结构房屋

11.3.1 本节适用于 6～8 度，砂浆砌筑的料石砌体（包括有垫片或无垫片）承重的房屋。

11.3.2 多层石砌体房屋的总高度和层数不宜超过表 11.3.2 的规定。

表 11.3.2 多层石房总高度（m）和层数限值

墙体类别	烈度					
	6		7		8	
	高度	层数	高度	层数	高度	层数
细、半细料石砌体（无垫片）	16	五	13	四	10	三
粗料石及毛料石砌体（有垫片）	13	四	10	三	7	二

注：房屋总高度的计算同表 7.1.2 注。

11.3.3 多层石砌体房屋的层高不宜超过 3m。

11.3.4 多层石砌体房屋的抗震横墙间距，不应超过表 11.3.4 的规定。

表 11.3.4 多层石房的抗震横墙间距（m）

楼、屋盖类型	烈度		
	6	7	8
现浇及装配整体式钢筋混凝土	10	10	7
装配整体式钢筋混凝土	7	7	4

11.3.5 多层石房，宜采用现浇或装配整体式钢筋混凝土楼、屋盖。

11.3.6 石墙的截面抗震验算，可参照本规范第 7.2 节；其抗剪强度应根据试验数据确定。

11.3.7 多层石房的下列部位，应设置钢筋混凝土构造柱：

1 外墙四角和楼梯间四角。

2 6 度隔开间的内外墙交接处。

3 7 度和 8 度每开间的内外墙交接处。

11.3.8 抗震横墙洞口的水平截面面积，不应大于全截面面积的 1/3。

11.3.9 每层的纵横墙均应设置圈梁，其截面高度不应小于 120mm，宽度宜与墙厚相同，纵向钢筋不应小于 4ϕ10，箍筋间距不宜大于 200mm。

11.3.10 无构造柱的纵横墙交接处，应采用条石无垫片砌筑，且应沿墙高每隔 500mm 设置拉结钢筋网片，每边每侧伸入墙内不宜小于 1m。

11.3.11 其他有关抗震构造措施要求，参照本规范第 7 章的规定。

12 隔震和消能减震设计

12.1 一 般 规 定

12.1.1 本章适用于在建筑上部结构与基础之间设置隔震层以隔离地震能量的房屋隔震设计，以及在抗侧力结构中设置消能器吸收与消耗地震能量的房屋消能减震设计。

采用隔震和消能减震设计的建筑结构，应符合本规范第3.8.1条的规定，其抗震设防目标应符合本规范第3.8.2条的规定。

注：1 本章隔震设计指在房屋底部设置的由橡胶隔震支座和阻尼器等部件组成的隔震层，以延长整个结构体系的自振周期、增大阻尼，减少输入上部结构的地震能量，达到预期防震要求。

2 消能减震设计指在房屋结构中设置消能装置，通过其局部变形提供附加阻尼，以消耗输入上部结构的地震能量，达到预期防震要求。

12.1.2 建筑结构的隔震设计和消能减震设计，应根据建筑抗震设防类别、抗震设防烈度、场地条件、建筑结构方案和建筑使用要求，与采用抗震设计的设计方案进行技术、经济可行性的对比分析后，确定其设计方案。

12.1.3 需要减少地震作用的多层砌体和钢筋混凝土框架等结构类型的房屋，采用隔震设计时应符合下列各项要求：

1 结构体型基本规则，不隔震时可在两个主轴方向分别采用本规范第5.1.2条规定的底部剪力法进行计算且结构基本周期小于1.0s；体型复杂结构采用隔震设计，宜通过模型试验后确定。

2 建筑场地宜为Ⅰ、Ⅱ、Ⅲ类，并应选用稳定性较好的基础类型。

3 风荷载和其他非地震作用的水平荷载标准值产生的总水平力不宜超过结构总重力的10%。

4 隔震层应提供必要的竖向承载力、侧向刚度和阻尼；穿过隔震层的设备配管、配线，应采用柔性连接或其他有效措施适应隔震层的罕遇地震水平位移。

12.1.4 需要减少地震水平位移的钢和钢筋混凝土等结构类型的房屋，宜采用消能减震设计。

消能部件应对结构提供足够的附加阻尼，尚应根据其结构类型分别符合本规范相应章节的设计要求。

12.1.5 隔震和消能减震设计时，隔震部件和消能减震部件应符合下列要求：

1 隔震部件和消能减震部件的耐久性和设计参数应由试验确定。

2 设置隔震部件和消能减震部件的部位，除按计算确定外，应采取便于检查和替换的措施。

3 设计文件上应注明对隔震部件和消能减震部件性能要求，安装前应对工程中所用的各种类型和规格的原型部件进行抽样检测，每种类型和每一规格的数量不应少于3个，抽样检测的合格率应为100%。

12.1.6 建筑结构的隔震设计和消能减震设计，尚应符合相关专门标准的规定。

12.2 房屋隔震设计要点

12.2.1 **隔震设计应根据预期的水平向减震系数和位移控制要求，选择适当的隔震支座（含阻尼器）及为抵抗地基微震动与风荷载提供初刚度的部件组成结构的隔震层。**

隔震支座应进行竖向承载力的验算和罕遇地震下水平位移的验算。

隔震层以上结构的水平地震作用应根据水平向减震系数确定；其竖向地震作用标准值，8度和9度时分别不应小于隔震层以上结构总重力荷载代表值的20%和40%。

12.2.2 建筑结构隔震设计的计算分析，应符合下列规定：

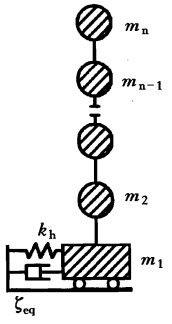


图 12.2.2 隔震结构计算简图

1 隔震体系的计算简图可采用剪切型结构模型（图 12.2.2）；当上部结构的质心与隔震层刚度中心不重合时应计入扭转变形的影响。隔震层顶部的梁板结构，对钢筋混凝土结构应作为其上部结构的一部分进行计算和设计。

2 一般情况下，宜采用时程分析法进行计算；输入地震波的反应谱特性和数量，应符合本规范第 5.1.2 条的规定；计算结果宜取其平均值；当处于发震断层 10km 以内时，若输入地震波未计及近场影响，对甲、乙类建筑，计算结果尚应乘以下列近场影响系数：5km 以内取 1.5，5km 以外取 1.25。

3 砌体结构及基本周期与其相当的结构可按本规范附录 L 简化计算。

12.2.3 隔震层由橡胶和薄钢板相间层叠组成的橡胶隔震支座应符合下列要求：

1 隔震支座在表 12.2.3 所列的压应力下的极限水平变位，应大于其有效直径的 0.55 倍和各橡胶层总厚度 3.0 倍二者的较大值。

2 在经历相应设计基准期的耐久试验后，隔震支座刚度、阻尼特性变化不超过初期值的 ±20%；徐变量不超过各橡胶层总厚度的 5%。

3 各橡胶隔震支座的竖向平均压应力设计值，不应超过表 12.2.3 的规定。

表 12.2.3 橡胶隔震支座平均压应力限值

建 筑 类 别	甲类建筑	乙类建筑	丙类建筑
平均压应力限值（MPa）	10	12	15

注：1 平均压应力设计值应按永久荷载和可变荷载组合计算，对需验算倾覆的结构应包括水平地震作用效应组合；对需进行竖向地震作用计算的结构，尚应包括竖向地震作用效应组合；

2 当橡胶支座的第二形状系数（有效直径与各橡胶层总厚度之比）小于 5.0 时应降低平均压应力限值：小于 5 不小于 4 时降低 20%，小于 4 不小于 3 时降低 40%；

3 外径小于 300mm 的橡胶支座，其平均压应力限值对丙类建筑为 12MPa。

12.2.4 隔震层的布置、竖向承载力、侧向刚度和阻尼应符合下列规定：

1 隔震层宜设置在结构第一层以下的部位，其橡胶隔震支座应设置在受力较大的位置，间距不宜过大，其规格、数量和分布应根据竖向承载力、侧向刚度和阻尼的要求通过计算确定。隔震层在罕遇地震下应保持稳定，不宜出现不可恢复的变形。隔震层橡胶支座在罕遇地震作用下，不宜出现拉应力。

2 隔震层的水平动刚度和等效粘滞阻尼比可按下列公式计算：

$$K_h = \Sigma K_j \tag{12.2.4-1}$$

$$\zeta_{eq} = \Sigma K_j \zeta_j / K_h \tag{12.2.4-2}$$

式中 ζ_{eq}——隔震层等效粘滞阻尼比；

K_h——隔震层水平动刚度；

ζ_j——j 隔震支座由试验确定的等效粘滞阻尼比，单独设置阻尼器时，应包括该阻尼器的相应阻尼比；

K_j——j 隔震支座（含阻尼器）由试验确定的水平动刚度，当试验发现动刚度与加载频率有关时，宜取相应于隔震体系基本自振周期的动刚度值。

3 隔震支座由试验确定设计参数时，竖向荷载应保持表 12.2.3 的平均压应力限值，对多遇地震验算，宜采用水平加载频率为 0.3Hz 且隔震支座剪切变形为 50% 的水平动刚度和等效粘滞阻尼比；对罕遇地震验算，直径小于 600mm 的隔震支座宜采用水平加载频率为 0.1Hz 且隔震支座剪切变形不小于 250% 时的水平动刚度和等效粘滞阻尼比，直径不小于 600mm 的隔震支座可采用水平加载频率为 0.2Hz 且隔震支座剪切变形为 100% 时的水平动刚度和等效粘滞阻尼比。

12.2.5 隔震层以上结构的地震作用计算，应符合下列规定：

1 水平地震作用沿高度可采用矩形分布；水平地震影响系数的最大值可采用本规范第 5.1.4 条规定的水平地震影响系数最大值和水平向减震系数的乘积。水平向减震系数应根据结构隔震与非隔震两种情况下各层层间剪力的最大比值，按表 12.2.5 确定。

表 12.2.5 层间剪力最大比值与水平向减震系数的对应关系

层间剪力最大比值	0.53	0.35	0.26	0.18
水平向减震系数	0.75	0.50	0.38	0.25

2 水平向减震系数不宜低于 0.25，且隔震后结构的总水平地震作用不得低于非隔震的结构在 6 度设防时的总水平地震作用；各楼层的水平地震剪力尚应符合本规范第 5.2.5 条最小地震剪力系数的规定。

3 9 度时和 8 度且水平向减震系数为 0.25 时，隔震层以上的结构应进行竖向地震作用的计算；8 度且水平向减震系数不大于 0.5 时，宜进行竖向地震作用的计算。

隔震层以上结构竖向地震作用标准值计算时，各楼层可视为质点，并按本规范第 5.3 节公式（5.3.1-2）计算竖向地震作用标准值沿高度的分布。

12.2.6 隔震支座的水平剪力应根据隔震层在罕遇地震下的水平剪力按各隔震支座的水平刚度分配；当按扭转耦联计算时，尚应计及隔震支座的扭转刚度。

隔震支座对应于罕遇地震水平剪力的水平位移，应符合下列要求：

$$u_i \leqslant [u_i] \tag{12.2.6-1}$$

$$u_i = \beta_i u_c \tag{12.2.6-2}$$

式中 u_i——罕遇地震作用下，第 i 个隔震支座考虑扭转的水平位移；

$[u_i]$——第 i 个隔震支座的水平位移限值；对橡胶隔震支座，不应超过该支座有效直径的 0.55 倍和支座各橡胶层总厚度 3.0 倍二者的较小值；

u_c——罕遇地震下隔震层质心处或不考虑扭转的水平位移；

β_i——第 i 个隔震支座的扭转影响系数，应取考虑扭转和不考虑扭转时 i 支座计算位移的比值；当隔震层以上结构的质心与隔震层刚度中心在两个主轴方向均无偏心时，边支座的扭转影响系数不应小于 1.15。

12.2.7 隔震层以上结构的隔震措施，应符合下列规定：

1 隔震层以上结构应采取不阻碍隔震层在罕遇地震下发生大变形的下列措施：

1） 上部结构的周边应设置防震缝，缝宽不宜小于各隔震支座在罕遇地震下的最大水平位移值的1.2倍。

2） 上部结构（包括与其相连的任何构件）与地面（包括地下室和与其相连的构件）之间，宜设置明确的水平隔离缝；当设置水平隔离缝确有困难时，应设置可靠的水平滑移垫层。

3） 在走廊、楼梯、电梯等部位，应无任何障碍物。

2 丙类建筑在隔震层以上结构的抗震措施，当水平向减震系数为0.75时不应降低非隔震时的有关要求；水平向减震系数不大于0.50时，可适当降低本规范有关章节对非隔震建筑的要求，但与抵抗竖向地震作用有关的抗震构造措施不应降低。此时，对砌体结构，可按本规范附录L采取抗震构造措施；对钢筋混凝土结构，柱和墙肢的轴压比控制应仍按非隔震的有关规定采用，其他计算和抗震构造措施要求，可按表12.2.7划分抗震等级，再按本规范第6章的有关规定采用。

表12.2.7 隔震后现浇钢筋混凝土结构的抗震等级

结构类型		7度		8度		9度	
框架	高度（m）	＜20	＞20	＜20	＞20	＜20	＞20
	一般框架	四	三	三	二	二	一
抗震墙	高度（m）	＜25	＞25	＜25	＞25	＜25	＞25
	一般抗震墙	四	三	三	二	二	一

12.2.8 隔震层与上部结构的连接，应符合下列规定：

1 隔震层顶部应设置梁板式楼盖，且应符合下列要求：

1） 应采用现浇或装配整体式混凝土板。现浇板厚度不宜小于140mm；配筋现浇面层厚度不应小于50mm。隔震支座上方的纵、横梁应采用现浇钢筋混凝土结构。

2） 隔震层顶部梁板的刚度和承载力，宜大于一般楼面梁板的刚度和承载力。

3） 隔震支座附近的梁、柱应计算冲切和局部承压，加密箍筋并根据需要配置网状钢筋。

2 隔震支座和阻尼器的连接构造，应符合下列要求：

1） 隔震支座和阻尼器应安装在便于维护人员接近的部位；

2） 隔震支座与上部结构、基础结构之间的连接件，应能传递罕遇地震下支座的最大水平剪力；

3） 抗震墙下隔震支座的间距不宜大于2.0m；

4） 外露的预埋件应有可靠的防锈措施。预埋件的锚固钢筋应与钢板牢固连接，锚固钢筋的锚固长度宜大于20倍锚固钢筋直径，且不应小于250mm。

12.2.9 隔震层以下结构（包括地下室）的地震作用和抗震验算，应采用罕遇地震下隔震支座底部的竖向力、水平力和力矩进行计算。

隔震建筑地基基础的抗震验算和地基处理仍应按本地区抗震设防烈度进行，甲、乙类建筑的抗液化措施应按提高一个液化等级确定，直至全部消除液化沉陷。

12.3 房屋消能减震设计要点

12.3.1 消能减震设计时，应根据罕遇地震下的预期结构位移控制要求，设置适当的消能部件。消能部件可由消能器及斜撑、墙体、梁或节点等支承构件组成。消能器可采用速度相关型、位移相关型或其他类型。

注：1 速度相关型消能器指粘滞消能器和粘弹性消能器等；

2 位移相关型消能器指金属屈服消能器和摩擦消能器等。

12.3.2 消能部件可根据需要沿结构的两个主轴方向分别设置。消能部件宜设置在层间变形较大的位置，其数量和分布应通过综合分析合理确定，并有利于提高整个结构的消能减震能力，形成均匀合理的受力体系。

12.3.3 消能减震设计的计算分析，应符合下列规定：

1 一般情况下，宜采用静力非线性分析方法或非线性时程分析方法。

2 当主体结构基本处于弹性工作阶段时，可采用线性分析方法作简化估算，并根据结构的变形特征和高度等，按本规范第 5.1 节的规定分别采用底部剪力法、振型分解反应谱法和时程分析法。其地震影响系数可根据消能减震结构的总阻尼比按本规范第 5.1.5 条的规定采用。

3 消能减震结构的总刚度应为结构刚度和消能部件有效刚度的总和。

4 消能减震结构的总阻尼比应为结构阻尼比和消能部件附加给结构的有效阻尼比的总和。

5 消能减震结构的层间弹塑性位移角限值，框架结构宜采用 1/80。

12.3.4 消能部件附加给结构的有效阻尼比，可按下列方法确定：

1 消能部件附加的有效阻尼比可按下式估算：

$$\zeta_a = W_c/(4\pi W_s) \tag{12.3.4-1}$$

式中 ζ_a——消能减震结构的附加有效阻尼比；

W_c——所有消能部件在结构预期位移下往复一周所消耗的能量；

W_s——设置消能部件的结构在预期位移下的总应变能。

2 不计及扭转影响时，消能减震结构在其水平地震作用下的总应变能，可按下式估算：

$$W_s = (1/2)\Sigma F_i u_i \tag{12.3.4-2}$$

式中 F_i——质点 i 的水平地震作用标准值；

u_i——质点 i 对应于水平地震作用标准值的位移。

3 速度线性相关型消能器在水平地震作用下所消耗的能量，可按下式估算：

$$W_c = (2\pi^2/T_1)\Sigma C_j \cos^2\theta_j \Delta u_j^2 \tag{12.3.4-3}$$

式中 T_1——消能减震结构的基本自振周期；

C_j——第 j 个消能器由试验确定的线性阻尼系数；

θ_j——第 j 个消能器的消能方向与水平面的夹角；

Δu_j——第 j 个消能器两端的相对水平位移。

当消能器的阻尼系数和有效刚度与结构振动周期有关时，可取相应于消能减震结构基

本自振周期的值。

4 位移相关型、速度非线性相关型和其他类型消能器在水平地震作用下所消耗的能量，可按下式估算：

$$W_c = \Sigma A_j \tag{12.3.4-4}$$

式中 A_j——第 j 个消能器的恢复力滞回环在相对水平位移 Δu_j 时的面积。

消能器的有效刚度可取消能器的恢复力滞回环在相对水平位移 Δu_j 时的割线刚度。

5 消能部件附加给结构的有效阻尼比超过20%时，宜按20%计算。

12.3.5 对非线性时程分析法，宜采用消能部件的恢复力模型计算；对静力非线性分析法，消能器附加给结构的有效阻尼比和有效刚度，可采用本章第12.3.4条的方法确定。

12.3.6 消能部件由试验确定的有效刚度、阻尼比和恢复力模型的设计参数，应符合下列规定：

1 速度相关型消能器应由试验提供设计容许位移、极限位移，以及设计容许位移幅值和不同环境温度条件下、加载频率为0.1～4Hz的滞回模型。速度线性相关型消能器与斜撑、墙体或梁等支承构件组成消能部件时，该支承构件在消能器消能方向的刚度可按下式计算：

$$K_b = (6\pi/T_1)C_v \tag{12.3.6-1}$$

式中 K_b——支承构件在消能器方向的刚度；

C_v——消能器的由试验确定的相应于结构基本自振周期的线性阻尼系数；

T_1——消能减震结构的基本自振周期。

2 位移相关型消能器应由往复静力加载确定设计容许位移、极限位移和恢复力模型参数。位移相关型消能器与斜撑、墙体或梁等支承构件组成消能部件时，该部件的恢复力模型参数宜符合下列要求：

$$\Delta u_{py}/\Delta u_{sy} \leqslant 2/3 \tag{12.3.6-2}$$

$$(K_p/K_s)(\Delta u_{py}/\Delta u_{sy}) \geqslant 0.8 \tag{12.3.6-3}$$

式中 K_p——消能部件在水平方向的初始刚度；

Δu_{py}——消能部件的屈服位移；

K_s——设置消能部件的结构楼层侧向刚度；

Δu_{sy}——设置消能部件的结构层间屈服位移。

3 在最大设计允许位移幅值下，经往复周期循环60圈后，消能器的主要性能衰减量不应超过10%、且不应有明显的低周疲劳现象。

12.3.7 消能器与斜撑、墙体、梁或节点等支承构件的连接，应符合钢构件连接或钢与钢筋混凝土构件连接的构造要求，并能承担消能器施加给连接节点的最大作用力。

12.3.8 与消能部件相连的结构构件，应计入消能部件传递的附加内力，并将其传递到基础。

12.3.9 消能器和连接构件应具有耐久性能和较好的易维护性。

13 非结构构件

13.1 一般规定

13.1.1 本章主要适用于非结构构件与建筑结构的连接。非结构构件包括持久性的建筑非结构构件和支承于建筑结构的附属机电设备。

注：1 建筑非结构构件指建筑中除承重骨架体系以外的固定构件和部件，主要包括非承重墙体，附着于楼面和屋面结构的构件、装饰构件和部件、固定于楼面的大型储物架等。

2 建筑附属机电设备指为现代建筑使用功能服务的附属机械、电气构件、部件和系统，主要包括电梯，照明和应急电源、通信设备，管道系统，采暖和空气调节系统，烟火监测和消防系统，公用天线等。

13.1.2 非结构构件应根据所属建筑的抗震设防类别和非结构地震破坏的后果及其对整个建筑结构影响的范围，采取不同的抗震措施；当相关专门标准有具体要求时，尚应采用不同的功能系数、类别系数等进行抗震计算。

13.1.3 当计算和抗震措施要求不同的两个非结构构件连接在一起时，应按较高的要求进行抗震设计。

非结构构件连接损坏时，应不致引起与之相连接的有较高要求的非结构构件失效。

13.2 基本计算要求

13.2.1 建筑结构抗震计算时，应按下列规定计入非结构构件的影响：

1 地震作用计算时，应计入支承于结构构件的建筑构件和建筑附属机电设备的重力。

2 对柔性连接的建筑构件，可不计入刚度；对嵌入抗侧力构件平面内的刚性建筑非结构构件，可采用周期调整等简化方法计入其刚度影响；一般情况下不应计入其抗震承载力，当有专门的构造措施时，尚可按有关规定计入其抗震承载力。

3 对需要采用楼面谱计算的建筑附属机电设备，宜采用合适的简化计算模型计入设备与结构的相互作用。

4 支承非结构构件的结构构件，应将非结构构件地震作用效应作为附加作用对待，并满足连接件的锚固要求。

13.2.2 非结构构件的地震作用计算方法，应符合下列要求：

1 各构件和部件的地震力应施加于其重心，水平地震力应沿任一水平方向。

2 一般情况下，非结构构件自身重力产生的地震作用可采用等效侧力法计算；对支承于不同楼层或防震缝两侧的非结构构件，除自身重力产生的地震作用外，尚应同时计及地震时支承点之间相对位移产生的作用效应。

3 建筑附属设备（含支架）的体系自振周期大于0.1s且其重力超过所在楼层重力的1%，或建筑附属设备的重力超过所在楼层重力的10%时，宜采用楼面反应谱方法。其中，与楼盖非弹性连接的设备，可直接将设备与楼盖作为一个质点计入整个结构的分析中得到设备所受的地震作用。

13.2.3 采用等效侧力法时，水平地震作用标准值宜按下列公式计算：

$$F = \gamma\eta\zeta_1\zeta_2\alpha_{max}G \tag{13.2.3}$$

式中 F——沿最不利方向施加于非结构构件重心处的水平地震作用标准值；

γ——非结构构件功能系数，由相关标准根据建筑设防类别和使用要求等确定；

η——非结构构件类别系数，由相关标准根据构件材料性能等因素确定；

ζ_1——状态系数；对预制建筑构件、悬臂类构件、支承点低于质心的任何设备和柔性体系宜取 2.0，其余情况可取 1.0；

ζ_2——位置系数，建筑的顶点宜取 2.0，底部宜取 1.0，沿高度线性分布；对本规范第 5 章要求采用时程分析法补充计算的结构，应按其计算结果调整；

α_{max}——地震影响系数最大值；可按本规范第 5.1.4 条关于多遇地震的规定采用；

G——非结构构件的重力，应包括运行时有关的人员、容器和管道中的介质及储物柜中物品的重力。

13.2.4 非结构构件因支承点相对水平位移产生的内力，可按该构件在位移方向的刚度乘以规定的支承点相对水平位移计算。

非结构构件在位移方向的刚度，应根据其端部的实际连接状态，分别采用刚接、铰接、弹性连接或滑动连接等简化的力学模型。

相邻楼层的相对水平位移，可按本规范第 5.5 节规定的限值采用；防震缝两侧的相对水平位移，宜根据使用要求确定。

13.2.5 采用楼面反应谱法时，非结构构件的水平地震作用标准值宜按下列公式计算：

$$F = \gamma\eta\beta_s G \tag{13.2.5}$$

式中 β_s——非结构构件的楼面反应谱值，取决于设防烈度、场地条件、非结构构件与结构体系之间的周期比、质量比和阻尼，以及非结构构件在结构的支承位置、数量和连接性质。通常将非结构构件简化为支承于结构的单质点体系，对支座间有相对位移的非结构构件则采用多支点体系，按专门方法计算。

13.2.6 非结构构件的地震作用效应（包括自身重力产生的效应和支座相对位移产生的效应）和其他荷载效应的基本组合，应按本规范第 5.4 节的规定计算；幕墙需计算地震作用效应与风荷载效应的组合；容器类尚应计及设备运转时的温度、工作压力等产生的作用效应。

非结构构件抗震验算时，摩擦力不得作为抵抗地震作用的抗力；承载力抗震调整系数，连接件可采用 1.0，其余可按相关标准的规定采用。

13.3 建筑非结构构件的基本抗震措施

13.3.1 建筑结构中，设置连接幕墙、围护墙、隔墙、女儿墙、雨篷、商标、广告牌、顶篷支架、大型储物架等建筑非结构构件的预埋件、锚固件的部位，应采取加强措施，以承受建筑非结构构件传给主体结构的地震作用。

13.3.2 非承重墙体的材料、选型和布置，应根据烈度、房屋高度、建筑体型、结构层间变形、墙体自身抗侧力性能的利用等因素，经综合分析后确定。

1 墙体材料的选用应符合下列要求：

1） 混凝土结构和钢结构的非承重墙体应优先采用轻质墙体材料。

2）单层钢筋混凝土柱厂房的围护墙宜采用轻质墙板或钢筋混凝土大型墙板，外侧柱距为12m时应采用轻质墙板或钢筋混凝土大型墙板；不等高厂房的高跨封墙和纵横向厂房交接处的悬墙宜采用轻质墙板，8、9度时应采用轻质墙板；

3）钢结构厂房的围护墙，7、8度时宜采用轻质墙板或与柱柔性连接的钢筋混凝土墙板，不应采用嵌砌砌体墙；9度时宜采用轻质墙板。

2 刚性非承重墙体的布置，应避免使结构形成刚度和强度分布上的突变。单层钢筋混凝土柱厂房的刚性围护墙沿纵向宜均匀对称布置。

3 墙体与主体结构应有可靠的拉结，应能适应主体结构不同方向的层间位移；8、9度时应具有满足层间变位的变形能力，与悬挑构件相连接时，尚应具有满足节点转动引起的竖向变形的能力。

4 外墙板的连接件应具有足够的延性和适当的转动能力，宜满足在设防烈度下主体结构层间变形的要求。

13.3.3 砌体墙应采取措施减少对主体结构的不利影响，并应设置拉结筋、水平系梁、圈梁、构造柱等与主体结构可靠拉结：

1 多层砌体结构中，后砌的非承重隔墙应沿墙高每隔500mm配置2ϕ6拉结钢筋与承重墙或柱拉结，每边伸入墙内不应少于500mm；8度和9度时，长度大于5m的后砌隔墙，墙顶尚应与楼板或梁拉结。

2 钢筋混凝土结构中的砌体填充墙，宜与柱脱开或采用柔性连接，并应符合下列要求：

1）填充墙在平面和竖向的布置，宜均匀对称，宜避免形成薄弱层或短柱；

2）砌体的砂浆强度等级不应低于M5，墙顶应与框架梁密切结合；

3）填充墙应沿框架柱全高每隔500mm设2ϕ6拉筋，拉筋伸入墙内的长度，6、7度时不应小于墙长的1/5且不小于700mm，8、9度时宜沿墙全长贯通；

4）墙长大于5m时，墙顶与梁宜有拉结；墙长超过层高2倍时，宜设置钢筋混凝土构造柱；墙高超过4m时，墙体半高宜设置与柱连接且沿墙全长贯通的钢筋混凝土水平系梁。

3 单层钢筋混凝土柱厂房的砌体隔墙和围护墙应符合下列要求：

1）砌体隔墙与柱宜脱开或柔性连接，并应采取措施使墙体稳定，隔墙顶部应设现浇钢筋混凝土压顶梁。

2）厂房的砌体围护墙宜采用外贴式并与柱可靠拉结；不等高厂房的高跨封墙和纵横向厂房交接处的悬墙采用砌体时，不应直接砌在低跨屋盖上。

3）砌体围护墙在下列部位应设置现浇钢筋混凝土圈梁：

——梯形屋架端部上弦和柱顶的标高处应各设一道，但屋架端部高度不大于900mm时可合并设置；

——8度和9度时，应按上密下稀的原则每隔4m左右在窗顶增设一道圈梁，不等高厂房的高低跨封墙和纵横跨交接处的悬墙，圈梁的竖向间距不应大于3m；

——山墙沿屋面应设钢筋混凝土卧梁，并应与屋架端部上弦标高处的圈梁连接。

4）圈梁的构造应符合下列规定：

——圈梁宜闭合，圈梁截面宽度宜与墙厚相同，截面高度不应小于180mm；圈梁的纵筋，6～8度时不应少于4ϕ12，9度时不应少于4ϕ14；

——厂房转角处柱顶圈梁在端开间范围内的纵筋，6～8度时不宜少于4ϕ14，9度时不宜少于4ϕ16，转角两侧各1m范围内的箍筋直径不宜小于ϕ8，间距不宜大于100mm；圈梁转角处应增设不少于3根且直径与纵筋相同的水平斜筋；

——圈梁应与柱或屋架牢固连接，山墙卧梁应与屋面板拉结；顶部圈梁与柱或屋架连接的锚拉钢筋不宜少于4ϕ12，且锚固长度不宜少于35倍钢筋直径，防震缝处圈梁与柱或屋架的拉结宜加强。

5）8度Ⅲ、Ⅳ类场地和9度时，砖围护墙下的预制基础梁应采用现浇接头；当另设条形基础时，在柱基础顶面标高处应设置连续的现浇钢筋混凝土圈梁，其配筋不应少于4ϕ12。

6）墙梁宜采用现浇，当采用预制墙梁时，梁底应与砖墙顶面牢固拉结并应与柱锚拉；厂房转角处相邻的墙梁，应相互可靠连接。

4 单层钢结构厂房的砌体围护墙不应采用嵌砌式，8度时尚应采取措施使墙体不妨碍厂房柱列沿纵向的水平位移。

5 砌体女儿墙在人流出入口应与主体结构锚固；防震缝处应留有足够的宽度，缝两侧的自由端应予以加强。

13.3.4 各类顶棚的构件与楼板的连接件，应能承受顶棚、悬挂重物和有关机电设施的自重和地震附加作用；其锚固的承载力应大于连接件的承载力。

13.3.5 悬挑雨篷或一端由柱支承的雨篷,应与主体结构可靠连接。

13.3.6 玻璃幕墙、预制墙板、附属于楼屋面的悬臂构件和大型储物架的抗震构造，应符合相关专门标准的规定。

13.4 建筑附属机电设备支架的基本抗震措施

13.4.1 附属于建筑的电梯、照明和应急电源系统、烟火监测和消防系统、采暖和空气调节系统、通信系统、公用天线等与建筑结构的连接构件和部件的抗震措施，应根据设防烈度、建筑使用功能、房屋高度、结构类型和变形特征、附属设备所处的位置和运转要求等，按相关专门标准的要求经综合分析后确定。

下列附属机电设备的支架可无抗震设防要求：

——重力不超过1.8kN的设备；

——内径小于25mm的煤气管道和内径小于60mm的电气配管；

——矩形截面面积小于0.38m^{2}和圆形直径小于0.70m的风管；

——吊杆计算长度不超过300mm的吊杆悬挂管道。

13.4.2 建筑附属机电设备不应设置在可能导致其使用功能发生障碍等二次灾害的部位；对于有隔振装置的设备，应注意其强烈振动对连接件的影响，并防止设备和建筑结构发生谐振现象。

建筑附属机电设备的支架应具有足够的刚度和强度；其与建筑结构应有可靠的连接和锚固，应使设备在遭遇设防烈度地震影响后能迅速恢复运转。

13.4.3 管道、电缆、通风管和设备的洞口设置，应减少对主要承重结构构件的削弱；洞口边缘应有补强措施。

管道和设备与建筑结构的连接，应能应允许二者间有一定的相对变位。

13.4.4 建筑附属机电设备的基座或连接件应能将设备承受的地震作用全部传递到建筑结构上。建筑结构中，用以固定建筑附属机电设备预埋件、锚固件的部位，应采取加强措施，以承受附属机电设备传给主体结构的地震作用。

13.4.5 建筑内的高位水箱应与所在的结构构件可靠连接；8、9 度时按本规范第 5.1.2 条规定需采用时程分析的高层建筑，尚宜计及水对建筑结构产生的附加地震作用效应。

13.4.6 在设防烈度地震下需要连续工作的附属设备，宜设置在建筑结构地震反应较小的部位；相关部位的结构构件应采取相应的加强措施。

附录A　我国主要城镇抗震设防烈度、设计基本地震加速度和设计地震分组

本附录仅提供我国抗震设防区各县级及县级以上城镇的中心地区建筑工程抗震设计时所采用的抗震设防烈度、设计基本地震加速度值和所属的设计地震分组。

注：本附录一般把“设计地震第一、二、三组”简称为“第一组、第二组、第三组”。

A.0.1　首都和直辖市

1　抗震设防烈度为8度，设计基本地震加速度值为0.20g：

北京（除昌平、门头沟外的11个市辖区），平谷，大兴，延庆，宁河，汉沽。

2　抗震设防烈度为7度，设计基本地震加速度值为0.15g：

密云，怀柔，昌平，门头沟，天津（除汉沽、大港外的12个市辖区），蓟县，宝坻，静海。

3　抗震设防烈度为7度，设计基本地震加速度值为0.10g：

大港，上海（除金山外的15个市辖区），南汇，奉贤

4　抗震设防烈度为6度，设计基本地震加速度值为0.05g：

崇明，金山，重庆（14个市辖区），巫山，奉节，云阳，忠县，丰都，长寿，壁山，合川，铜梁，大足，荣昌，永川，江津，綦江，南川，黔江，石柱，巫溪*

注：**1**　首都和直辖市的全部县级及县级以上设防城镇，设计地震分组均为第一组；

2　上标*指该城镇的中心位于本设防区和较低设防区的分界线，下同。

A.0.2　河北省

1　抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：廊坊（2个市辖区），唐山（5个市辖区），三河，大厂，香河，丰南，丰润，怀来，涿鹿

2　抗震设防烈度为**7**度，设计基本地震加速度值为0.15g：

第一组：邯郸（4个市辖区），邯郸县，文安，任丘，河间，大城，涿州，高碑店，涞水，固安，永清，玉田，迁安，卢龙，滦县，滦南，唐海，乐亭，宣化，蔚县，阳原，成安，磁县，临漳，大名，宁晋，下花园

3　抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：石家庄（6个市辖区），保定（3个市辖区），张家口（桥西区、桥东区），沧州（2个市辖区），衡水，邢台（2个市辖区），霸州，雄县，易县，沧县，张北，万全，怀安，兴隆，迁西，抚宁，昌黎，青县，献县，广宗，平乡，鸡泽，隆尧，新河，曲周，肥乡，馆陶，广平，高邑，内丘，邢台县，赵县，武安，涉县，赤城，涞源，定兴，容城，徐水，安新，高阳，博野，蠡县，肃宁，深泽，安平，饶阳，魏县，藁城，栾城，晋州，深州，武强，辛集，冀州，任县，柏乡，巨鹿，南和，沙河，临城，泊头，永年，崇礼，南宫*

第二组：秦皇岛（海港、北戴河），清苑，遵化，安国

4 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：正定，围场，尚义，灵寿，无极，平山，鹿泉，井陉，元氏，南皮，吴桥，景县，东光

第二组：承德(除鹰手营子外的2个市辖区)，隆化，承德县，宽城，青龙，阜平，满城，顺平，唐县，望都，曲阳，定州，行唐，赞皇，黄骅，海兴，孟村，盐山，阜城，故城，清河，山海关，沽源，新乐，武邑，枣强，威县

第三组：丰宁，滦平，鹰手营子，平泉，临西，邱县

A.0.3 山西省

1 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：太原（6个市辖区），临汾，忻州，祁县，平遥，古县，代县，原平，定襄，阳曲，太谷，介休，灵石，汾西，霍州，洪洞，襄汾，晋中，浮山，永济，清徐

2 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：大同（4个市辖区），朔州（朔城区），大同县，怀仁，浑源，广灵，应县，山阴，灵丘，繁峙，五台，古交，交城，文水，汾阳，曲沃，孝义，侯马，新绛，稷山，绛县，河津，闻喜，翼城，万荣，临猗，夏县，运城，芮城，平陆，沁源*，宁武*

3 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：长治（2个市辖区），阳泉（3个市辖区），长治县，阳高，天镇，左云，右玉，神池，寿阳，昔阳，安泽，乡宁，垣曲，沁水，平定，和顺，黎城，潞城，壶关

第二组：平顺，榆社，武乡，娄烦，交口，隰县，蒲县，吉县，静乐，盂县，沁县，陵川，平鲁

4 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第二组：偏关，河曲，保德，兴县，临县，方山，柳林

第三组：晋城，离石，左权，襄垣，屯留，长子，高平，阳城，泽州，五寨，岢岚，岚县，中阳，石楼，永和，大宁

A.0.4 内蒙自治区

1 抗震设防烈度为8度，设计基本地震加速度值为0.30g：

第一组：土默特右旗，达拉特旗*

2 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：包头(除白云矿区外的5个市辖区)，呼和浩特(4个市辖区)，土默特左旗，乌海(3个市辖区)，杭锦后旗，磴口，宁城，托克托*

3 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：喀喇沁旗，五原，乌拉特前旗，临河，固阳，武川，凉城，和林格尔，赤峰(红山*，元宝山区)

第二组：阿拉善左旗

4 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：集宁，清水河，开鲁，傲汉旗，乌特拉后旗，卓资，察右前旗，丰镇，扎兰屯，乌特拉中旗，赤峰（松山区），通辽*

第三组：东胜，准格尔旗

5 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：满洲里，新巴尔虎右旗，莫力达瓦旗，阿荣旗，扎赉特旗，翁牛特旗，兴

和，商都，察右后旗，科左中旗，科左后旗，奈曼旗，库伦旗，乌审旗，苏尼特右旗

第二组：达尔罕茂明安联合旗，阿拉善右旗，鄂托克旗，鄂托克前旗，白云

第三组：伊金霍洛旗，杭锦旗，四王子旗，察右中旗

A.0.5 辽宁省

1 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

普兰店，东港

2 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

营口（4个市辖区），丹东（3个市辖区），海城，大石桥，瓦房店，盖州，金州

3 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

沈阳（9个市辖区），鞍山（4个市辖区），大连（除金州外的5个市辖区），朝阳（2个市辖区），辽阳（5个市辖区），抚顺（除顺城外的3个市辖区），铁岭（2个市辖区），盘锦（2个市辖区），盘山，朝阳县，辽阳县，岫岩，铁岭县，凌源，北票，建平，开原，抚顺县，灯塔，台安，大洼，辽中

4 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

本溪（4个市辖区），阜新（5个市辖区），锦州（3个市辖区），葫芦岛（3个市辖区），昌图，西丰，法库，彰武，铁法，阜新县，康平，新民，黑山，北宁，义县，喀喇沁，凌海，兴城，绥中，建昌，宽甸，凤城，庄河，长海，顺城

注：全省县级及县级以上设防城镇的设计地震分组，除兴城、绥中、建昌、南票为第二组外，均为第一组。

A.0.6 吉林省

1 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

前郭尔罗斯，松原

2 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

大安*

3 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

长春（6个市辖区），吉林（除丰满外的3个市辖区），白城，乾安，舒兰，九台，永吉*

4 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

四平（2个市辖区），辽源（2个市辖区），镇赉，洮南，延吉，汪清，图们，珲春，龙井，和龙，安图，蛟河，桦甸，梨树，磐石，东丰，辉南，梅河口，东辽，榆树，靖宇，抚松，长岭，通榆*，德惠，农安，伊通，公主岭，扶余，丰满区

注：全省县级及县级以上设防城镇，设计地震分组均为第一组。

A.0.7 黑龙江省

1 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

绥化，萝北，泰来

2 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

哈尔滨（7个市辖区），齐齐哈尔（7个市辖区），大庆（5个市辖区），鹤岗（6个市辖区），牡丹江（4个市辖区），鸡西（6个市辖区），佳木斯（5个市辖区），七台河（3个市辖区），伊春（伊春区，乌马河区），鸡东，望奎，穆棱，绥芬河，东宁，宁安，五大连

池，嘉荫，汤原，桦南，桦川，依兰，勃利，通河，方正，木兰，巴彦，延寿，尚志，宾县，安达，明水，绥棱，庆安，兰西，肇东，肇州，肇源，呼兰，阿城，双城，五常，讷河，北安，甘南，富裕，龙江，黑河，青冈*，海林*

注：全省县级及县级以上设防城镇，设计地震分组均为第一组。

A.0.8 江苏省

1 抗震设防烈度为8度，设计基本地震加速度值为0.30g：

第一组：宿迁，宿豫*

2 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：新沂，邳州，睢宁

3 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：扬州（3个市辖区），镇江（2个市辖区），东海，沭阳，泗洪，江都，大丰

4 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：南京(11个市辖区)，淮安(除楚州外的3个市辖区)，徐州(5个市辖区)，铜山，沛县，常州(4个市辖区)，泰州(2个市辖区)，赣榆，泗阳，盱眙，射阳，江浦，武进，盐城，盐都，东台，海安，姜堰，如皋，如东，扬中，仪征，兴化，高邮，六合，句容，丹阳，金坛，丹徒，溧阳，溧水，昆山，太仓

第三组：连云港（4个市辖区），灌云

5 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：南通（2个市辖区），无锡（6个市辖区），苏州（6个市辖区），通州，宜兴，江阴，洪泽，建湖，常熟，吴江，靖江，泰兴，张家港，海门，启东，高淳，丰县

第二组：响水，滨海，阜宁，宝应，金湖

第三组：灌南，涟水，楚州

A.0.9 浙江省

1 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

岱山，嵊泗，舟山（2个市辖区），镇海区，北仑区

2 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

杭州（6个市辖区），宁波（3个市辖区），湖州，嘉兴（2个市辖区），温州（3个市辖区），绍兴，绍兴县，长兴，安吉，临安，奉化，鄞县，象山，德清，嘉善，平湖，海盐，桐乡，余杭，海宁，萧山，上虞，慈溪，余姚，瑞安，富阳，平阳，苍南，乐清，永嘉，泰顺，景宁，云和，庆元，洞头

注：全省县级及县级以上设防城镇，设计地震分组均为第一组。

A.0.10 安徽省

1 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：五河，泗县

2 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：合肥（4个市辖区），蚌埠（4个市辖区），阜阳（3个市辖区），淮南（5个市辖区），枞阳，怀远，长丰，六安（2个市辖区），灵璧，固镇，凤阳，明光，定远，肥东，肥西，舒城，庐江，桐城，霍山，涡阳，安庆（3个市辖区）*，铜陵县*

3 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：铜陵（3个市辖区），芜湖（4个市辖区），巢湖，马鞍山（4个市辖区），滁州（2个市辖区），芜湖县，砀山，萧县，亳州，界首，太和，临泉，阜南，利辛，蒙城，凤台，寿县，颍上，霍丘，金寨，天长，来安，全椒，含山，和县，当涂，无为，繁昌，池州，岳西，潜山，太湖，怀宁，望江，东至，宿松，南陵，宣城，郎溪，广德，泾县，青阳，石台

第二组：濉溪，淮北

第三组：宿州

A.0.11 福建省

1 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：金门*

2 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：厦门（7个市辖区），漳州（2个市辖区），晋江，石狮，龙海，长泰，漳浦，东山，诏安

第二组：泉州（4个市辖区）

3 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：福州（除马尾外的4个市辖区），安溪，南靖，华安，平和，云霄

第二组：莆田（2个市辖区），长乐，福清，莆田县，平潭，惠安，南安，马尾

4 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：三明(2个市辖区)，政和，屏南，霞浦，福鼎，福安，柘荣，寿宁，周宁，松溪，宁德，古田，罗源，沙县，尤溪，闽清，闽侯，南平，大田，漳平，龙岩，永定，泰宁，宁化，长汀，武平，建宁，将乐，明溪，清流，连城，上杭，永安，建瓯

第二组：连江，永泰，德化，永春，仙游，马祖

A.0.12 江西省

1 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

寻乌，会昌

2 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

南昌（5个市辖区），九江（2个市辖区），南昌县，进贤，余干，九江县，彭泽，湖口，星子，瑞昌，德安，都昌，武宁，修水，靖安，铜鼓，宜丰，宁都，石城，瑞金，安远，定南，龙南，全南，大余

注：全省县级及县级以上设防城镇，设计地震分组均为第一组。

A.0.13 山东省

1 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：郯城，临沭，莒南，莒县，沂水，安丘，阳谷

2 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：临沂（3个市辖区），潍坊（4个市辖区），菏泽，东明，聊城，苍山，沂南，昌邑，昌乐，青州，临朐，诸城，五莲，长岛，蓬莱，龙口，莘县，鄄城，寿光*，台儿庄，东营（河口区）

3 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：烟台（4个市辖区），威海，枣庄（除台儿庄外的4个市辖区），淄博（除博

山外的4个市辖区），平原，高唐，茌平，东阿，平阴，梁山，郓城，定陶，巨野，成武，曹县，广饶，博兴，高青，桓台，文登，沂源，蒙阴，费县，微山，禹城，冠县，莱芜（2个市辖区）*，单县*，夏津*

第二组：东营（东营区），招远，新泰，栖霞，莱州，日照，平度，高密，垦利，博山，滨州*，平邑*

4 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：德州，宁阳，陵县，曲阜，邹城，鱼台，乳山，荣成，兖州

第二组：济南（5个市辖区），青岛（7个市辖区），泰安（2个市辖区），济宁（2个市辖区），武城，乐陵，庆云，无棣，阳信，宁津，沾化，利津，惠民，商河，临邑，济阳，齐河，邹平，章丘，泗水，莱阳，海阳，金乡，滕州，莱西，即墨

第三组：胶南，胶州，东平，汶上，嘉祥，临清，长清，肥城

A.0.14 河南省

1 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：新乡（4个市辖区），新乡县，安阳（4个市辖区），安阳县，鹤壁（3个市辖区），原阳，延津，汤阴，淇县，卫辉，获嘉，范县，辉县

2 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：郑州（除上街外的5个市辖区），濮阳，濮阳县，长桓，封丘，修武，武陟，内黄，浚县，滑县，台前，南乐，清丰，灵宝，三门峡，陕县，林州*

3 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：洛阳（6个市辖区），焦作（4个市辖区），开封（5个市辖区），南阳（2个市辖区），开封县，许昌县，沁阳，博爱，孟州，孟津，巩义，偃师，济源，新密，新郑，民权，兰考，长葛，温县，荥阳，中牟，杞县*，许昌*，上街

4 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：商丘（2个市辖区），信阳（2个市辖区），漯河，平顶山（4个市辖区），登封，义马，虞城，夏邑，通许，尉氏，叶县，宁陵，柘城，新安，宜阳，嵩县，汝阳，伊川，禹州，郏县，宝丰，襄城，郾城，鄢陵，扶沟，太康，鹿邑，鄲城，沈丘，项城，淮阳，周口，商水，上蔡，临颍，西华，西平，栾川，内乡，镇平，唐河，邓州，新野，社旗，平舆，新县，驻马店，泌阳，汝南，桐柏，淮滨，息县，正阳，遂平，光山，罗山，潢川，商城，固始，南召，舞阳*

第二组：汝州，睢县，永城

第三组：卢氏，洛宁，渑池

A.0.15 湖北省

1 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

竹溪，竹山，房县

2 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

武汉（13个市辖区），荆州（2个市辖区），荆门，襄樊（2个市辖区），襄阳，十堰（2个市辖区），宜昌（4个市辖区），宜昌县，黄石（4个市辖区），恩施，咸宁，麻城，团风，罗田，英山，黄冈，鄂州，浠水，蕲春，黄梅，武穴，郧西，郧县，丹江口，谷城，老河口，宜城，南漳，保康，神农架，钟祥，沙洋，远安，兴山，巴东，秭归，当阳，建始，利川，公安，宣恩，咸丰，长阳，宜都，枝

江，松滋，江陵，石首，监利，洪湖，孝感，应城，云梦，天门，仙桃，红安，安陆，潜江，嘉鱼，大冶，通山，赤壁，崇阳，通城，五峰*，京山*

注：全省县级及县级以上设防城镇，设计地震分组均为第一组。

A.0.16 湖南省

1 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

常德（2个市辖区）

2 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

岳阳（2个市辖区），岳阳县，汨罗，湘阴，临澧，澧县，津市，桃源，安乡，汉寿

3 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

长沙（5个市辖区），长沙县，益阳（2个市辖区），张家界（2个市辖区），郴州（2个市辖区），邵阳（3个市辖区），邵阳县，泸溪，沅陵，娄底，宜章，资兴，平江，宁乡，新化，冷水江，涟源，双峰，新邵，邵东，隆回，石门，慈利，华容，南县，临湘，沅江，桃江，望城，溆浦，会同，靖州，韶山，江华，宁远，道县，临武，湘乡*，安化*，中方*，洪江*，云溪

注：全省县级及县级以上设防城镇，设计地震分组均为第一组。

A.0.17 广东省

1 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

汕头（5个市辖区），澄海，潮安，南澳，徐闻，潮州*

2 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

揭阳，揭东，潮阳，饶平

3 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

广州（除花都外的9个市辖区），深圳（6个市辖区），湛江（4个市辖区），汕尾，海丰，普宁，惠来，阳江，阳东，阳西，茂名，化州，廉江，遂溪，吴川，丰顺，南海，顺德，中山，珠海，斗门，电白，雷州，佛山（2个市辖区）*，江门（2个市辖区）*，新会*，陆丰*

4 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

韶关（3个市辖区），肇庆（2个市辖区），花都，河源，揭西，东源，梅州，东莞，清远，清新，南雄，仁化，始兴，乳源，曲江，英德，佛冈，龙门，龙川，平远，大埔，从化，梅县，兴宁，五华，紫金，陆河，增城，博罗，惠州，惠阳，惠东，三水，四会，云浮，云安，高要，高明，鹤山，封开，郁南，罗定，信宜，新兴，开平，恩平，台山，阳春，高州，翁源，连平，和平，蕉岭，新丰*

注：全省县级及县级以上设防城镇，设计地震分组均为第一组。

A.0.18 广西自治区

1 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

灵山，田东

2 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

玉林，兴业，横县，北流，百色，田阳，平果，隆安，浦北，博白，乐业*

3 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

南宁（6个市辖区），桂林（5个市辖区），柳州（5个市辖区），梧州（3个市辖区），钦州（2

个市辖区),贵港(2 个市辖区),防城港(2 个市辖区),北海(2 个市辖区),兴安,灵川,临桂,永福,鹿寨,天峨,东兰,巴马,都安,大化,马山,融安,象州,武宣,桂平,平南,上林,宾阳,武鸣,大新,扶绥,邕宁,东兴,合浦,钟山,贺州,藤县,苍梧,容县,岑溪,陆川,凤山,凌云,田林,隆林,西林,德保,靖西,那坡,天等,崇左,上思,龙州,宁明,融水,凭祥,全州

注:全自治区县级及县级以上设防城镇,设计地震分组均为第一组。

A.0.19 海南省

1 抗震设防烈度为 8 度,设计基本地震加速度值为 0.30g:

海口(3 个市辖区),琼山

2 抗震设防烈度为 8 度,设计基本地震加速度值为 0.20g:

文昌,定安

3 抗震设防烈度为 7 度,设计基本地震加速度值为 0.15g:

澄迈

4 抗震设防烈度为 7 度,设计基本地震加速度值为 0.10g:

临高,琼海,儋州,屯昌

5 抗震设防烈度为 6 度,设计基本地震加速度值为 0.05g:

三亚,万宁,琼中,昌江,白沙,保亭,陵水,东方,乐东,通什

注:全省县级及县级以上设防城镇,设计地震分组均为第一组。

A.0.20 四川省

1 抗震设防烈度不低于 9 度,设计基本地震加速度值不小于 0.40g:

第一组:康定,西昌

2 抗震设防烈度为 8 度,设计基本地震加速度值为 0.30g:

第一组:冕宁*

3 抗震设防烈度为 8 度,设计基本地震加速度值为 0.20g:

第一组:松潘,道孚,泸定,甘孜,炉霍,石棉,喜德,普格,宁南,德昌,理塘

第二组:九寨沟

4 抗震设防烈度为 7 度,设计基本地震加速度值为 0.15g:

第一组:宝兴,茂县,巴塘,德格,马边,雷波

第二组:越西,雅江,九龙,平武,木里,盐源,会东,新龙

第三组:天全,荥经,汉源,昭觉,布拖,丹巴,芦山,甘洛

5 抗震设防烈度为 7 度,设计基本地震加速度值为 0.10g:

第一组:成都(除龙泉驿、清白江的 5 个市辖区),乐山(除金口河外的 3 个市辖区),自贡(4 个市辖区),宜宾,宜宾县,北川,安县,绵竹,汶川,都江堰,双流,新津,青神,峨边,沐川,屏山,理县,得荣,新都*

第二组:攀枝花(3 个市辖区),江油,什邡,彭州,郫县,温江,大邑,崇州,邛崃,蒲江,彭山,丹棱,眉山,洪雅,夹江,峨嵋山,若尔盖,色达,壤塘,马尔康,石渠,白玉,金川,黑水,盐边,米易,乡城,稻城,金口河,朝天区*

第三组:青川,雅安,名山,美姑,金阳,小金,会理

6 抗震设防烈度为 6 度,设计基本地震加速度值为 0.05g:

第一组:泸州(3 个市辖区),内江(2 个市辖区),德阳,宣汉,达州,达县,大竹,邻水,渠

县，广安，华蓥，隆昌，富顺，泸县，南溪，江安，长宁，高县，珙县，兴文，叙永，古蔺，金堂，广汉，简阳，资阳，仁寿，资中，犍为，荣县，威远，南江，通江，万源，巴中，苍溪，阆中，仪陇，西充，南部，盐亭，三台，射洪，大英，乐至，旺苍，龙泉驿，清白江

第二组：绵阳（2个市辖区），梓潼，中江，阿坝，筠连，井研

第三组：广元（除朝天区外的2个市辖区），剑阁，罗江，红原

A.0.21 贵州省

1 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：望谟

第二组：威宁

2 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：贵阳（除白云外的5个市辖区），凯里，毕节，安顺，都匀，六盘水，黄平，福泉，贵定，麻江，清镇，龙里，平坝，纳雍，织金，水城，普定，六枝，镇宁，惠水，长顺，关岭，紫云，罗甸，兴仁，贞丰，安龙，册亨，金沙，印江，赤水，习水，思南*

第二组：赫章，普安，晴隆，兴义

第三组：盘县

A.0.22 云南省

1 抗震设防烈度不低于9度，设计基本地震加速度值不小于0.40g：

第一组：寻甸，东川

第二组：澜沧

2 抗震设防烈度为8度，设计基本地震加速度值为0.30g：

第一组：剑川，嵩明，宜良，丽江，鹤庆，永胜，潞西，龙陵，石屏，建水

第二组：耿马，双江，沧源，勐海，西盟，孟连

3 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：石林，玉溪，大理，巧家，江川，华宁，峨山，通海，洱源，宾川，弥渡，祥云，会泽，南涧

第二组：昆明（除东川外的4个市辖区），思茅，保山，马龙，呈贡，澄江，晋宁，易门，漾濞，巍山，云县，腾冲，施甸，瑞丽，梁河，安宁，凤庆*，陇川*

第三组：景洪，永德，镇康，临沧

4 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：中甸，泸水，大关，新平*，永善

第二组：沾益，个旧，红河，元江，禄丰，双柏，开远，盈江，永平，昌宁，宁蒗，南华，楚雄，勐腊，华坪，景东*

第三组：曲靖，弥勒，陆良，富民，禄劝，武定，兰坪，云龙，景谷，普洱

5 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：盐津，绥江，德钦，水富，贡山

第二组：昭通，彝良，鲁甸，福贡，永仁，大姚，元谋，姚安，牟定，墨江，绿春，镇沅，江城，金平

第三组：富源，师宗，泸西，蒙自，元阳，维西，宣威

6 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：威信，镇雄，广南，富宁，西畴，麻栗坡，马关

第二组：丘北，砚山，屏边，河口，文山

第三组：罗平

A.0.23 西藏自治区

1 抗震设防烈度不低于9度，设计基本地震加速度值不小于0.40g：

第二组：当雄，墨脱

2 抗震设防烈度为8度，设计基本地震加速度值为0.30g：

第一组：申扎

第二组：米林，波密

3 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：普兰，聂拉木，萨嘎

第二组：拉萨，堆龙德庆，尼木，仁布，尼玛，洛隆，隆子，错那，曲松

第三组：那曲，林芝（八一镇），林周

4 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：札达，吉隆，拉孜，谢通门，亚东，洛扎，昂仁

第二组：日土，江孜，康马，白朗，扎囊，措美，桑日，加查，边坝，八宿，丁青，类乌齐，乃东，琼结，贡嘎，朗县，达孜，日喀则*，噶尔*

第三组:南木林,班戈,浪卡子,墨竹工卡,曲水,安多,聂荣

5 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：改则，措勤，仲巴，定结，芒康

第二组：昌都，定日，萨迦，岗巴，巴青，工布江达，索县，比如，嘉黎，察雅，左贡，察隅，江达，贡觉

6 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：革吉

A.0.24 陕西省

1 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组:西安(8个市辖区),渭南,华县,华阴,潼关,大荔

第二组：陇县

2 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组:咸阳(2个市辖区,杨凌特区),宝鸡(2个市辖区),高陵,千阳,岐山,凤翔,扶风,武功,兴平,周至,眉县,宝鸡县,三原,富平,澄城,蒲城,泾阳,礼泉,长安,户县,蓝田,韩城,合阳

第二组:凤县

3 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：安康，平利，乾县，洛南

第二组：白水，耀县，淳化，麟游，永寿，商州，铜川（2个市辖区）*，柞水*

第三组：太白，留坝，勉县，略阳

4 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：延安，清涧，神木，佳县，米脂，绥德，安塞，延川，延长，定边，吴旗，

志丹，甘泉，富县，商南，旬阳，紫阳，镇巴，白河，岚皋，镇坪，子长*

第二组：府谷，吴堡，洛川，黄陵，旬邑，洋县，西乡，石泉，汉阴，宁陕，汉中，南郑，城固

第三组：宁强，宜川，黄龙，宜君，长武，彬县，佛坪，镇安，丹凤，山阳

A.0.25 甘肃省

1 抗震设防烈度不低于9度，设计基本地震加速度值不小于0.40g：

第一组：古浪

2 抗震设防烈度为8度，设计基本地震加速度值为0.30g：

第一组：天水（2个市辖区），礼县，西和，平川区

3 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：宕昌，文县，肃北，武都

第二组：兰州(4个市辖区)，成县，舟曲，徽县，康县，武威，永登，天祝，景泰，靖远，陇西，武山，秦安，清水，甘谷，漳县，会宁，静宁，庄浪，张家川，通渭，华亭

4 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：康乐，嘉峪关，玉门，酒泉，高台，临泽，肃南

第二组：白银（白银区），永靖，岷县，东乡，和政，广河，临潭，卓尼，迭部，临洮，渭源，皋兰，崇信，榆中，定西，金昌，两当，阿克塞，民乐，永昌，红古区

第三组：平凉

5 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：张掖，合作，玛曲，金塔，积石山

第二组：敦煌，安西，山丹，临夏，临夏县，夏河，碌曲，泾川，灵台

第三组：民勤，镇原，环县

6 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第二组：华池，正宁，庆阳，合水，宁县

第三组：西峰

A.0.26 青海省

1 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：玛沁

第二组：玛多，达日

2 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：祁连，玉树

第二组：甘德，门源

3 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：乌兰，治多，称多，杂多，囊谦

第二组：西宁（4个市辖区），同仁，共和，德令哈，海晏，湟源，湟中，平安，民和，化隆，贵德，尖扎，循化，格尔木，贵南，同德，河南，曲麻莱，久治，班玛，天峻，刚察

第三组：大通，互助，乐都，都兰，兴海

4 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第二组：泽库

A.0.27 宁夏自治区

1 抗震设防烈度为8度，设计基本地震加速度值为0.30g：

第一组：海原

2 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：银川（3个市辖区），石嘴山（3个市辖区），吴忠，惠农，平罗，贺兰，永宁，青铜峡，泾源，灵武，陶乐，固原

第二组：西吉，中卫，中宁，同心，隆德

3 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第三组：彭阳

4 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第三组：盐池

A.0.28 新疆自治区

1 抗震设防烈度不低于9度，设计基本地震加速度值不小于0.40g：

第二组：乌恰，塔什库尔干

2 抗震设防烈度为8度，设计基本地震加速度值为0.30g：

第二组：阿图什，喀什，疏附

3 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第一组：乌鲁木齐（7个市辖区），乌鲁木齐县，温宿，阿克苏，柯坪，米泉，昭苏，特克斯，库车，巴里坤，青河，富蕴，乌什*

第二组：尼勒克，新源，巩留，精河，乌苏，奎屯，沙湾，玛纳斯，石河子，独山子

第三组：疏勒，伽师，阿克陶，英吉沙

4 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：库尔勒，新和，轮台，和静，焉耆，博湖，巴楚，昌吉，拜城，阜康*，木垒*

第二组：伊宁，伊宁县，霍城，察布查尔，呼图壁

第三组：岳普湖

5 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：吐鲁番，和田，和田县，吉木萨尔，洛浦，奇台，伊吾，鄯善，托克逊，和硕，尉犁，墨玉，策勒，哈密

第二组：克拉玛依（克拉玛依区），博乐，温泉，阿合奇，阿瓦提，沙雅

第三组：莎车，泽普，叶城，麦盖堤，皮山

6 抗震设防烈度为6度，设计基本地震加速度值为0.05g：

第一组：于田，哈巴河，塔城，额敏，福海，和布克赛尔，乌尔禾

第二组：阿勒泰，托里，民丰，若羌，布尔津，吉木乃，裕民，白碱滩

第三组：且末

A.0.29 港澳特区和台湾省

1 抗震设防烈度不低于9度，设计基本地震加速度值不小于0.40g：

第一组：台中

第二组：苗栗，云林，嘉义，花莲

2 抗震设防烈度为8度，设计基本地震加速度值为0.30g：

第二组：台北，桃园，台南，基隆，宜兰，台东，屏东

3 抗震设防烈度为8度，设计基本地震加速度值为0.20g：

第二组：高雄，澎湖

4 抗震设防烈度为7度，设计基本地震加速度值为0.15g：

第一组：香港

5 抗震设防烈度为7度，设计基本地震加速度值为0.10g：

第一组：澳门

附录 B　高强混凝土结构抗震设计要求

B.0.1　高强混凝土结构所采用的混凝土强度等级应符合本规范第 3.9.3 条的规定；其抗震设计，除应符合普通混凝土结构抗震设计要求外，尚应符合本附录的规定。

B.0.2　结构构件截面剪力设计值的限值中含有混凝土轴心抗压强度设计值（f_c）的项应乘以混凝土强度影响系数（β_c）。其值，混凝土强度等级为 C50 时取 1.0，C80 时取 0.8，介于 C50 和 C80 之间时取其内插值。

结构构件受压区高度计算和承载力验算时，公式中含有混凝土轴心抗压强度设计值（f_c）的项也应按国家标准《混凝土结构设计规范》GB50010 的有关规定乘以相应的混凝土强度影响系数。

B.0.3　高强混凝土框架的抗震构造措施，应符合下列要求：

1　梁端纵向受拉钢筋的配筋率不宜大于 3%（HRB335 级钢筋）和 2.6%（HRB400 级钢筋）。梁端箍筋加密区的箍筋最小直径应比普通混凝土梁箍筋的最小直径增大 2mm。

2　柱的轴压比限值宜按下列规定采用：不超过 C60 混凝土的柱可与普通混凝土柱相同，C65～C70 混凝土的柱宜比普通混凝土柱减小 0.05，C75～C80 混凝土的柱宜比普通混凝土柱减小 0.1。

3　当混凝土强度等级大于 C60 时，柱纵向钢筋的最小总配筋率应比普通混凝土柱增大 0.1%。

4　柱加密区的最小配箍特征值宜按下列规定采用：混凝土强度等级高于 C60 时，箍筋宜采用复合箍、复合螺旋箍或连续复合矩形螺旋箍。

1）轴压比不大于 0.6 时，宜比普通混凝土柱大 0.02；

2）轴压比大于 0.6 时，宜比普通混凝土柱大 0.03。

B.0.4　当混凝土强度等级大于 C60 时，抗震墙约束边缘构件的配箍特征值宜比轴压比相同的普通混凝土抗震墙增加 0.02。

附录C　预应力混凝土结构抗震设计要求

C.1　一 般 要 求

C.1.1　本附录适用于6、7、8度时先张法和后张有粘结预应力混凝土结构的抗震设计，9度时应进行专门研究。

无粘结预应力混凝土结构的抗震设计，应符合专门的规定。

C.1.2　抗震设计时，框架的后张预应力构件宜采用有粘结预应力筋。

C.1.3　后张预应力筋的锚具不宜设置在梁柱节点核芯区。

C.2　预应力框架结构

C.2.1　预应力混凝土框架梁应符合下列规定：

1　后张预应力混凝土框架梁中应采用预应力筋和非预应力筋混合配筋方式，按下式计算的预应力强度比，一级不宜大于0.60；二、三级不宜大于0.75。

$$\lambda = \frac{A_p f_{py}}{A_p f_{py} + A_s f_y} \tag{C.2.1}$$

式中　λ——预应力强度比；

A_p、A_s——分别为受拉区预应力筋、非预应力筋截面面积；

f_{py}——预应力筋的抗拉强度设计值；

f_y——非预应力筋的抗拉强度设计值。

2　预应力混凝土框架梁端纵向受拉钢筋按非预应力钢筋抗拉强度设计值换算的配筋率不应大于2.5%，且考虑受压钢筋的梁端混凝土受压区高度和有效高度之比，一级不应大于0.25，二、三级不应大于0.35。

3　梁端截面的底面和顶面非预应力钢筋配筋量的比值，除按计算确定外，一级不应小于1.0，二、三级不应小于0.8，同时，底面非预应力钢筋配筋量不应低于毛截面面积的0.2%。

C.2.2　预应力混凝土悬臂梁应符合下列规定：

1　悬臂梁的预应力强度比可按本附录第C.2.1条1款的规定采用；考虑受压钢筋的混凝土受压区高度和有效高度之比可按本附录第C.2.1条2款的规定采用。

2　悬臂梁梁底和梁顶非预应力筋配筋量的比值，除按计算确定外，不应小于1.0，且底面非预应力筋配筋量不应低于毛截面面积的0.2%。

C.2.3　预应力混凝土框架柱应符合下列规定：

1　预应力混凝土大跨度框架顶层边柱宜采用非对称配筋，一侧采用混合配筋，另一侧仅配置普通钢筋。

2　预应力框架柱应符合本规范第6.2节调整框架柱内力组合设计值的相应要求。

3　预应力混凝土框架柱的截面受压区高度和有效高度之比，一级不宜大于0.25，二、三级不宜大于0.35。

4　预应力框架柱箍筋应沿柱全高加密。

附录D　框架梁柱节点核芯区截面抗震验算

D.1　一般框架梁柱节点

D.1.1　一、二级框架梁柱节点核芯区组合的剪力设计值，应按下列公式确定：

$$V_j = \frac{\eta_{jb}\Sigma M_b}{h_{b0} - a'_s}\left(1 - \frac{h_{b0} - a'_s}{H_c - h_b}\right) \tag{D.1.1-1}$$

9度时和一级框架结构尚应符合

$$V_j = \frac{1.15\Sigma M_{bua}}{h_{b0} - a'_s}\left(1 - \frac{h_{b0} - a'_s}{H_c - h_b}\right) \tag{D.1.1-2}$$

式中　V_j——梁柱节点核芯区组合的剪力设计值；

h_{b0}——梁截面的有效高度，节点两侧梁截面高度不等时可采用平均值；

a'_s——梁受压钢筋合力点至受压边缘的距离；

H_c——柱的计算高度，可采用节点上、下柱反弯点之间的距离；

h_b——梁的截面高度，节点两侧梁截面高度不等时可采用平均值；

η_{jb}——节点剪力增大系数，一级取1.35，二级取1.2；

ΣM_b——节点左右梁端反时针或顺时针方向组合弯矩设计值之和，一级时节点左右梁端均为负弯矩，绝对值较小的弯矩应取零；

ΣM_{bua}——节点左右梁端反时针或顺时针方向实配的正截面抗震受弯承载力所对应的弯矩值之和，根据实配钢筋面积（计入受压筋）和材料强度标准值确定。

D.1.2　核芯区截面有效验算宽度，应按下列规定采用：

1　核芯区截面有效验算宽度，当验算方向的梁截面宽度不小于该侧柱截面宽度的1/2时，可采用该侧柱截面宽度，当小于柱截面宽度的1/2时，可采用下列二者的较小值：

$$b_j = b_b + 0.5h_c \tag{D.1.2-1}$$

$$b_j = b_c \tag{D.1.2-2}$$

式中　b_j——节点核芯区的截面有效验算宽度；

b_b——梁截面宽度；

h_c——验算方向的柱截面高度；

b_c——验算方向的柱截面宽度。

2　当梁、柱的中线不重合且偏心距不大于柱宽的1/4时，核芯区的截面有效验算宽度可采用上款和下式计算结果的较小值。

$$b_j = 0.5(b_b + b_c) + 0.25h_c - e \tag{D.1.2-3}$$

式中　e——梁与柱中线偏心距。

D.1.3　节点核芯区组合的剪力设计值，应符合下列要求：

$$V_j \leqslant \frac{1}{\gamma_{RE}}(0.30\eta_j f_c b_j h_j) \tag{D.1.3}$$

式中 η_j——正交梁的约束影响系数，楼板为现浇，梁柱中线重合，四侧各梁截面宽度不小于该侧柱截面宽度的1/2，且正交方向梁高度不小于框架梁高度的3/4时，可采用1.5，9度时宜采用1.25，其他情况均采用1.0；

h_j——节点核芯区的截面高度，可采用验算方向的柱截面高度；

γ_{RE}——承载力抗震调整系数，可采用0.85。

D.1.4 节点核芯区截面抗震受剪承载力，应采用下列公式验算：

$$V_j \leqslant \frac{1}{\gamma_{RE}}\left(1.1\eta_j f_t b_j h_j + 0.05\eta_j N\frac{b_j}{b_c} + f_{yv}A_{svj}\frac{h_{b0}-a'_s}{s}\right) \qquad \text{(D.1.4-1)}$$

9度时

$$V_j \leqslant \frac{1}{\gamma_{RE}}\left(0.9\eta_j f_t b_j h_j + f_{yv}A_{svj}\frac{h_{b0}-a'_s}{s}\right) \qquad \text{(D.1.4-2)}$$

式中 N——对应于组合剪力设计值的上柱组合轴向压力较小值，其取值不应大于柱的截面面积和混凝土轴心抗压强度设计值的乘积的50%，当N为拉力时，取$N=0$；

f_{yv}——箍筋的抗拉强度设计值；

f_t——混凝土轴心抗拉强度设计值；

A_{svj}——核芯区有效验算宽度范围内同一截面验算方向箍筋的总截面面积；

s——箍筋间距。

D.2 扁梁框架的梁柱节点

D.2.1 扁梁框架的梁宽大于柱宽时，梁柱节点应符合本段的规定。

D.2.2 扁梁框架的梁柱节点核芯区应根据梁纵筋在柱宽范围内、外的截面面积比例，对柱宽以内和柱宽以外的范围分别验算受剪承载力。

D.2.3 核芯区验算方法除应符合一般框架梁柱节点的要求外，尚应符合下列要求：

1 按本附录式（D.1.3）验算核芯区剪力限值时，核芯区有效宽度可取梁宽与柱宽之和的平均值；

2 四边有梁的约束影响系数，验算柱宽范围内核芯区的受剪承载力时可取1.5，验算柱宽范围外核芯区的受剪承载力时宜取1.0；

3 验算核芯区受剪承载力时，在柱宽范围内的核芯区，轴向力的取值可与一般梁柱节点相同；柱宽以外的核芯区，可不考虑轴力对受剪承载力的有利作用；

4 锚入柱内的梁上部钢筋宜大于其全部截面面积的60%。

D.3 圆柱框架的梁柱节点

D.3.1 梁中线与柱中线重合时，圆柱框架梁柱节点核芯区组合的剪力设计值应符合下列要求：

$$V_j \leqslant \frac{1}{\gamma_{RE}}(0.30\eta_j f_c A_j) \qquad \text{(D.3.1)}$$

式中 η_j——正交梁的约束影响系数，按本附录D.1.3确定，其中柱截面宽度按柱直径采用；

A_j——节点核芯区有效截面面积，梁宽(b_b)不小于柱直径(D)之半时，取 $A_j = 0.8D^2$；

梁宽(b_b)小于柱直径(D)之半且不小于 $0.4D$ 时，取 $A_j = 0.8D(b_b + D/2)$。

D.3.2 梁中线与柱中线重合时，圆柱框架梁柱节点核芯区截面抗震受剪承载力应采用下列公式验算：

$$V_j \leqslant \frac{1}{\gamma_{RE}}\left(1.5\eta_j f_t A_j + 0.05\eta_j \frac{N}{D^2}A_j + 1.57 f_{yv} A_{sh} \frac{h_{b0} - a'_s}{s} + f_{yv} A_{svj} \frac{h_{b0} - a'_s}{s}\right) \tag{D.3.2-1}$$

9 度时

$$V_j \leqslant \frac{1}{\gamma_{RE}}\left(1.2\eta_j f_t A_j + 1.57 f_{yv} A_{sh} \frac{h_{b0} - a'_s}{s} + f_{yv} A_{svj} \frac{h_{b0} - a'_s}{s}\right) \tag{D.3.2-2}$$

式中 A_{sh}——单根圆形箍筋的截面面积；

A_{svj}——同一截面验算方向的拉筋和非圆形箍筋的总截面面积；

D——圆柱截面直径；

N——轴向力设计值，按一般梁柱节点的规定取值。

附录E　转换层结构抗震设计要求

E.1　矩形平面抗震墙结构框支层楼板设计要求

E.1.1　框支层应采用现浇楼板，厚度不宜小于180mm，混凝土强度等级不宜低于C30，应采用双层双向配筋，且每层每个方向的配筋率不应小于0.25%。

E.1.2　部分框支抗震墙结构的框支层楼板剪力设计值，应符合下列要求：

$$V_f \leqslant \frac{1}{\gamma_{RE}}(0.1 f_c b_f t_f) \tag{E.1.2}$$

式中　V_f——由不落地抗震墙传到落地抗震墙处按刚性楼板计算的框支层楼板组合的剪力设计值，8度时应乘以增大系数2，7度时应乘以增大系数1.5；验算落地抗震墙时不考虑此项增大系数；

b_f、t_f——分别为框支层楼板的宽度和厚度；

γ_{RE}——承载力抗震调整系数，可采用0.85。

E.1.3　部分框支抗震墙结构的框支层楼板与落地抗震墙交接截面的受剪承载力，应按下列公式验算：

$$V_f \leqslant \frac{1}{\gamma_{RE}}(f_y A_s) \tag{E.1.3}$$

式中　A_s——穿过落地抗震墙的框支层楼盖（包括梁和板）的全部钢筋的截面面积。

E.1.4　框支层楼板的边缘和较大洞口周边应设置边梁，其宽度不宜小于板厚的2倍，纵向钢筋配筋率不应小于1%，钢筋接头宜采用机械连接或焊接，楼板的钢筋应锚固在边梁内。

E.1.5　对建筑平面较长或不规则及各抗震墙内力相差较大的框支层，必要时可采用简化方法验算楼板平面内的受弯、受剪承载力。

E.2　筒体结构转换层抗震设计要求

E.2.1　转换层上下的结构质量中心宜接近重合（不包括裙房），转换层上下层的侧向刚度比不宜大于2。

E.2.2　转换层上部的竖向抗侧力构件（墙、柱）宜直接落在转换层的主结构上。

E.2.3　厚板转换层结构不宜用于7度及7度以上的高层建筑。

E.2.4　转换层楼盖不应有大洞口，在平面内宜接近刚性。

E.2.5　转换层楼盖与筒体、抗震墙应有可靠的连接，转换层楼板的抗震验算和构造宜符合本附录E.1对框支层楼板的有关规定。

E.2.6　8度时转换层结构应考虑竖向地震作用。

E.2.7　9度时不应采用转换层结构。

附录 F　配筋混凝土小型空心砌块抗震墙房屋抗震设计要求

F.1　一 般 要 求

F.1.1　本附录适用的配筋混凝土小型空心砌块抗震墙房屋的最大高度应符合表 F.1.1-1 规定，且房屋总高度与总宽度的比值不宜超过表 F.1.1-2 的规定；对横墙较少或建造于Ⅳ类场地的房屋，适用的最大高度应适当降低。

表 F.1.1-1　配筋混凝土小型空心砌块抗震墙房屋适用的最大高度（m）

最小墙厚（mm）	6 度	7 度	8 度
190	54	45	30

注：房屋高度超过表内高度时，应根据专门研究，采取有效的加强措施。

表 F.1.1-2　配筋混凝土小型空心砌块抗震墙房屋的最大高宽比

烈　　度	6 度	7 度	8 度
最大高宽比	5	4	3

F.1.2　配筋小型空心砌块抗震墙房屋应根据抗震设防分类、抗震设防烈度和房屋高度采用不同的抗震等级，并应符合相应的计算和构造措施要求。丙类建筑的抗震等级宜按表 F.1.2 确定：

表 F.1.2　配筋小型空心砌块抗震墙房屋的抗震等级

烈　　度	6 度		7 度		8 度	
高度（m）	≤24	>24	≤24	>24	≤24	>24
抗震等级	四	三	三	二	二	一

注：接近或等于高度分界时，可结合房屋不规则程度及和场地、地基条件确定抗震等级。

F.1.3　房屋应避免采用本规范第 3.4 节规定的不规则建筑结构方案，并应符合下列要求：

1　平面形状宜简单、规则，凹凸不宜过大；竖向布置宜规则、均匀，避免过大的外挑和内收。

2　纵横向抗震墙宜拉通对直；每个墙段不宜太长，每个独立墙段的总高度与墙段长度之比不宜小于 2；门洞口宜上下对齐，成列布置。

3　房屋抗震横墙的最大间距，应符合表 F.1.3 的要求：

表 F.1.3　抗震横墙的最大间距

烈　　度	6 度	7 度	8 度
最大间距（m）	15	15	11

F.1.4　房屋宜选用规则、合理的建筑结构方案不设防震缝，当需要防震缝时，其最小宽度应符合下列要求：

当房屋高度不超过 20m 时，可采用 70mm；当超过 20m 时，6 度、7 度、8 度相应每

增加 6m、5m 和 4m，宜加宽 20mm。

F.2 计 算 要 点

F.2.1 配筋小型空心砌块抗震墙房屋抗震计算时，应按本节规定调整地震作用效应；6 度时可不做抗震验算。

F.2.2 配筋小型空心砌块抗震墙承载力计算时，底部加强部位截面的组合剪力设计值应按下列规定调整：

$$V = \eta_{vw} V_w \tag{F.2.2}$$

式中 V——抗震墙底部加强部位截面组合的剪力设计值；

V_w——抗震墙底部加强部位截面组合的剪力计算值；

η_{vw}——剪力增大系数，一级取 1.6，二级取 1.4，三级取 1.2，四级取 1.0。

F.2.3 配筋小型空心砌块抗震墙截面组合的剪力设计值，应符合下列要求：

剪跨比大于 2

$$V \leqslant \frac{1}{\gamma_{RE}}(0.2 f_{gc} b_w h_w) \tag{F.2.3-1}$$

剪跨比不大于 2

$$V \leqslant \frac{1}{\gamma_{RE}}(0.15 f_{gc} b_w h_w) \tag{F.2.3-2}$$

式中 f_{gc}——灌芯小砌块砌体抗压强度设计值；满灌时可取2倍砌块砌体抗压强度设计值；

b_w——抗震墙截面宽度；

h_w——抗震墙截面高度；

γ_{RE}——承载力抗震调整系数，取 0.85。

注：剪跨比应按本规范式（6.2.9-3）计算。

F.2.4 偏心受压配筋小型空心砌块抗震墙截面受剪承载力，应按下列公式验算：

$$V \leqslant \frac{1}{\gamma_{RE}}\left[\frac{1}{\lambda - 0.5}(0.48 f_{gv} b_w h_w + 0.1N) + 0.72 f_{yh} \frac{A_{sb}}{s} h_{w0}\right] \tag{F.2.4-1}$$

$$0.5V \leqslant \frac{1}{\gamma_{RE}}\left(0.72 f_{yh} \frac{A_{sh}}{s} h_{w0}\right) \tag{F.2.4-2}$$

式中 N——抗震墙轴向压力设计值；取值不大于 $0.2 f_{gc} b_w h_w$；

λ——计算截面处的剪跨比，取 $\lambda = M / V h_w$；当小于 1.5 时取 1.5，当大于 2.2 时取 2.2；

f_{gv}——灌芯小砌块砌体抗剪强度设计值；可取 $f_{gv} = 0.2 f_{gc}^{0.55}$；

A_{sh}——同一截面的横向钢筋截面面积；

s——横向分布筋间距；

f_{yh}——横向分布筋抗拉强度设计值；

h_{w0}——抗震墙截面有效高度；

γ_{RE}——承载力抗震调整系数，取 0.85。

F.2.5 配筋小型空心砌块抗震墙跨高比大于 2.5 的连梁宜采用钢筋混凝土连梁，其截面

组合的剪力设计值和斜截面受剪承载力，应符合现行国家标准《混凝土结构设计规范》GB 50010 对连梁的有关规定。

F.3 抗震构造措施

F.3.1 配筋小型空心砌块抗震墙房屋的灌芯混凝土，应采用塌落度大、流动性和和易性好，并与砌块结合良好的混凝土，灌芯混凝土的强度等级不应低于 C20。

F.3.2 配筋小型空心砌块房屋的墙段底部（高度不小于房屋高度的 1/6 且不小于二层的高度），应按加强部位配置水平和竖向钢筋。

F.3.3 配筋小型空心砌块抗震墙横向和竖向分布钢筋的配置，应符合下列要求：

1 竖向钢筋可采用单排布置，最小直径 12mm；其最大间距 600mm，顶层和底层应适当减小。

2 横向钢筋宜双排布置，最小直径 8mm；其最大间距 600mm，顶层和底层不应大于 400mm。

3 竖向、横向的分布钢筋的最小配筋率，一级均不应小于 0.13%；二级的一般部位不应小于 0.10%，加强部位不宜小于 0.13%；三、四级均不应小于 0.10%。

F.3.4 配筋小型空心砌块抗震墙内竖向和水平分布钢筋的搭接长度不应小于 48 倍钢筋直径，锚固长度不应小于 42 倍钢筋直径。

F.3.5 配筋小型空心砌块抗震墙在重力荷载代表值作用下的轴压比，一级不宜大于 0.5，二、三级不宜大于 0.6。

F.3.6 配筋小型空心砌块抗震墙的压应力大于 0.5 倍灌芯小砌块砌体抗压强度设计值（f_{gc}）时，在墙端应设置长度不小于 3 倍墙厚的边缘构件，其最小配筋应符合表 F.3.6 的要求：

表 F.3.6 配筋小型空心砌块抗震墙边缘构件的配筋要求

抗震等级	加强部位纵向钢筋最小量	一般部位纵向钢筋最小量	箍筋最小直径	箍筋最大间距
一	3φ20	3φ18	φ8	200mm
二	3φ18	3φ16	φ8	200mm
三	3φ16	3φ14	φ8	200mm
四	3φ14	3φ12	φ8	200mm

F.3.7 配筋小型空心砌块抗震墙连梁的抗震构造，应符合下列要求：

1 连梁的纵向钢筋锚入墙内的长度，一、二级不应小于 1.15 倍锚固长度，三级不应小于 1.05 倍锚固长度，四级不应小于锚固长度且不应小于 600mm。

2 连梁的箍筋设置，沿梁全长均应符合框架梁端箍筋加密区的构造要求。

3 顶层连梁的纵向钢筋锚固长度范围内，应设置间距不大于 200mm 的箍筋，直径与该连梁的箍筋直径相同。

4 跨高比不大于 2.5 的连梁，自梁顶面下 200mm 至梁底面上 200mm 的范围内应增设水平分布钢筋；其间距不大于 200mm；每层分布筋的数量，一级不少于 2φ12，二～四级不少于 2φ10；水平分布筋伸入墙内的长度，不应小于 30 倍钢筋直径和 300mm。

5 配筋小型空心砌块抗震墙的连梁内不宜开洞，需要开洞时应符合下列要求：

1）在跨中梁高 1/3 处预埋外径不大于 200mm 的钢套管；

2）洞口上下的有效高度不应小于 1/3 梁高，且不小于 200mm；

3）洞口处应配置补强钢筋，被洞口削弱的截面应进行受剪承载力验算。

F.3.8 楼盖的构造应符合下列要求：

1 配筋小型空心砌块房屋的楼、屋盖宜采用现浇钢筋混凝土板；抗震等级为四级时，也可采用装配整体式钢筋混凝土楼盖。

2 各楼层均应设置现浇钢筋混凝土圈梁。其混凝土强度等级应为砌块强度等级的二倍；现浇楼板的圈梁截面高度不宜小于 200mm，装配整体式楼板的板底圈梁截面高度不宜小于 120mm；其纵向钢筋直径不应小于砌体的水平分布钢筋直径，箍筋直径不应小于 8mm，间距不应大于 200mm。

附录G　多层钢结构厂房抗震设计要求

G.0.1　多层钢结构厂房的布置应符合本规范第8.1.4～8.1.7条的有关要求，尚应符合下列规定：

1　平面形状复杂、各部分构架高度差异大或楼层荷载相差悬殊时，应设防震缝或采取其他措施。

2　料斗等设备穿过楼层且支承在该楼层时，其运行装料后的设备总重心宜接近楼层的支点处。同一设备穿过两个以上楼层时，应选择其中的一层作为支座；必要时可另选一层加设水平支承点。

3　设备自承重时，厂房楼层应与设备分开。

表G.0.1　楼层水平支撑设置要求

项次	楼面结构类型		楼面荷载标准值 $\leqslant 10kN/m^2$	楼面荷载标准值 $>10kN/m^2$ 或较大集中荷载
1	钢与混凝土组合楼面，现浇、装配整体式楼板与钢梁有可靠连接	仅有小孔楼板	不需设水平支撑	不需设水平支撑
		有大孔楼板	应在开孔周围柱网区格内设水平支撑	应在开孔周围柱网区格内设水平支撑
2	铺金属板（与主梁有可靠连接）		宜设水平支撑	应设水平支撑
3	铺活动格栅板		应设水平支撑	应设水平支撑

注：1　楼面荷载系指除结构自重外的活荷载、管道及电缆等；

2　各行业楼层面板开孔不尽相同，大小孔的划分宜结合工程具体情况确定；

3　6、7度设防时，铺金属板与主梁有可靠连接，可不设置水平支撑。

4　厂房的支撑布置应符合下列要求：

1）柱间支撑宜布置在荷载较大的柱间，且在同一柱间上下贯通，不贯通时应错开开间后连续布置并宜适当增加相近楼层、屋面的水平支撑，确保支撑承担的水平地震作用能传递至基础。

2）有抽柱的结构，宜适当增加相近楼层、屋面的水平支撑并在相邻柱间设置竖向支撑。

3）柱间支撑杆件应采用整根材料，超过材料最大长度规格时可采用对接焊缝等强拼接；柱间支撑与构件的连接，不应小于支撑杆件塑性承载力的1.2倍。

5　厂房楼盖宜采用压型钢板与现浇钢筋混凝土的组合楼板，亦可采用钢铺板。

6　当各榀框架侧向刚度相差较大、柱间支撑布置又不规则时，应设楼层水平支撑；其他情况，楼层水平支撑的设置应按表G.0.1确定。

G.0.2　厂房的抗震计算，除应符合本规范第8.2节有关要求外，尚应符合下列规定：

1　地震作用计算时，重力荷载代表值和可变荷载组合值系数，除应符合本规范第5

章规定外，尚应根据行业的特点，对楼面检修荷载、成品或原料堆积楼面荷载、设备和料斗及管道内的物料等，采用相应的组合值系数。

2 直接支承设备和料斗的构件及其连接，应计入设备等产生的地震作用：

1） 设备与料斗对支承构件及其连接产生的水平地震作用，可按下式确定：

$$F_s = \alpha_{max}\lambda G_{eq} \tag{G.0.2-1}$$

$$\lambda = 1.0 + H_x/H_n \tag{G.0.2-2}$$

式中 F_s——设备或料斗重心处的水平地震作用标准值；

α_{max}——水平地震影响系数最大值；

G_{eq}——设备或料斗的重力荷载代表值；

λ——放大系数；

H_x——建筑基础至设备或料斗重心的距离；

H_n——建筑基础底至建筑物顶部的距离。

2） 此水平地震作用对支承构件产生的弯矩、扭矩，取设备或料斗重心至支承构件形心距离计算。

3 有压型钢板的现浇钢筋混凝土楼板，板面开孔较小且用栓钉等抗剪连接件与钢梁连接时，可将楼盖视为刚性楼盖。

G.0.3 多层钢结构厂房的抗震构造措施，除应符合本规范第 8.3、8.4 节有关要求外，尚应符合下列要求：

1 多层厂房钢框架与支撑的连接可采用焊接或高强度螺栓连接，纵向柱间支撑和屋面水平支撑布置，应符合下列要求：

1） 纵向柱间支撑宜设置于柱列中部附近；

2） 屋面的横向水平支撑和顶层的柱间支撑，宜设置在厂房单元端部的同一柱间内；当厂房单元较长时，应每隔 3～5 个柱间设置一道。

2 厂房设置楼层水平支撑时，其构造宜符合下列要求：

1） 水平支撑可设在次梁底部，但支撑杆端部应与楼层轴线上主梁的腹板和下翼缘同时相连；

2） 楼层水平支撑的布置应与柱间支撑位置相协调；

3） 楼层轴线上的主梁可作为水平支撑系统的弦杆，斜杆与弦杆夹角宜在 30°～60°之间；

4） 在柱网区格内次梁承受较大的设备荷载时，应增设刚性系杆，将设备重力的地震作用传到水平支撑弦杆（轴线上的主梁）或节点上。

附录 H　单层厂房横向平面排架地震作用效应调整

H.1　基本自振周期的调整

H.1.1　按平面排架计算厂房的横向地震作用时，排架的基本自振周期应考虑纵墙及屋架与柱连接的固结作用，可按下列规定进行调整：

1　由钢筋混凝土屋架或钢屋架与钢筋混凝土柱组成的排架，有纵墙时取周期计算值的 80%，无纵墙时取 90%；

2　由钢筋混凝土屋架或钢屋架与砖柱组成的排架，取周期计算值的 90%；

3　由木屋架、钢木屋架或轻钢屋架与砖柱组成排架，取周期计算值。

H.2　排架柱地震剪力和弯矩的调整系数

H.2.1　钢筋混凝土屋盖的单层钢筋混凝柱厂房，按 H.1.1 确定基本自振周期且按平面排架计算的排架柱地震剪力和弯矩，当符合下列要求时，可考虑空间工作和扭转影响，并按 H.2.3 的规定调整：

1　7 度和 8 度；

2　厂房单元屋盖长度与总跨度之比小于 8 或厂房总跨度大于 12m；

3　山墙的厚度不小于 240mm，开洞所占的水平截面积不超过总面积 50%，并与屋盖系统有良好的连接；

4　柱顶高度不大于 15m。

注：1. 屋盖长度指山墙到山墙的间距，仅一端有山墙时，应取所考虑排架至山墙的距离；

2. 高低跨相差较大的不等高厂房，总跨度可不包括低跨。

H.2.2　钢筋混凝土屋盖和密铺望板瓦木屋盖的单层砖柱厂房，按 H.1.1 确定基本自振周期且按平面排架计算的排架柱地震剪力和弯矩，当符合下列要求时，可考虑空间工作，并按第 H.2.3 条的规定调整：

1　7 度和 8 度；

2　两端均有承重山墙；

3　山墙或承重（抗震）横墙的厚度不小于 240mm，开洞所占的水平截面积不超过总面积 50%，并与屋盖系统有良好的连接；

4　山墙或承重（抗震）横墙的长度不宜小于其高度；

5　单元屋盖长度与总跨度之比小于 8 或厂房总跨度大于 12m。

注：屋盖长度指山墙到山墙或承重（抗震）横墙的间距。

H.2.3　排架柱的剪力和弯矩应分别乘以相应的调整系数，除高低跨度交接处上柱以外的钢筋混凝土柱，其值可按表 H.2.3-1 采用，两端均有山墙的砖柱，其值可按表 H.2.3-2 采用。

H.2.4　高低跨交接处的钢筋混凝土柱的支承低跨屋盖牛腿以上各截面，按底部剪力法求得的地震剪力和弯矩应乘以增大系数，其值可按下式采用：

表 H.2.3-1　钢筋混凝土柱（除高低跨交接处上柱外）考虑空间工作和扭转影响的效应调整系数

屋盖	山墙		屋盖长度（m）											
			≤30	36	42	48	54	60	66	72	78	84	90	96
钢筋混凝土无檩屋盖	两端山墙	等高厂房			0.75	0.75	0.75	0.8	0.8	0.8	0.85	0.85	0.85	0.9
		不等高厂房			0.85	0.85	0.85	0.9	0.9	0.9	0.95	0.95	0.95	1.0
	一端山墙		1.05	1.15	1.2	1.25	1.3	1.3	1.3	1.3	1.35	1.35	1.35	1.35
钢筋混凝土有檩屋盖	两端山墙	等高厂房			0.8	0.85	0.9	0.95	0.95	1.0	1.0	1.05	1.05	1.1
		不等高厂房			0.85	0.9	0.95	1.0	1.0	1.05	1.05	1.1	1.1	1.15
	一端山墙		1.0	1.05	1.1	1.1	1.15	1.15	1.15	1.2	1.2	1.2	1.25	1.25

表 H.2.3-2　砖柱考虑空间作用的效应调整系数

屋盖类型	山墙或承重（抗震）横墙间距（m）										
	≤12	18	24	30	36	42	48	54	60	66	72
钢筋混凝土无檩屋盖	0.60	0.65	0.70	0.75	0.80	0.85	0.85	0.90	0.95	0.95	1.00
钢筋混凝土有檩屋盖或密铺望板瓦木屋盖	0.65	0.70	0.75	0.80	0.90	0.95	0.95	1.00	1.05	1.05	1.10

$$\eta = \zeta\left(1 + 1.7\frac{n_h}{n_0}\cdot\frac{G_{EL}}{G_{Eh}}\right) \tag{H.2.4}$$

式中　η——地震剪力和弯矩的增大系数；

ζ——不等高厂房低跨交接处的空间工作影响系数，可按表 H.2.4 采用；

n_h——高跨的跨数；

n_0——计算跨数，仅一侧有低跨时应取总跨数，两侧均有低跨时应取总跨数与高跨跨数之和；

G_{EL}——集中于交接处一侧各低跨屋盖标高处的总重力荷载代表值；

G_{Eh}——集中于高跨柱顶标高处的总重力荷载代表值。

表 H.2.4　高低跨交接处钢筋混凝土上柱空间工作影响系数

屋盖	山墙	屋盖长度（m）										
		≤36	42	48	54	60	66	72	78	84	90	96
钢筋混凝土无檩屋盖	两端山墙		0.7	0.76	0.82	0.88	0.94	1.0	1.06	1.06	1.06	1.06
	一端山墙	1.25										
钢筋混凝土有檩屋盖	两端山墙		0.9	1.0	1.05	1.1	1.1	1.15	1.15	1.15	1.2	1.2
	一端山墙	1.05										

H.3　吊车桥架引起的地震作用效应的增大系数

H.3.1　钢筋混凝土柱单层厂房的吊车梁顶标高处的上柱截面，由吊车桥架引起的地震剪

力和弯矩应乘以增大系数，当按底部剪力法等简化计算方法计算时，其值可按表 H.3.1 采用。

表 H.3.1　桥架引起的地震剪力和弯矩增大系数

屋盖类型	山　　墙	边　柱	高低跨柱	其他中柱
钢筋混凝土无檩屋盖	两端山墙	2.0	2.5	3.0
	一端山墙	1.5	2.0	2.5
钢筋混凝土有檩屋盖	两端山墙	1.5	2.0	2.5
	一端山墙	1.5	2.0	2.0

附录J　单层钢筋混凝土柱厂房纵向抗震验算

J.1　厂房纵向抗震计算的修正刚度法

J.1.1　纵向基本自振周期的计算

按本附录计算单跨或等高多跨的钢筋混凝土柱厂房纵向地震作用时，在柱顶标高不大于15m且平均跨度不大于30m时，纵向基本周期可按下列公式确定：

1　砖围护墙厂房，可按下式计算：

$$T_1 = 0.23 + 0.00025\psi_1 l\sqrt{H^3} \tag{J.1.1-1}$$

式中　ψ_1——屋盖类型系数，大型屋面板钢筋混凝土屋架可采用1.0，钢屋架采用0.85；

l——厂房跨度（m），多跨厂房可取各跨的平均值；

H——基础顶面至柱顶的高度（m）。

2　敞开、半敞开或墙板与柱子柔性连接的厂房，可按第1款式（J.1.1-1）进行计算并乘以下列围护墙影响系数：

$$\psi_2 = 2.6 - 0.002l\sqrt{H^3} \tag{J.1.1-2}$$

式中　ψ_2——围护墙影响系数，小于1.0时应采用1.0。

J.1.2　柱列地震作用的计算

1　等高多跨钢筋混凝土屋盖的厂房，各纵向柱列的柱顶标高处的地震作用标准值，可按下列公式确定：

$$F_i = \alpha_1 G_{eq}\frac{K_{ai}}{\Sigma K_{ai}} \tag{J.1.2-1}$$

$$K_{ai} = \psi_3\psi_4 K_i \tag{J.1.2-2}$$

式中　F_i——i柱列柱顶标高处的纵向地震作用标准值；

α_1——相应于厂房纵向基本自振周期的水平地震影响系数，应按本规范第5.1.5条确定；

G_{eq}——厂房单元柱列总等效重力荷载代表值，应包括按本规范第5.1.3条确定的屋盖重力荷载代表值、70%纵墙自重、50%横墙与山墙自重及折算的柱自重(有吊车时采用10%柱自重，无吊车时采用50%柱自重)；

K_i——i柱列柱顶的总侧移刚度，应包括i柱列内柱子和上、下柱间支撑的侧移刚度及纵墙的折减侧移刚度的总和，贴砌的砖围护墙侧移刚度的折减系数，可根据柱列侧移值的大小，采用0.2～0.6；

K_{ai}——i柱列柱顶的调整侧移刚度；

ψ_3——柱列侧移刚度的围护墙影响系数，可按表J.1.2-1采用；有纵向砖围护墙的四跨或五跨厂房，由边柱列数起的第三柱列，可按表内相应数值的1.15倍采用；

ψ_4——柱列侧移刚度的柱间支撑影响系数，纵向为砖围护墙时，边柱列可采用1.0，中柱

列可按表J.1.2-2采用。

表 J.1.2-1　围护墙影响系数

围护墙类别和烈度		柱列和屋盖类别				
		边柱列	中柱列			
			无檩屋盖		有檩屋盖	
240砖墙	370砖墙		边跨无天窗	边跨有天窗	边跨无天窗	边跨有天窗
	7度	0.85	1.7	1.8	1.8	1.9
7度	8度	0.85	1.5	1.6	1.6	1.7
8度	9度	0.85	1.3	1.4	1.4	1.5
9度		0.85	1.2	1.3	1.3	1.4
无墙、石棉瓦或挂板		0.90	1.1	1.1	1.2	1.2

表 J.1.2-2　纵向采用砖围护墙的中柱列柱间支撑影响系数

厂房单元内设置下柱支撑的柱间数	中柱列下柱支撑斜杆的长细比					中柱列无支撑
	≤40	41～80	81～120	121～150	>150	
一柱间	0.9	0.95	1.0	1.1	1.25	1.40
二柱间			0.9	0.95	1.1	1.40

2　等高多跨钢筋混凝土屋盖厂房，柱列各吊车梁顶标高处的纵向地震作用标准值，可按下式确定：

$$F_{ci} = \alpha_1 G_{ci} \frac{H_{ci}}{H_i} \tag{J.1.2-3}$$

式中　F_{ci}——i 柱列在吊车梁顶标高处的纵向地震作用标准值；

G_{ci}——集中于 i 柱列吊车梁顶标高处的等效重力荷载代表值，应包括按本规范第5.1.3条确定的吊车梁与悬吊物的重力荷载代表值和40%柱子自重；

H_{ci}——i 柱列吊车梁顶高度；

H_i——i 柱列柱顶高度。

J.2　柱间支撑地震作用效应及验算

J.2.1　斜杆长细比不大于200的柱间支撑在单位侧力作用下的水平位移，可按下式确定：

$$u = \Sigma \frac{1}{1 + \varphi_i} u_{ti} \tag{J.2.1}$$

式中　u——单位侧力作用点的位移；

φ_i——i 节间斜杆轴心受压稳定系数，应按现行国家标准《钢结构设计规范》采用；

u_{ti}——单位侧力作用下 i 节间仅考虑拉杆受力的相对位移。

J.2.2　长细比不大于200的斜杆截面可仅按抗拉验算，但应考虑压杆的卸载影响，其拉力可按下式确定：

$$N_t = \frac{l_i}{(1 + \psi_c \varphi_i) s_c} V_{bi} \tag{J.2.2}$$

式中　N_t——i 节间支撑斜杆抗拉验算时的轴向拉力设计值；

l_i——i 节间斜杆的全长；

ψ_c——压杆卸载系数，压杆长细比为60、100和200时，可分别采用0.7、0.6和0.5；

V_{bi}——i节间支撑承受的地震剪力设计值；

s_c——支撑所在柱间的净距。

J.2.3 无贴砌墙的纵向柱列，上柱支撑与同列下柱支撑宜等强设计。

J.3 柱间支撑端节点预埋件的截面抗震验算

J.3.1 柱间支撑与柱连接节点预埋件的锚件采用锚筋时，其截面抗震承载力宜按下列公式验算：

$$N \leqslant \frac{0.8 f_y A_s}{\gamma_{RE}\left(\frac{\cos\theta}{0.8\zeta_m\psi} + \frac{\sin\theta}{\zeta_r\zeta_v}\right)} \tag{J.3.1-1}$$

$$\psi = \frac{1}{1 + \frac{0.6e_0}{\zeta_r s}} \tag{J.3.1-2}$$

$$\zeta_m = 0.6 + 0.25t/d \tag{J.3.1-3}$$

$$\zeta_v = (4 - 0.08d)\sqrt{f_c/f_y} \tag{J.3.1-4}$$

式中 A_s——锚筋总截面面积；

γ_{RE}——承载力抗震调整系数，可采用1.0；

N——预埋板的斜向拉力，可采用全截面屈服点强度计算的支撑斜杆轴向力的1.05倍；

e_0——斜向拉力对锚筋合力作用线的偏心距，应小于外排锚筋之间距离的20%（mm）；

θ——斜向拉力与其水平投影的夹角；

ψ——偏心影响系数；

s——外排锚筋之间的距离（mm）；

ζ_m——预埋板弯曲变形影响系数；

t——预埋板厚度（mm）；

d——锚筋直径（mm）；

ζ_r——验算方向锚筋排数的影响系数，二、三和四排可分别采用1.0、0.9和0.85；

ζ_v——锚筋的受剪影响系数，大于0.7时应采用0.7。

J.3.2 柱间支撑与柱连接节点预埋件的锚件采用角钢加端板时，其截面抗震承载力宜按下列公式验算：

$$N \leqslant \frac{0.7}{\gamma_{RE}\left(\frac{\sin\theta}{V_{u0}} + \frac{\cos\theta}{\psi N_{u0}}\right)} \tag{J.3.2-1}$$

$$V_{u0} = 3n\zeta_r\sqrt{W_{min}bf_af_c} \quad \text{(J.3.2-2)}$$

$$N_{u0} = 0.8nf_aA_s \quad \text{(J.3.2-3)}$$

式中 n——角钢根数；

b——角钢肢宽；

W_{min}——与剪力方向垂直的角钢最小截面模量；

A_s——一根角钢的截面面积；

f_a——角钢抗拉强度设计值。

附录K　单层砖柱厂房纵向抗震计算的修正刚度法

K.0.1　本附录适用于钢筋混凝土无檩或有檩屋盖等高多跨单层砖柱厂房的纵向抗震验算。

K.0.2　单层砖柱厂房的纵向基本自振周期可按下式计算：

$$T_1 = 2\psi_T\sqrt{\frac{\Sigma G_s}{\Sigma K_s}} \tag{K.0.2}$$

式中　ψ_T——周期修正系数，按表K.0.2采用；

G_s——第 s 柱列的集中重力荷载，包括柱列左右各半跨的屋盖和山墙重力荷载，及按动能等效原则换算集中到柱顶或墙顶处的墙、柱重力荷载；

K_s——第 s 柱列的侧移刚度。

表K.0.2　厂房纵向基本自振周期修正系数

屋盖类型	钢筋混凝土无檩屋盖		钢筋混凝土有檩屋盖	
	边跨无天窗	边跨有天窗	边跨无天窗	边跨有天窗
周期修正系数	1.3	1.35	1.4	1.45

K.0.3　单层砖柱厂房纵向总水平地震作用标准值可按下式计算：

$$F_{Ek} = \alpha_1\Sigma G_s \tag{K.0.3}$$

式中　α_1——相应于单层砖柱厂房纵向基本自振周期 T_1 的地震影响系数；

G_s——按照柱列底部剪力相等原则，第 s 柱列换算集中到墙顶处的重力荷载代表值。

K.0.4　沿厂房纵向第 s 柱列上端的水平地震作用可按下式计算：

$$F_s = \frac{\psi_s K_s}{\Sigma\psi_s K_s}F_{Ek} \tag{K.0.4}$$

式中　ψ_s——反映屋盖水平变形影响的柱列刚度调整系数，根据屋盖类型和各柱列的纵墙设置情况，按表K.0.4采用。

表K.0.4　柱列刚度调整系数

纵墙设置情况		屋盖类型			
		钢筋混凝土无檩屋盖		钢筋混凝土有檩屋盖	
		边柱列	中柱列	边柱列	中柱列
砖柱敞棚		0.95	1.1	0.9	1.6
各柱列均为带壁柱砖墙		0.95	1.1	0.9	1.2
边柱列为带壁柱砖墙	中柱列的纵墙不少于4开间	0.7	1.4	0.75	1.5
	中柱列的纵墙少于4开间	0.6	1.8	0.65	1.9

附录 L　隔震设计简化计算和砌体结构隔震措施

L.1　隔震设计的简化计算

L.1.1　多层砌体结构及与砌体结构周期相当的结构采用隔震设计时，上部结构的总水平地震作用可按本规范第 5.2.1 条公式（5.2.1-1）简化计算，但应符合下列规定：

1　水平向减震系数，宜根据隔震后整个体系的基本周期，按下式确定：

$$\psi = \sqrt{2}\,\eta_2 (T_{gm}/T_1)^{\gamma} \tag{L.1.1-1}$$

式中　ψ——水平向减震系数；

η_2——地震影响系数的阻尼调整系数，根据隔震层等效阻尼按本规范第 5.1.5 条确定；

γ——地震影响系数的曲线下降段衰减指数，根据隔震层等效阻尼按本规范第 5.1.5 条确定；

T_{gm}——砌体结构采用隔震方案时的设计特征周期，根据本地区所属的设计地震分组按本规范第 5.1.4 条确定，但小于 0.4s 时应按 0.4s 采用；

T_1——隔震后体系的基本周期，不应大于 2.0s 和 5 倍特征周期的较大值。

2　与砌体结构周期相当的结构，其水平向减震系数宜根据隔震后整个体系的基本周期，按下式确定：

$$\psi = \sqrt{2}\,\eta_2 (T_g/T_1)^{\gamma} (T_0/T_g)^{0.9} \tag{L.1.1-2}$$

式中　T_0——非隔震结构的计算周期，当小于特征周期时应采用特征周期的数值；

T_1——隔震后体系的基本周期，不应大于 5 倍特征周期值；

T_g——特征周期；其余符号同上。

3　砌体结构及与其基本周期相当的结构，隔震后体系的基本周期可按下式计算：

$$T_1 = 2\pi \sqrt{G/K_h g} \tag{L.1.1-3}$$

式中　T_1——隔震体系的基本周期；

G——隔震层以上结构的重力荷载代表值；

K_h——隔震层的水平动刚度，可按本规范第 12.2.4 条的规定计算；

g——重力加速度。

L.1.2　砌体结构及与其基本周期相当的结构，隔震层在罕遇地震下的水平剪力可按下式计算：

$$V_c = \lambda_s \alpha_1 (\zeta_{eq}) G \tag{L.1.2}$$

式中　V_c——隔震层在罕遇地震下的水平剪力。

L.1.3　砌体结构及与其基本周期相当的结构，隔震层质心处在罕遇地震下的水平位移可按下式计算：

$$u_e = \lambda_s \alpha_1 (\zeta_{eq}) G/K_h \tag{L.1.3}$$

式中 λ_s——近场系数；甲、乙类建筑距发震断层 5km 以内取 1.5；5～10km 取 1.25；10km 以远取 1.0；丙类建筑可取 1.0；

$\alpha_1(\zeta_{eq})$——罕遇地震下的地震影响系数值，可根据隔震层参数，按本规范第 5.1.5 条的规定进行计算；

K_h——罕遇地震下隔震层的水平动刚度，应按本规范第 12.2.4 条的有关规定采用。

L.1.4 当隔震支座的平面布置为矩形或接近于矩形，但上部结构的质心与隔震层刚度中心不重合时，隔震支座扭转影响系数可按下列方法确定：

1 仅考虑单向地震作用的扭转时，扭转影响系数可按下列公式估计：

$$\beta_i = 1 + 12es_i/(a^2 + b^2) \qquad \text{(L.1.4-1)}$$

式中 e——上部结构质心与隔震层刚度中心在垂直于地震作用方向的偏心距；

s_i——第 i 个隔震支座与隔震层刚度中心在垂直于地震作用方向的距离；

a、b——隔震层平面的两个边长。

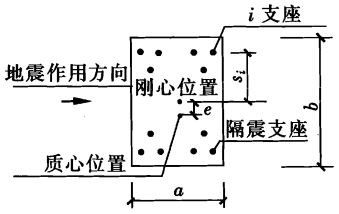


图 L.1.4 扭转计算示意图

对边支座，其扭转影响系数不宜小于 1.15；当隔震层和上部结构采取有效的抗扭措施后或扭转周期小于平动周期的 70%，扭转影响系数可取 1.15。

2 同时考虑双向地震作用的扭转时，扭转影响系数可仍按式（L.1.4-1）计算，但其中的偏心距值（e）应采用下列公式中的较大值替代：

$$e = \sqrt{e_x^2 + (0.85e_y)^2} \qquad \text{(L.1.4-2)}$$

$$e = \sqrt{e_y^2 + (0.85e_x)^2} \qquad \text{(L.1.4-3)}$$

式中 e_x——y 方向地震作用时的偏心距；

e_y——x 方向地震作用时的偏心距。

对边支座，其扭转影响系数不宜小于 1.2。

L.1.5 砌体结构按本规范第 12.2.5 条规定进行竖向地震作用下的抗震验算时，砌体抗震抗剪强度的正应力影响系数，宜按减去竖向地震作用效应后的平均压应力取值。

L.1.6 砌体结构的隔震层顶部各纵、横梁均可按承受均布荷载的单跨简支梁或多跨连续梁计算。均布荷载可按本规范第 7.2.5 条关于底部框架砖房的钢筋混凝土托墙梁的规定取值；当按连续梁算出的正弯矩小于单跨简支梁跨中弯矩的 0.8 倍时，应按 0.8 倍单跨简支梁跨中弯矩配筋。

L.2 砌体结构的隔震措施

L.2.1 当水平向减震系数不大于 0.50 时，丙类建筑的多层砌体结构，房屋的层数、总高度和高宽比限值，可按本规范第 7.1 节中降低一度的有关规定采用。

L.2.2 砌体结构隔震层的构造应符合下列规定：

1 多层砌体房屋的隔震层位于地下室顶部时，隔震支座不宜直接放置在砌体墙上，并应验算砌体的局部承压。

2 隔震层顶部纵、横梁的构造均应符合本规范第 7.5.4 条关于底部框架砖房的钢筋混凝土托墙梁的要求。

L.2.3 丙类建筑隔震后上部砌体结构的抗震构造措施应符合下列要求：

1 承重外墙尽端至门窗洞边的最小距离及圈梁的截面和配筋构造，仍应符合本规范第 7.1 节和第 7.3 节的有关规定。

2 多层烧结普通粘土砖和烧结多孔粘土砖房屋的钢筋混凝土构造柱设置，水平向减震系数为 0.75 时，仍应符合本规范表 7.3.1 的规定；7～9 度，水平向减震系数为 0.5 和 0.38 时，应符合表 L.2.3-1 的规定，水平向减震系数为 0.25 时，宜符合本规范表 7.3.1 降低一度的有关规定。

表 L.2.3-1 隔震后砖房构造柱设置要求

房屋层数			设置部位	
7 度	8 度	9 度		
三、四	二、三		楼、电梯间四角外墙四角，错层部位横墙与外纵墙交接处，较大洞口两侧，大房间内外墙交接处	每隔 15m 或单元横墙与外墙交接处
五	四	二		每隔三开间的横墙与外墙交接处
六、七	五	三、四		隔开间横墙（轴线）与外墙交接处，山墙与内纵墙交接处；9 度四层，外纵墙与内墙（轴线）交接处
八	六、七	五		内墙（轴线）与外墙交接处，内墙局部较小墙垛处；8 度七层，内纵墙与隔开间横墙交接处；9 度时内纵墙与横墙（轴线）交接处

3 混凝土小型空心砌块房屋芯柱的设置，水平向减震系数为 0.75 时，仍应符合本规范表 7.4.1 的规定；7～9 度，当水平向减震系数为 0.5 和 0.38 时，应符合表 L.2.3-2 的规定，当水平向减震系数为 0.25 时，宜符合本规范表 7.4.1 降低一度的有关规定。

表 L.2.3-2 隔震后混凝土小型空心砌块房屋芯柱设置要求

房屋层数			设置部位	设置数量
7 度	8 度	9 度		
三、四	二、三		外墙转角，楼梯间四角，大房间内外墙交接处；每隔 16m 或单元横墙与外墙交接处	外墙转角，灌实 3 个孔 内外墙交接处，灌实 4 个孔
五	四	二	外墙转角，楼梯间四角，大房间内外墙交接处，山墙与内纵墙交接处，隔三开间横墙（轴线）与外纵墙交接处	
六	五	三	外墙转角，楼梯间四角，大房间内外墙交接处；隔开间横墙（轴线）与外纵墙交接处，山墙与内纵墙交接处；8、9 度时，外纵墙与横墙（轴线）交接处，大洞口两侧	外墙转角，灌实 5 个孔 内外墙交接处，灌实 4 个孔 洞口两侧各灌实 1 个孔
七	六	四	外墙转角，楼梯间四角，各内墙（轴线）与外纵墙交接处；内纵墙与横墙（轴线）交接处；8、9 度时洞口两侧	外墙转角，灌实 7 个孔 内外墙交接处，灌实 4 个孔 内墙交接处，灌实 4～5 个孔洞口两侧各灌实 1 个孔

4 上部结构的其他抗震构造措施，水平向减系数为 0.75 时仍按本规范第 7 章的相应规定采用；7～9 度，水平向减震系数为 0.50 和 0.38 时，可按本规范第 7 章降低一度的相应规定采用；水平向减系数为 0.25 时可按本规范第 7 章降低二度且不低于 6 度的相应规定采用。

本规范用词用语说明

1 为了便于在执行本规范条文时区别对待，对要求严格程度不同的用词说明如下：

1）表示很严格，非这样做不可的用词：

正面词采用“必须”；反面词采用“严禁”。

2）表示严格，在正常情况下均应这样做的用词：

正面词采用“应”；反面词采用“不应”或“不得”。

3）表示允许稍有选择，在条件许可时首先这样做的用词：

正面词采用“宜”；反面词采用“不宜”；

表示有选择，在一定条件下可以这样做的，采用“可”。

2 规范中指定应按其他有关标准、规范执行时，写法为：“应符合……的规定”或“应按……执行”。