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GB 50011–2010

Code for Seismic Design of Buildings

建筑抗震设计规范

Issued on: May 31, 2010

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Jointly Issued by Ministry of Housing and Urban-Rural Construction of the People's Republic of China

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Chief Development Department: Ministry of Housing and Urban-Rural Development of the People's Republic of China

Approval Department: Ministry of Housing and Urban-Rural Development of the People's Republic of China

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NOTICE

This 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.

Announcement of Ministry of Housing and Urban-Rural Development of the People's Republic of China

No. 609

Announcement on Publishing the National Standard "Code for Seismic Design of Buildings"

The standard "Code for Seismic Design of Buildings" has been approved as a national standard with the serial number of GB 50011−2010 and shall be implemented on December 1, 2010. Herein, Articles 1.0.2, 1.0.4, 3.1.1, 3.3.1, 3.3.2, 3.4.1, 3.5.2, 3.7.1, 3.7.4, 3.9.1, 3.9.2, 3.9.4, 3.9.6, 4.1.6, 4.1.8, 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, 5.4.3, 6.1.2, 6.3.3, 6.3.7, 6.4.3, 7.1.2, 7.1.5, 7.1.8, 7.2.4, 7.2.6, 7.3.1, 7.3.3, 7.3.5, 7.3.6, 7.3.8, 7.4.1, 7.4.4, 7.5.7, 7.5.8, 8.1.3, 8.3.1, 8.3.6, 8.4.1, 8.5.1, 10.1.3, 10.1.12, 10.1.15, 12.1.5, 12.2.1 and 12.2.9 are compulsory ones and must be enforced strictly. The former standard "Code for Seismic Design of Buildings" GB 50011−2001 shall be abolished simultaneously.

Authorized by the Research Institute of Standard and Norms of the Ministry, this code is published and distributed by China Architecture & Building Press.

Ministry of Housing and Urban-Rural Development of the People's Republic of China

May 31, 2010

Foreword

According to the requirements of Document Jian Biao [2006] No. 77—"Notice on Printing and Distributing the Development and Revision Plan of Engineering Construction Standards and Codes in 2006 (Batch 1)" issued by the former Ministry of Construction (MOC), this code was revised from "Code for Seismic Design of Buildings" GB 50011−2001 by China Academy of Building Research (CABR) together with other design, survey, research and education institutions concerned.

During the process of revision, the editorial team summarized the relief experiences accumulated in Wenchuan Earthquake in 2008; adjusted the seismic precautionary Intensity; added the compulsory provisions on sites in mountainous areas, construction of framed structure filler wall, staircase of masonry structure and construction requirements of seismic structure; and raised the requirements on fabricated floor framing and steel bar elongation. Hereafter, the editorial team carried out studies on specific topics and some tests concerned, investigated and summarized the experiences and lessons from the strong earthquakes occurred in recent years home and abroad (including Wenchuan Earthquake), adopted the new research achievements of earthquake engineering, took the economic condition and construction practices in China into account, widely collected the comments from the relevant design, survey, research and education institutions as well as seismic administration authorities nationwide and finalized this code through repeated discussion, revision, substantiation and pilot design.

This newly-revised version comprises 14 Chapters and 12 Appendixes. Besides remaining those provisions partially revised in 2008, the main revisions at this edition are: supplement the provisions on the seismic measures against Intensity 7 (0.15*g*) and Intensity 8 (0.30*g*), and adjust the design earthquake grouping in accordance with "Seismic Ground Motion Parameter Zonation Map of China"; modify the soil liquefaction discriminating formula; adjust the damping adjustment parameter in Seismic Influence Coefficient Curve, damping ratio and bearing force seismic adjustment coefficient of steel structure, and horizontal shock absorbing coefficient calculation, and supplement the calculation method for horizontal and vertical earthquake action of large-span buildings; raise the requirements on the seismic design of concrete-framed house and bottom-framed masonry house; propose the seismic grade of steel structure house and adjust the provisions on seismic measures correspondingly; modify the seismic measures of multi-story masonry house, concrete-seismic-wall house, reinforced masonry house; expend the application scope of houses designed with seismic isolation, energy dissipation and shock absorption; add the principles on performance-based seismic design of buildings as well as the provisions on seismic design of large-span building, subterranean building, framed and trestled plant building, steel shotcrete-concrete frame and structure steel frame-reinforced concrete core-wall structure. Cancel the contents related to inner frame brickwork.

The provisions printed in bold type are compulsory ones and must be enforced strictly.

The Ministry of Housing and Urban-Rural Development of the People's Republic of China is in charge of the administration of this code and the explanation of the compulsory provisions. China Academy of Building Research (CABR) is responsible for the explanation of specific technical contents. All relevant organizations are kindly requested to sum up and accumulate your experiences in actual practices during the process of implementing this code. The relevant opinions and advice, whenever necessary, can be posted or passed on to the Management Group of the National Standard "Code for Seismic Design of Buildings" of the China Academy of Building Research (Address: No.

30, Beisanhuan East Road, Beijing City, 100013, China; E-mail: GB50011-cabr@163.com).

Chief Development Organization: China Academy of Building Research (CABR).

Participating Development Organizations: Institute of Engineering Mechanics (IEM) of China Earthquake Administration, China Architecture Design & Research Group, China Institute of Building Standard Design & Research, Beijing Institute of Architectural Design, China Electronics Engineering Design Institute, China Southwest Architectural Design and Research Institute, China Northwest Architectural Design and Research Institute, China Northeast Architecture Design and Research Institute, East China Architectural Design and Research Institute, Central-South Architectural Design Institute, The Architectural Design and Research Institute of Guangdong Province, Shanghai Institute of Architecture Design and Research, Institute of Building Design and Research of Xinjiang Uygur Autonomous Region, Yunnan Province Design Institute, Sichuan Architectural Design Institute, Shenzhen General Institute of Architectural Design and Research, Beijing Geotechnical Institute, Shanghai Tunnel Engineering and Rail Transit Design and Research Institute, China Construction (Shenzhen) Design international, Architecture Design General Institute of China Metallurgical Group Corporation, China National Machinery Industry Corporation, China IPPR International Engineering Corporation, Tsinghua University, Tongji University, Harbin Institute of Technology, Zhejiang University, Chongqing University, Yunnan University, Guangzhou University, Dalian University of Technology and Beijing University of Technology

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Contents

1 General Provisions

1.0.1 This code is formulated with a view to implementing the relevant laws and regulations on construction engineering and earthquake prevention and disaster reduction, carrying out the policy of "prevention first", as well as alleviating the seismic damage of buildings, avoiding casualties and reducing economic loss through earthquake protection of buildings.

As for the buildings adopting seismic design according to this code, the basic seismic precautionary objectives are as follows: under the frequent earthquake action with Intensity being less than the local seismic precautionary Intensity, the buildings with major structure undamaged or requiring no repair may continue to serve; under the earthquake action with Intensity being equivalent to the local seismic precautionary Intensity, the buildings with possible damage may continue to serve with common repair; under the rare earthquake action with Intensity being larger than the local seismic precautionary Intensity, the buildings shall not collapse or shall be free from such severe damage that may endanger human lives. If the buildings with special requirements in functions or other aspects are adopted with the seismic performance design, more concrete and higher seismic precautionary objectives shall be established.

1.0.2 All the buildings situated on zones of seismic precautionary Intensity 6 or above must be carried out with seismic design.

1.0.3 This code is applicable to the seismic design and the isolation and energy dissipation design of the buildings suited on zones of seismic precautionary Intensity 6, 7, 8 and 9. And the seismic performance design of buildings may be adopted with the basic methods specified in this code.

As for the buildings suited on zones where the seismic precautionary Intensity is above Intensity 9 and the industrial buildings for special purpose, their seismic design shall be carried out according to the relevant special provisions.

Note: For the purposes of this code, "seismic precautionary Intensity 6, 7, 8 and 9" hereinafter is referred to "Intensity 6, 7, 8 and 9".

1.0.4 The seismic precautionary Intensity must be determined in accordance with the documents (drawings) examined, approved and issued by the authorities appointed by the State.

1.0.5 Generally, the seismic precautionary Intensity of buildings shall be adopted with the basic seismic Intensity (the Intensity values corresponding to the design basic acceleration of ground motion value in this code) determined according to the "Seismic Ground Motion Parameter Zonation Map of China".

1.0.6 In addition to the requirements of this code, the seismic design of buildings also shall comply with those specified in the relevant current standards of the State.

2 Terms and Symbols

2.1 Terms

2.1.1 Seismic precautionary intensity

The seismic Intensity approved by the authority appointed by the State, which is used as the basis for the seismic precaution of buildings in a certain region. Generally, the seismic Intensity with the frequency over 10% in 50 years is adopted.

2.1.2 Seismic precautionary criterion

The rule for judging the seismic precautionary requirements, which is dependent on the seismic precautionary Intensity or the design parameters of ground motion and the precautionary category of buildings.

2.1.3 Seismic ground motion parameter zonation map

The map in which the whole county is divided into regions with different seismic precautionary requirements according to the ground motion parameter (earthquake action degree indicated by acceleration).

2.1.4 Earthquake action

The dynamic action of structure caused by ground motion, including horizontal earthquake action and vertical earthquake action.

2.1.5 Design parameters of ground motion

The seismic acceleration-time curve (speed and displacement), the response spectrum of acceleration, and the peak acceleration used in seismic design.

2.1.6 Design basic acceleration of ground motion

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

2.1.7 Design characteristic period of ground motion

The period value corresponding to the starting point of the descending branch reflecting such factors as the earthquake magnitude, epicentral distance and site class in the seismic influence coefficient curve used for seismic design, which is named as "characteristic period" for short.

2.1.8 Site

Locations of the project colonies, being with similar characteristics of response spectrum. The scope of site is equivalent to plant area, residential area and natural village or the plane area no less than 1.0km^2.

2.1.9 Seismic concept design of buildings

The process of making the general arrangement for the buildings and structures and of determining details, based on the design fundamental design principles and design concept obtained from the past experiences in earthquake disasters and projects.

2.1.10 Seismic measures

The seismic design contents except earthquake action calculation and resistance calculation, including the details of seismic design.

2.1.11 Details of seismic design

All the detailed requirements must be taken for the structural and nonstructural parts according to seismic concept design principles, requiring no calculation generally.

2.2 Symbols

2.2.1 Actions and effects

F_{Ek}, F_{Evk}——Standard values of total horizontal and vertical earthquake actions of structure respectively;

G_E, G_{eq}——Representative value of gravity load and the representative value of total equivalent gravity load of a structure (component) under earthquake action;

W_K——Standard value of wind load;

S_E——Earthquake effect (bending moment, axial force, shear force, stress and deformation);

S——Fundamental combination of the effects of earthquake action and other loads;

S_k——Effect of the standard value of action and load;

M——Bending moment;

N——Axial pressure;

V——Shear force;

p——Pressure on bottom of foundation;

u——Lateral displacement;

θ——Displacement angle of storey.

2.2.2 Material properties and resistance

K——Rigidity of structure (component);

R——Bearing capacity of structural component;

F, f_K, f_E——Design value, standard value and seismic design value of various material strength (including th e bearing capacity of base) respectively;

$[\theta]$——Allowable displacement angle of storey.

2.2.3 Geometric parameters

A——Cross-sectional area of component;

A_s——Cross-sectional area of steel reinforcement;

B——Total width of structure;

H——Total height of structure, or the column height;

L——Total length of structure (unit);

a——Distance;

a_s, a'_s——Minimal distance from the force concurrence point of all longitudinal tensile and compressive reinforcements to the margin of section;

b——Sectional width of component;

d——Depth or thickness of soil layer, or the diameter of steel reinforcement;

h——Depth of section of component;

l——Length or span of component;

t——Thickness of seismic wall or the thickness of floor 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;

γ_G, γ_E, γ_W——Partial factor of action;

γ_{RE}——Seismic adjustment coefficient of bearing capacity;

ζ——Calculation coefficient;

η——Enhancement or adjustment coefficient of earthquake action effect (internal force or deformation);

λ——Slenderness ratio of component, or the proportionality coefficient;

ξ_y——Yield strength coefficient of structure (component);

ρ——Reinforcement ratio or ratio;

φ——Stability coefficient of compressive component;

ψ——Combination value coefficient or the influence coefficient.

2.2.5 Others

T——Natural vibration period of structure;

N——Penetration blow count;

I_{lE}——Liquefaction index of base under earthquake;

X_{ji}——Vibration mode coordinate of displacement (relative displacement of the i^{th} mass point of the j^{th} vibration mode in x direction);

Y_{ji}——Vibration mode coordinate of displacement (relative displacement of the i^{th} mass point of the j^{th} vibration mode in y direction);

n——Total number, such as number of storeys, masses, reinforcement bars and spans, etc.;

v_{se}——Equivalent shear wave velocity of soil layer;

Φ_{ji}——Vibration mode coordinate of rotation (relative rotation of the i^{th} mass point of the j^{th} vibration mode in rotating direction).

3 Basic Requirements of Seismic Design

3.1 Category and Criterion for Seismic Precaution of Buildings

3.1.1 As for all the buildings adopting with seismic precaution, the category and criterion for their seismic precaution shall be determined according to the current national standard "Standard for Classification of Seismic Protection of Building Constructions" GB 50223.

3.1.2 Unless otherwise specified in this code concretely, the Category B, C and D buildings with seismic precautionary Intensity 6 may not be carried out the calculation of earthquake action.

3.2 Earthquake Effect

3.2.1 The earthquake effect suffered by the zones in which buildings are suited shall be characterized by adopting with the design basic acceleration and characteristic period of ground motion corresponding to the seismic precautionary Intensity.

3.2.2 The corresponding relationship between the values of seismic precautionary Intensity and design basic acceleration of ground motion shall be in accordance with those specified in Table 3.2.2. Unless otherwise stated in this code, the buildings in such zones where the design basic acceleration of ground motion is 0.15g and 0.30g shall be carried out with seismic design respectively according to the requirements of seismic precautionary Intensity 7 and 8.

Table 3.2.2 Corresponding Relationship Between Values of Seismic Precautionary Intensity and Design Basic Acceleration of Ground Motion

Seismic precautionary Intensity	6	7	8	9
Design basic acceleration value of ground motion	0.05g	0.10 (0.15)g	0.20 (0.30)g	0.40g

Note: g is the gravity acceleration.

3.2.3 The characteristic period of earthquake effect shall be determined according to the design earthquake groups and site class of the site of buildings. The design earthquakes in this code are totally divided into three groups, and their characteristic periods shall be adopted according to the relevant provisions in Chapter 5 of this code.

3.2.4 The seismic precautionary Intensity, design basic acceleration value of ground motion and design earthquake groups of the central areas in the main cities in China may be adopted according to Appendix A of this code.

3.3 Site and Base

3.3.1 To select a building site, a comprehensive assessment shall be taken on the favorable, common, unfavorable and hazardous sections to the seismic precaution based on the engineering need and the condition of seismic motion as well as the relevant data on engineering geology and seismogeology. As for the unfavorable sections, requirements on avoiding shall be proposed; if not, effective measures shall be taken. Category A and B buildings must not be constructed and the Category C buildings shall not be constructed in the hazardous sections.

3.3.2 If the building site is of Class I, it is still allowed to adopt details of seismic design for the

Category A and B buildings according to the requirements of local seismic precautionary Intensity and adopting details of seismic design for the Category C buildings according to the requirements one grade less than the local seismic precautionary Intensity, however, as for the seismic precautionary Intensity 6, buildings shall still be adopted with details of seismic design according to the local seismic precautionary Intensity.

3.3.3 If the building site is of Class III or IV, in the areas where the design basic acceleration of ground motion is $0.15g$ or $0.30g$, unless otherwise stated in this code, buildings should be adopted with details of seismic design according to the requirements of buildings belonging to each precautionary category respectively for seismic precautionary Intensity 8 ($0.20g$) and 9 ($0.40g$).

3.3.4 The design of base and foundation shall meet the following requirements:

1 Foundation of one same structural unit should not be built on the bases with entirely different features.

2 One same structural unit should not be adopted with natural base and pile foundation partially; if different types of foundations are adopted or the buried depth of foundation is different obviously, corresponding measures shall be taken at the relevant positions of foundation and superstructure according to the differential settlement of these two parts of base foundations under earthquake.

3 For the base consisted of soft clay, liquefied soil, newly filled soil or extremely non-uniform soil, corresponding measures shall be taken according to the differential settlement of base under earthquake and other adverse impacts.

3.3.5 The site and based foundation of buildings in mountainous areas shall meet the following requirements:

1 The investigation on the building sites in mountainous areas shall be taken with slop stability evaluation and prevention and treatment scheme suggestions; and slope project meeting the requirements of seismic precaution shall be set up in accordance with the geologic and orographic conditions, operation requirements and local conditions.

2 The slope design shall meet the requirements of the current national standard "Technical Code for Building Slope Engineering" GB 50330; and the relevant friction angle shall be corrected according to the precautionary Intensity in stability checking and calculation.

3 The building foundation near to slope shall be carried out with seismic stability design. An adequate distance shall be left at the edge of building foundation and the soil or severely weathered rock slope, the distance value shall be determined according to the precautionary Intensity, and proper measures shall be taken to avoid the foundation failure of base under earthquake.

3.4 Regularity of Building Configuration and Structural Assembly

3.4.1 The building design shall specify clear building configuration regularity according to the requirements of seismic concept design. The irregular buildings shall be taken with strengthening measures as required; the especially irregular buildings shall be taken with special strengthening measures through special study and demonstration; severely irregular buildings shall not be built.

Note: The building configuration refers to the variations in the plane form, vertical plane and vertical section of a building.

3.4.2 The building design shall attach importance to influence of the regularity of its plane, vertical plane and vertical section on the seismic performance and economical rationality, the regular building configuration shall be preferred, the plane layout of its lateral-force-resisting component should be regular and symmetric, the lateral rigidity should vary uniformly along the vertical direction, the sectional dimension and material strength of the vertical lateral-force-resisting components should be reduced gradually from bottom to top, and the sudden changes of lateral rigidity and bearing capacity shall be avoided.

The seismic design of irregular building shall comply with those specified in Article 3.4.4 of this code.

3.4.3 The plan and vertical irregularity of building configuration and its structural assembly shall be divided according to the following requirements:

1 The concrete house, steel structure house and steel-concrete structured house, having any plan irregularity types listed in Table 3.4.3-1 or any vertical irregularity types listed in Table 3.4.3-2 or other similar irregular types, shall be regarded as irregular buildings.

Table 3.4.3-1 Main Types of Plan Irregularity

Type of irregularity	Definition and reference index
Torsional irregularity	Under the action of specified horizontal force, the maximum elastic horizontal displacement or (storey drift) of storey is larger than 1.2 times of the elastic horizontal displacement (or storey drift) at both ends of this storey
Uneven irregularity	The sunken size of plane is larger than 30% of the overall size in the corresponding projection direction
Partial discontinuity of floor slab	The size of floor slab and the rigidity of plane change rapidly, for instance, the effective width of floor slab is less than 50% of the typical width of floor slab at this storey, or the opening area is larger than 30% of the floorage of this storey or great split-storey exists

Table 3.4.3-2 Main Types of Vertical Irregularity

Type of irregularity	Definition and reference index
Irregularity of lateral rigidity	The lateral rigidity of this storey is less than 70% of the adjacent upper storey or less than 80% of the average lateral rigidity of the adjacent three storeys; except for the top storey or the small buildings outside roof, the horizontal size of partial take-in is larger than 25% of the adjacent lower storey
Discontinuity of vertical lateral-force-resisting component	The internal force of vertical lateral-force-resisting components (columns, seismic walls and seismic bracing) is transmitted downward through horizontal transmission components (beam and truss)
Discontinuity of storey bearing capacity	The inter-storey shear capacity of lateral-force-resisting structure is less than 80% of the adjacent upper storey

2 The division of plan and vertical irregularity of masonry buildings, single-storey factory buildings, single-storey spacious buildings, large-span roof buildings and subterranean buildings shall meet those specified in the related chapters of this code.

3 A building, with several irregularity types or one certain irregularity type exceeding the reference index greatly, shall be regarded as especially irregular building.

3.4.4 Provided that the building configuration and its structural assembly are irregular, the earthquake action calculation and internal force adjustment shall be carried out in accordance with the following requirements and the weak positions shall be taken with effective details of seismic design:

1 The buildings meeting plan irregularity and vertical regularity shall be adopted with three-dimensional calculation model and comply with following requirements:

1) For buildings having torsional irregularity, the torsion effects shall be considered, and the maximum elastic horizontal displacement and storey drift of the vertical components respectively should be less than or equal to 1.5 times of the average values of elastic horizontal displacement and storey drift at both ends of this storey, and if the maximum storey drift is far less than the specified limit, the requirement may be loosened properly;

2) In case of the uneven irregularity or the partial discontinuity of floor slab exists, the calculation model meeting the practical rigidity changes in the floor level shall be adopted; for high Intensity or relatively big degree of irregularity, the influence of the local deformation of floor slab also should be taken into account;

3) For the buildings with dissymmetrical plane and uneven irregularity or partial discontinuity, the torsional displacement may be calculated in blocks depending on actual situation, and the position with large torsion shall be adopted with local internal force enhancement coefficient.

2 The buildings meeting plan irregularity and vertical irregularity shall be adopted with three-dimensional calculation model, the seismic shear force of storeys with small rigidity shall be multiplied by a enhancement coefficient no less than 1.15, the weak storeys shall be carried out with elasto-plastic deformation analysis according to the relevant regulations of this code and also shall meet the following requirements:

1) If one vertical lateral-force-resisting component is discontinuous, the seismic internal force transferred through this component to the horizontal transmission components shall be multiplied by a enhancement coefficient of 1.25~2.0 according to the Intensity, type of horizontal transmission component, stress condition and physical dimension, etc.;

2) In case of the irregularly of lateral rigidity, the lateral rigidity ratio between adjacent storeys shall comply with those specified in the relevant chapters of this code based on the structure type;

3) In case of the abrupt discontinuity of storey bearing capacity, the shear capacity of the lateral-force-resisting structure at weak storey shall not be less than 65% of that of the adjacent upper storey.

3 For the buildings meeting plan irregularity and vertical irregularity, the seismic measures no less than requirements of Item 1 and 2 in this article shall be adopted correspondingly according to the quantity and degree of the irregularity types. For especially irregular buildings, much more efficient strengthening measures shall be taken through special study or the weak positions shall be adopted with corresponding performance-based seismic design method.

3.4.5 For the buildings with complex configuration and irregular plan and vertical planes, a comparative analysis of such factors as degree of irregularity, condition of base foundation and technical economy shall be conducted to determine whether set seismic joints, and the following requirements shall be met respectively:

1 If no seismic joint is set, practical calculation model shall be adopted to analyze and distinguish the vulnerable positions due to stress concentration, deformation concentrated or earthquake twisting effect, so as to adopt corresponding strengthening measures.

2 To set seismic joints at proper positions, multiple regular lateral-force-resisting structural units should be formed. The seismic joint shall be left with adequate width according to the seismic precautionary Intensity, variety of structural material, structure type, height and height difference of structural units as well as the condition of possible earthquake twisting effect, and the superstructure on both sides of the seismic joint shall be separated completely.

3 To set expansion joint and settlement joint, their width shall comply with the requirements on seismic joint.

3.5 Structural System

3.5.1 The structural system shall be determined through comprehensive comparison in technology, economy and application conditions based on such factors as precautionary category, seismic precautionary Intensity, height, site conditions, base, structural materials and construction of the building.

3.5.2 The structural system shall meet the following requirements:

1 Clear calculation diagram and reasonable earthquake action transition ways shall be provided.

2 The entire structure shall be avoided from losing its seismic capacity or its bearing capacity to gravity load due to the failure of part of structure or components.

3 Necessary seismic capacity, favorable deformability and seismic energy dissipation ability.

4 The weak positions that may appear shall be taken with measures to improve their seismic capacity.

3.5.3 The structural system still should meet the following requirements:

1 Enough earthquake fortification lines should be arranged.

2 It should be with reasonable distribution of rigidity and bearing capacity to avoid forming weak positions due to partial weakening or abrupt changes and avoiding excessive stress concentration or plastic deformation concentration from happening.

3 The structure should have similar dynamic characteristics in the directions of two main axes.

3.5.4 The structural components shall meet the following requirements:

1 The masonry structures shall be built with reinforced concrete ring beams, constructional columns and core columns as required or be adopted with constrained masonry and reinforced masonry, etc..

2 As for the concrete structure components, the sectional dimension and the installation of principle bars and stirrups shall be under control in order to avoid the shear failure occurring before flexural failure, concrete crush before steel reinforcement yielding, and anchoring bond failure of steel reinforcement before steel reinforcement failure.

3 The prestressed concrete components shall be equipped with adequate nonprestressed reinforcement.

4 The size of steel structure component shall be controlled reasonably to avoid local instability

or whole instability of component.

5 The floor and roof of multi-storey and tall buildings should be preferred with cast-in-situ RC (reinforced concrete) slabs. While applying the prefabricated concrete floor slab or roof, measures for the floor system and structure shall be taken to ensure the integration of connection between prefabricated slabs.

3.5.5 The connection between components of the structure shall meet the following requirements:

1 The failure of connected nodes of components shall not occur before that of components they connect.

2 Anchorage failure of embedded parts shall not occur before that of the connection piece.

3 The connections of fabricated structural components shall ensure the integrality of the structure.

4 Prestressed reinforcements of prestressed concrete components should be anchored beyond the exterior face of the core of joint.

3.5.6 The seismic brace systems of single-storey fabricated factory building shall ensure the integrity and stability of the factory building during an earthquake.

3.6 Structural Analysis

3.6.1 The analysis for internal force and deformation of building structures on frequent earthquake level shall be carried out, unless otherwise provision is issued in this code. In this analysis, it may be assumed that the structure and its components are working at elastic state, so that the analysis of internal force and deformation may be adopted with the linear static/dynamic method.

3.6.2 For irregular building structures with obvious weak positions that may result in serious seismic damage, the elasto-plastic deformation analysis under the action of rare earthquake shall be carried out according to relevant provisions of this code. In this analysis, the elasto-plastic static analyzing method or elasto-plastic time history analyzing method may be adopted depending on the structural characteristics.

Where the specific provisions are specified in this code, the simplified methods calculating elasto-plastic deformations of the structures may be adopted, either.

3.6.3 When the gravity additional bending moment of structure under earthquake action is greater than 10% of original bending moment, the influence of gravity second-order effect shall be taken into consideration.

Note: The gravity additional bending moment is the product of the total gravity load at and above any one storey by the mean storey drift of this storey under earthquake; the original bending moment is the product of the seismic shear force of this storey by the storey height of the building.

3.6.4 In seismic analysis of structure, the floor and roof shall be assumed as the rigid, block rigid, semi-rigid or local flexible and flexible diaphragm depending on the deformation in slab plan and plan form, then the interaction behavior between lateral-force-resisting components may be determined according to the arrangement of lateral-force-resisting system, and then, the seismic internal forces of these components may be analyzed.

3.6.5 The structure having nearly symmetric distribution of masses and lateral rigidity and floor and roof regarded as rigid diaphragms as well as the structure with specific provisions of this code, may be carried out with seismic analysis by adopting two-dimensional structural model. All other structures shall adopt three-dimensional structural models to carry out the seismic analysis.

3.6.6 The seismic analyses of structures by computers shall meet the following requirements:

1 The determination of computation mode as well as necessary simplified calculation and treatment for a structure shall comply with the actual working condition of this structure, and the effects of stair components shall be involved in calculation.

2 The technical conditions of calculation software shall comply with this code and those specified in the relevant standards, and the contents and reference of special treatment shall also be clarified.

3 The analysis for internal force and deformation of complicate structures under frequent earthquake action shall be adopted with at least two different mechanical models, and the calculation results of these two models shall be analyzed and compared.

4 The rationality and validity of all the calculation results from the computer shall be analyzed, judged and affirmed, and after then they are permitted to be used in the project design.

3.7 Nonstructural Components

3.7.1 Nonstructural components include the nonstructural components and the auxiliary mechanical and electrical equipments of buildings, the connections of it and with the main structure body shall be carried out with seismic design.

3.7.2 The seismic design of nonstructural components shall be carried out by relevant professional personnel respeetively.

3.7.3 The nonstructural components attached to floor and roof structures as well as the non-bearing wall of the staircase shall be reliably connected or anchored to the major structure so that human injury or damage of important equipments induced by their collapse can be avoided.

3.7.4 For the arrangement of enclosure walls and partition walls of frame structures, their unfavorable effects on seismic performance of structure shall be estimated and the irrational arrangement of these walls that would cause damage to major structure shall be avoided.

3.7.5 Curtain wall and veneers shall be reliably connected to the major structure so that human injury due to their falling during earthquake can be avoided.

3.7.6 The supports and connections of the auxiliary mechanical and electrical equipments installed at the buildings shall meet the functional requirements under earthquake and shall not result in any damage to relevant portions.

3.8 Isolation and Energy-Dissipation

3.8.1 The isolation and energy-dissipation design may be applied to the buildings which have higher or special requirements on seismic safety and use function.

3.8.2 The buildings adopting the seismic isolation or energy-dissipation design may be designed higher than the basic fortification target stated in Article 1.0.1 of this code on meeting the influence of frequent earthquake, precautionary earthquake and rare earthquake.

3.9 Materials and Construction

3.9.1 The special requirements of seismic structures on materials and construction quality shall be clearly stated in the design documents.

3.9.2 The performance indexes of structural materials shall meet the following minimum requirements:

1 The materials of masonry structure shall meet the following requirements:

1) The strength grade of common brick and perforated brick shall not be less than MU10 and the strength grade of their masonry mortar shall not be less than M5;

2) The strength grade of small concrete hollow block shall not be less than MU7.5 and the strength grade of its masonry mortar shall not be less than Mb7.5.

2 The materials of concrete structures shall meet the following requirements:

1) The strength grades of concrete for frame-supported beams and columns as well as frame-supported beams and columns and node-core area assigned to seismic Grade 1 shall not be less than C30; and the strength grades of concrete for constructional columns, core columns, ring-beams and other components shall not be less than C20;

2) For the frames and diagonal bracing components (including the stair section) assigned to seismic Grade 1, 2 and 3, if ordinary reinforcements are used as their longitudinal bearing force reinforcements, then the ratio between the measured tensile strength and the measured yield strength of steel reinforcement shall not be less than 1.25, the ratio between the measured value and standard value of yield strength shall not be larger than 1.3, and the measured overall elongation of steel reinforcement under the maximum tensile stress shall not be less than 9%.

3 The steels of steel structures shall meet the following requirements:

1) The ratio between the measured yield strength and measured tensile strength of steels shall not be larger than 0.85;

2) The steels shall have obvious yield steps and their elongation rate shall not be less than 20%;

3) The steels shall have good weldability and qualified impact ductility.

3.9.3 The performance indexes of structural materials still should meet the following requirements:

1 The ordinary reinforcements with better ductility, tenacity and weldability should be preferred; for the strength grade of ordinary reinforcements, the longitudinal bearing force reinforcements should be selected the hot rolled reinforcements with seismic performance index no less than Grade HRB400 or may be adopted the Grade HRB335 hot rolled reinforcements; the stirrups should be selected the hot rolled reinforcements with seismic performance index no less than Grade HRB335 or may be adopted the Grade HPB300 hot rolled reinforcements.

Note: The inspection method of steel reinforcements shall comply with those specified in the current national standard "Code for Acceptance of Constructional Quality of Concrete Structures" GB 50204.

2 The concrete strength grade of concrete structures, like seismic wall, should not exceed C60, and should not exceed C60 for ntensity 9 and C70 for Intensity 8 for other components.

3 The steel type of steel structures should be selected the Grade Q235-B, C and D carbon structural steels and Grade Q345-B, C, D and E high strength low alloy structural steels; when reliable references are available, other types and grades of steels also may be adopted.

3.9.4 During the construction, if the longitudinal bearing force reinforcements in original design have to be replaced by those with higher strength grade, the conversion shall be made according to equal tensile capacity design values of such reinforcements, and shall also comply with requirement of minimum reinforcement ratio.

3.9.5 For steel structures adopting welded connections, if Intensity of welding restraint of joints is relatively big, the thickness of steel plate is not less than 40mm and the steel plates bear the tensile force along the plate thickness direction, then the size contraction rate in the thickness direction of thickness shall not be less than the permissible value of Grade Z15 specified in the national standard "Steel Plate with Through-thickness Characteristics" GB/T 5313.

3.9.6 As for the masonry seismic walls in buildings adopting reinforced concrete constructional columns and R.C. frames on ground floors and seismic walls, the wall shall be laid out prior to casting the constructional columns and frame beam-columns.

3.9.7 The horizontal construction joints of concrete wall and frame column shall be taken with measures to strengthen the bonding property of concrete. For the seismic Grade I wall and the connection part between transition storey slab and ground concrete wall, the shear bearing capacity of the section of horizontal construction joint shall be checked and calculated.

3.10 Performance-Based Design of Buildings

3.10.1 When performance-based seismic design is conducted for a building structure, the technical and economic feasibility comprehensive analysis and argumentation on the selected seismic performance target shall be conducted according to the following factors: precautionary category, precautionary Intensity, site conditions, structure type and irregularity, requirements on use functions of building and ancillary facilities, investment size, post-disaster loss and reconstruction easiness.

3.10.2 According to the actual requirement and possibility, the performance-based seismic design of building structure shall have pertinency and may select performance objectives respectively on the whole structure, partial or key positions, critical, important and secondary components of the structure, building components and supports for mechanical and electrical equipments.

3.10.3 The performance-based seismic design of building structure shall meet the following requirements:

1 The seismic motion level shall be selected. For the structures with the design life of 50 years, the earthquake action of frequent earthquake, rare earthquake and precautionary earthquake may be chosen, thereinto, the acceleration of the precautionary earthquake shall be adopted according to the design basic acceleration of ground motion listed in Table 3.2.2, and the maximum seismic influence coefficient of the precautionary earthquake may respectively be 0.12, 0.23, 0.34, 0.45, 0.68 and 0.90

for Intensity 6, Intensity 7 (0.10*g*), Intensity 7 (0.15*g*), Intensity 8 (0.20*g*), Intensity 8 (0.30*g*) and Intensity 9. For the structure with the design life of over 50 years, the earthquake action shall be properly adjusted through special study in consideration of the actual requirement and possibility. For the structures within 10km on both sides of the shock fracture, the ground motion parameter shall be considered in near-field influence; for those within 5km on both sides of the shock fracture, the ground motion parameter should be multiplied by a enhancement coefficient of 1.5, and the ground motion parameter of the structures outside of 5km should be multiplied by a enhancement coefficient of no less than 1.25.

2 The selected performance objectives, also the expected damaged condition or functions of use corresponding to different levels of ground motion, shall not be lower than the requirements of basic precautionary objective specified in Article 1.0.1 of this code.

3 The performance design indexes shall be selected. In the design, the specific indexes to improve the seismic bearing capacity and deformability of the structure or its vital parts respectively or simultaneously shall be selected; the uncertainty of action value selection of earthquake with different levels shall be considered and proper clearance shall be left. In the design, the requirements on the bearing capacity of horizontal and vertical components at different positions of a structure under different ground motion levels shall be determined (including not occurring brittle shear failure, forming plastic hinge, or reaching yield value or maintaining elasticity, etc.); the expected elasticity or elasto-plastic deformation conditions of different positions of a structure under different ground motion levels should be selected, and the high, medium and low requirements of ductile construction of the corresponding component also should be determined. If the bearing capacity of component is improved obviously, the corresponding ductile construction may be reduced properly.

3.10.4 The calculation of the performance-based seismic design of building structure shall meet the following requirements:

1 The analytical model shall correctly and reasonably reflect the transmission route of the earthquake action and reflect whether the integral or partial floor is at elasticity working state at different ground motion levels.

2 Linear method may be adopted for elasticity analysis; and equivalent linearization method (damping increase) and static or dynamic nonlinear analysis method may be respectively used for elasto-plastic analysis according to the elasto-plastic state of structure expected by the performance objective.

3 In relation to the elasticity analysis model, the nonlinear analysis model for the structure may be simplified properly, but the linear analysis results of above two models under frequent earthquake condition shall be basically consistent; the gravity second-order effect shall be considered and the elasto-plastic parameters shall be determined reasonably; the bearing capacity may be calculated according to the actual section and reinforcement of the component; the comparative analysis with the supposed calculated results of ideal elasticity may be carried out to find out the possible damaged parts and the elasto-plastic deformation degree of the component.

3.10.5 The reference objects and calculation methods for the performance-based seismic design of the structure and its components may be adopted according to those specified in Section M.1 of Appendix M in this code.

3.11 Seismic Response Observation System of Buildings

3.11.1 For the large-scale public buildings higher than 160m, 120m and 80m with seismic precautionary Intensity of 7, 8 and 9 respectively, the seismic response observation system of building structures shall be set as required, so the building design shall reserve spaces for the observation instruments and relevant circuits.

4 Site, Base and Foundation

4.1 Site

4.1.1 When selecting the building site, the favorable, common, unfavorable and hazardous sections to seismic protection of buildings shall be divided according to Table 4.1.1.

Table 4.1.1 Division of Favorable, Common, Unfavorable and Hazardous Sections

Section type	Geological, topographical and geomorphic description
Favorable section	Stable rock bed, stiff soil, or wide-open, even, compacted and homogeneous medium-stiff soil
Common section	Sections not belonging to the favorable, unfavorable and hazardous sections
Unfavorable section	Soft soil, liquefied soil; stripe-protruding spur; lonely tall hill, steep slops, steep step, river bank or boundary of side slops, soil layer having obviously heterogeneous cause of formation, rock character and state in plane (including abandoned river beds, loosened fracture zone of fault, and hidden swamp, creek, ditch and pit, as well as base formatted with excavated and filled), plastic loess with high moisture, ground surface with structural fissure, etc
Hazardous section	Places where landslide, collapse, land subsidence, ground fissure and debris flow may occur during the earthquake, as well as the positions in causative fault where ground dislocation may occur

4.1.2 The classification of building site class shall be subject to the equivalent shear wave velocity of soil layer and the cover layer thickness of site.

4.1.3 The measurement of the shear wave velocity of soil layer shall meet the following requirements:

1 At the stage of primary investigation of the site, for the same geologic units in large area, the number of drilling holes for testing the shear wave velocity of soil layer should not be less than 3.

2 At the stage of detailed investigation of the site, for every building, the number of drilling holes for testing the shear wave velocity of soil layer should not be less than 2; if the data varies significantly, the number may be increased properly. For the crowded building complex in one community, which are built in one same geologic unit, such number may be reduced properly but that for each tall building and each large-span spatial structure shall not be less than one.

3 For buildings assigned to Category D or to Category C with no more than 10 storeys and no more than 30m in height, if the measured shear wave velocity is not available, the shear wave velocity of each soil layer may be estimated within the range of shear wave velocity specified in Table 4.1.3 based on the name and character of rock-soil and according to the soil types listed in Table 4.1.3 and the local experiences.

Table 4.1.3 Classification of Soil Types and Scope of Shear Wave Velocity

Soil type	Name and character of rock-soil	Scope of shear wave velocity of soil layer (m/s)
Rock	Stiff, hard, and complete rocks	v_s>800
Stiff soil or soft sock	Broken and comparatively broken rock; soft and comparatively soft rock; compact gravel soil	800≥v_s>500
Medium-stiff soil	Medium dense or slightly dense detritus; dense or medium-dense gravel; coarse or medium sands; cohesive soil and silt with f_{ak}>150kPa; hard loess	500≥v_s>250

Table 4.1.3 (continued)

Soil type	Name and character of rock-soil	Scope of shear wave velocity of soil layer (m/s)
Medium-soft soil	Slightly dense gravel; coarse and medium sands; fine and mealy sands (excluding the loose sand), cohesive soil and silt with f_{ak}≤150kPa; filled soil with f_{ak}>130kPa; plastic young loess	$250 \geq \upsilon_s > 150$
Soft soil	Mud and muddy soil; loose sand; new sedimented cohesive soil and silt; filled soil with f_{ak}≤130kPa; flow plastic loess	$\upsilon_s \leq 150$

Note: f_{ak} is the characteristic value (kPa) of base bearing capacity obtained through load test or other methods; υ_s is the shear wave velocity of rock-soil.

4.1.4 The cover layer thickness at the building site shall be determined according to the following requirements:

1 Generally, this thickness shall be determined according to the distance from the ground surface to the top surface of a soil layer, under which the shear wave velocity is more than 500m/s and the shear wave velocity of the soil layers under it is no less than 500m/s.

2 If such soil layer with shear wave velocity is more than 2.5 times of that of the soil layers above it exists 5m under the ground surface and the shear wave velocity of this soil layer and those under it all is less than 400m/s, then the cover layer thickness may be determined according to the distance from the ground surface to the top surface of this soil layer.

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

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

4.1.5 The equivalent shear wave velocity of soil layer shall be calculated according to the following formula:

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

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

Where υ_{se}——Equivalent shear wave velocity of soil layer (m/s);

d_0——Calculation depth (m), which shall be taken as the smaller value between the cover layer thickness and 20m;

t——Travel time of shear wave between the ground and the calculation depth;

d_i——Thickness (m) of the i^{th} soil layer within the range of calculation depth;

υ_{si}——Shear wave velocity (m/s) of the i^{th} soil layer within the range of calculation depth;

n——Number of soil layers within the range of calculation depth.

4.1.6 The site class of buildings shall be classified into four class (Class I consists of two subclasses: I_0 and I_1) according to Table 4.1.6 based on the equivalent shear wave velocity of soil

layer and the cover layer thickness of the site. If reliable shear wave velocity and cover layer thickness are available and their values are near to the divisional line of site class listed in Table 4.1.6, then it shall be allowed to determine the characteristic period for earthquake action calculation with interpolation method.

Table 4.1.6 Cover Layer Thickness (m) of Various Building Sites

Shear wave velocity of rock or equivalent shear wave velocity of soil (m/s)	Site class				
	I_0	I_1	II	III	IV
$v_s>800$	0				
$800\geq v_s>500$		0			
$500\geq v_{se}>250$		<5	≥5		
$250\geq v_{se}>150$		<3	3~50	>50	
$v_{se}\leq150$		<3	3~15	15~80	>80

Note: In the above table, v_s refers to the shear wave velocity of rock.

4.1.7 If causative fault exists in the site, the impact of causative fault on the project shall be evaluated and the following requirements shall be met:

1 For the conditions meeting any one of the following requirements, the impact of causative fault dislocation on the surface structures may be neglected:

1) The seismic precautionary Intensity is less than 8;

2) Not holocene active faults;

3) For Intensity 8 and 9, the soil layer coverage for the hidden 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 article, the main fault zone shall be avoided in the selection of site. The avoidance distance should not be less than the minimum avoidance distance for causative fault specified in Table 4.1.7. If scattered Category C and D buildings with less than 3 storeys are required to be built within the scope of avoidance distance, the seismic measures of one intensity higher shall be taken, the integrity of foundation and superstructure shall be improved, and the building must not cross over any fault trace.

Table 4.1.7 Minimum Avoidance Distance of Causative Fault (m)

Intensity	Precautionary category of building			
	A	B	C	D
8	Special study	200m	100m	—
9	Special study	400m	200m	—

4.1.8 When Category C and above buildings are to be built in such unfavorable sections as the stripe-protruding spur, lonely tall hill, non-rocky or highly weathered rocky steep slop, river banks or boundary of slopes, not only the stability amplified action buildings under the earthquake action shall be guaranteed, but also the possible amplified action of unfavorable sections to the design parameters of ground motion shall be estimated, the maximum value of horizontal seismic influence coefficient shall be multiplied by the enhancement coefficient, and the value of the enhancement coefficient shall be determined from 1.1 to 1.6 depending on the specific conditions of the unfavorable section.

4.1.9 The geotechnical engineering investigation of the site shall be carried out as follows: The evaluation on the site class of building and the seismic stability of rock-soil (including landslide, collapse, liquification and seismic subsidence characteristics) shall be provided according to the favorable, common, unfavorable and hazardous sections to buildings classified for actual requirements. For the buildings requiring time history analysis method in addition, the soil profile, the cover layer thickness of site and the relevant kinetic parameters still shall be provided according to the design requirements.

4.2 Natural Base and Foundation

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

1 The buildings that may not be carried out with the superstructure seismic check as specified in this code.

2 The following buildings without soft cohesive soil layer within the main load-bearing layer of base:

1) Ordinary single-storey factory buildings and single-storey spacious buildings;

2) Masonry buildings;

3) Ordinary civil framed buildings and buildings with frame-seismic wall not exceeding 8 storeys and 24m in height;

4) Multi-storey frame factory buildings and multi-storey buildings with concrete seismic wall, of which the foundation load is equivalent to those specified in Item 3).

Note: The soft cohesive soil layer refers to the soil layer with characteristic value of base bearing capacity less than 80, 100 and 120 respectively for Intensity 7, 8 and 9.

4.2.2 To conduct seismic check for the natural base foundation, the characteristic combination of earthquake action effects shall be adopted, and the base seismic bearing capacity shall be the characteristic value of base bearing capacity multiplied by the adjustment coefficient of base seismic bearing capacity.

4.2.3 The seismic bearing capacity of base shall be calculated according to the following formula:

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

Where f_{aE}——Adjusted seismic bearing capacity of base;

ζ_a——Adjustment coefficient of base seismic bearing capacity, which shall be adopted according to Table 4.2.3;

f_a——Characteristic value of base bearing capacity after depth and width correction, which shall be adopted according to the current national standard "Code for Design of Building Foundation" GB 50007.

4.2.4 To check the vertical bearing capacity of natural base under the earthquake action, the mean pressure on bottom and the maximum pressure on edge of foundation according to the standard combination of earthquake action effects shall meet the requirements of the following formulae:

Table 4.2.3 Adjustment Coefficient of the Seismic Bearing Capacity of Base

Name and character of rock-soil	ζ_a
Rock, dense detritus, dense gravel, course and medium sands, cohesive soil and silt with $f_{ak}\geq300$kPa	1.5
Medium dense and slightly dense detritus, medium dense and slightly dense gravel, course and medium sands, dense and medium dense fine and mealy sands, cohesive soil and silt with 150kPa$\leq f_{ak}<$300kPa, and hard loess	1.3
Slightly dense fine and mealy sands, cohesive soil and silt with $100\leq f_{ak}<150$, and plastic loess	1.1
Mud, muddy soil, loose sand, miscellaneous fill soil, newly piled loess and streamed loess	1.0

$$p\leq f_{sE} \tag{4.2.4-1}$$

$$p_{max}\leq1.2f_{sE} \tag{4.2.4-2}$$

Where p——Mean pressure on foundational bottom according to the standard combination of earthquake action effects;

p_{max}——Maximum pressure on foundation edge according to the standard combination of earthquake action effects. For the tall buildings with height-width ratio above 4, the foundation bottom should not appear with any abscission zone (zero stress zone) under the earthquake action; for the other buildings, the area of the abscission zone (zero stress zone) between the foundation bottom and foundation soil shall not exceed 15% of the area of foundation bottom.

4.3 Liquefied Soil and Soft Soil Bases

4.3.1 The liquification evaluation of saturated sandy soil and saturated silt (excluding loess) and the base treatment may not be carried out under common conditions for Intensity 6, however, those of the Category B buildings that are sensitive to liquefaction settlement shall be carried out according to the requirements for Intensity 7 and those of Category B buildings for Intensity 7~9 may be carried out according to the requirements of local seismic precautionary Intensity.

4.3.2 If saturated sandy soil and saturated silt exist under ground, buildings, except for those for Intensity 6, all shall be carried out with the liquification evaluation; for the base with liquefied soil layer, corresponding measures shall be taken according to the precautionary category of building and liquification degree of base, in combination of the specific conditions.

Note: The requirements for the liquification evaluation of saturated soil specified in this article do not include loess and powdery clay.

4.3.3 The saturated sandy soil or silt (excluding loess), if meeting any one of the following conditions, may be preliminarily evaluated as non-liquefaction or the influence of liquification may not be considered.

1 The geologic time of soil is Epipleistocene of Quaternary (Q_3) or earlier, and they may be evaluated as non-liquefied soils for Intensity 7 and 8.

2 If the percentage content of sticky particles (particles with grain size less than 0.005mm) in silt is not less than 10%, 13% and 16% for Intensity 7, 8 and 9 respectively, the soil may be evaluated as non-liquefied soil.

Note: The sticky particle content used for liquification evaluation is measured by using hexametaphosphate as dispersion agent, and it shall be converted according to relevant regulations if other methods are adopted.

3 For buildings with shallowly-buried natural base, if the thickness of upper covered non-liquefied soil and the depth of underground water level meet any one of the following conditions, the liquification influence may not be considered.

$$d_u > d_0 + d_b - 2 \tag{4.3.3-1}$$

$$d_w > d_0 + d_b - 3 \tag{4.3.3-2}$$

$$d_u + d_w > 1.5d_0 + 2d_b - 4.5 \tag{4.3.3-3}$$

Where d_w——Depth of underground water level (m), which should be adopted according to the mean annual highest water level within the design reference period or may be adopted according to the annual highest water level in recent years;

d_u——Thickness of upper covered non-liquefied soil layer (m), in which the thickness of mud and muddy soil layers should be deducted;

d_b——Embedded depth of foundation (m), which shall be 2m if it is less than 2m;

d_0——Characteristic depth of liquefied soil (m), which may be adopted according to Table 4.3.3.

Table 4.3.3 Characteristic Depth of Liquefied Soil (m)

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

Note: If the underground water level in this regions is under variable condition, the characteristic depth shall be considered according to unfavorable conditions.

4.3.4 If the preliminary discrimination of saturated sandy soil and silt indicates further liquification evaluation is necessary, standard penetration test shall be adopted to discriminate the liquification condition of the soil within 20m (deep) under the ground; but for the buildings that may not be carried out with seismic capacity check of natural base and foundation as specified in Article 4.2.1, the liquification condition of the soil within only 15m (deep) under the ground may be evaluated. If the standard penetration blow count of saturated soil (without pole length correction) is less than or equal to the standard penetration blow count for liquification evaluation, the soil shall be evaluated as liquefied soil. If mature experiences are available, other evaluation methods may also be adopted.

Within the depth range of 20m under the ground, the critical value of standard penetration blow count for liquification evaluation may be calculated according to following formula:

$$N_{cr} = N_0 \beta [\ln(0.6d_s + 1.5) - 0.1d_w] \sqrt{3/\rho_c} \tag{4.3.4}$$

Where N_{cr}——Critical value of standard penetration blow count for liquification evaluation;

N_0——Reference value of standard penetration blow count for liquification evaluation, which may be adopted according to Table 4.3.4;

d_s——Depth of standard penetration point for saturated soil (m);

d_w——Underground water level (m);

ρ_c——Percentage content of sticky particles, which shall be taken as 3 if it is less than 3 or the soil is sandy soil;

β——Adjustment coefficient, which shall be taken as 0.80 for design earthquake Group 1, 0.95 for Group 2 and 1.05 for Group 3.

Table 4.3.4 Reference Value (N_0) of Standard Penetration Blow Count for Liquification Evaluation

Design basic acceleration of ground motion (g)	0.10	0.15	0.20	0.30	0.40
Referebce value of standard penetration blow count for liquification evaluation	7	10	12	16	19

4.3.5 For the base with liquefied sandy soil layer and silt layer, the depth and thickness of the liquefied soil layers shall be explored, the liquification index of each drilling hole shall be calculated according to the following formula, and the liquification grade of base shall be comprehensively classified according to Table 4.3.5:

$$I_{lE}=\sum_{i=1}^{n}[1-\frac{N_i}{N_{cri}}]d_iW_i \tag{4.3.5}$$

Where I_{lE}——Liquification index;

n——Total number of standard penetration test points for each drilling hole within the evaluated depth range;

N_i, N_{cri}——Measured value and the critical value of the standard penetration blow count at the i^{th} point respectively, and the critical value shall be taken if the measured value is larger than the critical value; if the liquification within a scope of 15m only needs to be evaluated, the measured value bellow 15m may be adopted according to critical value;

d_i——Thickness of soil layer represented by the i^{th} point, which may be taken as half the difference between the depth values at these two upper and lower standard penetration test points adjoining this standard penetration test point, however, its upper limit shall not be larger than the depth of underground water level and its lower limit shall not be deeper than the liquification depth;

W_i——Weight horizon influence function value (unit: m^{-1}) of unit thickness of the i^{th} soil layer, which shall be taken as 10 if the depth of midpoint of this layer is no larger than 5m, zero if the depth is equal to 20m, and be taken with value with linear interpolation if the depth is from 5m~20m.

Table 4.3.5 Corresponding Relationship between Liquification Grade and Liquification Index

Liquification grade	Light	Moderate	Serious
Liquification index I_{lE}	$0<I_{lE}\leq6$	$6<I_{lE}\leq18$	$I_{lE}>18$

4.3.6 If the liquefied sandy soil layer and silt layer is relatively even and uniform, the anti-liquefaction measures for base should be selected according to Table 4.3.6; the affect of gravity load of superstructure may also be considered, the anti-liquefaction measures shall be properly adjusted according to the estimated seismic subsidence amount due to liquification. The untreated liquefied soil layer should not be taken as the supporting course of natural base.

Table 4.3.6 Anti-liquefaction Measures

Seismic precaution of building	Liquification grade of base		
Precautionary category	Light	Moderate	Serious
B	Eliminating the liquefaction settlement partially or treating the foundation and superstructure	Eliminating the liquefaction settlement wholly, or eliminating the liquefaction settlement partially and treating the foundation and superstructure	Eliminating the liquefaction settlement wholly
C	Treating the foundation and superstructure, or taking no measures	Treating the foundation and superstructure, or taking measures of much higher requirement	Eliminating the liquefaction settlement wholly, or eliminating the liquefaction settlement partially and treating the foundation and superstructure
D	May not take measures	May not take measures	Treating the foundation and superstructure, or taking other economical measures

Note: Special studies shall be conducted for the anti-liquefaction measures of the base of Category A buildings, however, the anti-liquefaction measures should be greater than or equal to the corresponding requirements of Category B buildings.

4.3.7 The measures for eliminating the liquefaction settlement of base wholly shall meet the following requirements:

1 When pile foundation is used, the length (pile-tip not included) of the pile tip driven into the stable soil layer below the liquefaction depth shall be determined through calculation, which shall not be less than 0.8m for detritus, gravel, coarse and medium sands, stiff cohesive soil, and dense silt and should not be less than 1.5m for other non-rocky soil.

2 When deep foundation is used, the bottom of foundation shall be embedded in the stable soil layer below the liquefaction depth, and the embedded depth shall not be less than 0.5m.

3 When a compaction method (e.g. vibroflotation, vibration compaction, gravel pile compaction, and dynamic compaction) is used for strengthening, compaction shall be carried out down to the lower margin of liquefaction depth; after strengthening the gravel piles by vibroflotation or compaction, the standard penetration blow count of soil between piles should not be less than the critical value of standard penetration blow count for liquification evaluation as specified in Article 4.3.4 of this code.

4 The totally liquefied soil layer shall be replaced with non-liquefied soil, or the thickness of the upper covered non-liquefied soil layer shall be increased.

5 When treating by adopting compaction method or soil replacement method, the treatment width outside the foundation edge shall exceed 1/2 of the treatment depth bellow the foundation bottom and shall not be less than 1/5 of the foundation width.

4.3.8 The measures for partially eliminating the liquefaction settlement of base shall meet the following requirements:

1 The treatment shall be carried out to a depth so that the liquefaction index of base is reduced after treatment, and the liquefaction index should not be larger than 5; for central zone of large area raft foundation and box foundation, the liquefaction index after treatment hereof may be reduced to 4; for individual foundation and strip foundation, the liquification index also shall not be less than the larger value between characteristic depth of liquefied soil under the foundation bottom and the foundation width.

Note: The central zone refers to the zone located within the foundation outer edge, and within over 1/4 length of the corresponding direction along the length/width direction away from the outer edge.

2 After strengthening through vibroflotation or gravel pile compaction, the standard penetration blow count of soil between piles should not be less than the critical value of standard penetration blow count for liquification evaluation as specified in Article 4.3.4 of this code.

3 The treatment width outside the foundation edge shall meet the requirements of Clause 5 in Article 4.3.7 of this code.

4 Other measures to reduce seismic subsidence due to liquefaction shall be adopted, like thickening the upper covered non-liquefied soil layer and improving the peripheral drainage condition, etc..

4.3.9 The treatment of foundation and superstructure to reduce the liquification affect may be adopted with the following measures comprehensively:

1 Appropriate embedded depth of foundation shall be selected.

2 The bottom area of foundation shall be adjusted to reduce the eccentricity of foundation.

3 The integrity and stiffness of foundation shall be strengthened, for instance, adopting box foundation, raft foundation or reinforced concrete cross strip foundation, installing foundation ring beams in addition, etc..

4 The loads shall be reduced, the integral stiffness and uniform symmetry of superstructure shall be reinforced, the settlement joints shall be arranged reasonably, and the adoption of such structure form sensitive to differential settlement shall be avoided.

5 Adequate size shall be reserved at the position where pipelines pass through the buildings, or the flexible joints shall be adopted.

4.3.10 No permanent buildings should be constructed within the sections with possibility of lateral expansion of liquification or flowsliding, such as the fossil river course and sections near to the river bank, seacoast and side slope; otherwise, anti-sliding checks shall be carried out, and anti-sliding measures for the earth and anti-cracking measures for the structure shall be taken.

4.3.11 The seismic subsidence evaluation of soft cohesive soil layer in the base may be adopted with the following methods. The hazardness of the seismic subsidence of saturated powdery clay and the anti-seismic subsidence measures shall be determined through comprehensive study according to such factors as degree of subsidence and lateral deformation, the saturated powdery clay, for Intensity 8 (0.30g) and Intensity 9, with the plasticity index less than 15 and meeting the requirements of the following formula, may be judged as seismic subsidence soft soil.

$$W_s \geq 0.9W_L \tag{4.3.11-1}$$

$$I_l \geq 0.75 \tag{4.3.11-2}$$

Where W_s——Natural moisture content;

W_L——Moisture content on liquid limit, which is measured by adopting with the liquid and plastic limit methods jointly;

I_L——Liquidity index.

4.3.12 If soft cohesive soil layer and collapsible loess with high moisture content exist within the scope of main bearing layer of base, the strengthening treatment of pile foundation and base or the measures specified in Article 4.3.9 of this code may be adopted through comprehensive consideration in combination of the specific conditions, or the corresponding measures may also be adopted according to the seismic subsidence amount of soft soil.

4.4 Pile Foundations

4.4.1 The following buildings, of which the low-cap pile foundation mainly bearing vertical loads and there is no liquefied soil layer under the ground, no mud or muddy soil and no filled soil with characteristic value of base bearing capacity no larger than 100kPa surrounding the pile cap, may not be carried out with the seismic bearing capacity check of pile foundation:

1 The following buildings for Intensity 7 and 8:

1) Ordinary single-storey factory buildings and single-storey spacious buildings;

2) Ordinary civil framed buildings not exceeding 8 storeys and 24m in height;

3) Multi-storey framed factory buildings and multi-storey buildings with concrete seismic wall , for which the foundation load is equivalent to that specified in Item 2).

2 Buildings adopting pile foundation among those specified in Item 1 and Item 3 of Article 4.2.1 of this code.

4.4.2 The seismic check of low-cap pile foundation in non-liquefied soil shall meet the following requirements:

1 Both the characteristic values of vertical and horizontal seismic bearing capacity of individual pile may be increased by 25% than those of non-seismic design.

2 When the back filling soil surrounding the pile cap is tamped so that the dry density is no less than the requirements on filled soil as specified in the current national standard "Code for Design of Building Foundation" GB 50007; the front filled soil of cap and the pile may together undertake the horizontal earthquake action; however, the frictional force between the bottom surface of cap and the foundation soil shall not be taken into account.

4.4.3 The seismic check of such low-cap pile foundation with liquefied soil layer shall meet the following requirements:

1 If the cap is buried shallowly, the resisting force of soil surrounding the cap and the sharing action of rigid terrace to the horizontal earthquake action should not be taken into account.

2 If non-liquefied soil layers or non-soft soil layers in thickness no less than 1.5m or 1.0m respectively exist up and down the bottom surface of pile cap, the seismic check of pile may be carried out according to the following two kinds of conditions, and the design shall be carried out according to unfavorable conditions:

1) The pile shall be designed to bear all the earthquake action, its bearing capacity shall be taken according to Article 4.4.2 of this code, and both the frictional resistance and horizontal resisting force of pile in liquefied soil shall be multiplied by the reduction coefficient listed in Table 4.4.3.

Table 4.4.3 Reduction Coefficient for Liquification Affect of Soil Layer

Actual standard penetration blow count/critical standard penetration blow count	Depth d_s (m)	Reduction coefficient
≤0.6	$d_s \leq 10$	0
	$10 < d_s \leq 20$	1/3
>0.6~0.8	$d_s \leq 10$	1/3
	$10 < d_s \leq 20$	2/3
>0.8~1.0	$d_s \leq 10$	2/3
	$10 < d_s \leq 20$	1

2) The earthquake action shall be adopted according to 10% of the maximum value of horizontal seismic influence coefficient, the pile bearing capacity still shall be adopted according to Item 1 in Article 4.4.2 of this code, however, all the frictional resistance of liquefied soil layer and the frictional resistance of pile in non-liquefied soil within a depth range of 2m bellow the pile cap shall be deducted.

3 For the precast driven piles and other squeezing earth piles, if the average pile spacing is 2.5~4 times of the pile diameter and the pile quantity is no less than 5×5, the compaction effect of driving to soil and the favorable effect of pile body to the deformation restriction of liquefied soil shall be taken into account. Provided that the standard penetration blow count of the soil between driven piles meets the non-liquification requirements, the bearing capacity of individual pile may not be reduced, however, the stress dispersion angle outside the pile group shall be taken as zero when doing strength check for the supporting course of pile tip. The standard penetration blow count of soil between piles after driving should be determined through tests or also may be calculated according to following formula:

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

Where N_1——Standard penetration blow count after driving;

ρ——Displacement ratio of area of precast driven pile;

N_p——Standard penetration blow count before driving.

4.4.4 The space surrounding the cap of pile foundation in liquefied soil should be filled and tamped with solid dry soil, if it is filled with sandy soil or silt, then the standard penetration blow count of the soil layer shall not be less than the critical value of standard penetration blow count for liquification evaluation as specified in Article 4.3.4 of this code.

4.4.5 The reinforcement range of pile in liquefied soil and seismic subsidence soft soil shall cover from the pile top down to a level under the liquification depth and shall meet the depth required for eliminating the liquefaction settlement wholly. The longitudinal reinforcements shall be same as the pile top and the stirrups shall be thickened and densed.

4.4.6 In the sections where lateral expansion of liquification exists, the pile foundation, besides meet the other requirements of this section, also shall be considered with the lateral acting force when the soil flows, and the area bearing lateral thrust shall be calculated according to the width between the outer edges of side piles.

5 Earthquake Action and Seismic Checking for Structures

5.1 General Requirements

5.1.1 The earthquake actions of various building structures shall meet the following requirements:

1 Generally, the horizontal earthquake actions shall be at least considered and checked separately along the two major axial directions of the building structure; and the horizontal earthquake action in each direction shall be bear by the lateral-force-resisting components in this direction.

2 For the structures having the oblique lateral-force-resisting components, if the intersection angle is greater than 15°, the horizontal earthquake action along the direction of each lateral-force-resisting component shall be calculated respectively.

3 Structures having obvious asymmetric mass and rigidity distribution, the torsion affects caused by horizontal earthquake actions in two directions shall be considered; for other structures, it is permitted to count in the torsion affects by adjusting the earthquake action effect.

4 As for the large-span structures and long-cantilevered structures for Intensity 8 or 9 and the tall buildings for Intensity 9, the vertical earthquake action shall be calculated.

Note: For building structures adopting seismic isolation for Intensity 8 and 9, the vertical earthquake action shall be calculated according to the relevant regulations.

5.1.2 The seismic calculation of various building structures shall be adopted with the following methods:

1 For structures, which are not higher than 40m, mainly have shear deformation and a rather uniform distribution of mass and rigidity along the height direction, or for the structures similar as a single-mass system, the simplified methods, such as base shear method, may be adopted.

2 For building structures other than those as stated Item 1, the mode-decomposition response spectrum method should be adopted.

3 The especially irregular buildings, the Category A buildings and the tall buildings belonging to the height range listed in Table 5.1.2-1 shall have the additional calculation under frequent earthquakes by adopting with time-history analysis method; when adopting three groups of acceleration-time-history curves, the calculation result should be taken as the envelope value with time-history method and the greater value with mode-decomposition response spectrum method; when adopting 7 or more time-history curves, the calculation result may be taken as the mean value with time-history method and the greater value with mode-decomposition response spectrum method.

When the time-history analysis method is adopted, the actual strong earthquake records and the artificially simulated acceleration time-history curve shall be selected according to the building site class and the design earthquake group, thereinto, at least 2/3 of strong earthquake records shall be provided, the average seismic influence coefficient curve of several groups of time-history curves shall be in conformity with the seismic influence coefficient curve adopted for mode-decomposition response spectrum method in the level of statistics, and the maximum value of acceleration time history may be adopted according to Table 5.1.2-2. When analyzing with elastic time history, the structure base shear force calculated with each time-history curve shall not be less than 65% of the calculation result with mode-decomposition response spectrum method, and the mean value from several time-history curves shall not be less than 80% of the calculation result with mode-decomposition response spectrum method.

Table 5.1.2-1 Height Range of Buildings Adopting Time-history Analysis Method

Intensity and site class	Height range of building (m)
Intensity 7, and Class I and II sites for Intensity 8	>100
Class III and IV sites for Intensity 8	>80
Intensity 9	>60

Table 5.1.2-2 Maximum Value of Earthquake Acceleration Time History Used for Time-history Analysis (cm/s^2)

Earthquake effect	Intensity 6	Intensity 7	Intensity 8	Intensity 9
Frequent earthquake	18	35 (55)	70 (110)	140
Rare earthquake	125	220 (310)	400 (510)	620

Note: The values in parentheses are used for the zones where the design basic acceleration of ground motion is 0.15g and 0.30g.

4 The deformation calculation of a structure under rare earthquake shall be adopted with simplified elasto-plastic analysis method or elasto-plastic time-history analysis method according to those specified in Section 5.5 of this code.

5 For the space structures with very large horizon projection size, the seismic calculation shall be conducted in the input mode of simple point uniformity, multi-point and multiway simple point or multiway multi-point according to the structure form and supporting condition. In multi-point input calculation, the travelling earthquake wave effect and local site effect shall be taken into consideration. In Class I and II sites for Intensity 6 and 7, the seismic check of supporting structures, superstructures and foundations may be adopted with simplified methods, and the components of short edges may be multiplied by a additional earthquake action effect coefficient of 1.15~1.30 according to the different span and length of structures; in Class III and IV sites for Intensity 7 and the sites for Intensity 8 and 9, the seismic check shall be carried out by adopting with time-history analysis method.

6 The seismic isolation and energy dissipation/shock absorption design of building structures shall be adopted with the calculation methods specified in Chapter 12 of this code.

7 The design of subterranean building structures shall be adopted with the calculation methods specified in Chapter 14 of this code.

5.1.3 In the calculation of earthquake action, the representative value of the gravity load of building shall be taken as the sum of the standard value of deadweight of structure and its components and accessories and the combination value of variable loads. The coefficient for combination value of variable loads shall be adopted according to Table 5.1.3.

Table 5.1.3 Coefficient for Combination Values

Type of variable load	Coefficient for combination value
Snow load	0.5
Dust load on the roof	0.5
Live load on the roof	Not considered
Live load on floor, calculated according to actual conditions	1.0

Table 5.1.3 (continued)

Type of variable load		Coefficient for combination value
Live load on floor, calculated according to equivalent uniform load	Library and archives	0.8
	Other civil buildings	0.5
Gravity of objects suspended by crane	Crane with hard hook	0.3
	Crane with flexible hook	Not considered

Note: The crane with hard hook is with high hoisting capacity, and the coefficient for combination value shall be adopted depending on actual conditions.

5.1.4 The seismic influence coefficient of a building structure shall be determined according to the Intensity, site class, design earthquake group, and natural vibration period and damping ratio of structure; the maximum value of its horizontal seismic influence coefficient shall be adopted according to Table 5.1.4-1; the characteristic period shall be adopted according to Table 5.1.4-2 based on the site class and design earthquake group, and the characteristic period shall be increased by 0.05s to calculate the rare earthquake action.

Note: For the building structures with period larger than 6.0s, the seismic influence coefficient to be adopted shall be studied especially.

Table 5.1.4-1 Maximum Value of Horizontal Seismic Influence Coefficient

Earthquake effect	Intensity 6	Intensity 7	Intensity 8	Intensity 9
Frequent earthquake	0.04	0.08 (0.12)	0.16 (0.24)	0.32
Rare earthquake	0.28	0.50 (0.72)	0.90 (1.20)	1.40

Note: The values in parentheses are used for the zones where the design basic acceleration of ground motion is $0.15g$ and $0.30g$.

Table 5.1.4-2 Values of Characteristic Period (s)

Design earthquake group	Site class				
	I_0	I_1	II	III	IV
Group 1	0.20	0.25	0.35	0.45	0.65
Group2	0.25	0.30	0.40	0.55	0.75
Group 3	0.30	0.35	0.45	0.65	0.90

5.1.5 The damping adjustment and the form parameters of the seismic influence coefficient curve (Figure 5.1.5) of building structure shall meet the following requirements:

1 Unless otherwise specified, the damping ratio of building structure shall be taken as 0.05, the damping adjustment coefficient of seismic influence coefficient curve shall be taken as 1.0 and the form parameter shall meet the following requirements:

1) Linear ascending section, whose period is less than 0.1s.

2) Horizontal section, whose period is from 0.1s thought to characteristic period, the maximum value ($\alpha_{\max}$) shall be taken.

3) Curvilinear descending section, whose period is from the characteristic period thought to 5 times of the characteristic period, the attenuation index shall be taken as 0.9.

4) Linear descending section, whose period is from 5 times of the characteristic period thought to 6s, the adjustment coefficient for descending slope shall be taken as 0.02.

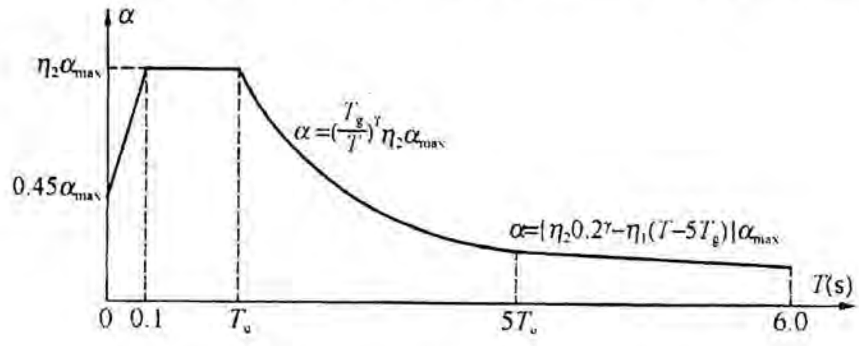


Figure 5.1.5 Seismic Influence Coefficient Curve

α—The seismic influence coefficient; α_{max}—The maximum value of seismic influence coefficient;

η_1—The adjustment coefficient for descending slope in the linear decreasing section; γ—The attenuation index;

T_g—The characteristic period; η_2—The damping adjustment coefficient; T—The natural vibration period of structure

2 If the damping ratio of building structure is not equal to 0.05 according to the relevant regulations, the damping adjustment coefficient and form parameter of the seismic influence coefficient curve shall meet the following requirements:

1) The attenuation index of curvilinear descending section shall be determined according to the following formula:

$$\gamma = 0.9 + \frac{0.05-\zeta}{0.3+6\zeta} \tag{5.1.5-1}$$

Where γ——Attenuation index of curvilinear descending section;

ζ——Damping ratio.

2) The adjustment coefficient for descending slope of attenuation index shall be determined according to the following formula:

$$\eta_1 = 0.02 + \frac{0.05-\zeta}{4+32\zeta} \tag{5.1.5-2}$$

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 coefficient shall be determined according to the following formula:

$$\eta_2 = 1 + \frac{0.05-\zeta}{0.08+1.6\zeta} \tag{5.1.5-3}$$

Where η_2——Damping adjustment coefficient, which shall be taken as 0.55 if it is less than 0.55.

5.1.6 The section seismic check of structure shall meet the following requirements:

1 The buildings for Intensity 6 (excluding the irregular buildings as well as the tall

buildings built on Class IV site) and the unfired earth houses and wood houses shall comply with the requirements of the relevant seismic measures, but shall be allowed to not be done with the section seismic check.

2 The irregular buildings for Intensity 6, the tall buildings built on Class IV site and the building structures of or above Intensity 7 (excluding the unfired earth houses and wood houses, etc) shall be carried out with the section seismic check under frequent earthquake action.

Note: As for the building structures adopting seismic isolation design, the seismic check shall comply with the relevant regulations.

5.1.7 In addition to the section seismic check under frequent earthquake action as required, the structures meeting those specified in Section 5.5 of this code still shall be carried out with the corresponding deformation check.

5.2 Calculation of Horizontal Earthquake Action

5.2.1 When the base shear force method is adopted, only one degree of freedom may be considered for each storey; the standard value of horizontal earthquake action of the structure shall be determined according to the following formulae (Figure 5.2.1):

Figure 5.2.1 Diagram of the Horizontal Earthquake Action of Structure

$$F_{EK}=\alpha_1 G_{eq} \quad (5.2.1\text{-}1)$$

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

$$\Delta F_n=\delta_n F_{EK} \quad (5.2.1\text{-}3)$$

Where F_{EK}——Standard value of the total horizontal earthquake action of a structure;

α_1——Value of horizontal seismic influence coefficient corresponding to the basic natural vibration period of structure, which shall be determined according to Article 5.1.4 and Article 5.1.5 of this code; and it should be taken as the maximum value of horizontal seismic influence coefficient for the multi-storey masonry building and the masonry buildings with bottom frame;

G_{eq}——Total equivalent gravity load of a structure, which shall be taken as the representative value of the total gravity load for single mass point and 85% of the representative value of total gravity load for multi-mass points;

F_i——Standard value of the horizontal earthquake action at the i^{th} mass point;

G_i, G_j——Respectively the representative values of the gravity loads concentrated on the i^{th} and j^{th} mass points, which shall be determined according to Article 5.1.3 of this code;

H_i, H_j——Respectively the calculated height of the i^{th} and j^{th} mass points;

δ_n——Additional earthquake action coefficient at top, which may be adopted according to Table 5.2.1 for the multi-storey reinforced concrete houses and the steel structure houses, and may be taken as 0.0 for other buildings;

ΔF_n——Additional horizontal earthquake action at top.

Table 5.2.1 Additional Earthquake Action Coefficient at Top

T_g(s)	T_1>1.4T_g	T_1≤1.4T_g
T_g≤0.35	0.08T_1+0.07	0.0
0.35<T_g≤0.55	0.08T_1+0.01	
T_g>0.55	0.08T_1－0.02	

Note: T_1 is the basic natural vibration period of the structure.

5.2.2 When the mode-decomposition response spectrum method is adopted, the structures not considered with coupling effect shall be calculated for the earthquake action and action effect according to the following requirements:

1 The standard value of the horizontal earthquake action of a structure at the i^{th} mass point of the j^{th} vibration mode shall be determined according to the following formulae:

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

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

Where F_{ji}——Standard value of horizontal seismic action at the i^{th} mass point of the j^{th} vibration mode;

α_j——Seismic influence coefficient corresponding to the natural vibration period of the j^{th} vibration mode, which shall be determined according to Article 5.1.4 and Article 5.1.5 of this code;

X_{ji}——Horizontal relative displacement of the i^{th} mass point of the j^{th} vibration mode;

γ_j——Participation coefficient of the j^{th} vibration mode.

2 Provided that the period ratio of adjacent vibration modes is less than 0.85, the horizontal earthquake action effects (bending moment, shear force, axial force and deformation) may be determined according to the following formula:

$$S_{Ek}=\sqrt{\Sigma S_j^2} \quad (5.2.2\text{-}3)$$

Where S_{EK}——Effect of the standard value of horizontal seismic action;

S_j——Effect of the standard value of horizontal earthquake action with the j^{th} vibration mode, which may be only taken with the first 2 or 3 vibration modes, however, the number of vibration modes shall be increased properly if the basic natural vibration period is larger than 1.5s or the height-width ratio of the building is larger than 5.

5.2.3 Under the horizontal earthquake action, the torsion coupling earthquake effect of the building structure shall meet the following requirements:

1 If no coupled torsion calculation is to be carried out for regular structures, the earthquake action effects of the components on bide sides parallel to the earthquake action direction shall be multiplied by an enhancement coefficient. Generally, the enhancement coefficient may be taken as 1.15 for the short side and as 1.05 for the long side; if the torsion rigidity is small, the enhancement coefficient for the surrounding components should not be less than 1.3. And the earthquake action effects of the components at corners should be multiplied by the enhancement coefficients in these two directions respectively.

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, and the earthquake action and action effect shall be calculated according to the following formulae. If sufficient references are available, the earthquake action effect also may be determined by adopting with simplified calculation methods.

1) The standard value of horizontal earthquake action of the i^{th} storey with the j^{th} vibration mode shall be determined according to the following formulae:

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

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

$$F_{tji}=\alpha_j\gamma_{tj}r_i^2\varphi_{ji}G_i \quad (5.2.3\text{-}1)$$

Where F_{xji}, F_{yji}, F_{tji}——Respectively the standard values of earthquake actions in the x direction, y direction and rotating direction of the i^{th} storey with the j^{th} vibration mode;

X_{ji}, Y_{ji}——Horizontal relative displacement of mass centre of the i^{th} storey with the j^{th} vibration mode respectively in the x and y directions;

φ_{ji}——Relative torsion angle of the i^{th} storey with the j^{th} vibration mode;

γ_i——Radius of rotation of the i^{th} storey, which may be taken as the square root of that the rotating moment of inertia around the center of the i^{th} storey divided by the mass of this storey;

γ_{tj}——Participation coefficient of the j^{th} vibration mode considering rotation effect, which may be determined according to the following formulae:

When only the earthquake action in x direction is considered

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

When only the earthquake action in y direction is considered

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

When the earthquake action oblique with the x direction is considered

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

Where γ_{xj}, γ_{yj}——Respectively the participation coefficients calculated according to Formula (5.2.3-2) and Formula (5.2.3-3);

θ——Included angle between the direction of earthquake action and the x direction.

2) The torsion coupling effect under the unidirectional horizontal earthquake action may be determined in accordance with following formulae:

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

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

Where S_{EK}——Torsion effect of the standard value of earthquake action;

S_j, S_k——Respectively the effect of the standard value of earthquake action in the j^{th} and k^{th} vibration modes, which may be taken with the first 8~15 vibration modes;

ζ_j, ζ_k——Respectively the damping ratio of the j^{th} and k^{th} vibration modes;

ρ_{jk}——Coupling coefficient of the j^{th} and k^{th} vibration modes;

λ_T——Ratio between the natural vibration periods of the k^{th} and j^{th} vibration modes.

3) The torsion coupling effect under the bi-directional horizontal earthquake actions may be determined with larger value of the following formulae:

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

Or

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

Where S_x, S_y——Respectively the torsion effect of unidirectional horizontal earthquake actions in the x and y direction calculated according to Formula (5.2.3-5).

5.2.4 When the base shear method is used, the earthquake action effects of penthouse, parapet wall and chimney on the roof should be multiplied by an enhancement coefficient of 3; such increased part of effect shall not be transmitted to the lower part of the structure. But the parts connected with this projecting part shall be considered. When modal analysis method is used, the projecting part may be considered as one mass. The enhancement coefficient for the earthquake action effect of projecting skylight frame of a single-storey factory building shall be adopted according to the relevant requirements of Chapter 9 of this code.

5.2.5 During the seismic check, the horizontal seismic shear force at any storey of the structure shall comply with the requirements of the following formula:

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

Where V_{eki}——**Storey shear of the i^{th} storey corresponding to the standard value of horizontal earthquake action;**

λ——**Coefficient of shear, which shall not be less than the minimum seismic shear force coefficient of storey as specified in Table 5.2.5 and also shall be multiplied by an enhancement coefficient of 1.15 for the weak storeys of such structures with vertical irregularity;**

G_j——**Representative value of gravity load of the j^{th} storey.**

Table 5.2.5 Minimum Seismic Shear Force Coefficient Value of a Storey

Type	Intensity 6	Intensity 7	Intensity 8	Intensity 9
Structures with obvious torsion effect or fundamental period less than 3.5s	0.008	0.016 (0.024)	0.032 (0.048)	0.064
Structures with fundamental period larger than 5.0s	0.006	0.012 (0.018)	0.024 (0.036)	0.048

Note: 1 For the structures with fundamental period between 3.5s and 5s, the minimum seismic shear force coefficient shall be determined according to interpolation method;

2 The values in parentheses are used for the zones where the design basic acceleration of ground motion is 0.15g and 0.30g respectively.

5.2.6 The horizontal seismic shear force of storeys of a structure shall be distributed according to the following principles:

1 For the rigid buildings and roof buildings, such as the cast-in-situ and monolithic-fabricated concrete floors and roofs, the horizontal seismic shear force should be distributed according to the proportion of equivalent rigidity of lateral-force-resisting components.

2 For the buildings with flexible floor and roof (such as wood floor and wood roof), the horizontal seismic shear force should be distributed according to the proportion of the representative values of gravity loads in the tributary area of lateral-force-resisting components.

3 For the buildings with such semi-rigid floor and roof as ordinary prefabricated concrete floor and roof, the horizontal seismic shear force may be taken as the mean value of these above two distribution results.

4 If the effects of space action, floor deformation, wall elasto-plastic deformation and torsion are considered, the above-mentioned distribution results may be adjusted properly according to the relevant provisions in this code.

5.2.7 As for the seismic calculation of a structure, the influence of the intersection of base and structure may not be taken into account; as for the reinforced concrete tall buildings that are built in the Class III and IV sites for Intensity 8 and Degree 9 and are adopted with box foundation, raft foundation with good rigidity and the combined pile-box foundation, if the basic natural vibration period of the structure is between 1.2 times and 5 times of the characteristic period and the influence of the intersection of base and structure is considered, then the supposed calculated horizontal seismic shear force of rigid base may be reduced according to the following requirements and the storey drift may be calculated according to the reduced storey shear force.

1 As for the structures with height-width ratio less than 3, the reduction coefficient for the horizontal seismic shear force of each storey may be calculated according to following formula:

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

Where ψ——Reduction coefficient for seismic shear force with consideration of the dynamic interaction of base and structure;

T_1——Basic natural vibration period (s) of the structure determined by assumption according to rigid base;

ΔT——Additional period with consideration of the dynamic interaction of base and structure, which may be selected according to those specified in Table 5.2.7.

Table 5.2.7 Additional Period (s)

Intensity	Site class	
	III	IV
8	0.08	0.20
9	0.10	0.25

2 As for the structures with height-width ratio not less than 3, the seismic shear force at bottom shall be reduced according to those specified in Item 1, that at the top of structure shall not be reduced, and that at the middle storeys shall be reduced according to the linear interpolation values.

3 The horizontal seismic shear force of each storey after reduction shall comply with those specified in Article 5.2.5 of this code.

5.3 Calculation of Vertical Earthquake Action

5.3.1 As for the tall buildings for Intensity 9, the standard value of their vertical earthquake action shall be determined according to the following formulae (Figure 5.3.1); the vertical earthquake action effect of storeys may be distributed according to the proportion of the representative values of gravity loads bear by each component and should be multiplied by an enhancement coefficient of 1.5.

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

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

Where F_{Evk}——Standard value of the total vertical earthquake action of a structure;

F_{vi}——Standard value of the vertical earthquake action at the i^{th} mass point;

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

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

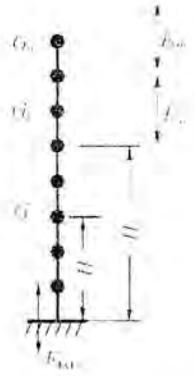


Figure 5.3.1 Diagram of the Vertical Earthquake Action of Structure

5.3.2 As for such regular flat lattice truss roof with span and length reprehensively less than those specified in Item 5 in Article 5.1.2 of this code as well as the roof truss, roof transverse beam and bracket with span larger than 24m, the standard value of their vertical earthquake action should be taken as the product of the representative value of their gravity load and the vertical earthquake action coefficient, in which, the vertical earthquake action coefficient may be adopted according to those specified in Table 5.3.2.

Table 5.3.2 Vertical Earthquake Action Coefficient

Structure type	Intensity	Site class		
		I	II	III, IV
Flat lattice truss and steel roof truss	8	Not considered (0.10)	0.08 (0.12)	0.10 (0.15)
	9	0.15	0.15	0.20
Reinforced concrete roof 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 parentheses are used for the zones where the design basic acceleration of ground motion is 0.30g.

5.3.3 As for the long-cantilever components and the large-span structures not specified in Article 5.3.2 of this code, the standard value of their vertical earthquake action may be taken 10% and 20% of the representative value of gravity load of this structure or component respectively for Intensity 8 and 9, and also may be taken as 15% of the representative value of gravity load of this structure or component when the design basic acceleration of ground motion is 0.30g.

5.3.4 The vertical earthquake action of the large-span space structure may also be calculated with the vertical model-decomposition response spectrum method. The vertical seismic influence coefficient hereof may be taken as 65% of the horizontal seismic influence coefficient specified in Article 5.1.4 and Article 5.1.5 of this code, however, the characteristic period shall be adopted according to the first design group.

5.4 Section Seismic Check

5.4.1 The fundamental combination of the earthquake action effect and other load effects of the structural components shall be calculated according to the following formula:

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

Where S——Design value of the internal force combination of structural components, including the design values of the combined bending moment, axial force and shear force, etc.;

γ_G——Partial coefficient of gravity load, which generally shall be taken as 1.2 and shall not be larger than 1.0 if the gravity load effect is favorable to the bearing capacity of component;

γ_{Eh}, γ_{Ev}——Respectively the partial coefficients of the horizontal and vertical earthquake actions, which shall be adopted according to Table 5.4.1;

γ_w——Partial coefficient of wind load, which shall be taken as 1.4;

S_{GE}——Effect of the representative value of gravity load, which may be adopted according to Article 5.1.3 of this code, however, it still shall include the effect of the standard value of suspender gravity if crane is used;

S_{Ehk}——Effect of the standard value of horizontal earthquake action, which still shall be multiplied by the corresponding enhancement coefficient or adjustment coefficient;

S_{Evk}——Effect of the standard value of vertical earthquake action, which still shall be multiplied by the corresponding enhancement coefficient or adjustment coefficient;

S_{wk}——Effect of standard value of wind load;

ψ_w——Combination value coefficient of wind load, which shall be 0.0 for ordinary structures and shall be 0.2 for such buildings that the wind load plays the control action.

Note: Generally, the subscript representing the horizontal direction is omitted in this code.

Table 5.4.1 Partial Coefficient of Earthquake Action

Earthquake action	γ_{Eh}	γ_{Ev}
Only calculating the horizontal earthquake action	1.3	0.0
Only calculating the vertical earthquake action	0.0	1.3
Simultaneously calculating the horizontal and vertical earthquake actions (mainly based on horizontal earthquake action)	1.3	0.5
Simultaneously calculating the horizontal and vertical earthquake actions (mainly based on vertical earthquake action)	0.5	1.3

5.4.2 The section seismic check of structural component shall be adopted with the following design expression:

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

Where γ_{RE}——Seismic adjustment coefficient of bearing capacity, which, unless otherwise

specified, shall be selected according to those specified in Table 5.4.2;

R——Design value of the bearing capacity of structural component.

Table 5.4.2 Seismic Adjustment Coefficient of Bearing Capacity

Material	Structural component	Stress state	γ_{RE}
Steel	Column, beam, brace, gusset plate, bolt and welded joint	Stable strength	0.75
	Column and brace		0.80
Masonry	Seismic wall with constructional columns and core columns at both ends	Shear	0.9
	Other seismic walls	Shear	1.0
Concrete	Beam	Bending	0.75
	Columns with axial compression ratio less than 0.15	Eccentric compression	0.75
	Columns with axial compression ratio no less than 0.15	Eccentric compression	0.80
	Seismic wall	Eccentric compression	0.85
	Various components	Shear, eccentric tension	0.85

5.4.3 If only the vertical earthquake action is calculated, the seismic adjustment coefficient of bearing capacity of various structural components all shall be taken as 1.0.

5.5 Seismic Deformation Check

5.5.1 The structures listed in Table 5.5.1 shall be carried out with seismic deformation check under the frequent earthquake action, and the maximum elastic layer drift in their storeys shall meet the requirements of the following formula:

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

Where Δu_e——Maximum elastic layer drift in a storey produced by the standard value of frequent earthquake action; except for the tall buildings mainly with flexural deformation, it may be calculated without reducing the integral flexural deformation of structure; the torsional deformation shall be considered, the partial coefficients of action all shall be taken as 1.0; and the sectional rigidity of reinforced concrete structure component may be taken as the elastic rigidity;

$[\theta_e]$——Limit for elastic storey drift rotation, which should be adopted according to Table 5.5.1;

h——Calculated storey height.

Table 5.5.1 Limit for Elastic Storey Drift

Structure type	$[\theta_e]$
Reinforced concrete frame	1/550
Reinforced concrete frame-seismic wall, slab column-seismic wall, and frame-core-tube	1/800
Reinforced concrete seismic wall, tube-in-tube	1/1000
Reinforced concrete frame-supported storey	1/1000
Multi-storey and tall steel structures	1/250

5.5.2 The elasto-plastic deformation check of the weak storey of a structure under rare earthquake action shall meet the following requirements:

1 The following structures shall be carried out with the elasto-plastic deformation check:

1) For Intensity 9 and Class III and IV sites for Intensity 8, the transverse bent frames of tall single-storey factory buildings with R.C. columns;

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

3) Structures with height greater than 150m;

4) Reinforced concrete structures and steel structures in Category A buildings and Category B buildings for Intensity 9;

5) The structures adopting with seismic-isolation and energy dissipating design.

2 The following structures should be carried out with the elasto-plastic deformation check:

1) The structures of such tall buildings belonging to the height range listed in Table 5.1.2-1 of this code and the vertical irregularity type listed in Table 3.4.3-2;

2) The reinforced concrete structures and the steel structures in Category B buildings at Class III and IV sites for Intensity 7 and sites for Intensity 8;

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

4) Other tall steel structures with height no larger than 150m;

5) The structures of irregular subterranean buildings as well as the subterranean space complex.

Note: The yield strength coefficient of a storey is the ratio between the storey shear capacity calculated according to the actual reinforcement and material strength standard value of RC components and the storey elastic seismic shear force calculated according to the standard action under rare earthquake action; as for the bent-frame columns, it refers to the ratio between the bend bearing capacity of normal section calculated according to the actual reinforcement area, material strength standard value and axial force and the elastic seismic bending moment calculated according to the standard action under rare earthquake action.

5.5.3 The elasto-plastic deformation of the weak storey (or location) of a structure under rare earthquake action may be calculated by adopting with the following methods:

1 The reinforced concrete frames and bent-frame structures with no more than 12 storeys and stable storey rigidity as well as the single-storey factory buildings with R.C. columns may be adopted with the simplified calculation methods specified in Article 5.5.4 of this code.

2 Building structures other than those specified in Item 1 all may be adopted with the static elasto-plastic analysis method or the elasto-plastic time-history analysis method.

3 The regular structures may be adopted with shear-bending storey model or plane member system model, and the irregular structures as per listed in Section 3.4 of this code shall be adopted with three-dimensional structural model.

5.5.4 The simplified calculation of the elasto-plastic storey drift of the weak storeys (or locations) of a structure should meet the following requirements:

1 The positions of the weak storeys (or locations) of the structure may be determined according to the following conditions:

1) As for the structure with storey yield strength coefficient uniformly distributed along the height, the bottom storey may be adopted;

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

3) For single-storey factory buildings, the upper portion of the columns may be taken as the weak locations.

2 The elasto-plastic storey drift may be calculated according to the following formulae:

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

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 storey drift;

Δu_y——Yield storey drift;

μ——Ductility coefficient of storey;

Δu_e——Storey drift under rare earthquake action, analyzed according to elasticity;

η_p——Enhancement coefficient for elasto-plastic storey drift, which may be adopted according to Table 5.5.4 if the yield strength coefficient of the weak storey (or location) is less than 80% of the mean value of this coefficient of its adjacent storeys (or locations) and may be adopted according to 1.5 times of the corresponding values listed in Table 5.5.4 if this yield strength coefficient is no larger than 50% of the above-mentioned mean value; as for other cases, it may be determined by interpolation method;

ξ_y——Yield strength coefficient of storey.

Table 5.5.4 Enhancement Coefficient for Elasto-plastic Storey Drift

Structure type	Total number of storeys or locations	ξ_y		
		0.5	0.4	0.3
Multi-storey uniform frame structure	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-storey factory building	Upper column	1.30	1.60	2.00

5.5.5 The elasto-plastic storey drift of the weak storeys (or locations) of a structure shall meet the following formula:

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

Where $[\theta_p]$——Limit for elasto-plastic storey drift rotation, which may be adopted according to Table 5.5.5; as for the R.C. frame structures, if their axial compression ratio is less than 0.40, then this limit may be increased by 10%; if the stirrup standard value along the full height of the column is 30% greater than the stirrup ratio specified in Table 6.3.9, this limit may be increased by 20%, however, it shall not be increased by over 25% totally;

h——Height of weak storey or the column height of single-storey factory building.

Table 5.5.5 Limit for Elasto-plastic Storey Drift Rotation

Structure type	$[\theta_p]$
Single-storey bent frame with R.C. columns	1/30
Reinforced concrete frame	1/50
Frame-seismic wall in masonry building with bottom frame	1/100
Reinforced concrete frame-seismic wall, slab column-seismic wall, and frame-core-tube	1/100
Reinforced concrete seismic wall, tube-in-tube	1/120
Multi-storey and tall steel structures	1/50

6 Multi-storey and Tall Reinforced Concrete Buildings

6.1 General Requirements

6.1.1 The structure type and maximum height of those cast-in-situ reinforced concrete buildings to which this chapter is applicable shall be in accordance with those specified in Table 6.1.1. As for structures that are irregular horizontally and vertically, the applicable maximum height should be reduced properly.

Note: The "seismic wall" in this chapter refers to the reinforced concrete shear wall in the lateral resistant system of structure, excluding the concrete wall only bearing gravity load.

Table 6.1.1 Applicable Maximum Height of Cast-in-situ Reinforced Concrete Buildings (m)

Structure type		Intensity				
		6	7	8 (0.2g)	8 (0.3g)	9
Frame structure		60	50	40	35	24
Frame-seismic wall		130	120	100	80	50
Seismic wall		140	120	100	80	60
Partial frame-support seismic wall		120	100	80	50	Inapplicable
Tube	Frame-core-tube	150	130	100	90	70
	Tube-in-tube	180	150	120	100	80
Slab-column-seismic wall		80	70	55	40	Inapplicable

Note: 1 The height of building refers to the height between the outdoor ground and the top of main roof slab (excluding partially protruding roof);

2 The frame-core-tube structure refers to the structure composed of the perimeter frames and the core tubes;

3 Partial frame-support seismic wall structure refers to the structures supported on the first storey or two storeys on bottom, excluding those only with individual frame-support wall;

4 The frames stated in the above table exclude the special-shaped column frames;

5 The slab-column-seismic wall structure refers to the structure adopting such a lateral resistant system consisted of slab columns, frames and seismic walls;

6 The applicable maximum height of Category B buildings may be determined according to the seismic precautionary Intensity of this region;

7 As for those buildings whose height is greater than the ones in the above table, special study and demonstration shall be conducted and effective strengthening measures shall be taken.

6.1.2 The reinforced concrete buildings shall be adopted with different seismic grades according to the precautionary category, Intensity, structure type and building height, and also shall meet the requirements of the corresponding calculation and design. The seismic grade of Category C buildings shall be determined according to Table 6.1.2.

6.1.3 The determination of the seismic grade of reinforced concrete building still shall meet the following requirements:

1 As for the frame structures set with a few seismic walls, if the seismic overturning moment bear by the bottom frame under the action of specified horizontal force is larger than 50% of the total seismic overturning moment of the structure, then the seismic grade of its frames shall be determined according to the frame structure and that of the seismic walls mat be identical to the seismic grade of these frames.

Note: The bottom storey refers to the storey where the partial fixing end to be calculated is set.

Table 6.1.2 Seismic Grades of Cast-in-situ Reinforced Concrete Buildings

Structure type			Precautionary Intensity									
			6		7			8			9	
Frame structure	Height (m)		≤24	>24	≤24	>24		≤24	>24		≤24	
	Frame		IV	III	III	II		II	I		I	
	Large-span frame		III		II			I			I	
Frame-seismic wall structure	Height (m)		≤60	>60	≤24	25~60	>60	≤24	25~60	>60	≤24	25~50
	Frame		IV	III	IV	III	II	III	II	I	II	I
	Seismic wall		III		III	II		II	I		I	
Seismic wall structure	Height (m)		≤80	>80	≤24	25~80	>80	≤24	25~80	>80	≤24	25~60
	Shear wall		IV	III	IV	III	II	III	II	I	II	I
Partial frame-support seismic wall structure	Height (m)		≤80	>80	≤24	25~80	>80	≤24	25~80			
	Seismic wall	Common part	IV	III	IV	III	II	III	II			
		Reinforced part	III	I	III	II	I	II	I			
	Frames of frame-supported storey		II		II		I	I				
Frame-core-tube structure	Frame		III		II			I			I	
	Core tube		II		II			I			I	
Tube-in-tube structure	Exterior tube		III		II			I			I	
	Interior tube		III		II			I			I	
Slab-column-seismic wall structure	Height (m)		≤35	>35	≤35	>35		≤35	>35			
	Columns of frame and slab column		III	II	II	II		I				
	Seismic wall		II	II	II	I		II	I			

Note: 1 If the building site is of Class I, except for Intensity 6, it shall be allowable to adopt details of seismic design according to the corresponding seismic grade one grade lower than the ones listed in the above table. However, the corresponding calculation requirements shall not be reduced;

2 When it is approximate or equal to height boundary, the seismic grade may be determined according to degree of irregularity of buildings as well as conditions of site and base;

3 The large-span frame refers to the frame with span no less than 18m;

4 When the frame-core-tube structure in height not exceeding 60m is designed according to the requirements of frame-seismic wall, the seismic grade of this structure shall be determined according to the requirements of frame-seismic wall structure as stated in the above table.

2 When the podium is connected with the main building, the seismic grade shall be determined according to the podium itself and its relevant range shall not be lower than that of the main building. And details of seismic design of the main building shall be strengthened appropriately at the adjacent up and down storey corresponding to the top-slab of podium. If the podium and the main building are separated, the seismic grade shall be determined according to the podium itself.

3 When the top-slab of the basement is used as the fixing location of superstructure, the seismic grade of the first storey underground shall be the same as that of the superstructure, the grade of the details of seismic design for other storeys lower than the first storey underground may be reduced by one grade storey by storey but shall not be less than Grade 4. For the part of the basement without superstructure, the grade of the details of seismic design may be taken as Grade 3 or 4 according to specific conditions.

4 **When** the Category A and B buildings are determined with the seismic grade of one grade higher as required but the height of building exceeds the corresponding upper limit as specified in Table 6.1.2 of this code, the details of seismic design that is much more efficient that Grade 1 shall be adopted.

Note: "Grade 1, 2, 3 and 4" in this chapter are short for "seismic grade 1, 2, 3 and 4".

6.1.4 The reinforced concrete buildings, requiring the arrangement of seismic joints, shall meet the following requirements:

1 The width of seismic joint shall respectively meet the following requirements:

1) For buildings with frame structures (including frame structure arranged with a few seismic walls), if the height of the building does not exceed 15m, the width of seismic joint shall not be less than 100mm; if not, the width, for Intensity 6, 7, 8 and 9 should be increased by 20mm respectively when the height is increased by every 5m, 4m, 3m and 2m.

2) For the buildings with frame-seismic wall structures, the width of seismic joint shall not be less than 70% of the value specified in Item 1 of this clause; for the buildings with seismic wall structures, the width shall not be less than 50% of the value specified in Item 1), however, the width for both two kinds of buildings should not be less than 100mm;

3) When the structures on the two sides of the seismic joint are of different types, the joint width shall be determined according to the structure type requiring wider seismic joint and the smaller building height.

2 For the buildings with frame structures for Intensity 8 and 9, if the height of structural layers on both sides of the seismic joint is different greatly, the stirrups of the frame columns on both sides of the seismic joint shall be thickened along the total height of the building, and at least two bump walls perpendicular to the seismic joint may be built on both sides of the seismic joint along the total height of the building as required. The arrangement of the bump walls should avoid increasing the torsion effect, the wall length may not be larger than 1/2 of the storey height, the seismic grade of these walls may be same as the frame structure; the internal force of frame component shall be adopted according to the unfavorable situations of two kinds of calculation models with or without bump walls.

6.1.5 In the frame structures and the frame-seismic wall structures, both the frames and the seismic walls shall be arranged in two directions, and the influence of eccentricity shall be taken into consideration if the eccentricity between the center lines of the column and the seismic wall as well as between the center lines of beam and column is larger than 1/4 of the column width.

The Category A and B buildings as well as the Category C buildings with height larger than 24m shall not be adopted with the single-span frame structures; and the Category C buildings with height no larger than 24m should not be adopted with single-span frame structures.

6.1.6 In the frame-seismic wall structures, slab-column-seismic wall structures and frame-supported storeys, the length-width ratio of such floors and roofs that are without large openings and between their seismic walls should not be less than those specified in Table 6.1.6; otherwise, the influence of the deformation in plane of the floor shall be taken into account.

Table 6.1.6 Length-width Ratio of Floor (Roof) between Seismic Walls

Type of floor and roof		Precautionary Intensity			
		6	7	8	9
Frame-seismic wall structure	Cast-in-situ or lapped floors and roofs	4	4	3	2
	Fabricated complete floors and roofs	3	3	2	Should not be adopted
Cast-in-situ floors and roofs of slab-column-seismic wall structures		3	3	2	—
Cast-in-situ floors and roofs of frame-supported storey		2.5	2.5	2	—

6.1.7 When adopting the fabricated complete floors and roofs are adopted, proper measures shall be taken so as to ensure the integrity of floors and roofs and their reliable connections with the seismic walls. When strengthening the fabricated complete floors and roofs by adopting with cast-in-situ surface course with reinforcements, the thickness shall not be less than 50mm.

6.1.8 The arrangement of seismic walls in the frame-seismic wall structures and the slab-column-seismic wall structures should meet the following requirements:

1 The seismic wall should be built through the overall height of the building.

2 The staircase should be built with seismic walls, by which large torsion effect should not be caused.

3 Both ends (excluding both sides of the opening) of the seismic wall should be built with end columns or be connected with the seismic wall in the other direction.

4 As for the long buildings, the longitudinal seismic walls with large rigidity should not be built at the end bays of the building.

5 The opening in seismic wall should be aligned up and down and the distance from the opening edge to the end column should not be less than 300mm.

6.1.9 The arrangement of seismic walls in the seismic wall structures and the partially frame-support seismic wall structures shall meet the following requirements:

1 Both ends (excluding both sides of the opening) of a seismic wall should be built with end columns or be connected with the seismic wall in the other direction; both ends (excluding both sides of the opening) of the floor-type wall of the frame-support part shall be built with end columns or be connected with the seismic wall in the other direction.

2 The long seismic walls should be set with connection beams with span-height ratio larger than 6 so as to form into openings, one seismic wall shall be divided into several sections in uniform length and the height-width ratio of each wall section should not be less than 3.

3 Length of wall columns should be stable along the total height of the structure; the big openings, if any, in seismic walls as well as those at the bottom reinforced part of Grade 1 and 2 seismic walls should be aligned up and down.

4 As for the partial frame-support seismic wall structure with rectangular plane, the lateral rigidity of the frame-supported storey shall not be less than 50% of that of the adjacent non-frame-supported storeys; the space between the floor-type seismic walls of the frame-supported storey should not be larger than 24m, the frame-supported storey should be adopted with symmetric plane layout and also should be set with seismic tubes; the seismic overturning moment bear by the bottom-frame part shall not be larger than 50% of the total seismic overturning moment of the structure.

6.1.10 The range of bottom reinforced part of earthquake resisting wall shall the following requirements:

1 The height of the bottom reinforced part shall be counted from the top-slab of basement.

2 As for the seismic walls of partial frame-support seismic wall structures, the height of the bottom reinforced part may be taken as the greater one between the height of the frame-supported storey and the two storeys above this frame-supported storey and 1/10 of the total height of floor-type seismic wall. As for the seismic walls of other structures, if the height of the building is larger than 24m, the height of its bottom reinforced part may be taken as the greater one between 1/10 of the height of these two bottom storeys and 1/10 of the total height of wall; if the height of the building is no lager than 24m, the bottom storey may be regarded as the bottom reinforced part.

3 When the calculated fixing end of the structure is located on or bellow the soleplate of underground storey, the bottom reinforced part still shall be extended downward to this calculation fixing end.

6.1.11 In one of the following cases for single column footing of the frame, the foundation tie beams should be installed along the directions of the two principal axes:

1 Grade 1 frames, as well as Grade 2 frames in Class II sites;

2 The compression stress values on the bottom surface of column footing caused by the representative value of gravity load are different greatly;

3 The embedded depth of foundation is large or the depth values of foundations are different greatly;

4 Soft cohesive soil layer, liquefaction soil layer or extremely non-uniform soil layer exist within the range of the main bearing layer of base;

5 Connection between the pile caps.

6.1.12 The foundations of seismic walls in the frame-seismic wall structures and slab-column-seismic wall structures as well as the foundations of the floor-type seismic walls in partial frame -support seismic wall structures shall be with excellent integrality and anti-rotational capability.

6.1.13 When the main building is connected with the podium and is adopted with natural base, in addition to those specified in Article 4.2.4 of this code, the bottom of main building foundation should be free from zero stress zones under the frequent earthquake action.

6.1.14 When the top-slab of basement is designed as the fixing position of superstructure, the basement shall meet the following requirements:

1 The top-slab of basement shall not be set with large openings; the top-slab within the scope related to the overground structures shall be adopted with the cast-in-situ beam and slab structure and that outside this scope should be adopted with cast-in-situ beam and slab structure; the thickness of the floor slab of the basement should not be less than 180mm, the concrete strength grade should not be less than C30, the double-layer two-way reinforcements shall be adopted and the reinforcement ratio in each direction of each layer should not be less than 0.25%.

2 The lateral rigidity of the first overground story of the structure should not be larger than 0.5 times of that of the underground storey within the relevant scope; and the seismic walls should be built around the basement to connect with the top-slab.

3 In addition to the requirements of seismic calculation, the beam-column joint of the basement top-slab corresponding to the overground frame column still shall meet one of the following requirements:

1) The longitudinal reinforcement on each side of column section at the first underground storey shall not less than 1.1 times of the longitudinal reinforcement corresponding to the first overground storey, and the total actual seismic bent bearing capacity of the reinforcements set at the upper ends of columns of the first underground story and the beam ends on the right and left sides of the joint shall be larger than 1.3 times of that at the lower ends of columns of the first overground story.

2) When the beams of the first underground story are with relatively big rigidity, the area of the longitudinal reinforcements on each side of the column section shall be larger than 1.1 times of those on each side of the corresponding column; meanwhile, the area of longitudinal reinforcements set both on the top and bottom surfaces at beam end shall be over 10% larger than the calculated values.

4 The sectional area of the longitudinal reinforcements of boundary components at ends of the seismic wall columns on the first underground storey shall not be less than that for the ends of the corresponding seismic wall columns on the first overground storey.

6.1.15 The staircase shall meet the following requirements:

1 The cast-in-situ reinforced concrete stairs should be adopted.

2 As for the frame structures, the layout of staircase shall not result in the especial irregularity of structural plan; when concreting the staircase components together with major structure, the influence of the staircase components on the earthquake action and effect shall be taken into consideration, and the staircase components shall be carried out with seismic capacity check; proper details of seismic design should be taken to reduce the influence of staircase components on the rigidity of major structure.

3 The tie of the filler walls on both sides of staircase with the columns shall be strengthened.

6.1.16 The filler walls for frames shall comply with those specified in Chapter 13 of this code.

6.1.17 The seismic design of the high strength concrete structures shall comply with those specified in Appendix B of this code.

6.1.18 The seismic design of the prestressed concrete structures shall comply with those specified in Appendix C of this code.

6.2 Essentials in Calculation

6.2.1 As for the reinforced concrete structures, the design value of the combined internal force of components shall be adjusted according to those specified in this section, the storey drift shall comply with the relevant regulations in Section 5.5 of this code. For the seismic check of component section, the design value of the non-seismic bearing capacity shall be divided by the seismic adjustment coefficient of bearing capacity specified in this code; for all the components not concerned in this chapter and appendixes of this code, their section seismic check shall meet the requirements of the relevant current codes for structural design.

6.2.2 At the beam-column joints of Grade 1, 2, 3 and 4 frames, except for the top layers of frames, those with axial compression ratio of column less than 0.15 as well as the joints of frame-supported beams with frame-supported columns, the design value of the combined bending moment at column ends shall meet the requirements of the following formula:

$$\sum M_c = \eta \sum M_b \qquad (6.2.2\text{-}1)$$

That of the Grade 1 frame structures as well as the Grade 1 frames for Intensity 9 may not meet the requirements of the above formula but shall meet the requirements of the following formula:

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

Where ΣM_c——Sum of the combined clockwise or counterclockwise bending moment on the upper and lower column end sections at one joint; the design value of the bending moment at the upper and lower column end may be distributed according to elasticity analysis;

ΣM_b——Sum of the design value of combined counterclockwise or clockwise bending moment on the sections of beam ends on right and left sides of a joint. If the beam ends on right and left sides of the Grade 1 frame joint are with negative moment, the bending moment of smaller absolute value shall be taken as zero;

ΣM_{bua}——Sum of counterclockwise or clockwise bending moment values on the sections of beam ends on right and left sides of a joint, corresponding to the actual seismic bending capacity of normal section, which shall be determined according to the actual area of steel reinforcements (including bearing reinforcements of beam and steel reinforcements of related floor slabs) and the standard value of material strength;

η_c——Enhancement coefficient for the bending moment at frame column end; as for the frame structures of Grade 1, 2, 3 and 4, the enhancement coefficient may be taken as 1.7, 1.5, 1.3 and 1.2 respectively; as for the frames in other types of structures, it may be taken as 1.4 for Grade 1 frames, 1.2 for Grade 2 frames and 1.1 for Grade 3 and 4 frames.

If the inflection point is outside the scope of storey height of column, the design value of the combined bending moment on the section at column end may be multiplied by such above-mentioned enhancement coefficient for bending moment at column end.

6.2.3 At the bottom storey of Grade 1, 2, 3 and 4 frame structures, the design value of combined bending moment on the lower end section of column shall be multiplied by an enhancement coefficient of 1.7, 1.5, 1.3 and 1.2 respectively. The longitudinal reinforcements for the columns at bottom storey shall be arranged according to the unfavorable situations at the upper and lower ends.

6.2.4 As for Grade 1, 2 and 3 frame beams and the connection beams of seismic walls, the design value of combined shear force on the beam end section shall be adjusted according to the following formula:

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

That of the Grade 1 frame structures as well as the Grade 1 frames for Intensity 9 may not be adjusted according to the above formula but shall meet the requirements of the following formula:

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

Where V——Design value of combined shear force on the end section of beam;

l_n——Clear span of a beam;

V_{Gb}——Design value of shear force on the end section of beam under the action of the representative value of its gravity load (for tall buildings for Intensity 9, the standard value of vertical earthquake action still shall be included), which is analyzed according to simple-supported beam;

M_b^l, M_b^r——Respectively the design value of combined counterclockwise or clockwise bending moment on the right and left sides of the beam. As for the Grade 1 frames, if both ends of the frame are with negative moment, the bending moment with smaller absolute value shall be taken as zero;

M_{bu}^l, M_{bua}^r——Respectively the actual counterclockwise or clockwise bending moment values on the right and left sides of the beam corresponding to the seismic bend bearing capacity of the normal section, which shall be determined according to the actual area of steel reinforcements (including bearing reinforcements of beam and steel reinforcements of related floor slabs) and the standard value of material strength;

η_{vb}——Enhancement coefficient for the shear force at beam end, which is taken as 1.3 for Grade 1, 1.2 for Grade 2 and 1.1 for Grade 3.

6.2.5 The design value of combined shear force of the Grade 1, 2, 3 and 4 frame columns and the frame-supported columns shall be adjusted according to the following formula:

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

That of the Grade 1 frame structures as well as the Grade 1 frames for Intensity 9 may not be adjusted according to the above formula but shall meet the requirements of the following formula:

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

Where V——Design value of combined shear force on the section at column end; for the frame-supported columns, the design value of shear force still shall comply with those specified in Article 6.2.10 of this code;

H_n——Net height of the column;

M_c^b, M_c^t——Respectively the design value of combined bending moment on the clockwise or counterclockwise sections at the upper and lower ends of a column, which shall comply with those specified in Article 6.2.2 and Article 6.2.3 of this code; as for frame-supported columns, the design value of bending moment still shall comply with those specified in Article 6.2.10 of this code;

M_{bcua}, M_{cua}^t——Respectively the bending moment values corresponding to the actual seismic bend bearing capacity of normal section in the clockwise or counterclockwise direction at the upper and lower ends of eccentrically loaded column, which shall be determined according to the actual area of steel reinforcements, the standard value of material strength and the axle pressure, etc.;

η_{vc}——Enhancement coefficient for shear force of column; as for the frame structures of Grade 1, 2, 3 and 4, the enhancement coefficient may be taken as 1.5, 1.3, 1.2 and 1.1 respectively; as for the frames in other types of structures, it may be taken as 1.4 for Grade 1, 1.2 for Grade 2 and 1.1 for Grade 3 and 4.

6.2.6 As for the corner columns of Grade 1, 2, 3 and 4 frames, the design values of combined bending moment and combined shear force adjusted according to Articles 6.2.2, 6.2.3, 6.2.5 and 6.2.10 of this code still shall be multiplied by an enhancement coefficient no less than 1.10.

6.2.7 For seismic walls, the design values of the combined internal forces of each wall column section shall be adopted according to the following requirements:

1 At the positions above the bottom reinforced part of Grade 1 seismic wall, the design value of combined bending moment of wall columns shall be multiplied by an enhancement coefficient, and the value of this coefficient may be taken as 1.2 and be adjusted correspondingly.

2 The columns of floor-type seismic walls in partial frame-support seismic wall structures shall be free from any small eccentric tension.

3 In coupled seismic wall, the wall columns should be free small eccentric tension; if any one wall column suffers the eccentric tension, the design values of the shear force and bending moment of the other wall column shall be multiplied by an enhancement coefficient of 1.25.

6.2.8 As for the bottom reinforced parts of Grade 1, 2 and 3 seismic walls, the design value of combined sectional shear force shall be adjusted according to the following formula:

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

That for Grade 1 seismic walls for Intensity 9 may not be adjusted according to the above formula but shall meet the requirements of the following formula:

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

Where V——Design value of combined sectional shear force at the bottom reinforced part of seismic wall;

V_w——Calculated value of combined sectional shear force at the bottom reinforced part of seismic wall;

M_{wua}——Bending moment of the bottom section of seismic wall, which is corresponding to the seismic bend bearing capacity and is calculated according to the actual area of longitudinal reinforcements, the standard value of material strength and the axial force; if any wing wall is built, the longitudinal reinforcements within the a scope of one times of the wing wall thickness on both sides of the wall shall be taken into account;

M_w——Design value of combined bending moment on the bottom section of seismic wall;

η_{vw}——Enhancement coefficient for shear force of seismic wall, which may be taken as 1.6 for Grade 1 seismic wall, 1.4 for Grade 2 seismic wall and 1.2 for Grade 3 seismic wall.

6.2.9 As for reinforced concrete structures, the design values of the combined sectional shear force of beams, columns, seismic walls and connection beams shall meet the following requirements:

Beams and connection beams with span-height ratio larger than 2.5 as well as the columns and seismic walls with shear span ratio larger than 2:

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

Connection beams with span-height ratio no larger than 2.5, columns and seismic walls with shear span ratio no larger than 2, frame-supported columns and frame-supported beams of the partial frame-support seismic wall structures as well as the bottom reinforced parts of floor-type seismic walls:

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

The shear span ratio shall be calculated according to the following formula:

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

Where λ——Shear span ratio, which shall be determined according to the calculated value M^c of the combined sectional bending moment at column end or wall end, the calculated value V^c of the combination shear force on the corresponding section and the effective height h_0 of this section, and shall be the greater value of the calculated results for the upper and lower ends; as for the frame column with the inflection point located at the middle of column height, it may be calculated according to the ratio between the net height of column and 2 times of the section height of column;

V——Design value of combined sectional shear force at the beam end, column end or wall end, which is adjusted according to those specified in Articles 6.2.4, 6.2.5, 6.2.6, 6.2.8 and 6.2.10 of this code;

f_c——Design value of the axial compressive strength of concrete;

b——Sectional width of beam and column or the sectional width of seismic wall column; the sectional width of the round section column may be calculated according to the square section column in the equal area;

h_0——Effective height of section, which may be the length of wall column for the seismic wall.

6.2.10 The frame-supported columns of partial frame-support seismic wall structure still shall meet the following requirements:

1 The frame-supported columns shall bear the minimum seismic shear force; if the quantity of frame-supported columns is no less than 10, the sum of the seismic shear force bear by the columns shall not be less than 20% of the total seismic shear force at bottom of the structure; otherwise, the seismic shear force bear by each column shall not be less than 2% of the total seismic shear force at the bottom of structure. The seismic bending moment of frame-supported columns shall be adjusted correspondingly.

2 Due to the earthquake action, the additional axial force of Grade 1 and 2 frame-supported columns shall be multiplied by an enhancement coefficient of 1.5 and 1.2 respectively; when calculating the axial compression ratio, this additional axial force may not be multiplied by the enhancement coefficient.

3 As for the Grade 1 and 2 frame-supported columns, the design value of the combined bending moment at the upper ends of top layer columns and the lower ends of bottom layer columns shall be respectively multiplied by an enhancement coefficient of 1.5 and 1.25, and the middle joints of frame-supported columns shall meet the requirements in Article 6.2.2 of this code.

4 The center line of frame-supported beam should be coincided with the center line of frame-supported column.

6.2.11 The bottom reinforced part of the Grade 1 floor-type seismic wall in partial frame-support seismic wall structures still shall meet the following requirements:

1 When the tie bars with diameter not less than 8mm are set with a space no larger than 400mm between two rows of steel reinforcements beyond the boundary components, the shear capacity check of seismic wall may be included in the shear bearing action of the concrete.

2 When large eccentric tension appears in the bottom section of wall column, additional cross anti-sliding diagonal bars should be placed at the bottom section of wall column, and the seismic shear force bear by these anti-skidding diagonal bars may be adopted according to 30% of the design value of shear force at the bottom section of wall column.

6.2.12 The top-storey floor with frame-supported columns of partial frame-support seismic wall structures shall comply with those specified in Section E.1 in Appendix E of this code.

6.2.13 For the seismic calculation of reinforced concrete structures, the following requirements still shall be met:

1 As for the frame-seismic wall structures and frame-core-tube structures with basically uniform distribution of lateral rigidity along vertical configuration, the value of shear force bear by the frame at any storey shall not be less than the smaller value between 20% of the total seismic shear force at bottom of the structure and 1.5 times of the maximum value among the seismic shear force of storeys in the frame part (calculated according to the frame-seismic wall structure and frame-core-tube structure).

2 To calculate the seismic internal force of seismic wall, the rigidity of connection beam may be reduced and the reduction coefficient should not be less than 0.50.

3 When calculating the internal force and deformation of the seismic wall structure, partial frame-support seismic wall structure, frame-seismic wall structure, frame-core-tube structure, tube-in-tube structure and slab-column-seismic wall structure, the seismic walls shall be included in the interaction of the end wing walls.

4 As for the frame structure set with a few seismic walls, the seismic shear force value of the frame part should be adopted with the greater value between the calculation results with frame structure model and frame-seismic wall structure model.

6.2.14 The seismic check of the joint core zone of frame shall meet the following requirements:

1 The joint core zones of the Grade 1, 2 and 3 frames shall be carried out with seismic check; the joint core zones of Grade 4 frames may not be carried out with seismic check but shall meet the requirements of the details of seismic design.

2 The method for section seismic check of core zone shall comply with those specified in Appendix D of this code.

6.3 Details of Seismic Design for Frame Structures

6.3.1 The sectional dimensions of beam should meet the following requirements:

1 The width of section should not be less than 200mm;

2 The height-width ratio of the section should not be larger than 4;

3 The ratio between the clear span and the height of section should not be less than 4.

6.3.2 The flat beams with width larger than column width shall meet the following requirements:

1 Floors and roofs with flat beams shall be casted on the spot, the centerlines of the beams and columns should be coincided, and the flat beams shall be arranged in two directions. The sectional dimensions of the flat beam not only shall meet the following requirements, but also shall comply with the requirements on deflection and crack width as stated in the relevant current codes.

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

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

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

Where b_c——Section width of column, which shall be 1.8 times of the column diameter for the column of round section;

b_b, h_b——Respectively the width and height of the section of beam;

d——Diameter of longitudinal reinforcement for the column.

2 The flat beams should not be used for Grade 1 frame structures.

6.3.3 Arrangement of reinforcements for beams shall meet the following requirements:

1 The ratio between the height and effective height of the concrete compressive area of beam end counted into the compression reinforcement shall not be larger than 0.25 for Grade 1 and shall not be larger than 0.35 for Grade 2 and 3.

2 In addition to being determined through calculation, the ratio between the amount of used longitudinal steel reinforcements at the bottom surface and top surface of the beam end section still shall not be less than 0.5 for Grade 1 and not be less than 0.3 for Grade 2 and 3.

3 The length of the stirrup densified area at the beam end, the maximum space between stirrups and the minimum diameter of stirrup shall be adopted according to Table 6.3.3; if the longitudinal tension reinforcement ratio at the beam end exceeds 2%, the values of the minimum diameter of stirrup shall be increased by 2mm.

Table 6.3.3 Length of Stirrup Densified Area, Maximum Space and Minimum Diameter of Stirrups at Beam End

Seismic grade	Length of densified area (the greater value shall be taken) (mm)	Maximum space of stirrups (the minimum value shall be taken) (mm)	Minimum diameter of stirrup (mm)
1	$2h_b$, 500	$h_b/4$, $6d$, 100	10
2	$1.5h_b$, 500	$h_b/4$, $8d$, 100	8
3	$1.5h_b$, 500	$h_b/4$, $8d$, 150	8
4	$1.5h_b$, 500	$h_b/4$, $8d$, 150	6

Note: 1 d is the diameter of longitudinal steel reinforcement and h_b is the height of section of beam;

2 If the diameter of the stirrup is larger than 12mm, the quantity is no less than 4 and the limb distance is no larger than 150mm, the maximum space of stirrups for Grade 1 and 2 may be increased properly but shall not be larger than 150mm.

6.3.4 Arrangement of steel reinforcements for beams still shall meet the following requirements:

1 The ratio of longitudinal tension reinforcements at the beam end should not be larger than 2.5%. For Grade 1 and 2, the reinforcements at the top surface and bottom surface along the full length of the beam shall not be less than 2ø14 and also shall not be 1/4 of the larger sectional area of the longitudinal steel reinforcements at both ends of the top surface and bottom surface of beam respectively; for Grade 3 and 4, the reinforcements shall not be less than 2ø12.

2 Diameter of each longitudinal steel reinforcement, which extending through the central column in Grade 1, 2 and 3 frame beams, shall neither be larger than 1/20 of the sectional dimension of column with rectangular section in this direction for the frame structures nor be larger than 1/20 of the chord length of column with round section where the longitudinal steel reinforcement locates; as for the frames of other structure types, this diameter should not be larger than 1/20 of the sectional dimension of column with rectangular section in this direction or 1/20 of the chord length of column with round section where the longitudinal steel reinforcement locates.

3 The spacing of stirrup legs in the densified area at beam end should not be larger than the greater value between 200mm and 20 times of the stirrup diameter for Grade 1, should not be larger

than the greater value between 250mm and 20 times of the stirrup diameter for Grade 2 and 3, and should not be larger than 300mm for Grade 4.

6.3.5 The sectional dimension of column should meet the following requirements:

1 The width and height of section should not be less than 300mm for Grade 4 or columns not exceeding 2 storeys, should not be less than 400mm for Grade 1, 2 and 3 and columns exceeding 2 storeys; the diameter of circular column should not be less than 350mm for Grade 4 or columns not exceeding 2 storeys and should not be less than 450mm for Grade 1, 2 and 3 and columns exceeding 2 storeys.

2 The shear span ratio should be larger than 2.

3 The length ratio of the short and long sides of section should not be larger than 3.

6.3.6 The axial compression ratio of column should not exceed those specified in Table 6.3.6; as for the tall buildings built at Class IV site, the axial compression ratio limit shall be reduced properly.

Table 6.3.6 Limit for Axial Compression Ratio of Column

Structure type	Seismic grade			
	1	2	3	4
Frame structure	0.65	0.75	0.85	0.90
Frame-seismic wall, slab-column-seismic wall, frame-core-tube and tube-in-tube	0.75	0.85	0.90	0.95
Partial frame-support seismic wall	0.6	0.7	—	

Note: 1 The axial compression ratio refers to the ratio between the design value of axial pressure of column combination and the product of the total cross-sectional area with the design value of axial compressive strength of concrete; as for such structures without seismic action calculation as specified in this code, the axial compression ratio may be calculated with the design value of combined axial force under no seismic action;

2 The limits listed in the above table are applicable to the columns with shear span ratio larger than 2 and concrete strength grade no larger than C60; as for the columns with shear span ratio no larger than 2, the axial compression ratio limit shall be reduced by 0.05; as for the columns with shear span ratio less than 1.5, the axial compression ratio limit shall be studies especially and special construction measures shall be taken;

3 The axial compression ratio limit may be increased by 1.10 in the following cases: cross compound stirrups are adopted along the total height of column, the spacing of stirrup legs is no larger than 200mm, the space between stirrups is no larger than 100mm and the stirrup diameter is no less than 12mm; or compound spiral stirrups are adopted along the total height of column, the spacing of screws is no larger than 100mm, the spacing of stirrup legs is no larger than 200mm and the stirrup diameter is no less than 12mm; or continuous compound rectangular spiral stirrups are adopted along the total height of column, the clear spacing of screws is no larger than 80mm, the spacing of stirrup legs is no larger than 200mm and the stirrup diameter is no less than 10mm; and the minimum stirrup characteristic values of these three kinds stirrups all shall be determined according to Table 6.3.9 of this code based on the increased axial compression ratio;

4 Additional core column shall be set at the middle part of the column section, thereinto, the total area of the added longitudinal steel reinforcements shall not be less than 0.8% if the cross-sectional area of column and the axial compression ratio limit may be increased by 0.05; if this measure is adopted together with the measure in Note 3, the axial compression ratio limit may be increased by 0.15, but the volume stirrup ratio still may be determined according to the requirements that the axial compression ratio is increased by 0.10;

5 The axial compression ratio of column shall not be larger than 1.05.

6.3.7 Arrangement of steel reinforcements for columns shall meet the following requirements:

1 The minimum total reinforcement ratio of longitudinal bearing force reinforcements for columns shall be adopted according to Table 6.3.7-1, meanwhile, the reinforcement ratio of each side shall not be less than 0.2%; as for the taller buildings built at Class IV site, the minimum total reinforcement ratio shall be increased by 0.1%.

Table 6.3.7-1 Minimum Total Reinforcement Ratio (Percentage) of Longitudinal Steel Reinforcements in Column Section

Type	Seismic grade			
	1	2	3	4
Central column and side column	0.9 (1.0)	0.7 (0.8)	0.6 (0.7)	0.5 (0.6)
Corner column and frame-supported column	1.1	0.9	0.8	0.7

Note: 1 The values bracketed in the table are applicable to the columns of frame structure;

2 If the standard value of reinforcement strength is less than 400MPa, the values in table shall be increased by 0.1; if the standard value of reinforcement strength is 400MPa, the values in table shall be increased by 0.05;

3 If the concrete strength grade is higher than C60, the above-mentioned values shall be increased by 0.1 correspondingly.

2 The stirrups for columns shall be densified within the specified scope, the spacing and diameter of stirrup in the densified area shall meet the following requirements:

1) Generally, the maximum spacing and the minimum diameter of stirrup shall be adopted according to Table 6.3.7-2

Table 6.3.7-2 Maximum Spacing and Minimum Diameter of Stirrups in the Stirrup Densified Area of Column

Seismic grade	Maximum spacing of stirrups (the smaller value shall be taken, mm)	Minimum diameter of stirrup (mm)
1	$6d$, 100	10
2	$8d$, 100	8
3	$8d$, 150 (100 at column root)	8
4	$8d$, 150 (100 at column root)	6 (8 at column root)

Note: 1 d is the minimum diameter of stirrup for column;

2 The column root refers to the stirrup densified area at the lower end of column at the bottom storey.

2) If the diameter of stirrup for Grade 1 frame column is larger than 12mm and the spacing of stirrup legs is no larger than 150mm as well as the diameter of stirrup for Grade 2 frame column is no less than 10mm and the spacing of stirrup legs is no larger than 200mm, except for the lower ends of columns at the bottom storey, the maximum spacing of stirrups may be taken as 150mm; if the sectional dimension of Grade 3 frame column is no larger than 400mml the minimum diameter of stirrup may be taken as 6mm; if the shear span ratio of Grade 4 frame column is no larger than 2, the diameter of stirrup shall not be less than 8mm.

3) As for the frame-supported columns and the frame columns with shear span ratio no larger than 2, the stirrup spacing shall not be larger than 100mm.

6.3.8 Arrangement of longitudinal steel reinforcements for columns still shall meet the following requirements:

1 The longitudinal steel reinforcements for columns should be arranged symmetrically.

2 As for the columns with the side length of section larger than 400mm, the spacing of longitudinal steel reinforcements should not be larger than 200mm.

3 The total reinforcement ratio of column shall not be larger than 5%; as for the columns of the Grade 1 frames with shear span ratio no larger than 2, the longitudinal steel reinforcement ratio on each side should not be larger than 1.2%.

4 As for the side columns, corner columns as well as the end columns of seismic wall, the gross cross-sectional area of the longitudinal steel reinforcements in the column under small eccentric tension shall be increased by 25% of the calculated value.

5 The banding joints of column longitudinal steel reinforcements shall be kept clear from the stirrup densified area at the column end.

6.3.9 The arrangement of stirrups for columns still shall meet the following requirements:

1 The stirrup densifying scope of a column shall be adopted according to the following requirements:

1) At the column end, the maximum value among the section height (diameter of circular column), 1/6 of the net height of column and 500mm shall be taken;

2) As for the columns at the bottom storey, the densified area shall be at the lower end of column, in a scope no less than 1/3 of the net height of column;

3) The areas of 500mm up and down the rigid ground respectively;

4) As for the columns with shear span ratio no larger than 2, the columns formed by setting filler wall, the columns with the ratio between net height and section height no larger than 4, frame-supported columns as well as corner columns of Grade 1 and 2 frames, the total height of the column are shall be taken as the densified area.

2 In the stirrup densified area of column, the spacing of stirrup legs should not be larger than 200mm for Grade 1 and should not be larger than 300mm for Grade 4. The stirrups or tie bars should be arranged in two directions for every other longitudinal steel reinforcement for confinement; if tie bar compound stirrups are adopted, the tie bars shall abut against the longitudinal steel reinforcement and hook with the stirrups.

3 The volume stirrup ratio in the stirrup densified area of column shall be adopted according to the following requirements:

1) The volume stirrup ratio in the stirrup densified area of column shall meet the requirements of the following formula:

$$\rho_v \geq \lambda_v f_c / f_{yv} \tag{6.3.9}$$

Where ρ_v——Volume stirrup ratio in the stirrup densified area of column, which shall not be less than 0.8% for Grade 1, 0.6% for Grade 2 and 0.4% for Grade 3 and 4; to calculate the volume stirrup ratio of compound spiral stirrups, the volume of spiral stirrups shall be multiplied by a reduction coefficient of 0.80;

f_c——Design value of the axial compressive strength of concrete, which shall be calculated according to C35 if the strength grade is less than C35;

f_{yv}——Design value of the tensile strength of stirrup or tie bar;

λ_v——Minimum stirrup characteristic value, which should be adopted according to Table 6.3.9.

Table 6.3.9 Minimum Stirrup Characteristic Value in Stirrup Densified Area of Column

Seismic grade	Stirrup type	Axial compression ratio of column								
		≤0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.05
1	Ordinary stirrup and compound stirrup	0.10	0.11	0.13	0.15	0.17	0.20	0.23	—	—
	Spiral stirrup, compound or continuous compound rectangular spiral stirrup	0.08	0.09	0.11	0.13	0.15	0.18	0.21	—	—
2	Ordinary stirrup and compound stirrup	0.08	0.09	0.11	0.13	0.15	0.17	0.19	0.22	0.24
	Spiral stirrup, compound or continuous compound rectangular spiral stirrup	0.06	0.07	0.09	0.11	0.13	0.15	0.17	0.20	0.22
3, 4	Ordinary stirrup and compound stirrup	0.06	0.07	0.09	0.11	0.13	0.15	0.17	0.20	0.22
	Spiral stirrup, compound or continuous compound rectangular spiral stirrup	0.05	0.06	0.07	0.09	0.11	0.13	0.15	0.18	0.20

Note: "Ordinary stirrup" refers to single rectangular stirrup and single circular stirrup, "compound stirrup" refers to the rectangular, polygonal and circular stirrups or the stirrup composed of tie bars; "compound spiral stirrup" refers to spiral stirrup, rectangular, polygonal and circular stirrups or the stirrup composed of tie bars; "continuous compound rectangular spiral stirrup" refers to a stick of full-length stirrup processed with steel reinforcements.

2) The frame-supported columns should be adopted with compound spiral stirrups or cross compound stirrups, the minimum stirrup characteristic value shall be 0.02 larger than the values stated in Table 6.3.9 and the volume stirrup ratio shall not be less than 1.5%.

3) The columns with shear span ratio no larger than 2 should be adopted with compound spiral stirrups or cross compound stirrups, the volume stirrup ratio shall not be less than 1.2% and also shall not be less than 1.5% for Grade 1 for Intensity 9.

4 The arrangement of stirrups in the stirrup non-densified area of column shall meet the following requirements:

1) The volume stirrup ratio in the stirrup non-densified area of column should not be less than 50% of the densified area.

2) The stirrup spacing shall not be larger than 10 times of the diameter of longitudinal steel reinforcement for Grade 1 and 2 frame columns and shall not be larger than 15 times for the Grade 3 and 4 frame columns.

6.3.10 The maximum spacing and minimum diameter of the stirrup in the joint core zone of frame should be adopted according to Article 6.3.7 of this code; the stirrup characteristic value in the joint core zones of Grade 1, 2 and 3 frames should not be less than 0.12, 0.10 and 0.08 respectively and the volume stirrup ratio should not be less than 0.6%, 0.5% and 0.4% respectively. In the joint core zone of frame with the column shear span ratio no larger than 2, the volume stirrup ratio should not be less than the larger volume stirrup ratio at the upper and lower column ends in the core zone.

6.4 Details of Seismic Design for Seismic Wall Structures

6.4.1 The thickness of seismic wall shall not be less than 160mm and should not be less than 1/20 of the storey height or non-support part length for Grade 1 and 2, and shall not be less than 140mm and should not be less than 1/25 of the storey height or non-support part length for Grade 3 and 4; if no end column or wing wall is set, this thickness should not be less than 1/16 of the storey height or non-support part length for Grade 1 and 2 and should not be less than 1/20 of the storey height or non-support part length for Grade 3 and 4.

The wall thickness at the bottom reinforced part shall not be less than 200mm and should not be less than 1/16 of the storey height or non-support part length for Grade 1 and 2, and shall not be less than 160mm and should not be less than 1/20 of the storey height or non-support part length for Grade 3 and 4; if no end column or wing wall is set, the wall thickness should not be less than 1/12 of the storey height or non-support part length for Grade 1 and 2, and should not be less than 1/16 of the storey height or non-support part length for Grade 3 and 4.

6.4.2 As for the Grade 1, 2 and 3 seismic walls, the axial compression ratio of wall columns under the action of the representative value of gravity load should not be larger than 0.4 for Grade 1 for Intensity 9 and should not be larger than 0.5 for Intensity 7 and 8; this ratio also should not be larger than 0.6 for Grade 2 and 3.

Note: The axial compression ratio of wall column refers to the ratio between the design value of the axial pressure of wall and the product of the gross cross-sectional area of wall with the design value of axial compressive strength of concrete.

6.4.3 The reinforcements for the vertically and transversely distributed steel reinforcements of seismic wall shall meet the following requirements:

1 The minimum reinforcement ratio of the vertically and transversely distributed steel reinforcements of Grade 1, 2 and 3 seismic walls all shall not be less than 0.25% and that of the distribution reinforcements of Grade 4 seismic wall shall not be less than 0.20%.

Note: As for the Grade 4 seismic walls with height less than 24m and very small shear pressure ratio, the minimum reinforcement ratio of the vertically distributed reinforcements may be taken as 0.15%.

2 For the floor-type seismic walls of partial frame-support seismic wall structures, the reinforcement ratio of steel reinforcements vertically and transversely distributed in the bottom reinforced part shall not be less than 0.3%.

6.4.4 Arrangement of the vertically and transversely distributed steel reinforcements for seismic walls still shall meet the following requirements:

1 The spacing of the vertically and transversely distributed steel reinforcements for seismic walls should not be larger than 300mm; in the bottom reinforced part of the floor-type seismic walls of partial frame-support seismic wall structures, the spacing of vertically and transversely distributed steel reinforcements should not be larger than 200mm.

2 If the thickness of seismic wall is larger than 140mm, the vertically and transversely distributed steel reinforcement shall be arranged in double rows, the spacing of the tie bars between the double-row distributed steel reinforcements should not be larger than 600mm and the diameter hereof shall not be less than 6mm.

3 The diameter of the vertically and transversely distributed steel reinforcements for seismic

walls all should not be larger than 1/10 of wall thickness and shall not be less than 8mm; the diameter of vertical steel reinforcement should not be less than 10mm.

6.4.5 Boundary components, including concealed columns, end columns and wing walls, shall be arranged at both ends of the seismic wall and both sides of the opening and shall meet the following requirements:

1 As for the seismic wall structures, the structural boundary components may be arranged at both ends of the wall columns of Grade 1, 2 and 3 seismic walls with the axial compression ratio of the base wall column section no larger than those specified in Table 6.4.5-1 and of the Grade 4 seismic walls, the range of structural boundary components may be adopted according to Figure 6.4.5-1, and the reinforcements of structural boundary components not only shall meet the requirements on bend bearing capacity, but also should meet those specified in Table 6.4.5-2.

Table 6.4.5-1 Maximum Axial Compression Ratio of Structural Boundary Components for Seismic Walls

Seismic grade or Intensity	Grade 1 (Intensity9)	Grade 1 (Intensity 7 and8)	Grade 2 and 3
Axial compression ratio	0.1	0.2	0.3

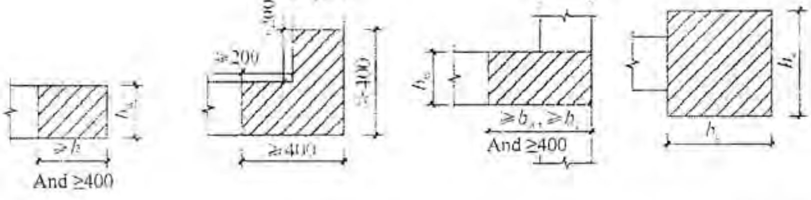
Table 6.4.5-2 Reinforcement Requirements of Structural Boundary Components for Seismic Wall

Seismic grade	Bottom reinforced part			Other parts		
	Minimum longitudinal steel reinforcement (larger value)	Stirrup		Minimum longitudinal steel reinforcement (larger value)	Tie bar	
		Minimum diameter (mm)	Maximum vertical spacing (mm)		Minimum diameter (mm)	Maximum vertical spacing (mm)
1	$0.010A_c$, 6ø16	8	100	$0.008A_c$, 6ø14	8	150
2	$0.008A_c$, 6ø14	8	150	$0.006A_c$, 6ø12	8	200
3	$0.006A_c$, 6ø12	6	150	$0.005A_c$, 4ø12	6	200
4	$0.005A_c$, 4ø12	6	200	$0.004A_c$, 4ø12	6	250

Note: 1 A_c is the cross-sectional area of boundary component;

2 The horizontal spacing of tie bars at the other parts shall not be larger than 2 times of the spacing of longitudinal reinforcements; and stirrups should be adopted at the corners;

3 If the end columns bear the concentrated load, the diameter and spacing of the longitudinal steel reinforcements and stirrups for them shall meet the corresponding requirements of column.



(a) Concealed column (b) Wing column (c) End column

Figure 6.4.5-1 Range of Structural Boundary Components for Seismic Wall

2 As for the Grade 1, 2 and 3 seismic walls with the axial compression ratio in the bottom section of wall column at bottom storey larger than those specified in Table 6.4.5-1 as well as the seismic walls of partial frame-support seismic wall structures, restraining boundary components shall be set at the bottom reinforced part and the adjacent upper storey, and the structural boundary components may be set at other positions above the bottom reinforced part and the adjacent upper storey. The length of restraining boundary components along the wall column, the stirrup characteristic value, and the stirrups and longitudinal steel reinforcements should meet the requirements of Table 6.4.5-3 (Figure 6.4.5-2).

Table 6.4.5-3 Range and Reinforcement Requirements of Restraining Boundary Components for Seismic Wall

Item	Grade 1 (Intensity 9)		Grade 1 (Intensity 8)		Grade 2 and 3	
	$\lambda \le 0.2$	$\lambda > 0.2$	$\lambda \le 0.3$	$\lambda > 0.3$	$\lambda \le 0.4$	$\lambda > 0.4$
l_c (Concealed column)	$0.20h_w$	$0.25h_w$	$0.15h_w$	$0.20h_w$	$0.15h_w$	$0.20h_w$
l_c (Wing column and end column)	$0.15h_w$	$0.20h_w$	$0.10h_w$	$0.15h_w$	$0.10h_w$	$0.15h_w$
λ_v	0.12	0.20	0.12	0.20	0.12	0.20
Longitudinal steel reinforcement (larger value)	$0.012A_c$, 8ø16		$0.012A_c$, 8ø16		$0.010A_c$, 6ø16 (6ø14 for Grade 3)	
Vertical spacing of stirrups or tie bars	100mm		100mm		150mm	

Note: 1 If the length of wing wall for seismic wall is less than 3 times of the wall thickness or the side length of the section of end column is less than 2 times of the wall thickness, the above table shall be looked up according to the condition of no wing wall or end column;

2 l_c is the length of restraining boundary component along the wall column and shall neither be less than the wall thickness nor 400mm; if wing wall or end column is adopted, the length shall neither be less than the thickness of the wing wall or the sum of section height of end column along the direction of wall column with 300mm;

3 λ_v is the stirrup characteristic value of restraining boundary component, the volume stirrup ratio may be calculated according to Formula (6.3.9) in this code, and the cross-sectional area of the horizontally-distributed steel reinforcements meeting the construction requirements and with reliable anchoring at wall ends may be included properly;

4 h_w is the length of seismic wall column;

5 λ is the axial compression ratio of wall column;

6 A_c is the cross-sectional area of the shaded area of restraining boundary components in Figure 6.4.5-2.

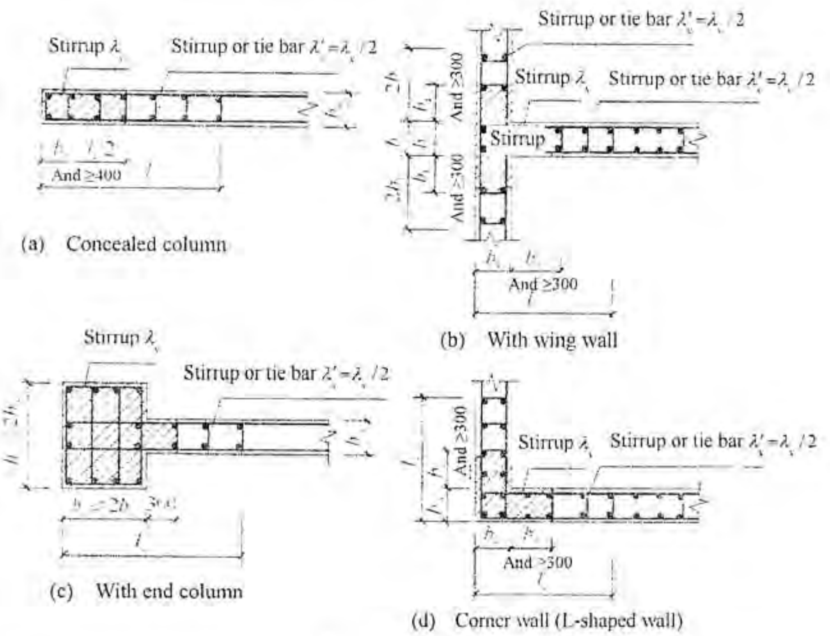


Figure 6.4.5-2 Restraining Boundary Components of Seismic Wall

6.4.6 If the wall column length of seismic wall is no larger than 3 times of the wall thickness, the seismic wall shall be designed according to the relevant requirements of column; if the thickness of the rectangular wall column is no larger than 300mm, the stirrups still shall be densified along the total height.

6.4.7 The tall connection beams with small span-height ratio, horizontal joint may be set to form double or multiple connection beams or other construction may be adopted to strengthen the shear bearing capacity. The longitudinal steel reinforcements for the connection beams at top storey shall be extended into the anchorage length scope of wall, and stirrups shall be arranged.

6.5 Details of Seismic Design for Frame-seismic Wall Structures

6.5.1 As for the frame-seismic wall structures, the thickness of seismic wall and the installation of frames shall meet the following requirements:

1 The thickness of seismic wall shall not be less than 160mm and should not be less than 1/20 of the storey height or non-support part length, the thickness of the seismic wall for bottom reinforced part shall not be less than 200mm and should not be less than 1/16 of the storey height or non-support part length.

2 If end columns are adopted, the wall shall be set with concealed beams at the floor, the section height of concealed beam should not be less than the greater value between wall thickness and 400mm; the end column should have the same section as the frame column at the same storey and also shall meet the requirements for frame columns as stated in Section 6.3 of this code; the end columns at the bottom reinforced part of seismic wall and the end columns abutting against the opening of seismic wall should be set with densified stirrups along the total height according to the requirements on stirrup densified area.

6.5.2 The ratio of vertically and transversely distributed steel reinforcements for seismic wall all shall not be less than 0.25%, the diameter of steel reinforcement should not be less than 10mm, the spacing of steel reinforcements should not be larger than 300mm, the steel reinforcements shall be arranged in double rows and tie bars shall be arranged between these two rows.

6.5.3 If the floor beam is connected with the outer plane of seismic wall, it should not be supported on the connection beam at opening; the seismic wall connected to the beam should be designed along the axial direction of the beam, the longitudinal reinforcements of the beam shall be anchored into the wall; or the support-wall column or concealed column may be ser at the position of support beam, and their sectional dimension and reinforcements shall be determined through calculation.

6.5.4 The other details of seismic design of the frame-seismic wall structures shall comply with the relevant requirements of Section 6.3 and Section 6.4 of this code.

Note: As for the frame structures set with a few seismic walls, the details of seismic design of these seismic walls still may be carried out according to the requirements on seismic wall as stated in Section 6.4 of this code

6.6 Requirements for Seismic Design of Slab-column-seismic Wall Structures

6.6.1 As for the seismic walls of slab-column-seismic wall structures, in addition to this section, the details of seismic design still shall comply with the relevant regulations specified in Section 6.5 of this code; the details of seismic design of columns (including the end columns of seismic wall) and beams shall comply with the relevant regulations specified in Section 6.3 of this code.

6.6.2 The structural layout of slab-column-seismic wall still shall meet the following requirements:

1 The thickness of seismic wall shall not be less than 180mm and should not be less than 1/20 of the storey height or non-support part length; if the height of buildings is larger than 12m, the wall thickness shall not be less than 200mm.

2 Beam frames shall be designed around the building, and margin frame beams should be designed around the building elevator opening.

3 For Intensity 8, slab column joints with supporting board or column cap should be used, and the thickness (including board thickness) of supporting board or column cap at root should not be less than 16 times of the diameter of column longitudinal reinforcement. The side length of supporting board or column cap should not be less than 4 times of the sum of board thickness and corresponding side length of column section.

4 The ceiling board of first underground storey of a building should be adopted with beam and slab structure.

6.6.3 The seismic calculation of slab-column-seismic wall structure shall meet the following requirements:

1 If the height of the building is larger than 12m, the seismic wall shall bear all the earthquake action of the structure; if not, the seismic wall should bear all the earthquake action of the structure. The slab columns and frames at each storey shall be able to bear no less than 20% of the seismic shear force of this storey.

2 When the slab-column structure under earthquake action is analyzed according to equal-substitute plane frame, the width of the equal-substitute beam should be 1/4 of the spacing of columns on both sides perpendicular to the direction of equal-substitute plane frame.

3 The slab column joints shall be carried out with seismic check for their punching bearing capacity, in which the punching caused by the unbalanced bending moment shall be included, the design value of the punching counterforce caused by the unbalanced bending moment of earthquake action combination at joint shall be multiplied by an enhancement coefficient, and this enhancement coefficient for Grade 1, 2 and 3 slab columns may be taken as 1.7, 1.5 and 1.3 respectively.

6.6.4 The construction of slab column joints of slab-column-seismic wall structure shall meet the following requirements:

1 The flat slab without column cap shall be set with concealed beams in the slab strip on column, the width of concealed beam may be taken as the column width plus no more than 1.5 times of slab thickness on both sides of the column. The area of the upper steel reinforcements of concealed beam support shall not be less than 50% of that for the slab strip on column, the quantity of the steel reinforcements at lower part of concealed beam should not be less than 1/2 of that of the upper steel reinforcements; the stirrup diameter shall not be less than 8mm, the stirrup spacing should not be larger than 3/4 of the slab thickness, the spacing of stirrup legs should not be larger than 2 times of the slab thickness, and the stirrups shall be densified at both ends of the concealed beam.

2 The steel reinforcements at the bottom of slabs on such column without cap should be connected away from the column surface by no less than 2 times of the slab thickness, and the steel reinforcements, if overlapped, should be set with hooks at their ends perpendicular to the slab surface.

3 The gross cross-sectional area of the continuous steel reinforcements at slab bottom through the column section along the directions of two major axes shall meet requirements of the following formula:

$$A_s \geq N_G / f_y \tag{6.6.4}$$

Where A_s——Gross cross-sectional area of continuous steel reinforcements at slab bottom;

N_G——Design value of axial pressure of column under the action (the vertical earthquake action also should be considered for Intensity 8) of the representative value of gravity load on the slab of this storey;

f_y——Design value of tensile strength of the steel reinforcements for floor slab.

4 The slab column joints shall be set with shear resistant studs or punching resistant steel reinforcements.

6.7 Requirements for Seismic Design of Tube Structures

6.7.1 The frame-core-tube structures shall meet the following requirements:

1 The floor between core tube and frame should be adopted with the beam slab system; if only part of the storeys is adopted with flat slab system, proper strengthening measures shall be taken.

2 Except for the strengthening storeys as well as their adjacent storey up and down, the maximum value of seismic shear force of the storeys in frames calculated and analyzed according to frame-core-tube should not be less than 10% of the total seismic shear force at bottom of the structure, otherwise, the seismic shear force of the core-tube wall shall be improved appropriately and the details of seismic design of boundary components shall be strengthened properly; the seismic shear force bear by the frame part of any storey shall not be less than 15% of the total seismic shear force at bottom of the structure.

3 The design of strengthening storey shall meet the following requirements:

1) The strengthening storey shall not be adopted for Intensity 9;

2) The girder or truss of the strengthening storey shall be penetrated with the wall column in core tube; the connection of the girder or truss with the surrounding frame columns should be adopted with hinged connection or semi-rigid connection;

3) The influence of the deformation of strengthening storey shall be considered for the integral analysis of structure;

4) In term of the construction procedures and connection details, measures shall be taken to reduce the influence of the vertical temperature deformation and axial compression of structure on the strengthening storey.

6.7.2 As for the core tube of frame-core-tube structure and inner tube of tube-in-tube structure, their seismic walls not only shall meet the relevant regulations specified in Section 6.4 of this code, but also shall meet the following requirements:

1 The thickness of seismic wall and the vertically and transversely distributed steel reinforcements shall comply with those specified in Section 6.5 of this code; at the bottom reinforced part of the tube and the adjacent upper storey, the wall thickness should not be changed of the lateral rigidity is free from sudden changes.

2 The boundary components at corners of the Grade 1 and 2 tubes in frame-core-tube structure should be strengthened according to the following requirements: stirrups should be adopted in the range of restraining boundary components at the bottom reinforced part, and the length of the restraining boundary components along the wall column should be 1/4 of the section height of wall column, restraining boundary components should be set within the total height above the bottom reinforced part according to the requirements of seismic wall column.

3 The door opening of inner tube should not be near to corners.

6.7.3 The floor girder should not be supported on the connection beam of inner tube. The connection of floor girder with the wall plane of inner tube or core tube, if adopted, shall meet those specified in Article 6.5.3 of this code.

6.7.4 The connection beams with span-height ratio no larger than 2 in Grade 1 and 2 core tubes and inner tubes may be adopted with cross concealed column reinforcements and ordinary stirrups if the section width of beam is no less than 400mm; if the section width is not less than 200mm but less than 400mm, besides ordinary stirrups, the oblique cross constructional reinforcements also may be added.

6.7.5 The seismic design of the transfer storey of tube structure shall comply with those specified in Section E.2 of this code.

7 Multi-storey Masonry Buildings and Multi-storey Masonry Buildings with R.C. Frames on Ground Floors

7.1 General Requirements

7.1.1 This chapter is applicable to multi-storey buildings bearing with masonry such as common bricks (including fired, autoclaved and concrete common bricks), perforated bricks (including fired and concrete perforated bricks) and small concrete hollow blocks, as well as and masonry buildings with R.C. frames on ground floors or two storeys from ground floors.

The seismic design of buildings with small reinforced concrete hollow blocks shall meet the requirements specified in Appendix F of this code.

Note: 1 For masonry buildings with fired bricks, autoclaved bricks and concrete bricks by adopting non-clay, the material property of blocks shall be provided with reliable test data; they may be implemented according to the corresponding requirements of buildings with common bricks and perforated bricks specified in this chapter;

2 In this chapter, "small concrete hollow block" is hereinafter referred to as "small block";

3 The seismic design for non-spacious single-storey masonry buildings may be carried out according to principles specified in this chapter.

7.1.2 The height and storey number of multi-storey buildings shall meet the following requirements:

1 In general conditions, the total height and storey number of buildings shall not exceed those specified in Table 7.1.2.

Table 7.1.2 Storey Number and Total Height Limit of Buildings (m)

Building category		Minimum thickness of seismic wall (mm)	Intensity and design basic acceleration of ground motion											
			6		7				8				9	
			0.05g		0.10g		0.15g		0.20g		0.30g		0.40g	
			Height	Storey number	Height	Storey number	Height	Storey number	Height	Storey number	Height	Storey number	Height	Storey number
Multi-storey masonry building	Common brick	240	21	7	21	7	21	7	18	6	15	5	12	4
	Perforated brick	240	21	7	21	7	18	6	18	6	15	5	9	3
	Perforated brick	190	21	7	18	6	15	5	15	5	12	4	—	—
	Small block	190	21	7	21	7	18	6	18	6	15	5	9	3
Masonry buildings with R.C. frames on ground floors and seismic wall	Common brick and perforated brick	240	22	7	22	7	19	6	16	5	—	—	—	—
	Perforated brick	190	22	7	19	6	16	5	13	4	—	—	—	—
	Small block	190	22	7	22	7	19	6	16	5	—	—	—	—

Note: 1 The total height of buildings refers to the height from the outdoor ground to the top of the main roof slab or the cornice; the height of the semi basement is counted from the indoor ground of the basement; the height of the whole basement and the semi basement with good built-in conditions shall be allowed to count from the outdoor ground; the height of the sloping roof with attics shall be counted to the position at 1/2 height of the pediment wall;

2 If the height difference of the indoor and outdoor is greater than 0.6m, the total height of buildings shall be allowed to be properly increased over the data given in the table; but the increment shall be less than 1.0m;

3 Category B multi-storey masonry buildings still shall look up in the table according to the precautionary Intensity in the area, the storey number shall reduce one and the total height shall lower 3m; masonry buildings with R.C. frames on ground floors and seismic walls shall not be adopted;

4 Masonry buildings with small blocks in this table exclude masonry buildings of small concrete hollow block with reinforcement.

2 For multi-storey masonry buildings with fewer transverse walls, the total height shall reduce 3m than those specified in Table 7.1.2, and the storey number shall be reduced by one correspondingly; for multi-storey masonry buildings with very few transverse walls at each storey, the storey number still shall be reduced one more.

Note: The fewer transverse walls refer to that rooms with bay which greater than 4.2m in the same storey occupy 40% above of the total area of the storey; hereinto, the very few transverse walls refer to that rooms with bay which not greater than 4.2m occupy less than 20% of the total area of the storey and rooms with bay which greater than 4.8m occupy 50% above of the total area of the storey.

3 Category C multi-storey masonry buildings with fewer transverse walls for Intensity 6 and 7, if the strengthening measures are taken according to the requirements and they meet the requirements of seismic capacity, it shall be allowed that the height and storey number are still adopted according to those specified in Table 7.1.2.

4 As for masonry buildings with autoclaved lime-sand bricks and autoclaved fly ash bricks, the storey number of buildings shall be reduced one than buildings with common bricks and the total height shall be reduced 3m if the shear strength of masonry only reaches 70% that of the masonry with common clay bricks. The requirements of the storey number and total height of buildings are the same to that of buildings with common bricks.

7.1.3 The storey height of buildings bearing with multi-storey masonry shall not exceed 3.6m.

For the bottom of masonry buildings with R.C. frames on ground floors and seismic walls, the storey height shall not exceed 4.5m; the storey height of the ground floor shall not exceed 4.2m if the seismic walls with constrained masonry are adopted.

Note: The storey height shall not exceed 3.9m for the buildings with common bricks by taking strengthening measures such as the constrained masonry if there is indeed in need of the functions of use.

7.1.4 The maximum ratio of the total height and total width of multi-storey masonry buildings should be in accordance with those specified in Table 7.1.4.

Table 7.1.4 Maximum Height-width Ratio of Buildings

Intensity	6	7	8	9
Maximum height-width ratio	2.5	2.5	2.0	1.5

Note: 1 The total width of buildings with single-side corridors excludes the corridor width;

2 The height-width ratio should be reduced properly if the architectural plane is close to the square.

7.1.5 The spacing of seismic transverse walls of buildings shall not exceed those specified in Table 7.1.5:

Table 7.1.5 Spacing of Seismic Transverse Walls of Buildings

Building category		Intensity			
		6	7	8	9
Multi-storey masonry building	Cast-in-situ and monolithic-fabricated concrete floors and roofs	15	15	11	7
	Fabricated reinforced concrete floors and roofs	11	11	9	4
	Wooden roof	9	9	4	—
Bottom frame-seismic wall masonry buildings	Upper storeys	Same as the multi-storey masonry building			—
	Ground floor or two storeys from the ground floor	18	15	11	—

Note: 1 For the top storey of multi-storey masonry buildings, the maximum transverse wall spacing except that of the wooden roof shall be allowed to broaden properly, but the corresponding strengthening measure shall be taken;

2 If the thickness of seismic transverse walls of perforated bricks is 190mm, the maximum transverse wall spacing shall be reduced 3m than the value given in the table.

7.1.6 The partial dimension limit of masonry wall sections in multi-storey masonry buildings should in accordance with those specified in Table 7.1.6:

Table 7.1.6 Partial Dimension Limit of Buildings (m)

Position	Intensity 6	Intensity 7	Intensity 8	Intensity 9
Minimum width of wall between bearing windows	1.0	1.0	1.2	1.5
Minimum distance from the end of bearing external wall to the edge of door/window opening	1.0	1.0	1.2	1.5
Minimum distance from the end of non-bearing external wall to the edge of door/window opening	1.0	1.0	1.0	1.0
Minimum distance from the exposed corner of the internal wall to the edge of door/window opening	1.0	1.0	1.5	2.0
Maximum height of non-anchoring parapet walls (non-exit-and-entrance positions)	0.5	0.5	0.5	0.0

Note: 1 The partial strengthening measures shall be taken to make up if the partial dimension is not enough, and the minimum width should not be less than 1/4 of the storey height and 80% of tabular data;

2 Parapet walls at the position of exits and entrances shall be provided with anchorages.

7.1.7 The architectural layout and structural system of multi-storey masonry buildings shall meet the following requirements:

1 The structural system that bearing by transverse walls or jointly bearing by longitudinal and transverse walls shall be adopted prior. The structural system that mixed-bearing by masonry walls and concrete walls shall not be adopted.

2 The layout of longitudinal and transverse masonry seismic walls shall meet the following requirements:

1) They should be uniform, symmetrical and aligned along the plane inside, and shall be continued up and down along the vertical; and the number of longitudinal and transverse walls should not differ largely;

2) The concave-convex dimension of the plane profile shall not exceed 50% of the typical

dimension; strengthening measures shall be taken at corners of buildings if it exceed 25% of the typical dimension;

3) The dimension of the partial big opening of the floor-slab should not exceed 30% of the floor-slab width and openings shall not be punched simultaneously at the both sides of the wall;

4) If the height difference of the floor-slab of the building split-storey exceeds 500mm, the layout shall be calculated according to two storeys; strengthening measures shall be taken for walls on split-storey positions;

5) The width of walls between windows on the same axes should be uniform, the area of the opening of the wall surface should not be greater than 55% of the total area of the wall surface for Intensity 6 and 7, should not be greater than 50% of the total area of the wall surface for Intensity 8 and 9;

6) Inner longitudinal walls shall be arranged in the middle of the building along the width direction, and the accumulative total length should not be less than 60% of the total length of the building (wall sections with height-width ratio greater than 4 are not counted in).

3 The seismic joint should be set, the wall shall be arranged at both sides of the joint, the joint width shall be determined according to the Intensity and building height adopting within 70mm~100mm if one of the following conditions exists:

1) The height difference of vertical plane in the building is 6m above;

2) The building is with split-storey and the floor-slab height difference is larger than 1/4 of the storey height;

3) The structural rigidity and quality of each part is completely different.

4 The staircase should not be set at the end of or at the corner of the building.

5 The corner window shall not be arranged at the corner of the building.

6 The cast-in-situ reinforced concrete floor and roof should be adopted for buildings with less transverse walls and larger spans.

7.1.8 The structural layout of masonry buildings with R.C. frames on ground floors and seismic walls shall meet the following requirements:

1 Besides the exceptional wall section nearby the staircase, the upper masonry wall and the bottom frame beam or seismic wall all shall be aligned.

2 Bottom of buildings. The certain quantity of seismic walls shall be arranged along longitudinal and transverse directions and they shall be arranged evenly and symmetrically. The masonry seismic wall with constrained common brick masonry or small block masonry that built in the frames shall be allowed to adopt for masonry buildings with R.C. frames on ground floors and seismic walls whose total storey number do not exceed four for Intensity 6.But the additional axial force and shear force that masonry walls pressed on frames shall be counted in and the seismic check for ground floors shall be carried out. The reinforced concrete seismic wall and constrained masonry seismic wall shall not be adopted simultaneously in the same

direction; for rest conditions, the reinforced concrete seismic wall shall be adopted for Intensity 8, the reinforced concrete seismic wall or the seismic wall of small block masonry with reinforcement shall be adopted for Intensity 6 and 7.

3 In the longitudinal and transverse directions of masonry buildings with R.C. frames on ground floors and seismic walls, the ratio of lateral rigidity counted in the effect of the constructional column in the second storey and the lateral rigidity of the ground floor shall not be greater than 2.5 for Intensity 6 and 7, not be greater than 2.0 for Intensity 8 and all shall not be less than 1.0.

4 In the longitudinal and transverse directions of masonry buildings with R.C. frames on ground floors and seismic walls, the lateral rigidity of the ground floor and the second storey of the bottom shall be close to each other; the ratio of lateral rigidity counted in the effect of the constructional column in the third storey and the lateral rigidity of the second storey of the bottom shall not be greater than 2.0 for Intensity 6 and 7, not be greater than 1.5 for Intensity 8 and all shall not be less than 1.0.

5 Seismic walls of masonry buildings with R.C. frames on ground floors and seismic walls shall be arranged foundations of good integrity such as strip foundations and raft foundations.

7.1.9 Besides meeting the requirements of this chapter, the reinforced concrete structure part of masonry buildings with R.C. frames on ground floors and seismic walls still shall meet the relevant requirements of Chapter 6 of this code; here, the seismic grade of bottom concrete frames shall be adopted respectively according to Grade 3, 2 and 1 for Intensity 6, 7 and 8; the seismic grade of concrete walls shall be adopted respectively according to Grade 3, 3 and 2 for Intensity 6, 7 and 8.

7.2 Essentials in Calculation

7.2.1 The base shear method may be adopted for the seismic calculation of multi-storey masonry buildings and masonry buildings with R.C. frames on ground floors and seismic walls, and the earthquake action effect shall be adjusted according to the requirements of this section.

7.2.2 For masonry buildings, the section seismic capacity check may be carried out on the wall section with greater subordinate area or less vertical stress.

7.2.3 When carrying out the seismic shear force distribution and the cross section check, the storey equivalent lateral rigidity of masonry wall sections shall be determined according to the following principles:

1 The calculation of rigidity shall take account of the effect of height-width ratio. only the shear deformation may be calculated if the height-width ratio is less than 1; the bend and shear deformation shall be simultaneously calculated if the height-width ratio is not greater than 4 and not less than 1; the equivalent lateral rigidity may take 0.0 if the height-width ratio is greater than 4.

Note: The height-width ratio of wall sections refers to the ratio of storey height and the length of the wall; the height-width ratio of small wall sections at the window hole edge of the opposite door refers to the ratio of the hole net height and the hole sidewall width.

2 Wall sections should be divided according to openings of doors and windows; the rigidity calculated according to gross wall surfaces of small opening wall sections arranged with constructional columns may be that the punch rate multiplies the opening influence coefficient of wall sections specified in Table 7.2.3:

Table 7.2.3 Influence Coefficient of Opening in Wall Section

Punch rate	0.10	0.20	0.30
Influence coefficient	0.98	0.94	0.88

Note: 1 The punch rate is the ratio of the horizontal section are of the opening and the horizontal gross sectional area of the wall section; the wall section that the net width of adjacent openings less than 500mm shall be taken as the opening;

2 If the opening center line deviates larger than 1/4 of the wall section length from the wall section center line, the influence coefficient in the table shall be reduced by 0.9; if the ceiling height of the door opening is larger than 80% of the storey height, values listed in the table are not applicable; if the window opening height is larger than 50% of the storey height, it is treated as a door opening.

7.2.4 The earthquake action effect of masonry buildings with R.C. frames on ground floors and seismic walls shall be adjusted according to the following requirements:

1 For masonry buildings with R.C. frames on ground floors and seismic walls, the design value of the longitudinal and transverse seismic shear force on the ground floor all shall multiply the enhancement coefficient; the value shall be allowed to adopt within the range of 1.2~1.5; the bigger value of the lateral rigidity ratio of the second storey and ground floor shall be taken.

2 For masonry buildings with R.C. frames on two storeys from ground floors and seismic walls, the design value of the longitudinal and transverse seismic shear force on the ground floor and the second storey also shall all multiply the enhancement coefficient; the value shall be allowed to adopt within the range of 1.2~1.5; the bigger value of the lateral rigidity ratio of the third storey and the second storey shall be taken.

3 The design value of the longitudinal and transverse seismic shear force on the ground floor or two storeys from the ground floor shall be undertaken by the seismic wall along the direction and be distributed according to the lateral rigidity ratio of each wall.

7.2.5 In masonry buildings with R.C. frames on ground floors and seismic walls, the earthquake action effect of frames on ground floors should be determined by adopting the following methods:

1 The seismic shear force and axial force of frame columns on ground floors should be adjusted according to the following requirements:

1) The design value of seismic shear force bearing by frame columns may be distributed and determined according to the effective lateral rigidity ratio of each lateral-force-resisting component; the value of the effective lateral rigidity of the frame shall not be reduced; the value of the effective lateral rigidity of the concrete wall or the masonry wall with reinforcement small concrete block may multiply the reduction coefficient 0.30; the value of the effective lateral rigidity of the seismic wall with constrained common brick masonry or small block masonry may multiply the reduction coefficient 0.20;

2) The additional axial force caused by the seismic overturning moment shall be counted in the axial force of the frame column; the upper brick building may be taken as the rigid body; the seismic overturning moment bearing by each axe on the ground floor may be distributed and determined approximately according to the effective lateral rigidity ratio of the seismic wall and frame on the ground floor;

3) If the length-width ratio of the floor between seismic walls is greater than 2.5, the

seismic shear force and axial force bearing by each axe of frame columns still shall be accounted into the influence of deformation in the plane of the floor.

2 The suitable calculation diagram shall be adopted when the seismic combination internal force is calculated for the reinforced concrete bressummer of masonry buildings with R.C. frames on ground floors and seismic walls. The relevant calculation parameters such as the moment coefficient and axial force coefficient may be adjusted and the adverse effect of wall fissuring made for the combination action shall be counted in when earthquake happening if the combination action of the upper wall and the bressummer is considered.

7.2.6 The design value of the seismic shear strength of different types of masonry damaged along the stepped cross section shall be determined according to the following formula:

$$f_{vE} = \xi_N f_v \tag{7.2.6}$$

Where f_{vE}——**Design value of the seismic shear strength of masonry damaged along the stepped cross section;**

f_v——**Design value of the shear strength of masonry with the non-seismic design;**

ξ_N——**Normal stress influence coefficient of the seismic shear strength of masonry, which shall be selected according to Table 7.2.6.**

Table 7.2.6 Normal Stress Influence Coefficient of Masonry Strength

Masonry type	σ_0/f_v							
	0.0	1.0	3.0	5.0	7.0	10.0	12.0	≥16.0
Common brick and perforated brick	0.80	0.99	1.25	1.47	1.65	1.90	2.05	—
Small block	—	1.23	1.69	2.15	2.57	3.02	3.32	3.92

Note: σ_0 is the mean compression stress of masonry section corresponding to the representative value of the gravity load.

7.2.7 The section seismic shear bearing capacity of wall with common brick and perforated brick shall be checked and calculated according to the following requirements:

1 In general situations, it shall be checked and calculated according to the following formula:

$$V \leq f_{vE} A / \gamma_{RE} \tag{7.2.7-1}$$

Where V——Design value of shear force of the wall;

f_{vE}——Design value of the seismic shear strength damaged along the stepped cross section of brick masonry;

A——Wall cross-sectional area, the perforated brick shall take the gross sectional area;

γ_{RE}——Seismic adjustment coefficient of bearing capacity, it shall be adopted for the bearing wall according to those specified in Table 5.4.2 of this code; it shall be adopted by 0.75 for the self-bearing wall.

2 The wall adopted with horizontal reinforcements shall be checked and calculated according to the following formula:

$$V \leq \frac{1}{\gamma_{RE}}(f_{vE}A + \xi_s f_{yh} A_{sh}) \tag{7.2.7-2}$$

Where f_{yh}——Design value of the horizontal reinforcement tensile strength;

A_{sh}——Total horizontal reinforcement area of vertical sections in storey walls, its reinforcement ratio shall not be less than 0.07% and not be greater than 0.17%;

ξ_s——Reinforcement participation service factor may be selected according to those specified in Table 7.2.7.

Table 7.2.7 Reinforcement Participation Service Factor

Height-width ratio of wall	0.4	0.6	0.8	1.0	1.2
ξ_s	0.10	0.12	0.14	0.15	0.12

3 When the check according to Formula 7.2.7-1 and 7.2.7-2 cannot satisfy the requirements, the raising effect on shear bearing capacity, made by constructional columns (which the cross section is not less than 240mm×240mm (240mm×190mm if the wall thickness is 190mm), and the spacing is not larger than 4m) uniformly arranged in the middle of the wall segment, may be counted in, and the shear bearing capacity may be checked according the following simplified method:

$$V \leq \frac{1}{\gamma_{RE}}[\eta_c f_{vE}(A - A_c) + \xi_c f_t A_c + 0.08 f_{yc} A_{sc} + \xi_s f_{yh} A_{sh}] \tag{7.2.7-3}$$

Where A_c——Total cross-sectional area of the middle constructional column (for the transverse wall and the inner longitudinal wall, A_c will be taken as 0.15A if A_c>0.15A; for the outer longitudinal wall, it will be taken as 0.25A if A_c>0.25A);

f_t——Tensile strength design value of the concrete axle center in the central constructional column;

A_{sc}——Total cross-sectional area of the longitudinal steel reinforcement of the central constructional column (take 1.4% if the reinforcement ratio is not less than 0.6% and greater than 1.4%);

f_{yh}, f_{yc}——Respectively the design value of the tensile strength of wall horizontal reinforcement and constructional column reinforcement;

ξ_c——Participation service factor of the central constructional column; take 0.5 if one column is arranged in the center, and take 0.4 if more than one columns are arranged;

η_c——Constrained correction coefficient of the wall; take 1.0 in general situations, take 1.1 if the constructional column spacing is not greater than 3.0m;

A_{sh}——Total horizontal reinforcement area of the vertical section of the storey wall, take 0.0 if there is without horizontal reinforcement.

7.2.8 The cross-sectional seismic shear bearing capacity of the small block wall shall be checked according to the following formula:

$$V \leq \frac{1}{\gamma_{RE}}[f_{vE}A + (0.3f_t A_c + 0.05 f_y A_s)\xi_c] \tag{7.2.8}$$

Where f_t——Tensile strength design value of the concrete axle center of core column;

A_c——Total cross-sectional area of the core column;

A_s——Total cross-sectional area of the steel reinforcements for core column;

f_y——Tensile strength design value of the steel reinforcement for core column;

ξ_c——Participation service factor of the core column, which may be selected according to Table 7.2.8.

Note: When setting the core column and constructional column simultaneously, the cross-section of the constructional column may be regarded as the cross-section of the core column; the steel reinforcement of the constructional column may be regarded as the steel reinforcement of the core column.

Table 7.2.8 Participation Service Factor of Core Column

Pore-filling ratio ρ	ρ<0.15	0.15≤ρ<0.25	0.25≤ρ<0.5	ρ≥0.5
ξ_c	0.0	1.0	1.10	1.15

Note: The pore-filling ratio refers to the ratio of the number of the core column (including the number of the constructional column and the infill pore) and the total number of the pore.

7.2.9 For the masonry wall with common bricks or small blocks built-in frames in masonry buildings with R.C. frames on ground floors and seismic walls, if it meets the structure requirements of Article 7.5.4 and Article 7.5.5 of this code, its seismic check shall meet the following requirements:

1 For the axial force and shear force of the frame column on the ground floor, the additional axial force and shear force caused by the brick wall or small block wall shall be counted in; the value may be determined according to the following formulae:

$$N_f = V_w H_f / l \tag{7.2.9-1}$$

$$V_f = V_w \tag{7.2.9-2}$$

Where V_w——Design value of shear force of wall bearing, take the relatively big value of the wall in both sides of the column;

N_f——Additional axle pressure design value of the frame column;

V_f——Additional shear force design value of the frame column;

H_f, l——Storey height and span of the frame respectively.

2 For the frame column with common brick walls or small block walls built-in frames as well as the both ends frame column, the seismic shear bearing capacity shall be checked and calculated according to the following formula:

$$V \leq \frac{1}{\gamma_{REc}} \sum (M_{yc}^u + M_{yc}^l) / H_0 + \frac{1}{\gamma_{REw}} \sum f_{vE} A_{w0} \tag{7.2.9-3}$$

Where V——Design value of shear force on the frame column with common brick walls or small block walls built-in frames as well as the both ends frame column;

A_{w0}——Calculated area of the horizontal section of the brick wall or small block wall, take 1.25 times actual section if there is no opening; take the net area of the section if there is opening, but the section area of the wall column with width less than 1/4 of the opening height shall not be counted in;

$M_{yc}^{u}+M_{yc}^{l}$——Respectively the design value for the bend bearing capacity of normal section at the top and bottom ends of the frame column on the ground floor, it may be calculated by taking the equality sign according to related formulae of the non-seismic design specified in the current national standard "Code for Design of Concrete Structures" GB 50010;

H_0——Calculated height of frame column on the ground floor, take 2/3 of the column net height if there are masonry walls at both sides, or take column net height in rest conditions;

γ_{REc}——Seismic adjustment coefficient of bearing capacity of the frame column on the ground floor, it may adopt 0.8;

γ_{REw}——Seismic adjustment coefficient of bearing capacity of built-in common brick walls or small block walls, it may adopt 0.9.

7.3 Details of Seismic Design of Multi-storey Brick Buildings

7.3.1 For various multi-storey brick masonry buildings, the cast-in-situ reinforced concrete constructional columns (hereinafter referred to as "constructional column") shall be arranged according to the following requirements:

1 The arranged position of the constructional column generally shall meet the requirements specified in Table 7.3.1.

2 Multi-storey buildings with gallery corridor type and single-side corridor type. The constructional column shall be arranged according to those specified in Table 7.3.1 and according to the storey number increased by one of buildings. And the longitudinal wall at both sides of the single-side corridor all shall be treated as the external wall.

3 Buildings with fewer transverse walls, the constructional column shall be arranged according to those specified in Table 7.3.1 according to the storey number increased by one of buildings. If buildings with fewer transverse walls are gallery corridor type or single-side corridor type, the constructional column shall be arranged according to the requirements of Item 2 of this article; but if the constructional column does not exceed four storeys for Intensity 6, three storeys for Intensity 7 and two storeys for Intensity 8, it shall be treated as the storey number increased by two.

4 Buildings with very a few transverse walls at each storey, the constructional columns shall be arranged accoding to the storey number increased by two.

5 As for the masonry buildings adopted with autoclaved lime-sand bricks and autoclaved fly ash bricks, if the shear strength of masonry only reaches 70% of that of the masonry with

common clay bricks, the constructional columns shall be arranged in accordance with the requirements of Item 1~4 of this article based the storey number increased by one; but it shall be treated according to the storey number increased by two if the building does not exceed four storeys for Intensity 6, three storeys for Intensity 7 or two storeys for Intensity 8.

Table 7.3.1 Arrangement Requirements of Constructional Columns of Multi-Storey Brick Masonry Buildings

Storey number of building				Arrangement position	
Intensity 6	Intensity 7	Intensity 8	Intensity 9		
4 and 5	3 and 4	2 and 3		Four corners at the staircase and elevator room. the corresponding wall at up and down ends at the inclined ladder of the staircase; four corners of external walls and corresponding corners; joints of transverse walls and outer longitudinal walls at split-storey positions; joints of internal and external walls in big rooms; both sides of bigger openings	Every 12m or joints of unit transverse walls and outer longitudinal walls; joints of inner transverse walls and outer longitudinal walls at the corresponding opposite side of staircases
6	5	4	2		Joints of transverse walls(axes) and external walls in separate rooms; joints of gable walls and inner longitudinal walls
7	≥6	≥5	≥3		Joints of internal walls (axes) and external walls; partial smaller piers of internal walls; joints of inner longitudinal walls and transverse walls (axes)

Note: Bigger opening in inner wall refers to the one no less than 2.1m; if the external walls is set with constructional columns at its joint with the inner wall, the opening in it may be widened properly, but the wall close to opening shall be strengthened.

7.3.2 The constructional columns of multi-storey brick masonry buildings shall meet the following constructional requirements:

1 The minimum section of the constructional columns may be 180mm×240mm (180mm×190mm if the wall thickness is 190mm); the longitudinal steel reinforcement should adopt 4ø12; the stirrup spacing should not be greater than 250mm and the stirrups shall be properly densified at the upper and lower ends of the constructional column; as for the buildings exceeding six storeys for Intensity 6 and 7, exceeding five storeys for Intensity 8 and buildings for Intensity 9, the longitudinal still reinforcements of constructional column should adopt 4ø14; the stirrup spacing shall not be greater than 200mm; the section and reinforcement of the constructional columns at four corners of the building shall be properly increased.

2 Joints of constructional columns and walls shall be built into the horse tooth trough; 2ø6 horizontal reinforcement and tie meshes composed by spot welding in ø4 distributed short reinforcement plane or ø4 mesh reinforcement steel fabric composed by spot welding shall be arranged every 500mm along the wall height; the length of each side stretched into the wall should not be less than 1m. The above-mentioned tie reinforcement meshes shall be arranged along the horizontal full length of the wall for 1/3 of storeys on the ground floor for Intensity 6 and 7, 1/2 of storeys on the ground floor for Intensity 8 and whole storeys for Intensity 9.

3 Joints of constructional columns and ring-beams, longitudinal reinforcements of constructional columns shall be drilled through from the inside of longitudinal reinforcements of ring-beams to guarantee longitudinal reinforcements of constructional columns through up and down.

4 The foundation of the constructional column may not be set alone but it shall stretch 500mm into the outdoor subsurface or it shall be connected with the foundation ring-beam whose buried depth is less than 500mm.

5 When heights and storey numbers of buildings are close to the limit given in Table 7.1.2 of this code, constructional column spacing in transverse walls shall still meet the following requirements:

1) The constructional column spacing inside the transverse wall should not be greater than twice the storey height; the constructional column spacing of 1/3 storeys from the under part shall be reduced properly;

2) Other strengthening measures shall be arranged if the bay of the outer longitudinal wall is greater than 3.9m. The constructional column spacing of the inner longitudinal wall should not be greater than 4.2m.

7.3.3 The arrangement of the cast-in-situ reinforced concrete ring-beam of multi-storey brick masonry buildings shall meet the following requirements:

1 As for fabricated reinforced concrete floors and roofs or the wooden roofs, the ring-beams shall be arranged according to those specified in Table 7.3.3; as for those adopting longitudinal wall as the bearing wall, the spacing of ring-beams on the seismic transverse wall shall be properly thickened properly than the requirements of Table 7.3.3.

2 Additional ring-beams shall be allowed to not arrange for cast-in-situ or assembled monolithic reinforced concrete buildings and buildings whose roofs and walls have reliable connection; but reinforcements shall be strengthened where the floor-slab along the perimeter zone of the seismic wall and they shall be reliably connected with corresponding constructional column steel reinforcements.

Table 7.3.3 Cast-in-situ Reinforced Concrete Ring-beam Arrangement Requirements of Multi-storey Brick Masonry Buildings

Wall Type	Intensity		
	6 and 7	8	9
External wall and inner longitudinal wall	Roof and floor of each storey	Roof and floor of each storey	Roof and floor of each storey
Inner transverse wall	As above; the roof spacing shall not be greater than 4.5m; floor spacing shall not be greater than 7.2m; corresponding position of constructional column	As above; all the transverse wall at each storey and the spacing shall not be greater than 4.5m; corresponding position of constructional column	As above; all the transverse wall at each storey

7.3.4 The cast-in-situ concrete ring-beam structure of multi-storey brick masonry buildings shall meet the following requirements:

1 Ring-beams shall be closed; they shall be lapped up and down if there are openings. Ring-beams should be set at the same elevation of the precast slab or abutted against the slab bottom;

2 If there is no transverse wall in the ring-beam spacing required in Article 7.3.3 of this code, the ring-beam shall be replaced by using the beam or reinforcement in slab joints;

3 The cross-section height of the ring-beam shall not be less than 120mm, the reinforcement shall be in accordance with those specified in Table 7.3.4; for the foundation ring-beam added according to the requirements of Item 3 in Article 3.3.4 of this code, the cross-section height shall not be less than 180mm, the reinforcement shall not be less than 4ø12.

Table 7.3.4 Ring-beam Reinforcement Requirements of Multi-storey Brick Masonry Buildings

Reinforcement	Intensity		
	6 and 7	8	9
Minimum longitudinal steel reinforcement	4ø10	4ø12	4ø14
Maximum spacing of stirrups (mm)	250	200	150

7.3.5 The floors and roofs of multi-storey brick masonry buildings shall meet the following construction requirements:

1 The length that the cast-in-situ reinforced concrete floor-slab or roof slab stretched into the longitudinal and transverse wall all shall not be less than 120mm.

2 For fabricated reinforced concrete floor slab or roof slab, if the ring-beams are not arranged at the same elevation of the slab, the length that the slab end stretched into the external wall shall not be less than 120mm; the length that the slab end stretched into the internal wall shall not be less than 100mm, or the slab wall shall be connected by adopting the hard-strut framework; it shall not be less than 80mm on the beam or shall be connected by adopting the hard-strut framework.

3 The side edge of the precast slab close to the external wall shall be tied with the wall or ring-beam if the slab span is greater than 4.8m and parallels to the external wall.

4 The floor of big room at the end of the building. For the roof of buildings for Intensity 6 and the floor and roof for Intensity 7~9, if the ring-beam is arranged at the slab bottom, the reinforced concrete precast slab shall be tied with each other and shall be tied with the beam and wall or ring-beam.

7.3.6 Reinforced concrete beams or roof trusses of the floor and roof shall be reliably connected with walls and columns (including constructional columns) or ring-beams; independent brick columns shall not be adopted. Strengthening measures such as combination masonry shall be adopted for supporting components whose spans are not be less than 6m girder and bearing capacity requirements shall be satisfied.

7.3.7 For big rooms with length greater than 7.2m for Intensity 6 and 7 and joints of outer wall corners and internal and external walls for Intensity 8 and 9, 2ø6 full length reinforcement and tie meshes composed by spot welding in ø4 distributed short reinforcement plane or ø4 mesh reinforcement steel fabric composed by spot welding shall be arranged every 500mm along the wall height.

7.3.8 The staircase still shall meet the following requirements:

1 2ø6 full length steel reinforcements and tie meshes composed by spot welding in ø4 distributed short reinforcement plane or ø4 mesh reinforcement steel fabric composed by spot welding shall be arranged every 500mm along the wall height for the staircase wall of the top storey; the reinforced concrete with the thickness of 60mm and longitudinal steel reinforcement that shall not be less than 2ø10 or reinforced brick strips no less than 3 strips shall be arranged on the stair landing or half height of the storey for the staircase wall of other storeys for Intensity 7~9. Each strip of reinforcement shall not be less than 2ø6, the mortar strength grade shall neither be less than M7.5 nor be less than the mortar strength grade of the wall on the same storey.

2 The girder supporting length in the staircase and the exposed corner at the hallway internal wall shall not be less than 500mm and the girder shall be connected with the ring-beam.

3 The fabricated staircase shall be reliably connected with the beam of the landing slab, the fabricated staircase shall not be adopted for Intensity 8 and 9; the staircase with overhang-in-wall stair-tread or the vertical fin of the stair-tread insert into the wall shall not be adopted; the brick setting breast board without reinforcement shall not be adopted.

4 For the building and elevator room with the outthrust roof, the constructional column shall be stretched to the top and be connected with the top ring-beam. 2ø6 full length reinforceement and tie meshes composed by spot welding in ø4 distributed short reinforcement plane or ø4 mesh reinforcement steel fabric composed by spot welding shall be arranged every 500mm along the wall height for the entire wall.

7.3.9 Roof trusses of buildings with gable roofs shall be reliably connected with ring-beams of top storeys, purlins or roof slabs shall be reliably connected with walls and roof trusses; eaves tiles at the exit and entrance of buildings shall be anchored with roof components. When adopting hard purlin roofs, stepped piers bearing gable walls should be built on top of inner longitudinal walls in the top storey in addition, and constructional columns should be set.

7.3.10 Brick lintels shall not be adopted at door and window openings; bearing length of lintel shall not be less than 240mm for Intensity 6~8 and shall not be less than 360mm for Intensity 9.

7.3.11 The precast balcony shall be reliably connected with the ring-beam and the cast-in-situ sheet strip of the floor-slab for Intensity 6 and 7; the precast balcony shall not be adopted for Intensity 8 and 9.

7.3.12 The post-built non-bearing masonry partition wall, flue, air duct and refuse channel shall comply with the relevant requirements of Section 13.3 of this code.

7.3.13 The same type of foundations (or pile caps) should be adopted for fat the same construction unit; their bottom should be burried at the same elevation, or else the foundation ring-beam shall be added and shall be gradually sloped according to the step ratio of 1:2.

7.3.14 Category C multi-storey brick masonry buildings shall be adopted with the following strengthhening measures if there are fewer transverse walls and the total height and storey number are close to or reach the limits specified in Table 7.1.2 of this code:

1 The maximum bay dimension of buildings should not be greater than 6.6m.

2 The number of the transverse wall malposition in the same construction unit should not exceed 1/3 of the total number of the transverse wall and the continuous malposition should not greater than two walls; constructional columns shall be added for all the joints of malposition walls and cast-in-situ reinforced concrete slabs shall be adopted for buildings and roof slabs.

3 The width of openings in transverse walls and inner longitudinal walls should not be greater than 1.5m; the width of openings in outer longitudinal walls should not be greater than 2.1m or half of the bay dimension; and opening positions in internal and external walls shall not influence the whole connections of inner and outer longitudinal walls and transverse walls.

4 The strengthened cast-in-situ reinforced concrete ring-beams shall be arranged at the floor and roof elevation for all the longitudinal and transverse walls; the section height of ring-beams should not be less than 150mm, the up and down longitudinal steel reinforcements respectively shall not be less than 3ø10, the stirrup shall not be less than ø6, the spacing shall not be greater than 300mm.

5 Constructional columns that meet the following requirements shall all be added in the central section of all the joints of longitudinal and transverse walls and transverse walls: spacing of columns inside longitudinal and transverse walls should not be greater than 3.0m; the minimum section dimension should not be less than 240mm×240mm (or 240mm×190mm if the wall thickness is 190mm); the reinforcement should in accordance with those specified in Table 7.3.14.

Table 7.3.14 Longitudinal Steel Reinforcement and Stirrup Arrangement Requirements of Added Constructional Columns

Position	Longitudinal steel reinforcement			Stirrup		
	Maximum reinforcement ratio (%)	Minimum reinforcement ratio (%)	Minimum diameter (mm)	Range of densified area (mm)	Spacing of densified areas (mm)	Minimum diameter (mm)
Corner column	1.8	0.8	14	Total height	100	6
Side column			14	Upper end 700 and lower end 500		
Central column	1.4	0.6	12			

6 The floor slab and roof slab of the same construction unit shall be arranged at the same elevation.

7 The windowsill elevation at the ground and top storeys of buildings shall be arranged with horizontal cast-in-situ reinforced concrete strips along the full length of longitudinal and transverse walls; the height of section hereof shall not be less than 60mm, and the width shall not be less than the wall thickness; the longitudinal steel reinforcement is not less than 2ø10; the diameter of the transverse distribution reinforcement is not less than ø6 and the spacing is not greater than 200mm.

7.4 Details of Seismic Design of Multi-storey Concrete Block Buildings

7.4.1 For multi-storey small block buildings, the reinforced concrete core columns shall be arranged according to those specified in Table 7.4.1. For multi-storey buildings with gallery corridor and single-side corridor, buildings with fewer transverse walls and buildings with very a few transverse walls on each storey, the core columns shall be arranged in accordance with those specified in Table 7.4.1 respectively according to the corresponding requirements on adding the storey number specified in Item 2, 3 and 4 in Article 7.3.1 of this code.

7.4.2 The core columns of multi-storey small block buildings shall meet the following structure requirements:

1 The core column section of small block buildings should not be less than 120mm×120mm.

2 The concrete strength grade of the core column shall not be less than Cb20.

3 The vertical steel dowel of the core column shall penetrate the wall and be connected with the ring-beam; the steel dowel shall not be less than 1ø12; the steel dowel shall not be less than 1ø14 if it exceeds five storeys for Intensity 6 and 7, four storeys for Intensity 8 and it is Intensity 9.

Table 7.4.1 Core Column Arrangement Requirements of Multi-storey Small Block Buildings

Storey number of buildings				Arrangement position	Arrangement quantity
Intensity 6	Intensity 7	Intensity 8	Intensity 9		
4 and 5	3 and 4	2 and 3		Outer corners of external walls, four corners at buildings and elevator rooms, corresponding walls at up and down ends at inclined ladders of staircases; Joints of internal and external walls in big rooms; Joints of transverse walls and outer longitudinal walls at split-storey positions; Joints of every 12m or unit transverse walls and outer longitudinal walls	Full-grouting 3 holes at corners of external walls; Full-grouting 4 holes at joints of internal and external walls; Full-grouting 2 holes at corresponding walls at up and down ends at inclined ladders of staircases
6	5	4		As Above; Joints of transverse walls (axes) and outer longitudinal walls in separate rooms	
7	6	5	2	As above; Joints of each internal wall (axes) and outer longitudinal walls; Joints of inner longitudinal walls and transverse walls (axes) as well as both sides of openings	Full-grouting 5 holes at corners of external walls; Full-grouting 4 holes at joints of internal and external walls; Full-grouting 4~5 holes at joints of internal walls; Respectively full-grouting 1 hole at both sides of openings
	7	≥6	≥3	As Above; The inner core column spacing of the transverse wall is no greater than 2m	Full-grouting 7 holes at corners of external walls; Full-grouting 5 holes at joints of internal and external walls; Full-grouting 4~5 holes at joints of internal walls; Respectively full-grouting 1 hole at both sides of openings

Note: At positions such as corners of external walls, joints of internal and externals and four corners of building elevator rooms, reinforced concrete construction columns shall be allowed to replace a part of core columns..

4 The core column shall stretch into 500mm of the outdoor subsurface or be connected with the foundation ring-beam whose buried depth is less than 500mm.

5 The core column arranged for improving the wall seismic shear bearing capacity should be evenly arranged in the wall, the maximum clear distance should not be greater than 2.0m.

6 Steel tie bar meshes shall be arranged at wall joints of multi-storey small block buildings or connections of core columns and walls; meshes may be made by spot welding with 4mm-diameter steel bars; spacing along the wall height is not greater than 600mm; and meshes shall be arranged along the horizontal full length of walls. For 1/3 storeys on bottom for Intensity 6 and 7, 1/2 storeys on bottom for Intensity 8, and all storeys for Intensity 9, spacing along wall heights of above-mentioned steel tie bar meshes are not greater than 400mm.

7.4.3 The reinforced concrete constructional columns used for replacing core columns in small block buildings shall meet the following structure requirements:

1 The constructional column section should not be less than 190mm×190mm, the longitudinal steel reinforcement should adopt 4ø12; the stirrup spacing should not be greater than 250mm and the stirrups shall be properly densified at the upper and lower ends of the constructional column; as for the buildings exceeding five storeys for Intensity 6 and 7, exceeding four storeys for Intensity 8 and buildings for Intensity 9, the longitudinal steel reinforcements of the constructional column should adopt 4ø14; the stirrup spacing shall not be greater than 200mm; the section and reinforcement of the constructional column at corners of external walls shall be properly increased.

2 Joints of constructional columns and block walls shall be built into the horse tooth trough; for masonry block holes adjacent to constructional columns, should be in filled for Intensity 6, shall be in filled for Intensity 7, shall be in filled and inserted bars for Intensity 8 and 9. ø4 tie mesh reinforce-ement steel fabric composed by spot welding shall be arranged every 600mm along the wall height among constructional columns and block walls and shall be arranged along the horizontal full length of walls. For 1/3 storeys on bottom for Intensity 6 and 7, 1/2 storeys on bottom for Intensity 8, and all storeys for Intensity 9, spacing along wall heights of above-mentioned tie mesh reinforcement steel fabric are not greater than 400mm.

3 Joints of constructional columns and ring-beams, longitudinal steel reinforcements of constructional columns shall be drilled through from the inside longitudinal steel reinforcements of ring-beams to guarantee longitudinal steel reinforcements of constructional columns through up and down.

4 The foundation of the constructional column may not be set alone but it shall stretch 500mm into the outdoor subsurface or it shall be connected with the foundation ring-beam whose buried depth is less than 500mm.

7.4.4 The arrangement position of cast-in-situ reinforced concrete ring-beams of multi-storey small block buildings shall be implemented according to the ring-beam requirements of multi-storey brick masonry buildings specified in Article 7.3.3 of this code, the ring-beam width shall not be less than 190mm, the reinforcement shall not be less than 4ø12, and the stirrup spacing shall not be greater than 200mm.

7.4.5 For the storey number of multi-storey small block buildings, if it does exceed five **storeys** for Intensity 6, four storeys for Intensity 7, three storeys for Intensity 8 and for Intensity 9, the full length horizontal cast-in-situ reinforced concrete strip shall be arranged along longitudinal and transverse walls at the windowsill elevation of the ground floor and top storey; thereof the section height is not less than 60mm, the longitudinal steel reinforcement is not less than 2ø10 and the distributed tie steel bar shall be existed; the concrete strength grade shall not be less than C20.

For horizontal cast-in-situ concrete strip, the groove shape block may be used to replace the formwork without changing the longitudinal steel reinforcement and tie steel bar.

7.4.6 Category C multi-storey small block buildings, when the transverse wall is less and the total height and the storey number is close to or reaches the limit specified in Table 7.1.2 of this code, shall meet the relevant requirements in Article 7.3.14 of this code; hereinto, the constructional column in the central wall may be replaced by the core column; and the quantity of pouring hole of core column shall not be less than 2, the diameter of the steel dowel into each hole shall not be less than 18mm.

7.4.7 Other details of seismic design on small block buildings still shall meet the relevant requirements specified from Article 7.3.5 to Article 7.3.13 of this code. Hereinto, tie mesh reinforcement steel fabric spacing of walls shall meet the corresponding requirements of this section, respectively take 600mm and 400mm.

7.5 Details of Seismic Design of Multi-storey Masonry Buildings with R.C. Frames on Ground Floors

7.5.1 The reinforced concrete constructional columns or core columns shall be arranged for the upper part wall of masonry buildings with R.C. frames on ground floors and seismic walls, and the buildings shall meet the following requirements:

1 The arrangement positions of reinforced concrete construction columns and core columns shall be respectively arranged according to the requirements of Article 7.3.1 and Article 7.4.1 of this code based on the total storey number of the building.

2 In addition to the following requirements, the structure of constructional columns and core columns also shall meet the requirements of Article 7.3.2, 7.4.2 and 7.4.3 of this code:

1) The constructional column section in brick masonry walls should not be less than 240mm×240mm (or 240mm×190mm if the wall thickness is 190mm);

2) The longitudinal steel reinforcement of constructional columns should not be less than 4ø14, the stirrup spacing should not be greater than 200mm; the steel dowel in each hole of core columns shall not be less than 1ø14, ø4 welded steel fabric meshes shall be arranged every 400mm along wall heights among core columns.

3 Constructional columns and core columns shall be connected with ring-beams at each storey, or reliably pulled and tied with cast-in-situ floor-slabs.

7.5.2 The construction of the transition layer wall shall be in accordance with the following requirements:

1 The center line of the upper masonry wall should coincide with that of the frame beam on the ground and the seismic wall; the constructional column or core column should penetrate frame column up and down.

2 For transition layer, constructional columns or core columns shall be arranged on positions corresponding to the frame columns on the ground floor and constructional columns of concrete walls or constrained masonry walls; spacing of the constructional columns in walls should not be greater than the storey height; core columns shall be arranged according to the requirements of Table 7.4.1 of this code and the maximum spacing should not be greater than 1m.

3 Longitudinal steel reinforcements of constructional columns in transition layers should not be less than 4ø16 for Intensity 6 and 7; it should not be less than 4ø18 for Intensity 8. Longitudinal steel reinforcements of core columns in transition layers should not be less than 1ø16 for each hole for Intensity 6 and 7; it should not be less than 1ø18 for each hole for Intensity 8. Generally, longitudinal steel reinforcements shall be anchored into the lower frame columns or concrete walls; when longitudinal steel reinforcements are anchored into bressummers, the relevant positions of bressummers shall be strengthened.

4 The masonry wall in transition layers at windowsill elevation shall be arranged horizontal cast-in-situ reinforced concrete strips along the full length of longitudinal and transverse walls; the section height hereof is not less than 60mm, and the width is not less than the wall thickness. The longitudinal steel reinforcement is not less than 2ø10, and the diameter of transverse distribution reinforcement is not less than 6mm and the spacing is not greater than 200mm. Furthermore, for the brick masonry wall among adjacent constructional columns, 2ø6 horizontal reinforcement along full length and tie meshes composed by spot welding in ø4 distributed short reinforcement plane or ø4 mesh reinforcement steel fabric composed by spot welding shall be arranged every 360mm along the wall height; and they shall be anchored into constructional columns; ø4 horizontal mesh reinforcement steel fabric composed by spot welding along full length shall be arranged every 400mm along the wall height among core columns of small block masonry walls.

5 Constructional columns or simple-pore core columns with the section area no less than 120mm×240mm (or 120mm×190mm if the wall thickness is 190mm) should be arranged at both sides of openings (door openings with width no less than 1.2m and window openings with width no less than 2.1m) of masonry walls in transition layers.

6 If masonry seismic walls in transition layers are not aligned with frame beams or walls on the ground floor, wall-bearing transfer beams shall be arranged in frames on the ground floor, and higher strengthening measures than that of Item 4 of this article shall be taken for brick walls or block walls in transition layers.

7.5.3 When reinforced concrete walls are adopted at the bottom of masonry buildings with R.C. frames on ground floors and seismic walls, the section and construction shall meet the following requirements:

1 The wall periphery shall be arranged with frames composed of beams (or concealed beams) and frame columns (or frame columns); the section width of frame beams should not be less than 1.5 times wallboard thickness; the section height should not be less than 2.5 times wallboard thickness; the section height of frame columns should not be less than 2 times wallboard thickness.

2 The wallboard thickness should not be less than 160mm and shall not be less than 1/20 of wallboard clear height; the wall should be set openings to form several wall sections, the height-width ratio of each wall section should not be less than 2.

3 The reinforcement ratio of the vertical and transverse distribution steel reinforcement for walls all shall not be less than 0.30% and shall adopt double row arrangement; the spacing of tie bar among double-row distributed steel reinforcements shall not be greater than 600mm, the diameter shall not be less than 6mm.

4 The boundary component of walls may be arranged according to the requirements about general positions specified in Section 6.4 of this code.

7.5.4 When constrained brick masonry walls are adopted at the ground floor of Intensity 6 brick buildings with frames on the ground floor and seismic walls, the structure hereof shall be in accordance with the following requirements:

1 The brick wall thickness shall not be less than 240mm, the masonry mortar strength grade shall not be less than M10 and the frames shall be casted after building the wall.

2 2ø8 horizontal reinforcement and tie meshes composed by spot welding in ø4 distributed short reinforcement plane shall be arranged every 300mm along frame columns and horizontal full length of brick walls; the reinforced concrete horizontal tie beams connected with frame columns still shall be arranged on the half height of the wall.

3 The reinforced concrete construction columns shall be added inside walls if the wall length is greater than 4m and at both sides of openings.

7.5.5 When constrained small block walls are adopted for the bottom storey of concrete block buildings with frames on the ground floor and seismic walls for Intensity 6, the structure hereof shall be in accordance with the following requirements:

1 The wall thickness shall not be less than 190mm, the masonry mortar strength grade shall not be less than Mb10 and the frames shall be casted after wall building.

2 2ø8 horizontal reinforcement and tie meshes composed by spot welding in ø4 distributed short reinforcement plane shall be arranged every 400mm along frame columns and horizontal full length of block walls; the reinforced concrete horizontal tie beams connected with frame columns still shall be arranged on the half height of the wall; the tie beam section shall not be less than 190mm× 190mm, the longitudinal steel reinforcement shall not be less than 4ø12; the stirrup diameter shall not be less than ø6 and the spacing shall not be greater than 200mm.

3 Core columns shall be arranged on both sides of door and window openings on walls; core columns shall be added in walls when the wall length is larger than 4m, and the core column shall meet the relevant requirements of Article 7.4.2 of this code; reinforced concrete constructional columns should be used to replace core columns for other positions; and the reinforced concrete constructional columns shall meet the relevant provisions of Article 7.4.3 of this code.

7.5.6 The frame columns in masonry buildings with R.C. frames on ground floors and seismic walls shall be in accordance with the following requirements:

1 The column section shall not be less than 400mm×400mm; the circular column diameter shall not be less than 450mm.

2 The axial compression ratio of the column should not be greater than 0.85 for Intensity 6, should not be greater than 0.75 for Intensity 7 and should not be greater than 0.65 for Intensity 8.

3 If the standard Intensity value of steel reinforcement is lower than 400MPa, the minimum total reinforcement ratio of the longitudinal steel reinforcements for central columns shall not be less than 0.9% for Intensity 6 and 7 and not be less than 1.1% for Intensity 8; and that for side columns, corner columns and concrete seismic wall end columns shall not be less than 1.0% for Intensity 6 and 7 and not be less than 1.2% for Intensity 8.

4 The stirrup diameter of columns shall not be less than 8mm for Intensity 6 and 7; it shall not be less than 10mm for Intensity 8, and the stirrup shall be densified at total height with the spacing no greater than 100mm.

5 The design value of the combined bending moment at the toppest and lowest ends on columns shall be multiplied by the enhancement coefficient, and the enhancement coefficient of Grade 1, 2 and 3 shall be adopted respectively by 1.5, 1.25 and 1.15.

7.5.7 The floor of masonry building with R.C. frames on ground floors and seismic walls shall be in accordance with the following requirements:

1 The cast-in-situ reinforced concrete slab shall be adopted for base slabs of transition

layers. The slab thickness shall not be less than 120mm; less or smaller openings shall be opened. When the dimension of opening is larger than 800mm, edge beams shall be arranged at the periphery of openings.

2 On other storeys, the cast-in-situ ring-beams shall be arranged when adopting fabricated reinforced concrete floor slabs; when cast-in-situ reinforced concrete floor slabs are adopted, additional ring-beam shall be allowed to not be arranged. But the reinforcement of the floor-slab along the seismic wall periphery shall be strengthened and connected stably with corresponding constructional columns.

7.5.8 For reinforced concrete bressummers of masonry building with R.C. frames on ground floors and seismic walls, the section and structure shall be in accordance with the following requirements:

1 The section width of beams shall not be less than 300mm. The section height shall not be less than 1/10 of the span.

2 The stirrup diameter shall not be less than 8mm and the spacing shall not be greater than 200mm; the stirrup spacing shall not be greater than 100mm in following ranges: within the range at 1.5 times beam depth at beam ends and less than 1/5 beam clear span; openings of upper walls; and respectively 500mm on both sides of the opening and no less than the beam depth.

3 The waist reinforcement shall be arranged along the beam depth, the quantity shall not be less than 2ø4, and the spacing shall not be greater than 200mm.

4 The longitudinal bearing steel bar and waist reinforcement of beam shall be anchored in columns in accordance with the requirements of tension reinforcement. The anchorage length in columns of the longitudinal steel reinforcement on upper part of support shall meet the relevant requirements of concrete frame-support beams.

7.5.9 The material strength grade of masonry buildings with R.C. frames on ground floors and seismic walls shall meet the following requirements:

1 The concrete strength grade of frame columns, concrete walls and bressummers shall not be less than C30.

2 The strength grade of masonry block materials in transition storeys shall not be less than MU10; the mortar strength grade of brick masonry shall not be less than M10; the masonry mortar strength grade of block masonry shall not be less than Mb10.

7.5.10 Other details of seismic design for masonry buildings with R.C. frames on ground floors and seismic walls shall comply with the relevant requirements of Section 7.3, Section 7.4 and Chapter 6 of this code.

8 Multi-Storey and Tall Steel Buildings

8.1 General Requirements

8.1.1 The structure type and maximum height of steel structure civil buildings to which this chapter is applicable shall be in accordance with those specified in Table 8.1.1. For steel structure that is irregular in horizontal and vertical directions, the applicable maximum height should be properly reduced.

Note: 1 The seismic design of steel support-concrete frame and steel frame-concrete tube structures shall meet the requirements of Appendix G of this code;

2 The seismic design of multi-storey steel factory buildings shall meet the requirements of Section H.2 in Appendix H of this code.

Table 8.1.1 Applicable Maximum Height of Steel Buildings (m)

Structure type	Intensity 6 and 7 (0.10g)	Intensity 7 (0.15g)	Intensity 8 (0.20g)	Intensity 8 (0.30g)	Intensity 9 (0.40g)
Frame	110	90	90	70	50
Frame-concentrically-supported	220	200	180	150	120
Frame-eccentrically-supported (ductility wallboard)	240	220	200	180	160
Tube (frame tube, tube in tube, truss tube and bundled tube) and giant frame	300	280	260	240	180

Note: 1 The height of buildings refers to the height from the outdoor ground to the top of main roof slab (excluding partial outthrust of roof);

2 The professional study and demonstration shall be carried out and effective strengthening measures shall be taken for buildings whose height exceeds that given in the table;

3 The tube in the table excludes the concrete tube.

8.1.2 The maximum height-width ratio of steel structure civil buildings applicable in this chapter should not exceed those specified in Table 8.1.2.

Table 8.1.2 Applicable Maximum Height-width Ratio of Steel Structure Civil Buildings

Intensity	6 and 7	8	9
Maximum height-width ratio	6.5	6.0	5.5

Note: The height-width ratio may be calculated based on the large base plate if it exists at the bottom of tower buildings.

8.1.3 Steel buildings shall be adopted with different seismic grades according to the precautionary category, Intensity and building height, and also shall meet the requirements of the corresponding calculation and details of design. The seismic grade of Category C buildings shall be determined according to those specified in Table 8.1.3.

Table 8.1.3 Seismic Grade of Steel Buildings

Building height	Intensity 6	Intensity 7	Intensity 8	Intensity 9
≤50m		4	3	2
>50m	4	3	2	1

Note: 1 If the height is close to or equivalent to the height boundary, the seismic grade shall be allowed to determine by considering the degree of irregularity, site and base condition of buildings;

2 Generally, the seismic grade of components shall be the same as the structure; when the bearing capacity of each component at some position all meet the internal force requirements of 2 times combined earthquake action, the seismic grade of components for Intensity 7~9 shall be allowed to determine by reducing one Intensity.

8.1.4 If the steel buildings need to set with seismic joints, the joint width shall not be less than 1.5 times of that for corresponding buildings with reinforced concrete structures.

8.1.5 Steel buildings of seismic grade 1 and 2 should be arranged with energy-dissipated supports or tubes, such as the eccentrically support, reinforced concrete seismic wallboard with vertical joints, reinforced concrete wallboard with built-in steel support, and buckling constrained support.

When adopting frame structures, single-span frames shall not adopted for Category A and B buildings and tall Category C buildings; single-span frames should not be adopted for multi-storey Category C buildings.

Note: "Grade 1, 2, 3 and 4" in this chapter is short for "seismic Grade 1, 2, 3 and 4".

8.1.6 Steel buildings adopting composite frame and support structures shall meet the following requirements:

1 The arrangement in two directions of supported frames should all be basically symmetric; the length-width ratio of floor among supported frames should not be greater than 3.

2 The concentrically-support should be adopted for steel structure for Grade 3 and 4 with height no greater than 50m; the energy-dissipated support such as the eccentrically support and buckling constrained support may also be adopted.

3 The cross support should be adopted for the concentrically-supported frame, the zigzag support or single diagonal support may also be adopted, but the K-shaped support should not be adopted; the support axes should meet at the intersection point of component axes of beam columns; the eccentricity that deviate the intersection point shall not exceed the width of the support strut component and the additional bending moment produced herefrom shall be counted in. Two groups of diagonal in different directions of dip shall be arranged simultaneously if the only-tensile single diagonal system is adopted for the central support, and the projected area difference in horizontal direction shall not be greater than 10% for the cross-section area of the single diagonal in different directions in each group.

4 At least one end of each support for the eccentrically supported frame shall be connected with the frame beam, the energy-dissipated beam section shall be formed among the intersection points of the support and beam and columns or between the intersection points of another support in the same span and the beam.

5 When adopting buckling constrained supports, zigzag supports and single diagonal supports of twined arrangement should be adopted and no K-shaped or X-shaped supports should be used. The angle between the support and the column should within 35°~55°. When the buckling constrained support is compressed, its design parameter and performance test as well as the calculation method of a kind of energy-dissipated component may be designed according to the relevant requirements.

8.1.7 Composite steel-frame-tube structures may be set with the strengthening layer composed of tube overhanging arms or overhanging arms and periphery trusses if necessary.

8.1.8 The floors of steel buildings shall meet the following requirements:

1 The cast-in-situ reinforced concrete composite floor slabs of profiled steel slabs or reinforced concrete floor slabs should be adopted and shall be reliably connected with the steel beams.

2 For steel structures no greater than 50m for Intensity 6 and 7, the reinforced concrete floor slab of fabricated integral type still may be adopted, the fabricated floor slab or other floor of light type may also be adopted; but the built-in fitting of floor-slabs and steel beams shall be welded, or other measures that can guarantee the floor integrity shall be taken.

3 A horizontal support may be set up if necessary for the floor of transfer storey or the floor-slab with a big opening.

8.1.9 The basement arrangement of steel buildings shall meet the following requirements:

1 When arranging the basement, the supports (seismic wallboards) arranged vertically and continuously in frame-supported (seismic wallboard) structures shall be extended to the foundation; the steel-frame columns shall be at least extended to first underground storey and the vertical load on the frame shall be directly transferred to the foundation.

2 Steel buildings greater than 50m shall be arranged basements. Its embedded depth of foundation should not be less than 1/15 of the total height of buildings when natural subsoil are adopted; the buried depth of platform on piles should not be less than 1/20 of the total height of buildings when pile foundations are adopted.

8.2 Essentials in Calculation

8.2.1 The earthquake action effect of steel structures shall be adjusted according to the requirements of this section; the storey deformation of steel structures shall meet the relevant requirements of Section 5.5 of this code. When seismic checking for component sections and connections, the non-seismic design value of bearing capacity shall divide by specified seismic adjustment coefficient of bearing capacity of this code; others that are not specified in this chapter shall meet the requirements of the current and relevant design codes and standards.

8.2.2 The damping ratio of the steel structure seismic calculation should meet the following requirements:

1 Calculation under frequent earthquakes, the damping ratio may take 0.04 if the height is not greater than 50m; may take 0.03 if the height is greater than 50m and less than 200m; should take 0.02 if the height is not less than 200m.

2 When the seismic overturning moment bearing by the eccentrically-supported frames is larger than 50% of the total seismic overturning moment of structures, its damping ratio may be correspondingly added 0.005 than that specified in Item 1 of this article.

3 Elasto-plastic analysis under rare earthquakes, the damping ratio may take 0.05.

8.2.3 The internal force and deformation analysis in earthquake action for steel structures shall meet the following requirements:

1 The gravity second-order effect on steel structures shall be counted in according to the requirements of Article 3.6.3 of this code. The hypothetical horizontal force shall be appended on the top of each storey according to the relevant requirements of the current national standard "Code for Design of Steel Structures" GB 50017 when carrying out the elasticity analysis of the second-order effect.

2 Frame beams may be designed according to the internal force on the section of beam ends. For I-shaped section columns, the influence of shear deformation for beam-to-column joint zone put on the lateral displacement of structures should be calculated; for box column frames, concentrically-supported frames and steel structures not exceed 50m, their storey drift calculation may be left out of the influence on the shear deformation for beam-to-column connection panel zone and be analyzed approximately according to the frame axes.

3 The diagonal of steel frame supported structures may be calculated according to the hinge bar at ends; the earthquake storey shear force obtained by the rigidity distribution calculation for the frame part shall multiply adjustment coefficient to reach the smaller one of the two values (one is the value that no less than 25% of the total seismic shear force on the structure bottom, the other is the value that 1.8 times shear force of the maximum storey in the frame part).

4 When the diagonal axes of concentrically-supported frames deviate the intersection points of beam and column axes, if the deviation doesn't exceed the width of support components, the frames still may be analyzed as the central support frame; but the additional bending moment produced herefrom shall be calculated.

5 In eccentrically-supported frames, the internal force design value of connected components with energy-dissipated beams shall be adjusted according to the following requirements:

1) The axial force design value of support diagonals shall take the product of the support diagonal axial force and enhancement coefficient when the energy-dissipated beams connected with the support diagonals reach the shear bearing capacity; the enhancement coefficient shall not be less than 1.4 for Grade 1, 1.3 for Grade 2 or 1.2 for Grade 3;

2) The internal force design value of frame beams in the same span on the energy-dissipated beams shall take the product of the internal force of the frame beams and the enhancement coefficient when the energy-dissipated beams reach the shear bearing capacity; the enhancement coefficient shall not be less than 1.3 for Grade 1, 1.2 for Grade 2 or 1.1 for Grade 3;

3) The internal force design value of frame columns shall take the product of the column internal force and the enhancement coefficient when the energy-dissipated beams reach the shear bearing capacity; the enhancement coefficient shall not be less than 1.3 for Grade 1, 1.2 for Grade 2 or 1.1 for Grade 3.

6 The reinforced concrete wallboards with steel supports inside and reinforced concrete wallboards with vertical joints shall be calculated according to the relevant requirements; reinforced concrete wallboards with vertical joints may only bear the shear force produced by the horizontal load but not the pressure produced by the vertical load.

7 The steel frame column in transfer components of steel structures, the earthquake internal force shall multiply the enhancement coefficient, the value may adopt 1.5.

8.2.4 If the top flange of steel frame beams is connected with the composite floor slabs by adopting with shear resistant connecting fittings, the monolithic stability under earthquake action may not be checked.

8.2.5 The seismic capacity check for joints of steel frames shall meet the following requirements:

1 In addition to one of the following conditions, the full plastic bearing capacity of the right and left beam ends and the upper and lower column ends of the joint shall meet the requirements of the following formulae:

1) The shear bearing capacity of the storey where the column is located is 25% greater that of the adjacent upper storey;

2) The column axial compression ratio is not greater than 0.4 or $N_2 \leq \varphi A_c f$ (N_2 is the design value of composite axial force in 2 times earthquake action);

3) Joints connected with support diagonals.

Beams with uniform cross section

$$\Sigma W_{\mathrm{pc}}(f_{\mathrm{yc}} - N/A_{\mathrm{c}}) \geq \eta \Sigma W_{\mathrm{pb}} f_{\mathrm{yb}} \tag{8.2.5-1}$$

Beams with variable cross section of end flange

$$\Sigma(W_{\mathrm{pc}}(f_{\mathrm{yc}} - N/A_{\mathrm{c}}) \geq \Sigma(\eta W_{\mathrm{pb1}} f_{\mathrm{yb}} + V_{\mathrm{pb}} s) \tag{8.2.5-2}$$

Where W_{pc}, W_{pb}——Respectively the plastic modulus of columns and beams meeting at joints;

W_{pb1}——Plastic section modulus of beams of sections where the plastic hinge of beams is located;

f_{yc}, f_{yb}——Respectively the steel yield strength of columns and beams;

N——Column axial force of seismic array;

A_{c}——Cross-sectional area of frame columns;

η——Strong column coefficient, it shall take 1.15 for Grade 1, 1.10 for Grade 2 and 1.05 for Grade 3;

V_{pb}——Shear force of beams with plastic hinge;

s——Distance from the plastic hinge to column surface, the plastic hinge may take the minimum part of flange on variable cross section at beam ends.

2 The yield bearing capacity of joints zone shall meet the following requirements:

$$\psi(M_{\mathrm{pb1}} + M_{\mathrm{pb2}})/V_{\mathrm{p}} \leq (4/3) f_{\mathrm{yv}} \tag{8.2.5-3}$$

I-shaped section column

$$V_{\mathrm{p}} = h_{\mathrm{b1}} h_{\mathrm{c1}} t_{\mathrm{w}} \tag{8.2.5-4}$$

Box section column

$$V_p = 1.8h_{b1}h_{c1}t_w \tag{8.2.5-5}$$

Section column of round tube

$$V_p = (\pi/2)h_{b1}h_{c1}t_w \tag{8.2.5-6}$$

3 The joint zone of I-shaped section column and box section column shall be checked and calculated according to the following formulae:

$$t_w \geq (h_b + h_c)/90 \tag{8.2.5-7}$$

$$(M_{b1} + M_{b2})/V_p \leq (4/3)f_v/\gamma_{RE} \tag{8.2.5-8}$$

Where M_{pb1}, M_{pb2}——Respectively full plastic bend bearing capacity of two side beams in joints zone;

V_p——Volume of joint zone;

f_v——Shear strength design value of steels;

f_{yv}——Yield shear strength of steels, take 0.58 times the yield strength of steels;

——Reduction coefficient; it shall take 0.6 for Grade 3 and 4 and 0.7 for Grade 1 and 2;

h_{b1}, h_{c1}——Respectively the distance among the thickness midpoints of beam flanges and the thickness midpoints of column flanges (or the tube wall on the diameter line of steel tube);

t_w——Web thickness of columns in joint zone;

M_{b1}, M_{b2}——Respectively the moment design value of two side beams in joint zone;

γ_{RE}——Seismic adjustment coefficient of bearing capacity in joint zone, it shall take 0.75.

8.2.6 The seismic capacity check for concentrically-supported frame components shall meet the following requirements:

1 The compression bearing capacity of support diagonals shall be checked according to the following formulae:

$$N(\varphi A_{br}) \leq f/\gamma_{RE} \tag{8.2.6-1}$$

$$=1/(1+0.35\lambda_n) \tag{8.2.6-2}$$

$$\lambda_n = (\lambda/\pi)\sqrt{f_{ay}/E} \tag{8.2.6-3}$$

Where N——Axial force design value of support diagonals;

A_{br}——Sectional area of support diagonals;

φ——Stability coefficient of axial compression components;

ψ——Strength reduction factor when cyclic loading;

λ, λ_n——Slenderness ratio of support diagonals and the regularization slenderness ratio;

E——Elasticity modulus of steels with support diagonals;

f, f_{ay}——Respectively the steel strength design value and yield strength;

γ_{RE}——Seismic adjustment coefficient of bearing capacity for support stability disruption.

2 The frame beams of inverted-V shaped and V-shaped supports shall keep continuous at the connections of supports; the gravity load and the bearing capacity under the action of unbalanced force during the support buckling shall be checked and calculated according to beams which the support point action is not counted in; the unbalanced force shall be calculated according to 0.3 times the minimum yield bearing capacity of tensioned supports and the maximum buckling bearing capacity of compressed supports. The inverted-V shaped and V-shaped support may be alternately arranged along the vertical direction or by adopting zipper columns.

Note: This item may not be implemented for beams of the top storey and rooms with exposed-roofs.

8.2.7 The seismic capacity check for eccentrically-supported frame components shall meet the following requirements:

1 The shear bearing capacity of energy-dissipated beams shall meet the following requirements:

When at $N \leq 0.15Af$

$$V \leq \varphi V_l / \gamma_{RE} \tag{8.2.7-1}$$

$$V_l = 0.58 A_w f_{av} \quad \text{or} \quad V_l = 2M_{lp} / a \quad \text{the smaller value shall be taken}$$

$$A_w = (h - 2t_f)t_w$$

$$M_{lp} = fW_p$$

When at $N > 0.15Af$

$$V \leq \varnothing V_{lc} / \gamma_{RE} \tag{8.2.7-2}$$

$$V_{lc} = 0.58 A_w f_{ay} \sqrt{1 - [N/(Af)]^2}$$

Or $$V_{lc} = 2.4 M_{lp} [1 - N/(Af)] / a \quad \text{the smaller value shall be taken}$$

Where N, V——Respectively the design values of axial force and shear force of energy-dissipated beams;

V_l, V_{lc}——Respectively the shear bearing capacity of energy-dissipated beams under the action of shear bearing capacity and with the axial force influence included;

M_{lp}——Full plastic bends bearing capacity of energy-dissipated beams;

A, A_w——Respectively the cross-sectional area of energy-dissipated beams and web plates;

W_p——Plastic section modulus of energy-dissipated beams;

a, h——Respectively the net length and section height of energy-dissipated beams;

t_w, f_f——Respectively the web thickness and flange thickness of energy-dissipated beams;

f, f_{ay}——Compression strength design value and yield strength of energy-dissipated beams of steels;

$\varnothing$——Coefficient, it may take 0.9;

γ_{RE}——Seismic adjustment coefficient of bearing capacity of energy-dissipated beams, it shall take 0.75.

2 The connected bearing capacity of support diagonals and energy-dissipated beams shall not be less than that of supports. The connection of supports and beams shall be designed according to the resist compression and bending connection if supports need to resist bending moments.

8.2.8 The connection calculation of lateral-force-resisting components of steel structures shall meet the following requirements:

1 The bearing capacity design value for the connection of lateral-force-resisting components of steel structures shall not be less than that of linked components; and the connected high-strength bolts shall not glide.

2 The ultimate bearing capacity for the connection of lateral-force-resisting components of steel structures shall be greater than the yield bearing capacity of the linked components.

3 The ultimate bearing capacity for the rigid connection of beams and columns shall be checked according to the following formulae:

$$M_u^j \geq \eta_j M_p \tag{8.2.8-1}$$

$$M_u^j \geq 1.2(2M_p / l_n) + V_{Gb} \tag{8.2.8-2}$$

4 The connection ultimate bearing capacity of supports and frames and the splicing ultimate bearing capacity of beams, columns and supports shall be checked and calculated according to the following formulae:

Connection and splicing of supports $$N_{ubr}^j \geq \eta_j A_{br} f_y \tag{8.2.8-3}$$

Splicing of beams $$M_{ub,sp}^j \geq \eta_j M_p \tag{8.2.8-4}$$

Splicing of columns $$M_{ub,sp}^j \geq \eta_j M_{pc} \tag{8.2.8-5}$$

5 The connection ultimate bearing capacity of column feet and foundations shall be checked and calculated according to the following formula:

$$M^{\mathrm{j}}_{\mathrm{u,\,base}} \geq \eta_{\mathrm{j}} M_{\mathrm{pc}} \tag{8.2.8-6}$$

Where M_p, M_{pc}——Respectively the plastic bend bearing capacity of beams and that of columns considered the axial force influence;

V_{Gb}——Design value of section shear force at beam ends analyzed according to simple beams when beams are under the action of gravity load representative value (the normal value of vertical earthquake action still shall be included for tall buildings for Intensity 9);

l_n——Clear span of a beam;

A_{br}——Sectional area of support struts;

$M^{\mathrm{j}}_{\mathrm{u}}, V^{\mathrm{j}}_{\mathrm{u}}$——Respectively the ultimate bending and shear bearing capacity of connections;

$N^{\mathrm{j}}_{\mathrm{ubr}}, M^{\mathrm{j}}_{\mathrm{ub,\,sp}}, M^{\mathrm{j}}_{\mathrm{uc,\,sp}}$——Respectively the ultimate compression (tension) and bend bearing capacity for connection and splicing of supports, splicing of beams and splicing of columns;

$M^{\mathrm{j}}_{\mathrm{u,\,base}}$——Ultimate bend bearing capacity of the column foot.

η_j——Connection coefficient, it may be selected according to those specified in Table 8.2.8.

Table 8.2.8 Connection Coefficient of Seismic Design of Steel Structure

Base material designation	Connection of beam column		Support connection and component splicing		Column foot	
	Welding	Bolted connection	Welding	Bolted connection		
Q235	1.40	1.45	1.25	1.30	Embedded	1.2
Q345	1.30	1.35	1.20	1.25	Epibolic	1.2
Q345GJ	1.25	1.30	1.15	1.20	Exposed	1.1

Note: 1 The steels with yield strength greater than Q345 are adopted according to the requirements of Q345;

2 The GJ steels with yield strength greater than Q345 GJ are adopted according to the requirements of Q345 GJ;

3 When welding web plate bolting, the connection coefficient shall be taken respectively according to the connection type in the table.

8.3 Details for Steel Frame Structures

8.3.1 The slenderness ratio of frame columns shall not greater than $60\sqrt{235/f_{ay}}$ for Grade 1, $80\sqrt{235/f_{ay}}$ for Grade 2, $100\sqrt{235/f_{ay}}$ for Grade 3 or $120\sqrt{235/f_{ay}}$ for Grade 4.

8.3.2 The width-to-thickness ratio of plates of frame beams and columns shall be in accordance with those specified in Table 8.3.2:

Table 8.3.2 Width-to-thickness Ratio Limit of Plate of Frame Beam and Column

Plate name		Grade 1	Grade 2	Grade 3	Grade 4
Column	Overhung part of I-shaped section flange	10	11	12	13
	Web plate of I-shaped section	43	45	48	52
	Box section wall plate	33	36	38	40
Beam	Overhung part of I-shaped section and box section flange	9	9	10	11
	Part of box section flange between two web plates	30	30	32	36
	Web plate of I-shaped section and box section	$72-120N_b/(Af)$ ≤ 60	$72-100N_b/(Af)$ ≤ 65	$80-110N_b/(Af)$ ≤ 70	$85-120N_b/(Af)$ ≤ 75

Note: 1 The listed value in the table is applicable to steel Q235; it shall multiply $\sqrt{235/f_{ay}}$ if steels of other designations are adopted.

2 $N_b/(Af)$ is the axial compression ratio of the beam.

8.3.3 The lateral supports of beam and column components shall meet the following requirements:

1 The compression flanges of beam and column components shall be arranged with lateral supports as required.

2 As for the beam and column components with plastic hinged section, the upper and lower flanges shall be set with lateral supports.

3 The slenderness ratio of components between the adjacent two lateral support points shall meet the relevant requirements of the current national standard "Code for Design of Steel Structures" GB 50017.

8.3.4 The connection structure of beams and columns shall meet the following requirements:

1 The connection of beams and columns should adopt the column-penetration type.

2 When a column is connected to beams by rigid connection in two orthogonal directions, the box section should be adopted, and partition plate shall be arranged at the connection of beam flanges; when the partition plate adopts slag welding, the column wall plate thickness should not be less than 16mm; I-shaped columns may be adopted or penetrating partition plates may be used if the column wall plate thickness is less than 16mm. When a column is connected to beams by rigid connection in only one direction, I-shaped columns should be adopted, and the column web plates shall be placed within the plane of the rigid-connection frame.

3 The rigid connection of beams with I-shaped columns (around strong axis) and box columns (Figure 8.3.4-1) shall meet the following requirements:

1) The full penetration groove weld shall be adopted between the beam flange and the column flange; the V-shaped cut impact ductility of welded joints shall be tested for Grade 1 and 2, the Charpy impact ductility is no less than 27J at −20℃;

2) The horizontal ribbed stiffeners (partition plates)shall be arranged at corresponding positions of columns at beam flanges, the thickness of ribbed stiffeners (partition plates) shall not be less than that of beam flanges; the strength of ribbed stiffeners (partition

plates) are the same as that of beam flanges;

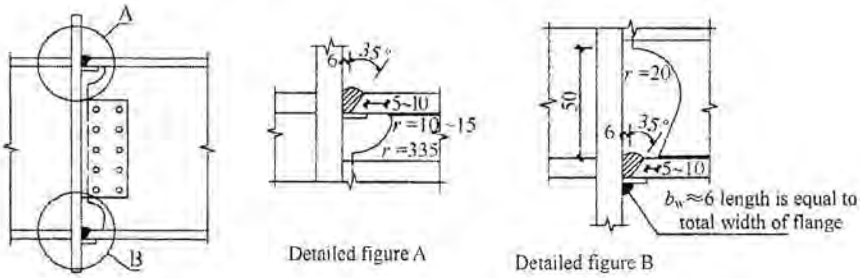


Figure 8.3.4-1 Field Connection of Frame Beam and Column

3) Connections of high-strength bolts of friction-type and column connection plates should be adopted for web plates of beams (gas shielded welding also may be used to weld if plates are qualified after the technological test and the field welding quality can be guaranteed); welding holes shall be arranged at angles of web plates, the hole shape shall completely separate full penetration groove weld joints of web plate ends and flanges of beams and columns;

4) The welding of web connection plates and columns, the twin fillet welding seam shall be adopted if the plate thickness is not greater than 16mm, the effective throat thickness shall meet the requirements of equal strength and is not less than 5mm; the K-shaped groove butt weld shall be adopted if the plate thickness is greater than 16mm. The gas shielded welding should be adopted for the above-mentioned weld, and the contour welding shall be adopted for slab ends;

5) For Grade 1 and 2, the end enlarged connection, beam end combined with cover plate or bone connection that can shift the plastic hinge from the beam end should be adopted.

4 When the rigid connection of cantilever beams and columns are adopted for frame beams (Figure 8.3.4-2), the all welded connection shall be adopted for cantilever beams and columns, here the shape of welding holes on upper and lower flanges should be the same; the flanges welding with web bolted connections or all bolted connections may be adopted for field splice of beams.

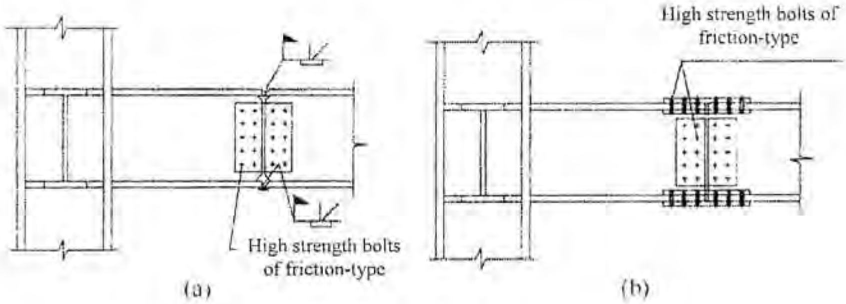


Figure 8.3.4-2 Connection of Frame Column and Cantilever Section of Beam

5 The partition arranged at the corresponding position of the beam flange for the box column shall be connected with wallboards by adopting the full penetration butt weld. The full penetration butt weld connection shall be adopted for the horizontal ribbed stiffener of the I-shaped column and the column flange; the fillet weld connection may be adopted for the horizontal ribbed stiffener of the I-shaped column and the web plate.

8.3.5 If the web thickness of the joint zone doesn't meet those specified in Item 2 and 3 of Article 8.2.5 of this code, measures such as thickening the column web plate or sticking the weld repair stiffening plate shall be taken. The thickness of the stiffening plate and its welded joints shall be designed according to the requirements of the shear force shared by the delivery stiffening plate.

8.3.6 As for the rigid connection of beams and columns, within the range of respectively 500mm up and down the beam flange, the full penetration groove weld shall be adopted for the attachment weld between the column flange and the column web plate or the box column wallboard.

8.3.7 The distance from the frame column joint to the upside frame beam may take the smaller one of 1.3m and the half column clear height.

The full penetration weld shall be adopted for butt joint of upper and lower columns; within the range of respective 100mm upper and lower of splicing joints of columns, the full penetration weld shall be adopted for welds among I-shaped column flanges and web plates as well as wallboards of box column corners.

8.3.8 The embedded type of rigid connection column foot of steel structures should be adopted, the Epibolic type may also be adopted; the exposed type may also be adopted if the height is less than 50m for Intensity 6 and 7.

8.4 Details of Seismic Design of Steel Frame-concentrically-supported Structures

8.4.1 The limits for slenderness ratio of member bars and for width-to-thickness ratio of plates of the center supports shall meet the following requirements:

1 When the slenderness ratio of supported member bars is designed according to compression bars, it shall not be less than $120\sqrt{235/f_{ay}}$; the design of tie bars shall not be adopted for concentrically supports for Grade 1, 2 and 3; the slenderness ratio shall not be greater than 180 when the design of tie bars is adopted for Grade 4.

2 The plate width-to-thickness ratio of support member bars shall not be greater than the specified limit in Table 8.4.1. The strength and stability of gusset plates shall be noticed when connecting by adopting gusset plates.

Table 8.4.1 Width-to-thickness Ratio Limit of Concentrically-supported Plate for Steel Structure

Plate name	Grade 1	Grade 2	Grade 3	Grade 4
Overhung part of flange	8	9	10	13
Web plate of I-shaped section	25	26	27	33
Box section wallboard	18	20	25	30
Wall ratio and outside diameter of round tube	38	40	40	42

Note: The listed value is applicable to steel Q235. Shall multiply $\sqrt{235/f_{ay}}$ when other designations of steels are adopted, the round tube shall multiply $235/f_{ay}$.

8.4.2 The structure of concentrically-supported joints shall meet the following requirements:

1 The support should be made by adopting H-shaped steels for Grade 1, 2 and 3; rigid structure may be adopted for the both ends and frames; the ribbed stiffeners shall be arranged at joints of beam columns and supports; when the welded I-shaped section support is adopted for Grade 1 and 2, the full penetration continuous weld should be adopted to connect the flanges and web plates.

2 The support bar end should be made into circular arc at joints of supports and frames.

3 The lateral support shall be arranged at the intersection of beams and V-shaped supports or zigzag supports; the lateral slenderness ratio (λ_y) and supporting force between the above-mentioned supporting point and that of the beam end shall meet the requirements about the plastic design specified in the current national standard "Code for Design of Steel Structures" GB 50017.

4 If the gusset plates are adopted for connections of supports and frames, they shall meet the requirements about that gusset plates have angles no less than 30° on each side of connecting member bars, specified in the current national standard "Code for Design of Steel Structures" GB 50017; the distance from the support end to the nearest built-in point of the gusset plate (the end of gusset plate and the attachment weld of frame component) along the axial direction of the support member bar shall not be less than 2 times thickness of the gusset plate.

8.4.3 For the frame part of frame-concentrically-supported structures, when the building height is not greater than 100m and the seismic shear force on frame parts distributed by calculation is not greater than 25% of the total seismic shear force on the structure bottom, the details of seismic design for Grade 1, 2 and 3 may be adopted according to the corresponding requirements that reduce one grade according to frame structures. Other details of seismic design shall meet the requirements of details of seismic design for frame structures specified in Section 8.3 of this code.

8.5 Details for Seismic Design of Steel Frame-eccentrically-supported Structures

8.5.1 The steel yield strength of energy-dissipated beams on eccentrically-supported frames shall not be greater than 345MPa. For the energy-dissipated beams as well as the non-energy-dissipated beams in the same span with energy-dissipated beam, the width-to-thickness ratio of plates shall not be greater than the limits specified in Table 8.5.1.

Table 8.5.1 Limits for Width-to-thickness Ratio of Plates of the Eccentrically-supported-frame-beams

Plate name		Width-to-thickness ratio limit
Overhung part of flange		8120
Web plate	Where $N/(Af)\le 0.14$	$90\,[1-1.65\,N/(Af)]$
	Where $N/(Af)>0.14$	$33\,[2.3-N/(Af)]$

Note: The listed value is applicable to steel Q235. Shall multiply $\sqrt{235/f_{ay}}$ when materials are other designations of steels. $N/(Af)$ is the axial compression ratio of the beam.

8.5.2 The slenderness ratio of support member bars on eccentrically-supported frames shall not be greater than 120, the width-to-thickness ratio of plates of support member bars shall not exceed the width ratio limit of the elastic design on the axial compression components specified in the current national standard "Code for Design of Steel Structures" GB 50017.

8.5.3 The structure of energy-dissipated beams of shall meet the following requirements:

1 When at $N>0.16Af$, the length of energy-dissipated beams of shall meet the following requirements:

When $\rho(A_w/a)<0.3$

$$a<1.6M_{lp}/V_l \tag{8.5.3-1}$$

When $\rho(A_w/a) \geq 0.3$

$$a \leq [1.15-0.5\rho(A_w/A)1.6M_{lp}/V_l] \tag{8.5.3-2}$$

$$\rho = N/V \tag{8.5.3-3}$$

Where a——Length of energy-dissipated beams;

ρ——Ratio of design value of axial force and shear force on energy-dissipated beams.

2 The web plate of energy-dissipated beams shall not be sticked on welding stiffening plate and shall not be punched holes.

3 The ribbed stiffener shall be prepared at both sides of the web plate at joints of energy-dissipated beams and supports, the height of the ribbed stiffener shall be the height of the beam web plate, the ribbed stiffener width of one side shall not be less than $(b_f/2-t_w)$, the thickness shall not be less than the bigger value of $0.75t_w$ and 10mm.

4 The middle ribbed stiffener shall be arranged on the web plate of energy-dissipated beams according to the following requirements:

1) When $a \leq 1.6M_{lp}/V_l$, the spacing of the ribbed stiffener shall not be greater than $(30t_w-h/5)$;

2) When $2.6M_{lp}/V_l<a \leq 5M_{lp}/V_l$, the middle ribbed stiffener shall be prepared $1.5b_f$ away from the end of the energy-dissipated beams, and the spacing of the middle ribbed stiffener shall not be greater than $(52t_w-h/5)$;

3) When $1.6M_{lp}/V_l<a \leq 2.6M_{lp}/V_l$, the spacing of the middle ribbed stiffener should be linearly-inserted between the above-mentioned two;

4) When $a>5M_{lp}/V_l$, the middle ribbed stiffener may not be prepared;

5) The height of the middle ribbed stiffener and the web plate of energy-dissipated beams shall be the same; the single-side ribbed stiffener may be prepared if the section height of energy-dissipated beams is not greater than 640mm; the ribbed stiffener shall be prepared at both sides if the section height of energy-dissipated beams is greater than 640mm, hereof, the ribbed stiffener width of one side shall not be less than $(b_f/2-t_w)$, the thickness shall not be less than t_w and 10mm.

8.5.4 The connection of energy-dissipated beams with columns shall meet the following requirements:

1 To connect the energy-dissipated beams and columns, the beam length shall not be greater than $1.6M_{lp}/V_l$, and the requirements of relevant standards shall be met.

2 The full penetration butt weld of groove shall be adopted for the connection of energy-dissipated beam flanges and column flanges; the fillet weld (gas shielded welding) shall be adopted for the connection of web plates and columns for energy-dissipated beams; the bearing capacity of the fillet weld shall not be less than the axial force and shear force of web plates as well as the bearing capacity acted with the bending moment simultaneously of energy-dissipated beams.

3 When energy-dissipated beams connecting with column web plates, the full penetration butt weld of groove shall be adopted for the connection of energy-dissipated beam flanges and horizontal stiffening plates; the fillet weld (gas shielded welding) shall be adopted for the connection of web plates and column connection plates; the bearing capacity of the fillet weld shall not be less than the axial force and shear force of web plates as well as the bearing capacity acted with the bending moment simultaneously of energy-dissipated beams.

8.5.5 The lateral support shall be arranged at upper and lower flanges at both ends of energy-dissipated beams; the axial force design value of supports shall not be less than 6% of the design value of the axial bearing capacity for flanges of energy-dissipated beams, that is $0.06b_f t_f f$.

8.5.6 The lateral support shall be arranged at upper and lower flanges of non-energy-dissipated beams for eccentrically-supported-frame beams, the axial force design value of supports shall not be less than 2% of the design value of the axial bearing capacity for beam flanges, which is $0.02\ b_f t_f f$.

8.5.7 For the frame part of frame-eccentrically-supported structures, if the building height is not greater than 100m and the earthquake action on frame parts distributed through calculation is not greater than 25% of the total seismic shear force on the structure bottom, the details of seismic design for Grade 1, 2 and 3 may be adopted according to the corresponding requirements that are reduced by one grade according to frame structures. Other details of seismic design shall meet the requirements on details of seismic design of the frame structures specified in Section 8.3 of this code.

9 Single-storey Factory Buildings

9.1 Single-storey Factory Buildings with R.C. Columns

(I) General Requirements

9.1.1 This section is mainly applicable to the assembled type single-storey factory buildings with R.C. columns and the structural layout shall be in accordance with the following requirements:

1 Multi-span factory building should be equal in height and length and the high-low span factory building should not be adopted with structural layout featured by one-end opening.

2 Auxiliary buildings and structure of a factory building should not be arranged at the corners of the buildings or close to the seismic joints.

3 When the factory building body is complex or there are auxiliary houses and structures, seismic joints should be adopted; At the longitudinal and traverse span intersection part of factory building, the seismic joint width of large column-grid factory building or factory building without inter-column support may be 100mm~150mm and for the other conditions, it may be 50mm~90mm.

4 The transition span between two main buildings shall at least be provided with seismic joint at one end in order to be detached from the main building.

5 The iron ladder of crane in factory building shall not be laid close to the seismic joint; the iron ladder of crane at each span of multi-span factory building should not be erected near the same transverse axial line.

6 The operating platform and rigid workshop in factory building should be detached from the major factory building structure.

7 Different structure forms shall not be adopted in the same structural unit of factory building; the factory building end shall be installed with roof truss and with the gable wall shall not be used to bear load; transverse wall and bent frame shall not be used together to bear load in factory building unit.

8 The column space in factory building should be equal; the lateral rigidity of each colonnade should be uniform; when the column is removed, seismic measures shall be taken.

Note: The seismic design for concrete frame-bent structure factory building shall be in accordance with the requirements of Section H.1 in Appendix H of this code.

9.1.2 The installation of factory building skylight truss shall be in accordance with the following requirements:

1 Skylight should be adopted with the wind-sheltered skylight with smaller part extending the roof and sunken skylight should be adopted if possible or where it is for Intensity 9 .

2 The skylight protruding roof should be adopted with steel skylight truss; when it is for Intensity 6~8, reinforced concrete skylight truss with rectangular section member bar may be adopted.

3 The skylight truss should not be installed from the first bay of factory building structural unit; when it is for Intensity 8 and 9, the skylight truss should be installed from the third bay of factory building unit end.

4 The skylight roof, front sheet and side sheet should be adopted with light sheets; the front sheet shall not replace the end skylight truss.

9.1.3 The installation of factory building roof truss shall be in accordance with the following requirements:

1 Factory building should be adopted with steel roof truss or the roof truss with pre-stressed concrete and reinforced concrete of lower gravity center.

2 When the span is not greater than 15m, roof beam with reinforced concrete may be adopted.

3 When the span is greater than 24m or it is Intensity 8 with site class III and IV and Intensity 9, the steel roof truss shall be adopted in priority.

4 When the column space is 12m, pre-stressed concrete bracket (beam) may be adopted; when steel roof truss is adopted, steel bracket (beam) may also be adopted.

5 The roof protruding the roof skylight truss should not be open web roof truss of pre-stressed concrete or reinforced concrete.

6 When it is Intensity 8 (0.30*g*) and Intensity 9, factory building with span greater than 24m should not be adopted with large-scale roof slab.

9.1.4 Installation of factory building column shall be in accordance with the following requirements:

1 When it is Intensity 8 and Intensity 9, rectangular or I-shaped section column or double leg column with diagonal web should be adopted instead of I-shape column of thin wall, I-shape column with open pore on web plate, I-shape column of prefabricated web plate or pipe column.

2 The column within the range of above 500mm form column root to indoor ground level and the column on the stepped column should be adopted rectangular section.

9.1.5 The arrangement of the enclosure wall and masonry parapet wall of the factory building, material selection as well as seismic structural measures shall be in accordance with relevant requirements in Section 13.3 of this code.

(II) Essentials in Calculation

9.1.6 Traverse and longitudinal seismic check may be omitted, where single-storey factory building adopts seismic structural measures according to the requirements of this code and any of the following conditions is met:

1 Single-span and equal-height multi-span factory building (expect zigzag-type factory building) with gable walls at both sides of the construction unit, for Intensity 7 at Class I and II site, column height no larger than 10m.

2 Open-air crane trestle for Intensity 7, Intensity 8 (0.20*g*) at Class I and II site.

9.1.7 The traverse seismic calculation of factory building shall be carried out with the following methods:

1 For the factory building without or with concrete purline roof, the traverse elastic deformation of roof, in general situation, shall be taken into account and shall be analyzed according to the multi-mass three-dimensional structure; when the conditions specified in Appendix J of this code are met, it may be calculated according to the plane bent frame and the seismic shear force and bending moment shall be adjusted according to those specified in Appendix J.

2 For the light roof factory building, where the column spaces are equal, it may be calculated according to the plane bent frame.

Note: The light roof in this section refers to the purline roof of profile steel sheet, corrugated iron and so on.

9.1.8 The longitudinal seismic calculation of factory building shall be carried out with the following methods:

1 The light roof factory building without or with concrete purline roof and with integrated support system may be adopted with the following methods:

1) Typically, the longitudinal elastic deformation of roof and the effective rigidity of enclosure wall and partition wall should be taken into account; when it is unsymmetrical, torsion influence should also be taken into account and the three-dimensional structure analysis shall be carried out according to the multi-mass.

2) As for single-span or equal-height multi-span reinforced concrete column factory building with column top elevation and average span less than or equal to 15m and 30m respectively, it should be calculated according to the modifying rigidity method as specified in Section K.1 of Appendix K in this code.

2 For the single span factory building with longitudinal walls symmetrically laid out and the light roof multi-span factory building, it may be calculated independently according to the colonnade piece by piece.

9.1.9 The traverse seismic calculation of skylight truss protruding roof may be adopted with the following methods:

1 The traverse seismic calculation of the triple hinged arch reinforced concrete with diagonal and the steel skylight truss may be adopted with base shear method; when the span is greater than 9m or for Intensity 9, the earthquake action effect of concrete skylight truss shall be multiplied by enhancement coefficient and its value may be 1.5.

2 The traverse earthquake action in other situations may be adopted with mode-decomposition response spectrum method.

9.1.10 The longitudinal seismic calculation of skylight truss protruding roof may be carried out with the following methods:

1 The longitudinal seismic calculation of skylight truss may be adopted with three-dimensional structure analysis method and the elastic deformation of roof surface and the effective rigidity of longitudinal wall shall be taken into account.

2 The longitudinal earthquake action calculation of skylight truss for factory building of single span with column height not exceeding 15m and of equal-height multi-span concrete without purline roof may be adopted with base shear method; however, the earthquake action effect of skylight truss shall be multiplied by enhancement coefficient of effect and the value may be taken according to the following requirements:

1) For single span, side span roofs or mid-span roof with longitudinal internal partition wall:

$$\eta = 1 + 0.5n \tag{9.1.10-1}$$

2) For other mid-span roofs:

$$\eta = 0.5n \tag{9.1.10-2}$$

Where η——Effect enhancement coefficient;

n——Span number of factory building; when there are more than four spans, it shall be taken as four spans.

9.1.11 For the large column grid factory building with column space in two principal axis not less than 12m, without overhead crane or column support, the seismic check of column section shall simultaneously calculate the horizontal earthquake action of the two principal axis directions as well as the additional bending moment due to personal drift.

9.1.12 The section area of longitudinal tension steel reinforcement supporting low-span column corbel (column shoulder) in non-equal-height factory building shall be determined according to the following formula:

$$A_s \geq \left(\frac{N_G \alpha}{0.85 h_0 f_y} + 1.2 \frac{N_E}{f_y} \right) \gamma_{RE} \tag{9.1.12}$$

Where A_s——Section area of longitudinal horizontal tension steel reinforcement;

N_G——Design pressure value caused by the representative value of gravity load on column corbel surface;

α——Distance from the gravity action point to the near edge of the lower column bracket; when it is less than $0.3h_0$, it shall be equal $0.3h_0$;

h_0——Effective height of the maximum corbel vertical section;

N_E——Designed horizontal pull value of seismic array on column corbel surface;

f_y——Design value of steel reinforcement tensile strength;

γ_{RE}——Anti-seismic adjustment coefficient of bearing force, which may adopt 1.0.

9.1.13 For the earthquake action effect of cross support diagonal between columns and the anti-seismic of support connection joints shall be checked according to those specified in Section K.2 of Appendix K in this code. The lower joints of lower column inter-column support shall be positioned above the top surface of foundation according to those specified in Article 9.1.23 of this code; the oblique section shear bearing capacity of longitudinal colonnade column root should be checked.

9.1.14 Wind-resistant column and roof truss column of the factory building, as well as seismic calculation with regard to the influence of working platform shall be in accordance with the following requirements:

1 For the wind-resistant column of high and large gable wall, where it is for Intensity 8 and

Intensity 9, section seismic capacity check outside the plane shall be carried out.

2 When wind-resistant column is connected with bottom chord of roof truss, the connection joints shall be set at the Transverse support of the bottom chord and the section and connecting joints of bottom chord transverse support bar shall be carried out with seismic capacity check.

3 When the operating platform and rigid internal partition wall are connected with the major structure of factory building, calculation diagrams corresponding with actual load on factory building shall be adopted and the additional earthquake action influence of operating platform and rigid internal partition wall on factory building shall be taken into account. As for the bent-frame column with constrained dislocation and with shear span ratio not greater than 2, the shear bearing capacity of oblique section shall be calculated in accordance with those specified in the current national standard "Code for Design of Concrete Structures" GB 50010 and corresponding details of seismic design shall be carried out according to those specified in Article 9.1.25 of this code.

4 As for Intensity 8 at Class III and IV site, for the arch with work column, broken line-type roof truss or the roof truss with longer joint space of top chord and lager vectorheight, the top chord should be checked for the torsion resistance.

(III) Details of Seismic Design

9.1.15 The connection and support layout of components with purline roof shall be in accordance with the following requirements:

1 The purline shall be well welded with the concrete roof truss (roof beam) and adequate support length shall be provided.

2 The double-ridge purline shall be tied at 1/3 section of span.

3 The profile steel sheet shall be reliably connected with purline and corrugated irons and asbestos tiles shall be tied with purlines.

4 Support layout should be in accordance with those specified in Table 9.1.15.

Table 9.1.15 Layout of Support with Purline Roof

<table>
<tr><th rowspan="2" colspan="2">Support name</th><th colspan="3">Intensity</th></tr>
<tr><th>6, 7</th><th>8</th><th>9</th></tr>
<tr><td rowspan="4">Roof truss support</td><td>Transverse support of top chord</td><td>It shall set a support for the bay at unit end</td><td>It shall set a support in the unit end bay and the column support bay with unit length greater than 66m;
It shall additionally set a partial support at both ends within the range of skylight opening</td><td rowspan="3">It shall set a support in the unit end bay and the column support bay with unitIt shall additionally set a partial top chord Transverse support at both ends within the range of skylight opening. length greater than 42m</td></tr>
<tr><td>Transverse support of bottom chord</td><td colspan="2" rowspan="2">Same with non-seismic design</td></tr>
<tr><td>Mid-span vertical support</td></tr>
<tr><td>End vertical support</td><td colspan="3">When end height of roof truss is greater than 900mm, it shall set a support for unit end bay and column support bay</td></tr>
</table>

Table 9.1.15 (continued)

Support name		Intensity		
		6, 7	8	9
Skylight truss support	Transverse support of top chord	It shall set a support for the bay at unit skylight end	It shall set a support in unit skylight end bay and every other 30m	It shall set a support in unit skylight end bay and every other 18m
	Vertical support at both sides	It shall set a support in unit skylight end bay and every other 36m		

9.1.16 The connection and the support layout of components without purline roof shall be in accordance with the following requirements:

1 Large scale roof slab shall be well- welded with roof truss (roof beam) and the connected weld length between roof slab and roof truss (roof beam) near colonnade should not be less than 80mm.

2 For the end bay of factory building unit with skylight for Intensity 6 and 7 and each bay for Intensity 8 and 9, the top surfaces of adjacent large scale roof slabs at both sides of the vertical roof truss direction shall be well welded with each other.

3 The embedded parts of large scale roof slab terminal bottom surface where it is for Intensity 8 and 9 should be adopted with angle steels and shall be well welded with the main reinforcement.

4 Nonstandard roof slab should be adopted with assembled monolithic joint or it shall be well welded with roof truss (roof beam) after cutting off the four corners of slab.

5 For the anchor bar of embedded parts at the top surface of roof truss (roof beam) end, where it is for Intensity 8, it should be greater than or equal to 4ø10; when it is for Intensity 9, it should be greater than or equal to 4ø12.

6 Support layout should be in accordance with those specified in Table 9.1.16-1; when there is intermediate well type skylight, it shall be in accordance with those specified in Table 9.1.16-2; when roof beam is adopted by factory building roof with span not greater than 15m for Intensity 8 and 9, a vertical support may be set at both ends of factory building unit; the roof support layout of lean-to roof beam should be carried out according to that with the roof truss end height greater than 900mm.

Table 9.1.16-1 Layout of Support Without Purline Roof

Support name		Intensity		
		6, 7	8	9
Roof truss support	Transverse support of top chord	When the span of roof truss is less than 18m, it shall be in accordance with the non-seismic design; when the span is not less than 18m, a support shall be set in the unit end bay of factory building	A support shall be set in unit end bay and column support bay and an additional partial support shall be set at both ends within the range of skylight opening	

Table 9.1.16-1 (continued)

<table>
<tr><th colspan="3" rowspan="2">Support name</th><th colspan="3">Intensity</th></tr>
<tr><th>6, 7</th><th>8</th><th>9</th></tr>
<tr><td rowspan="6">Roof truss support</td><td colspan="2">Full-length horizontal tie bar of top chord</td><td rowspan="5">Same with non-seismic design</td><td>A support shall be set not greater than 15m along the span of roof truss; however, it may be set only within the range of skylight opening for assembled monolithic roof; when cast-in-place ring beam is provided for enclosure wall on the top chord height of roof truss, it may not be additionally set at the end</td><td>A support shall be set not greater than 12m along the span of roof truss. However, it may be set only within the range of skylight opening for assembled monolithic roof; when cast-in-place ring beam is provided for enclosure wall on the top chord height of roof truss, it may not be additionally set at the end</td></tr>
<tr><td colspan="2">Transverse support of bottom chord</td><td rowspan="2">Same with non-seismic design</td><td rowspan="2">Same with transverse support of top chord</td></tr>
<tr><td colspan="2">Mid-span vertical support</td></tr>
<tr><td rowspan="2">Vertical support at both ends</td><td>Roof truss end height ≤900mm</td><td>It shall set a support in unit end bay</td><td>It shall set a support in unit skylight end bay and every other 48m</td></tr>
<tr><td>Roof truss end height >900mm</td><td>It shall set a support in each unit end bay</td><td>It shall set a support in each unit end bay and each column support bay</td><td>It shall set a support in each unit end bay, column support bay and every other 30m</td></tr>
<tr><td rowspan="2">Skylight truss support</td><td colspan="2">Vertical support at both sides of skylight</td><td>It shall set a support in each unit skylight end bay of factory building and every other 30m</td><td>It shall set a support in each unit skylight end bay of factory building and every other 24m</td><td>It shall set a support in each unit skylight end bay of factory building and every other 18m</td></tr>
<tr><td colspan="2">Transverse support of top chord</td><td>Same with non-seismic design</td><td>When the skylight spans ≥9m, it shall set a support in each unit skylight end bay and column support bay</td><td>It shall set a support in each unit end bay and each column support bay</td></tr>
</table>

Table 9.1.16-2 Layout Support with Intermediate Well Skylight and Without Purline Roof

<table>
<tr><th colspan="2">Support name</th><th>Intensity 6 and 7</th><th>Intensity 8</th><th>Intensity 9</th></tr>
<tr><td colspan="2">Transverse support of top chord
Transverse support of bottom chord</td><td>It shall set a support in each unit end bay of factory building</td><td colspan="2">A support shall be set in each unit end bay of factory building and each column support bay</td></tr>
<tr><td colspan="2">Full-length horizontal tie bar of top chord</td><td colspan="3">Set at top chord joints of roof truss mid-span within the range of skylight</td></tr>
<tr><td colspan="2">Full-length horizontal tie bar of the bottom chord</td><td colspan="3">Set at the bottom chord joint of roof truss within the range of skylight and at both sides of skylight</td></tr>
<tr><td colspan="2">Mid-span vertical support</td><td colspan="3">The transverse support bay of top chord shall be set at the corresponding position with the full-length tie bar of bottom chord</td></tr>
<tr><td>Vertical support at both ends</td><td>Roof truss end height ≤900mm</td><td colspan="2">Same with non-seismic design</td><td>There is transverse support bay of top chord and the space is not greater than 48m</td></tr>
</table>

Table 9.1.16-2 (continued)

Support name		Intensity 6 and 7	Intensity 8	Intensity 9
Vertical support at both ends	Roof truss end height >900mm	It shall set a support in each unit end bay of factory building	There is transverse support bay of top chord and the space is not greater than 48m	There is transverse support bay of top chord and the space is not greater than 30m

9.1.17 Roof support shall also be in accordance with the following requirements:

1 Full length horizontal pressure bar of top chord shall be installed at the crest point of roof truss within the range of skylight opening; When it is for Intensity 8 at Class III and IV site and Intensity 9, the upper joint of trapezoid roof truss end shall be installed with full length horizontal pressure bar along the longitudinal direction of factory building.

2 The mid-span vertical support space of roof truss in span direction, for Intensity 6~8, shall not be greater than 15m; for Intensity 9, shall not be greater than 12m; when only one support is set at the mid-span, it shall be set at the roof truss ridge; when two supports are set, it shall be laid out uniformly in span direction.

3 The full-length horizontal tie bar of roof truss top and bottom chords shall be installed in cooperation with the vertical support.

4 For the factory building with column space not less than 12m and roof span being 6m, longitudinal horizontal support of bottom chord shall be set in the bracket (beam) section and its adjacent bays.

5 Supporting member bar of roof should be made of profile steels.

9.1.18 Wallboards at both sides of concrete skylight truss protruding roof shall be connected with the skylight upright column with bolts.

9.1.19 The section and the reinforcement of concrete roof truss shall be in accordance with the following requirements:

1 The reinforcement of the first chord bay of top chord and the trapezoid roof truss end vertical bar, for Intensity 6 and 7, should be greater than or equal to 4ø12; for Intensity 8 and 9, should be greater than or equal to 4ø14.

2 The section width of end vertical bar for trapezoid roof truss should be the same with that of the top chord.

3 As for the small vertical column supporting roof slab at the top chord end of arch and mansard roof truss, the section should not be less than 200mm×200mm; the height should not be larger than 500mm. The main reinforcement should be adopted with ∏ form, it should be greater than or equal to 4ø12 for Intensity 6 and 7 and should be greater than or equal to 4ø14 for Intensity 8 and 9; ø6 may be adopted for the stirrup and the space should not be larger than 100mm.

9.1.20 The stirrup of factory building column shall be in accordance with the following requirements:

1 The stirrup of column within the following range shall be thickened:

1) Column head, 500mm below the column top and not less than the dimension of longer side of column section;

2) Upper column, from the corbel surface to a distance of 300mm above the crane beam surface of a stepped column;

3) Corbel (column shoulder), total height;

4) Column root, from the bottom of the lower column to a distance of 500mm above the indoor ground level;

5) The connecting joints between column support and column as well as positions where column deflection are constrained by the platform, a distance of 300mm above and below of the joints.

2 The space of stirrup in the densified area shall not be larger than 100mm, and the space between legs and the minimum diameter of stirrups shall meet those specified in Table 9.1.20.

Table 9.1.20 Maximum Distance and Minimum Diameter of Stirrup in the Densified area of Column

Intensity and Site class		Intensity 6, Intensity 7 at Class I, II site	Intensity 7 at Class III and IV site, Intensity 8 at Class I and II site	Intensity 8 at Class III and IV site, Intensity 9
Maximum space for legs of stirrup (mm)		300	250	200
Minimum diameter of stirrup	General column head and column root	ø6	ø8	ø8 (ø10)
	Column top of corner column	ø8	ø10	ø10
	Corbel of upper column and column root with support	ø8	ø8	ø10
	Column head with support and position at which column deflection is constrained	ø8	ø10	ø12

Note: Values in parentheses are applied to column root.

3 As for the laterally-constrained bent frame columns with shear span ratio of no larger than 2, the structure of the embedded steel plate on the column top and the densified area structure of column stirrup shall meet the following requirements:

1) The length of the embedded steel plate on the column top along the bent frame plane should be equal to the section height of the column top, and shall not be less than 1/2 of the section height or 300mm;

2) As for the installation position of the roof truss, the eccentricity on the top of the column should be reduced; the eccentricity of axial force on the top of the column shall not be larger than 1/4 of the section height;

3) When the eccentricity on the plane of column top axial force bent frame is at the range of 1/6~1/4 of the section height, the volume reinforcement ratio of stirrup in stirrup densified area of column top should not be less than 1.2% for Intensity 9; 1.0% for Intensity 8; 0.8% for Intensity 6 and 7;

4) Stirrup in the densified area should be equipped with four-leg stirrups with no larger than 200mm leg distance.

9.1.21 The section and reinforcement structure for columns of factory building with large-sized column-grid shall meet the following requirements:

1 The column section should adopt the square or nearly square rectangular shapes, and the side length should not be less than 1/18~1/16 of the total height of the column.

2 The column axial-compression-ratio of heavy weight roof factory building should not be larger than 0.8 for Intensity 6 and 7, 0.7 for Intensity 8, and 0.6 for Intensity 9.

3 Longitudinal steel reinforcements should be arranged symmetrically along the perimeters of the column section, the space should not be larger than 200mm; and the steel reinforcements with larger diameter should be arranged at the corner of section.

4 The stirrup of column head and column root should be densified and shall be in accordance with the following requirements:

1) For the column root, the densified scope from the foundation top surface 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 column top and the distance shall not be less than the longer side dimension of the column section;

2) Stirrup diameter, space and leg distance shall be in accordance with those specified in Article 9.1.20 of this code.

9.1.22 Wind resistance column of gable wall shall be in accordance with the following requirements:

1 For the stirrup within the scope of 300mm below the wind-resisting column top and 300mm above the corbel (column shoulder), the diameter should not be less than 6mm, the space should not be greater than 100mm, and the space of legs should not be greater than 250mm.

2 Longitudinal tension steel reinforcement should be installed at the variable section corbel (column shoulder) of wind resistance column.

9.1.23 The installation and structure of column support in factory building shall be in accordance with the following requirements:

1 The column support layout in factory building shall be in accordance with the following requirements:

1) Generally, upper and lower column supports shall be installed in the middle of factory building unit and the lower column support shall be matched with the upper column support;

2) When there are cranes or for Intensity 8 and 9, support of upper columns should be installed at both ends of the factory building unit;

3) When the factory building unit is long or for Intensity 8 at Class III and IV site or for Intensity 9, two column supports may be arranged in 1/3 length scope measured from both ends of the factory unit.

2 Column support shall be of profile steel; the support should be cross type and the intersection angle between its diagonal and horizontal plane should not be greater than 55 Degree.

3 The slenderness ratio of supporting member bar should not exceed those specified in Table 9.1.23.

4 The location of the lower joint and structural measures for the lower column support shall ensure to directly transfer the earthquake action to the foundation; for Intensity 6 and 7 (0.10g), if the earthquake action cannot be transferred directly to the foundation, strengthening measures shall be adopted with regard to the adverse effects of the support on the column and the foundation.

Table 9.1.23 Maxima Slenderness Ratio of Cross Support Diagonal

Position	Intensity			
	Intensity 6, Intensity 7 at Class I and II site	Intensity 7 at Class III and IV site, Intensity 8 at Class I and II site	Intensity 8 at Class III and IV site, Intensity 9 at Class I and II site	Intensity 9 at Class III and IV site
Upper column support	250	250	200	150
Lower column support	200	150	120	120

5 Gusset plate shall be installed at the crossing point of Cross Support; the thickness shall not be less than 10mm; the diagonal shall be welded with the cross joint plate as well as the end joint plate.

9.1.24 As for the middle columns of multi-span factory buildings with no less than 18m span for Intensity 8, and each column of multi-span factory buildings for Intensity 9, the full-length horizontal pressure bars should be installed on the column top. This compression bar may be installed by combining with the full-length horizontal tie bars at the support of the trapezoid roof truss. The clearance between the end of the reinforced concrete tie bar and the roof truss shall be filled with concrete.

9.1.25 The connected joint of factory building structural components shall be in accordance with the following requirements:

1 The roof truss (roof beam) and the column top should be connected with bolts where it is for Intensity 8 and steel plate hinges or bolts where it is for Intensity 9; the thickness of end bearing plate for Roof truss (roof beam) should not be less than 16mm.

2 The anchoring bars of embedded parts on column top should not be less than 4ø14 for Intensity 8 and 4ø16 for Intensity 9. For columns with column support, shear plate shall be installed additionally for the embedded parts on column top.

3 Embedded plates shall be installed on the top wind resistance column of gable wall in order to reliably connect the column top with the top chord (top flange of roof beam) of end roof truss. The connection position shall be located at the connection point between the top chord transverse support and the roof truss; when it is discrepant, sub-web member or profile steel cross-beam may be additionally installed in the support.

4 The embedded parts in the corbel (column shoulder) of the middle-column supporting the lower-span roof shall be welded with the longitudinal steel reinforcement bearing the calculated horizontal tension in the corbel (column shoulder). And the welded steel reinforcement shall not be less than 2ø12 for Intensity 6 and 7, 2ø14 for Intensity 8, and 2ø16 for Intensity 9.

5 For Intensity 8 at Class III or IV site and for Intensity 9, the angle steels together with end plate should be used as anchoring pieces of the embedded parts at the joint of the column support and the column. And in other cases, the hot-rolled reinforcement no less than HRB335 may be adopted, but the anchorage length shall be no less than 30 times of the anchoring bar diameter and the added end plate.

6 The crane sidewalk plate, the small roof plate filling the space between the end roof truss and the gable wall, the gutter plate, the filling masonry underneath the skylight end plate and side plate as well as other components shall have reliable connection with its supporting structure.

9.2 Single-storey Steel Factory Buildings

(I) General Requirements

9.2.1 This section is mainly applicable to steel column, steel roof truss or steel roof beam bearing single-storey factory buildings. The seismic design for single-storey factory buildings with light steel structures shall meet the special requirements.

9.2.2 The structural system of factory building shall be in accordance with the following requirements:

1 The transverse lateral resistant system of factory building may be adopted with rigid-connection frame, hinged frame, portal rigid frame or other structural systems. The longitudinal lateral resistant system of factory building shall be adopted with column support where it is for Intensity 8 and 9; column support or rigid-connection frame should be adopted where it is for Intensity 6 and 7.

2 When overhead traveling crane is set in the factory building, the connection between the members of crane beam system and the frame column of factory building shall be able to reliably transfer vertical horizontal earthquake action.

3 The roof shall be installed with complete roof support system. When the transverse beam of the roof is hinged with the column top, bolted connection should be adopted.

9.2.3 The factory building plane, reinforced concrete roof board and skylight truss may be arranged in accordance with the relevant requirements of Section 9.1 "Single-storey Factory Buildings with R.C.Columns" in this code. When seismic joints are set, the joint width should not be less than 1.5 times of that of single-storey factory building with reinforced concrete column.

9.2.4 The enclosure wall boards of factory buildings shall be in accordance with the relevant requirements of Section 13.3 in this code.

(II) Seismic Check

9.2.5 In the seismic calculation of factory buildings, the earthquake action shall be calculated with calculation model corresponding to the actual working conditions of factory building structures, according to the roof height difference and crane arrangement conditions.

The damping ratio of single-storey factory building may be adopted with 0.045~0.05 according to the types of roofs and enclosure walls.

9.2.6 In calculation of factory building earthquake action, the values for dead weight and rigidity of enclosure wall body shall be taken according to the following requirements:

1 The entire dead weight of prefabricated concrete wallboard flexibly connecting the light wallboard or the column shall be counted in; however, its rigidity shall not be counted in;

2 For masonry enclosure walls closely built to and tied to the columns, the total dead weight of walls shall be counted in. When the earthquake action is calculated longitudinally along the wall, the conversion rigidity and conversion coefficient of common brick masonry wall shall be taken into account, it shall adopt respectively 0.6, 0.4 and 0.2 for Intensity 7, 8 and 9.

9.2.7 The horizontal seismic calculation of factory building may be carried out with the following methods:

1 Typically, spatial analysis method should be adopted with regard to elastic deformation of roof;

2 As for the light roof factory building with regular plane and uniform lateral rigidity, it may be calculated according to the plane framework. Base shear method may be adopted for the equal-height factory buildings; mode-decomposition response spectrum method shall be adopted for high-low span factory buildings.

9.2.8 The longitudinal seismic calculation of factory building shall be carried out with the following methods:

1 As for factory buildings with light sheet enclosure walls or large-size wall boards flexibly connected with columns, it may be calculated with base shear method. The earthquake action of each longitudinal colonnade may be distributed according to the following principles:

1) The light roof may be distributed according to ratio of gravity load representative value born by longitudinal colonnade;

2) For reinforced concrete roof without purline roof, the distribution may be carried out according to the rigidity ratio of the longitudinal colonnade;

3) For reinforced concrete roof with purlines, the average value of the above mentioned two results may be adopted.

2 As for factory buildings with common brick mansonry enclosure walls which are built close to and tied with columns, it shall be calculated according to the requirements of Section 9.1 of this code.

3 As for the colonnade set with column support, the earthquake action effect after flexion of supporting members shall be counted in.

9.2.9 Seismic calculation of the factory building roof components shall be in accordance with the following requirements:

1 The web members vertically supporting the truss shall be able to bear and transmit horizontal earthquake action of the roof. The bearing capacity of its connection shall be larger than that of the web member and shall meet the structural requirements.

2 The counter diagonals of both roof transverse horizontal support and longitudinal horizontal support may be designed as the draw bar and adopt the same section areas.

3 For Intensity 8 and 9, as for the bracket of roof beam with larger than 24m support span as well as the roof beam with larger equipment loads, their vertical earthquake action shall be calculated according to Section 5.3 of this code.

9.2.10 The combined action of tension/pressure bars shall be taken into consideration for X-support, V-support or Λ-support between columns. Their earthquake actions and check calculations may be calculated as draw bar according to the requirements of Section K.2 in Appendix K of this code, and the influence of intersecting compression bars shall be taken into consideration, but the compression bar unloading coefficient should adopt 0.30 instead.

As for the connection of Cross Support end, Intensity reduction shall be counted in for single angle steel support; single-side eccentric connection must not be adopted for Intensity 8 and 9. In Cross Support, if one bar breaks crossed joint plate shall be strengthened, and their bearing capacity shall not be less than 1.1 times of that of member bars.

Stress ratio for supporting member bar section should not be larger than 0.75.

9.2.11 The bearing capacity for structural components of factory building shall be calculated according to the following requirements:

1 The splicing positions of frame upper columns shall select areas with smaller bending moment. Its bearing capacity shall not be less than the internal force at the splicing part calculated according to the bulk section plastic yielding state at both sides of upper columns, and must not be less than 0.5 times of tensile yielding bearing capacity of column total section.

2 The splicing of rigid-connection frame roof transom, where it is located beyond the maximum stress zone of transom, should be designed according to the strength same with that of the spliced section.

3 Rigid connection between solid-web roof beam and column as well as splicing between beam end beam and beam shall be designed at elastic stage by adopting combined internal force of earthquake. The rigid connection of beam columns and the limit bend bearing capacity for beam and beam spicing shall be in accordance with the following requirements:

1) In general. it shall be checked with regard to the connection coefficient according to those specified in Article 8.2.8 of this code about the rigid connection of steel structure beam columns and the spicing of beams. Herein, where the maximum stress zone is located at the upper column, total plastic bend bearing capacity shall be adopted with the smaller value among solid web beam and upper column;

2) When the roof beams adopt plate width-to-thickness ratio at the elastic design stage of steel structures, beam-column rigid connection and beam-beam splicing shall be able to reliably transfer precautionary Intensity combination of seismic internal force or it shall be checked according to No.1 of this item.

Plastic deformation should not occur to the plate connecting the roof truss top chord and the column of rigid-connection frame under precautionary seismic.

4 The connection of column support and component shall not be less than 1.2 times of the plastic bearing capacity of supporting member bar.

(III) Details of Seismic Design

9.2.12 The roof support of factory building shall be in accordance with the following requirements:

1 The layout for supports without purline roof should be in accordance with those specified in Table 9.2.12-1.

2 The layout for supports with purline roof should be in accordance with those specified in

Table 9.2.12-2.

Table 9.2.12-1 Support System Layout of Support Roof without Purline

Support name			Intensity		
			6, 7	8	9
Roof truss support	Transverse support of top and bottom boom		When the span of roof truss is less than 18m, it shall be in accordance with the non-seismic design; when the span is not less than 18m, a support shall be set in the unit end bay of factory building	It shall set a support in the unit end bay and upper column support bay of factory building; a partial upper cord support shall be additionally set at both ends within the range of skylight opening; when the roof truss end is braced on the top chord of roof truss, its Transverse support of bottom chord shall be the same with non-seismic design	
	Full-length horizontal tie bar of upper chord		Same with non-seismic design	It shall be set on the ridge, at vertical support section of the skylight truss, at the joint of the Transverse support and at both ends of the roof truss	
	Full-length horizontal tie bar on the bottom chord			It shall be set on the joint of the roof truss vertical brace; when the roof truss is in rigid connection with the column, it shall be set on the inter-joint of the roof truss end according to the external slenderness ratio of control bottom chord plane no larger than 150	
	Vertical support	Span of roof truss is less than 30m		A support shall be set at the roof truss end in each bay at both ends of unit, and in each support bay of upper column	Same with Intensity 8, and it shall be set on the end of roof truss every other 42m
		Span of roof truss is more than or equal to 30m		It shall be set at the roof truss end in the end bay, at 1/3 span of the roof truss and in the support bay of upper column corresponding to the Transverse supports of top and bottom chord	Same with Intensity 8, and it shall be set on the end of roof truss every other 36m
Longitudinal skylight truss support	Transverse support of top chord		A support shall be set in each bay at both ends of skylight truss unit	It shall set a support in each unit end bay of skylight truss and in each column support bay	
	Vertical support	Mid-span	It shall be set where the span is not less than 12m and the number shall be the same at both sides	It shall be set where the span is not less than 9m and the number shall be the same at both sides	
		Both sides	It shall be set in the unit end bay of skylight truss and every other 36m	It shall be set in the unit end bay of skylight truss and every other 30m	It shall be set in the unit end bay of skylight truss and every other 24m

3 When the light roofs adopt the rigid frame systems of solid web roof beam and column rigid connection, roof horizontal support may be arranged on the top flange plane of roof beam. The lower flange of roof beam shall be equipped with knee-support lateral support. The other side of knee support can be connected with the roof purline. See the requirements in Table 9.2.12 for the arrangement of roof Transverse support and longitudinal skylight truss support.

Table 9.2.12-2 Layout of Support System with Purline Roof

Support name		Intensity		
		6, 7	8	9
Roof truss support	Transverse support of top chord	It shall set a support in each unit end bay of factory building and every other 60m	It shall set a support in each unit end bay of factory building and in each column support bay of upper column	Same with Intensity 8, it shall set a partial top chord Transverse support additionally at both ends within the range of skylight opening
	Transverse support of bottom chord	Same with the non-seismic design; when the roof truss end bearing is at the bottom chord of roof truss, it shall be the same with the Transverse support of upper chord		
	Mid-span vertical support	Same with non-seismic design		It shall additionally set a support at mid-span where the span of roof truss is greater than or equal to 30m
	Vertical support at both sides	It shall set a support in each Unit End bay of factory building and in each column support bay where the roof truss end height is greater than 900mm		
	Full-length horizontal tie bar of the bottom chord	Same with non-seismic design	It shall be set at both ends and vertical support of the roof truss; when the roof truss is in rigid connection with the column, it shall be set at the roof truss end inter-joint according to the external slenderness ratio of control bottom chord plane no larger than 150	
Longitudinal support of skylight truss	Transverse support of top chord	A support shall be set in each bay at both ends of skylight truss unit	A support shall be set in each bay at both ends of skylight truss unit and every other 54m	A support shall be set in each bay at both ends of skylight truss unit and every other 48m
	Vertical support at both sides	It shall set a support in each unit end bay of skylight truss and every other 42m	It shall set a support in each unit end bay of skylight truss and every other 36m	It shall set a support in each unit end bay of skylight truss and every other 24m

4 Layout for longitudinal horizontal support of roof shall also meet the following requirements:

1) When roof structure of bracket support the roof transom is adopted, longitudinal horizontal support shall be installed along the full unit length of factory building;

2) For the high-low span factory building, longitudinal horizontal support shall be installed at the end support section of low-span roof transom along the full length of roof;

3) When brackets are adopted to brace the roof transom between partial columns of longitudinal colonnade, longitudinal horizontal support of roof shall be installed along the bracket column and by at least expanding one column at both sides;

4) When the full-length longitudinal horizontal supports along the structural unit are installed, they shall form sealed horizontal support system with transverse horizontal supports. The longitudinal horizontal support space of multi-span factory building roofs should not exceed two spans and shall not exceed three spans. High span and low span should form relatively independent sealed support system according to their respective elevations.

5 The profile steels should be adopted for supporting member bars. When cross support is arranged, the slenderness ratio limit value of supporting member bar may adopt 350.

9.2.13 As for the slenderness ratio of factory building frame columns, it should not be larger than 150 when the axial force ratio is less than 0.2; it should not be larger than $120\sqrt{235/f_{ay}}$ when the axial force ratio is no less than 0.2.

9.2.14 The width-to-thickness ratio of frame column and beam plate piece in factory building shall meet the following requirements:

1 For the heavy-weight roof factory building, the plate width-to-thickness ratio limit may be adopted according to those specified in Article 8.3.2 of this code and it shall be respectively adopted with IV, III and II grades of seismic for Intensity 7, 8 and 9.

2 The width-to-thickness ratio limit of plate in plastic energy consuming zone may be determined based on its bearing capacity according to the performance objectives. The width-to-thickness ratio limits of plate outside the plastic energy consuming zone may be adopted with the width-to-thickness ratio limit of plate at elastic design stage according to the current "Code for Design of Steel Structures" GB 50017.

Note: Width-to-thickness ratio of web plate may be reduced by setting longitudinal ribbed stiffener.

9.2.15 Column support shall be in accordance with the following requirements:

1 As for each longitudinal colonnade of factory building unit, one lower column support shall be arranged in the middle of factory building unit. When Intensity 7 factory building unit length is larger than 120m (150m where adopting light protective materials), when Intensity 8 and 9 factory building units are larger than 90m (120m where adopting light enclosing materials), one column support shall be arranged in each 1/3 section of factory building unit; when column numbers are no larger than 5 and factory building length is less than 60m, lower column supports may also be arranged at both sides of factory building units. Upper column support shall be arranged at both sides of factory building units and between columns with lower column supports.

2 Column support should adopt X-support, may adopt V-support, Λ-support or other types if constrained. The included angle between the X-support diagonal and horizontal plane, as well as gusset plate thickness at the junction of support diagonals shall meet the requirements of Section 9.1 in this code.

3 The slenderness ratio limit of column supporting member bar shall be in accordance with those specified in the current national standard "Code for Design of Steel Structures" GB 50017.

4 Whole bar of profile steel should be adopted for column support; when the hot rolled profile steel exceeds the maximum length specification, lengthening by force such as splicing may be adopted.

5 If conditions permit, energy-dissipation support may be adopted.

9.2.16 The column root shall be able to reliably transfer bearing capacity of column shaft and should be embedded type, plug in type or encased column roots; when it is Intensity 6 and 7, exposed column root may also be adopted. The column root design shall be in accordance with the following requirements:

1 The embedment depth for solid-web steel column adopting with embedded type, plug in type column roots shall be determined by calculating and shall not be less than 2.5 times of the section height of steel column.

2 The embedment length of lattice type column adopting plug in type column root shall be determined by calculating; its minimum insertion depth shall not be less than 2.5 times of the single-limb section height (or external diameter) and shall not be less than 0.5 times of the total column width.

3 When encased column root is adopted, the reinforced concrete encased height of solid-web, H-section column should not be less than 2.5 times of the steel structure section height; the reinforced concrete encased height of box section column or round tube section column should not be less than 3.0 times of the section height of steel structure or the section diameter of round tube.

4 When exposed column root is adopted, the bearing capacity of column root should not be less than 1.2 times of the bearing capacity for column section plastic yielding. The anchor bolts of column root should not be used to bear horizontal shear of column bottom and the column bottom shear force shall be born by the friction force between steel base plate and foundation or by setting shear key and other measures. The anchor bolts of column root shall be reliably anchored.

9.3 Single-storey Factory Buildings with Brick Columns

(I) General Requirements

9.3.1 This Section is applicable to the following small and medium single-storey factory buildings supported by brick columns (pier) built by fired common bricks (clay brick and shale brick) and concrete common brick:

1 Single-span, equal-height multi-span overhead free crane.

2 Span is not greater than 15m and column top elevation is not greater than 6.6m.

9.3.2 Structural layout of factory building shall meet the following requirements and should be in accordance with the relevant requirements of Article 9.1.1 of this code:

1 Brick bearing gable wall shall be installed at both ends of factory building.

2 Seismic brick wall should be adopted for the vertical and horizontal internal partition wall which is connected to and with equal height with the column.

3 Seismic joint installation shall meet the following requirements:

1) Light roof factory building may not be installed with seismic joint;

2) Seismic joint should be installed between the factory building with reinforced concrete roof and the buildings or structures bay built close to factory building and the seismic joint width may be adopted with 50mm~70mm; double columns or double walls shall be installed at the seismic joint.

4 The skylight shall not be connected to the end bay of factory building unit and shall not be adopted with end brick wall to support.

Note: The light roof in this chapter refers to the roofs of wooden house, light steel roof truss, profiled steel sheet, corrugated iron and so on.

9.3.3 Structural system of factory building shall also be in accordance with the following requirements:

1 Factory building roof should be adopted light roof.

2 When it is for Intensity 6 and 7, crisscross section brick column without reinforcement may be adopted; when it is for Intensity 8, brick column without reinforcement shall not be adopted.

3 For the independent longitudinal brick colonnade of factory building, seismic wall with the same height with column may be installed between columns to bear the longitudinal earthquake action; for the independent brick column top not installed with seismic wall shall be installed with full length horizontal pressure bar.

4 Seismic wall should be adopted for longitudinal and transversal internal partition wall; light wall should be adopted for non-bearing cross wall, non-integral masonry and longitudinal partition wall not reaching the top; when non-light wall is adopted, additional seismic shear force of partition wall on column and connected joint of partition and roof truss (roof beam) shall be counted in. Measures shall be taken for independent longitudinal and transverse internal partition walls to make sure the stability outside the planes and the top shall be installed with capping beam of cast-in-place reinforced concrete.

(II) Essentials in Calculation

9.3.4 For the single-storey factory buildings with brick columns adopting details of seismic design according to those specified in this section, where any of the following conditions is met, it may not carry out seismic check on transverse or longitudinal sections.

1 For the single-span and equal-height multi-span factory building of brick columns with both ends of structural unit provided with gable walls, column top elevation not exceeding 4.5m for Intensity 7 (0.10g) Class I and II site, transverse and longitudinal seismic check may not be carried out.

2 For Intensity 7 (0.10*g*) at Class I or II site, the column top elevation no larger than 6.6m, lengthwise outer wall whose thickness is no less than 240mm and opening section area is no larger than 50% installed at both sides, single-span factory building, both gable wall has been installed, the seismic check in longitudinal directions may not be carried out.

9.3.5 The horizontal seismic calculation of factory building may be carried out with the following methods:

1 Light roof factory building may be calculated according to the plane bent frame.

2 The factory building assigned to reinforced concrete roof or wood roof with fully covered sheathings may be calculated according to plane bent frame and the work of space has been considered; the earthquake action effect shall be adjusted according to Appendix J of this code.

9.3.6 Longitudinal seismic calculation of factory building may be adopted with the following methods:

1 Factory building with reinforced concrete roof should be calculated with mode-decomposition response spectrum method.

2 Equal-height and multi-span factory building with brick columns of reinforced concrete roof may be calculated according to the modifying rigidity method specified in Appendix K of this code.

3 For single-span factory buildings and multi-span factory building with light roof that longitudinal walls arranged symmetrically, they may be calculated separately by colonnade fragmentation

9.3.7 The transverse and longitudinal seismic calculation of skylight truss protruding roof shall be in accordance with those specified in Article 9.1.9 and Article 9.1.10 of this code.

9.3.8 Seismic check of eccentric compression brick columns shall be in accordance with the following requirements:

1 For brick columns without reinforcement, the eccentricity of the seismic composite axial force design value should not exceed 0.9 times of the distance from the section center to the section edge in the direction where the axial force locates; and the seismic adjustment coefficient of bearing capacity may adopt 0.9.

2 The reinforcement of composite brick column shall be determined through calculation and the seismic adjustment coefficient of bearing capacity may adopt 0.85.

(III) Details of Seismic Design

9.3.9 The supports of light roofs such as steel roof truss, profiled steel sheet and corrugated iron may be installed according to those specified in Table 9.2.12-2 of this code. The Transverse supports of top and bottom chords shall be arranged in the second bay at both sides. The support arrangement of wood roofs should meet the requirements of Table 9.3.9. The support and roof truss or skylight truss shall adopt bolted connection. The side columns of wood skylight truss should adopt full-length wood splint or iron plate as well as adopt bolts to strengthen the connection between side column and roof truss top chord.

Table 9.3.9 Layout of Support in Wooden Roof

Support name		Intensity		
		6, 7	8	
		All kinds of roofs	Fully covered sheathing	Sparsely covered sheathing or without sheathing
Roof truss support	Transverse support of top chord	Same with non-seismic design		It shall set a support in the second bay at both ends of building unit and every other 20m where the span of roof truss is greater than 6m
	Transverse support of bottom chord	Same with non-seismic design		
	Vertical support of midspan	Same with non-seismic design		
Skylight truss support	Vertical support at both sides of skylight	Same with non-seismic design	It should not install skylight	
	Transverse support of top chord	Design		

9.3.10 The purline and the gable horizontal beam shall be reliably connected and the shelf length shall not be less than 120mm; when conditions permitting, the roof structure for purlines protruding the gable wall may be adopted.

9.3.11 Measures for reinforced concrete roof structures shall be in accordance with the relevant requirements of Section 9.1 of this code.

9.3.12 A cast-in-place closed ring-beam shall be installed along the external wall and the bearing internal wall of building at the elevation of the column; for Intensity 8, a ring-beam shall be also installed in each 3~4m along height of the wall. The ring beam section height shall not be less than 180mm and its reinforcement shall not be less than 4ø12. Foundation ring-beam shall also be installed, where the base is weak cohesive soil, liquefied soil, newly-filled soil or an extremely non-uniform soil layer. When the ring-beam also serves as the lintel or used to resist uneven-settlement, besides meeting the seismic requirements, its section dimension and reinforcements shall also be determined according to the calculation of actual state.

9.3.13 The gable wall shall be installed with horizontal beam of cast-in-place reinforced concrete along the roof and shall be anchored with roof components; the section and reinforcement of gable wall column should not be less than that of the bent frame column and the wall column shall be extended to the wall top and connected with horizontal beam or roof component.

9.3.14 The cushion block between roof truss (roof beam) and wall top ring beam or column top shall be connected with bolts or by welding; the thickness of column top cushion block shall not be less than 240mm and two layers of reinforcing meshes with diameter of not less than 8mm and space not greater than 100mm shall be configured; the ring beam on the wall top shall be block cast with cushion block on column top.

9.3.15 Brick column structure shall be in accordance with the following requirements:

1 Brick strength grade shall not be less than MU10; mortar strength grade shall not be less than M5 and concrete strength grade in composite brick column shall not be less than C20.

2 Damp-proof course of brick column shall adopt waterproof mortar.

9.3.16 Horizontal section areas of the brick column factory building with reinforced concrete roof and the gable wall opening should be less than or equal to 50% of the gross section areas; reinforced concrete construction column shall be installed at both sides of gable wall and transverse wall for Intensity 8; the section dimension of constructional column may adopt 240mm×240mm; the vertical steel reinforcement shall not be less than 4ø12; the stirrup may adopt ø6 and the space should be between 250mm~300mm.

9.3.17 Structure of brick masonry wall shall be in accordance with the following requirements:

1 For brick column factory buildings with reinforced concrete roof without purlines where it is for Intensity 8, 1ø8 vertical reinforcement should be embedded for brick enclosure wall top every other 1m along wall and inserted into top ring beam.

2 When it is for Intensity 7 and the wall top height is greater than 4.8m or for Intensity 8, external wall turning of constructional column shall not be installed and 2ø6 steel reinforcement shall be configured every 500mm along the wall height at the connection section of bearing internal transverse wall and external longitudinal wall and each side of steel reinforcement shall be inserted into the wall by not less than 1m.

3 Details of seismic design for parapet wall protruding roof shall meet the relevant requirements of Section 13.3 of this code.

10 Large-span Buildings

10.1 Single-storey Spacious Buildings

(I) General Requirements

10.1.1 This Section is applicable to the public building composed of spacious single-storey hall and attached buildings.

10.1.2 Seismic joint should not be installed between hall, ante-hall and stage, and also may not be installed between hall and attached buildings at both sides; however, connection shall be strengthened where no seismic joint is set.

10.1.3 Bearing structure of hall roof in single-storey spacious building shall not adopt brick columns in the following conditions:

1 Halls for Intensity 7 (0.15*g*), Intensity 8 and 9.

2 Cantilever platform is installed in hall.

3 Hall span is greater than 12m or the column top height is greater than 6 m for Intensity 7 (0.10*g*).

4 Hall span is greater than 15m or column top height is greater than 8m for Intensity 6.

10.1.4 For the bearing structure of hall roof in single-storey spacious building, the bearing structure of the hall roof in single-storey spacious buildings, besides complying with the requirements in Article 10.1.3, may also be installed with composite wall columns of reinforced concrete and brick at the supporting point of longitudinal wall roof truss in the hall, but brick wall columns without reinforcement shall not be adopted.

10.1.5 Transverse lateral rigidity shall be strengthened for antehall structural layout and steel reinforced concrete column shall be adopted for wall column at front door and for detached column in antehall.

10.1.6 The transverse wall at the joint between antehall and hall, hall and stage shall be strengthened for the lateral rigidity and a certain number of reinforced concrete seismic walls shall be installed.

10.1.7 The other requirements for hall are detailed in Chapter 9 of this code and the attached building shall meet the relevant requirements of this code.

(II) Essentials in Calculation

10.1.8 The seismic calculation of single-storey spacious building may be divided into several independent structures, such as antehall, stage, and hall and attached buildings and then shall be carried out according to the concerned stipulation of this code and the mutual influence shall be counted in.

10.1.9 Seismic calculation of single-storey spacious building may be adopted with base shear method and the seismic influence coefficient may adopt the maximum value.

10.1.10 The standard value of longitudinal horizontal earthquake action for hall may be calculated according to following formula:

$$F_{Ek}=\alpha_{max}G_{ep} \tag{10.1.10}$$

Where F_{Ek}——Longitudinal horizontal earthquake action standard value of longitudinal wall or colonnade at one side of hall;

G_{eq}——Representative value of equivalent gravity load. It includes the dead weight of half both hall roof and adjacent attached building roof and 50% of the snow load standard value as well as the conversion dead weight of longitudinal wall or colonnade at one side.

10.1.11 Transverse seismic calculation of hall should be in accordance with the following principles:

1 For the hall without attached building at both sides, a typical bay shall be selected to calculate for each hall with cantilever platform and without cantilever platform; when those specified in Chapter 9 of this code are met, space works may also be counted in.

2 When there is attached building at both sides, proper calculation methods shall be selected according to the structure types of attached buildings.

10.1.12 Check for strength outside the plane shall be carried out for tall gable wall where it is Intensity 8 and 9.

(III) Details of Seismic Design

10.1.13 Roof structure of hall shall be in accordance with those specified in Chapter 9 of this code.

10.1.14 Steel reinforced concrete column and composite brick column in hall shall be in accordance with the following requirements:

1 The top of composite brick-columns vertical reinforcements shall be anchored into the reinforced concrete ring-beam at the bottom of the roof truss. The longitudinal steel reinforcements of composite brick-column, besides to be determined by calculation, shall not be less than 4ø14 for Intensity 6 at Class III or IV site on each side and Intensity 7 (0.10*g*) at Class I or II site, or 4ø16 for Intensity 7 (0.10*g*) at Class III or IV site on each side.

2 Steel reinforced concrete column shall be designed according to frame column with seismic grade not less than grade 2 and its reinforcement quantity shall be determined by calculation.

10.1.15 Transverse wall on axial line between antehall and hall, hall and stage shall be in accordance with the following requirements:

1 Reinforced concrete frame column or constructional column shall be installed at both ends of transverse wall, at longitudinal beam supporting point and on both sides of large opening.

2 Transverse wall built between frame columns shall be partially designed into the reinforced concrete seismic wall with seismic grade not less than grade 2.

3 Column and beam of stage shall adopt reinforced concrete structure; upper bearing masonry wall of girder for stage shall be installed with upright columns of space no greater than

4m and ring beams of space not greater than 3m. The section dimension of upright column and ring beam as well as the tying of reinforcement and surrounding masonry shall meet the requirements of multi-storey masonry building.

4 Wall on stage beam shall adopt light partition wall for Intensity 9.

10.1.16 Cast-in-place ring beam shall be arranged on the elevation part of hall column (wall) top and a ring beam should be installed additionally about every other 3m along the wall height. When the trapezoid roof truss height is greater than 900mm, a ring beam shall be additionally installed at the top chord elevation part. The section height of ring beam should not be less than 180mm; the width should be the same with wall thickness; longitudinal steel reinforcement shall not be less than $4\phi12$ and stirrup space should not be larger than 200mm.

10.1.17 When no seismic joint is installed between hall and attached building at both sides of hall, sealed ring beam shall be installed at the identical elevation part and shall be pulled through at the connection part; steel tie bar meshes shall be installed in horizontal mortar joint every other 400mm along the wall height for the wall connection part and each side into the human wall should not be 1m.

10.1.18 Overhang cantilever platform shall be provided with reliable anchor and measures to prevent overturning.

10.1.19 Reinforced concrete horizontal beam shall be installed along the gable wall roof and the gable wall and roof components shall be anchored; the gable wall shall be installed with steel reinforced concrete column or composite column and its section and reinforcement should respectively not less than that of the bent frame column or the combination column of longitudinal wall and shall be extended to gable wall top and connected with horizontal beam.

10.1.20 The tall gable wall at connection part of hall and antehall for stage back wall shall use the operating platform or storey as the horizontal support.

10.2 Large-span Roof Buildings

(I) General Requirements

10.2.1 This section is applicable to large-span steel roof buildings composed of arch, plane truss, spatial truss, wire frame, reticulated shell, beam string structure, suspend-dome and other basic forms.

As for the seismic design of large-span steel roof buildings adopting non-conventional type and whose span is larger than 120m, structural unit length is larger than 300m or cantilevered length is larger than 40m, special study and argumentation shall be carried out and effective reinforcement measures shall be adopted.

10.2.2 The type and layout of roof and its support structure shall be in accordance with the following requirements:

1 Shall be able to effectively transfer roof earthquake action to the lower support structure.

2 Shall be provided with reasonable rigidity and bearing capacity distribution; arrangement of roofs and their supports should be even and symmetrical.

3 Two space force transmission systems with balanced rigidity along horizontal direction should firstly be adopted.

4 Weak positions, over-large internal force, concentrated deformation due to partial weakening or mutation should be avoided for structural layout. Measures to boost its seismic capacity shall be taken for the possible weakened positions.

5 Light roof systems should be adopted.

6 Lower support structures shall be arranged reasonably to avoid generating excessive earthquake twisting effect on roofs.

10.2.3 Structural layout of roof system shall respectively meet the following requirements:

1 Structural layout of unidirectional force transmission system shall meet the following requireements:

1) Reliable support shall be installed with reliable support among major structures (truss, arch and beam string structure) to guarantee effective transmission of horizontal earthquake action along the direction which is vertical to major structures;

2) When lower chord joint support is adopted for the truss support, longitudinal trusses shall be set among supports or other reliable measures shall be taken to avoid twisting outside the plane appear on the truss support.

2 Structural layout of space force transmission system shall meet the following requirements:

1) As for structures whose plane form is rectangular with three sides support one side opening, the opening side shall be strengthened and adequate rigidity shall be guaranteed;

2) Sealed horizontal support shall be set along peripheral supports for two-direction orthogonal spatial wire frames and bi-directional beam string structures;

3) Single-storey reticulated shell shall adopt rigid connection joints.

Note: Unidirectional force transmission system refers to the structure form of plane arch, unidirectional plane truss, unidirectional spatial truss, unidirectional beam string structure, etc.; space force transmission system refers to the structure forms of wire frame, reticulated shell, bidirectional spatial truss, bidirectional beam string structure and suspend-dome, etc.

10.2.4 When different structure forms are adopted with roof by zone, member bars and joints in border areas shall be strengthened; or seismic joints may be set and the joint width should not be less than 150mm.

10.2.5 Nonstructural components, such as enclosing system of roof, suspended ceiling and suspender shall be reliably connected with structure and their seismic measures shall be in accordance with the relevant requirements of Chapter 13 in this code.

(II) Essentials in Calculation

10.2.6 Earthquake action calculation may not be carried out for the following roof structures; however, it shall be in accordance with the relevant requirements of seismic precautionary measures in this section:

1 Horizontal and vertical earthquake action along truss shall be carried out for unidirectional plane truss and unidirectional spatial truss structures with rise-span ratio less than 1/5.

2 Grid structure may not be carried out with earthquake action calculation for Intensity 7.

10.2.7 Calculation model for seismic analysis on roof structure shall be in accordance with the following requirements:

1 Calculation model shall be reasonably determined and the connection assumption between roof and main supporting position shall correspond with structure.

2 Calculation model shall be counted in the cooperative action between roof structure and substructure.

3 Earthquake action on support component of unidirectional force transmission system should be calculated according to the entire model of roof structure.

4 The influence of geometrical rigidity should be counted in for the earthquake action calculation model of beam string structures and suspend-domes.

10.2.8 In cooperative analysis on roof steel structure and lower support structure, damping ratio shall meet the following requirements:

1 When the lower support structures are steel structures or where the roofs are directly braced on the ground, damping ratio may be 0.02.

2 When the lower support structure is concrete structure, the damping ratio may adopt 0.025~0.035.

10.2.9 Horizontal earthquake action of roof structure shall be in accordance with the following requirements:

1 As for the unidirectional force transmission system, horizontal earthquake action may be calculated respectively along the major structure direction and the direction vertical to structure.

2 At least two principal axis directions shall be selected to calculate the horizontal earthquake action for space force transmission system; for the roof structure with two or more principal axis or obviously non symmetrical quality and rigidity, the calculation direction of horizontal earthquake action shall be increased.

10.2.10 Generally, frequent earthquake action calculation of roof structure may be adopted with mode-decomposition response spectrum method; the structure with complex body or larger span may also be adopted with multi-directional earthquake response spectrum method or time-history analysis method to supplement the calculation. For the regular wire frame, plane truss and spatial truss structure with peripheral support or combination of peripheral support and multipoint support, the simplified calculation of vertical earthquake action may be carried out according to those specified in Article 5.3.2 of this code.

10.2.11 Earthquake action effect combination of roof structural components shall be in accordance with the following requirements:

1 As for the check calculation of unidirectional force transmission system and major structural components, the combination of horizontal earthquake effect along the direction of major structures and vertical earthquake effect may be adopted. As for the check calculation of support members among major structures, only horizontal earthquake effect vertical to the direction of major structure may be counted in.

2 As for general structure, combination of three-dimensional earthquake action effects may be carried out.

10.2.12 The combined deflection value of large-span roof structures under the gravity load representative value and frequent vertical earthquake action standard value should not exceed the limit in Table 10.2.12.

Table 10.2.12 Deflexion Limit of Large-span Roof Structure

Structural system	Roof structure (short span l_1)	Cantilever structure (cantilever span l_2)
Plane truss, spatial truss, wire frame and beam string structure	$l_1/250$	$l_2/125$
Arch and single-storey reticulated shell	$l_1/400$	—
Double-storey reticulated shell and suspend-dome	$l_1/300$	$l_2/150$

10.2.13 Check for strength of roof component section shall not only meet relevant requirements in Section 5.4 of this code but also shall be in accordance with the following requirements:

1 Seismic combination internal force design value of key member bars shall multiply by enhancement coefficient, it shall adopt 1.1, 1.15 and 1.2 respectively for Intensity 7, 8 and 9.

2 Earthquake action effect combination design values of key joints shall multiply by enhancement coefficient; it should adopt 1.15, 1.2 and 1.25 respectively for Intensity 7, 8 and 9.

3 Travel cable in pretension structure shall not sag under frequent earthquake action.

Note: For space force transmission system, key member bars refers to the member bars close to support: namely: the chord and web member within 2 zones (grids); the chord and web member within range of 1/10 span close to the support shall adopt the minimum scope of both; key member bars refer to chord and web member in the section directly adjacent to the support. Key joints refer to joints connected with key member bars.

(III) Details of Seismic Design

10.2.14 Slenderness ratio of roof steel member bar should be in accordance with those specified in Table 10.2.14:

Table 10.2.14 Slenderness Ratio Limit of Steel Member Bar

Member bar type	Tension	Compression	Bending	Tension-bending
General member bar	250	180	150	250
Key member bar	200	150 (120)	150 (120)	200

Note: 1 Values in parentheses are applied to Intensity 8 and 9;

2 The data in the table is not applicable travel cable and other flexible components.

10.2.15 Seismic structures of roof component joints shall be in accordance with the following requirements:

1 When joint plates are applied to connect member bars, the joint plate thickness should not be less than 1.2 times of the maximum wall thickness of connecting member bar.

2 When tubular joints are applied, member bars in the direction of lager internal force shall be thoroughly connected. The wall thickness of thoroughly-connected member bar shall not be less than the wall thickness of each member bar welded on it.

3 When welded spherical joints are adopted, the sphere wall thickness shall not be less than 1.3 times of the maximum wall thickness of connecting member bar.

4 Member bar should be intersected on the joint center.

10.2.16 Seismic structure of support shall be in accordance with the following requirements:

1 Adequate strength and rigidity shall be provided and the support shall not be broken under the loading action ahead of member bar and other joints as well as shall not produce non-negligible deformation. The structural form of support joint shall transmit force reliably, connectcd simply and shall be in accordance with the calculation assumption.

2 As for horizontally slide-able support, the roof sliding shall not exceed the supporting surface under rare earthquake, and limit measures shall be taken.

3 Supports only bearing vertical pressure under frequent earthquakes for Intensity 8 and 9 should be adopted with tension-compression structure.

10.2.17 When seismic isolation and seismic absorbing support are applied for roof structure, the performance parameter, durability and associated structures shall meet the relevant requirements of Chapter 12 in this code.

11 Earth, Wood and Stone Houses

11.1 General Requirements

11.1.1 Building and structural layout of earth, wood and stone houses shall be in accordance with the following requirements:

1 Plane layout of building shall avoid corners or protruding.

2 Layout of longitudinal and horizontal bearing walls should be uniformly symmetrical, line up in plane and continuous in vertical direction; width of wall between windows should be uniform on the identical axial line.

3 The stories of multi-storey houses shall not split-leveled; plate and single side cantilever stairway shall not be adopted.

4 Bearing components made of different materials shall not be adopted in the identical height.

5 Masonry shall not be built on the cantilever beam outside eave.

11.1.2 Tying measures shall be taken at the lower position of wood houses and roof building;

1 Vertical diagonal support shall be installed for both end bay roof truss and mid-separated room roof truss;

2 Longitudinal full-through horizontal tie bar shall be arranged at the eave height; wall cables shall be adopted to connect tie bars with transverse walls as well as wooden beam and bottom chord of roof truss; longitudinal horizontal tie bar end should be butted by wood splint and square timber, angle iron and other materials may be adopted for wall cable;

3 Wall cables shall be adopted to tie gable wall and pediment wall with wood roof truss, wood-frame or purline;

4 Wall crest of internal partition wall shall be tied with beam or bottom chord of roof truss.

11.1.3 Bearing length of wood building and roof components shall not be less than those specified in Table 11.1.3:

Table 11.1.3 Minimum Bearing Length of Wood Building and Roof Components (mm)

Component name	Wood roof truss and wood beam	Butted wood joist and wooden purline		Overlapped wood joist and wooden purline
Position	On wall	On roof truss	On wall	On roof truss and on wall
Bearing length and connecting type	240 (wooden cushion pads)	60 (wooden splint and bolt)	120 (wooden splint and bolt)	Fully overlapping

11.1.4 Bearing length of lintel at door and window opening shall not be less than 240mm for Intensity 6~8 and shall not be less than 360mm for Intensity 9.

11.1.5 When cold-spreading tile roof is adopted, tack hole should be arranged at both corners of under-tile arc edge and iron tail and rafter may be adopted to fasten; lime or cement mortar rolling methods should be adopted for bonding firmly the cover tile and under tile.

11.1.6 The out-roof height of chimney, parapet wall and other easily collapse-able components protruding roof of earth, wood and stone houses shall not be greater than 600mm for Intensity 6 and 7 and shall not be greater than 500mm for Intensity 8 (0.20*g*) as well as shall not be greater than 400mm for Intensity 8 (0.30*g*) and for Intensity 9. Tying measures shall be taken.

Note: Chimney height on sloping roof shall calculated from chimney foot.

11.1.7 Structured materials of earth, wood and stone houses shall be in accordance with the following requirements:

1 Wood members shall be made of those dry and rot-free woods with straight streak, fewer knots.

2 Unfired earth wall soil shall be the cohesive soil with little dirt.

3 Stone shall be solid and free from weathering, delamination and crack.

11.1.8 Construction of earth, wood and stone houses shall be in accordance with the following requirements:

1 180° hook shall be installed at the terminal of HPB300 steel reinforcement.

2 Exposed ironworks shall be carried out with rust prevention treatment.

11.2 Unfired Earth Houses

11.2.1 This Section is applicable to the house with unfired adobe, lime soil and rammed bearing wall as well as the cave-house and the earth arch house for Intensity 6 and 7 (0.10*g*).

Note: 1 Lime soil wall refers to the soil wall mixed with lime (or other binding materials) and adobe wall mixed with lime;

2 Soil cave dwelling refers to the cliff dwelling excavated in the non disturbed original soils.

11.2.2 Unfired earth house height and load-bearing transverse wall space shall be in accordance with the following requirements:

1 Unfired earth houses should be constructed into single storey; lime soil wall houses may be constructed into two stories but the total height shall not be greater than 6m.

2 Cornice height of single storey unfired earth house should not be larger than 2.5m.

3 Load-bearing transverse wall space of single-storey unfired earth house should not be larger than 3.2m.

4 Clear span of cave dwelling should not be larger than 2.5m.

11.2.3 Unfired earth house roof shall be in accordance with the following requirements:

1 Light roofing materials shall be adopted.

2 Purline roof should be adopted with double sloping roof or arc roof and wood washer shall be installed at the purline bearing part; End purline shall protrude the eave; purlines on internal wall shall be fully overlapped, butted with splint or connected with dovetail tenon and clasp nail.

3 Wire nail, clasp nail and steel wire shall be adopted to connect the components of wood roof.

4 Wood roof truss and wooden beam should be fully overlapped on the external wall and wood ring beam or wooden cushion pad shall be arranged at the bearing part; the wooden cushion pad length, width and thickness should not respectively be less than 500mm, 370mm and 60mm; mortar or clay stone cushion courses shall be laid under the wooden cushion pad.

11.2.4 Bearing wall of unfired earth house shall be in accordance with the following requirements:

1 Door and window opening width of bearing wall shall not be greater than 1.5m for Intensity 6 and 7.

2 Wood lintel should be adopted for door and window opening; when the lintel is composed of multiple wooden poles, wood plate, clasp nail, lead wire and other materials shall be adopted to connect all the wooden poles into integrity.

3 Internal and external walls shall be rammed or laid alternatively layer by layer simultaneously. At the intersection area of four outside wall corners and four inside wall coners, a layer of tying meshes knitted with bamboo reinforcement, batten, twigs of the chaste tree and so on shall be laid along a wall height of about every other 500mm; each edge extending into wall shall not be less than 1000mm or shall be extended to the opening of door and window; the mesh tying part shall be banded at the intersection part; or other measures to strengthen integrity shall be taken.

11.2.5 Every kind of unfired earth house base shall be tamped and shall be rubble stone, slab-stone, chiseled pebble or common brick foundations; the foundation wall shall be built by cement lime mortar or cement mortar. Dado moisture protection treatment should be carried out for external wall (wall foundation should be covered with damp-proof course).

11.2.6 Adobes should be shaped with cohesive soil wet method and should be blended with grass, reed and other tying materials; adobes shall be horizontally built and should be adopted with clay mortar or clay stone mortar.

11.2.7 Ring beams shall be arranged for each storey of lime soil wall houses which shall be pulled through on transverse wall; offset piers should be additionally built at both sides of pediment wall for internal longitudinal wall top.

11.2.8 Multi-span of earth arch houses shall be connected and each arch springing shall be supported on the solid cliff or on artificial earth wall; arch ring thickness should be 300mm~400mm and formwork masonry shall be taken instead of retroversion adheresion laying; door and window shall not be arranged on the external supporting wall and arch ring.

11.2.9 Earth cave dwelling shall keep clear of those areas with landslide and landslip; cliff for excavating cave dwelling shall be provided with compact and stable soil, gentle slope without obvious vertical joint; front wall of adobe or other materials should not be laid before cliff cave; storey-cave should not be excavated, otherwise adequate space shall be maintained and the up and down shall be not aligned.

11.3 Wood Houses

11.3.1 This Section is applicable to the buildings for column-and-tie wood-frame, wood column truss and wood wooden beam, etc. for Intensity 6~9.

11.3.2 Wood house shall not be supported by wood column, brick column or brick wall all together; end roof truss (wooden beam) shall be arranged for gable wall and shall not be adopted with purline roof.

11.3.3 Wood house height shall be in accordance with the following requirements:

1 Wood column truss and column-and-tie wood frame house should be less than or equal to two storied for Intensity 6~8; total height should be less than or equal to 6m; single storey should be built for Intensity 9 and the height shall not be greater than 3.3m.

2 Wood column and beam houses should be built into single storey and the height should be less than or equal to 3m.

11.3.4 The large-span spacious houses such as assembly hall, theater, granary, etc. should be adopted with three-span wood bent frame with four columns on ground.

11.3.5 Support layout of wood roof truss roof shall be in accordance with the relevant requirements of Article 9.3 in this code; however, the roof truss supports at both ends of houses should be installed in end bay.

11.3.6 Diagonal support shall be installed between wood column and roof truss (or beam) for wood column truss and wood column beam building; dwelling houses with many transverse partitions shall be installed with diagonal support in the non-seismic partition; wooden splint should be adopted for diagonal support and shall be led off to the top chord of roof truss.

11.3.7 Penetrating ties shall be installed on the upper and lower column ends of wood column and on the storey bottom along the transverse and longitudinal directions of column-and-tie wood frame house; 1~2 diagonal supports or bridging shall be installed between each longitudinal colonnade.

11.3.8 Component connection of wood houses shall be in accordance with the following requirements:

1 Concealed dovetail shall be inserted into the bottom chord of roof truss for column top and the column top shall be connected with U-irons; when it is for Intensity 8 or 9, column root shall be anchored with foundation with ironworks or by other measures. Depth of plinth embedded into ground shall not be less than 200mm.

2 Diagonal support and roof support structures shall be connected with main components with bolts; other wood components, except column-and-tie wood component, should be connected with bolts.

3 Full nailing shall be adopted at the overlapping part between rafter and purline in order to strengthen the roof integrity. Vertical diagonal support and other measures shall be installed along the building longitudinal direction above column cornice in order to strengthen the longitudinal stability.

11.3.9 Wood components shall be in accordance with the following requirements:

1 Top diameter of wood column should not be less than 150mm; simultaneously longitudinal and horizontal slotting at the identical height of column shall be avoided and the slotting areas on the identical section of column shall not be greater than 1/2 of the total section area.

2 Column shall be free from joints.

3 Penetrating ties shall be through all the columns of wood-frame.

11.3.10 Enclosure wall shall be in accordance with the following requirements:

1 Tying of enclosure wall and wood column shall be in accordance with the following requirements:

1) No. 8 steel wire shall be adopted to tie horizontal tying bar or tying mesh in wall with wood column;

2) Reinforcement brick ring beam, reinforcement mortar strip and wood column shall be tied with ø6 steel reinforcement or No. 8 steel wire.

2 Enclosure wall opening widths built by adobes shall meet the requirements of Section 11.2 in this code. Opening width of enclosure wall, transverse wall and internal longitudinal wall built by bricks should not be larger than 1.5m and the opening width on longitudinal wall should not be larger than 1.8 m or should be half of the bay dimension.

3 Enclosure wall built by adobes and bricks shall not completely envelop the wood column and shall be laid close to the external side of wood column.

11.4 Stone Houses

11.4.1 This Section is applicable to the building supported by dressed stone masonry (including with or without washer) built by mortar for Intensity 6~8.

11.4.2 Total height and stories of multi-storey stone masonry building shall not be greater than those specified in Table 11.4.2.

Table 11.4.2 Total Height (m) and Storey Limit for Multi-storey Stone Masonry Building

Wall type	Intensity					
	6		7		8	
	Height	Stories	Height	Stories	Height	Stories
Fine and half fine dressed stone masonry (without washer)	16	5	13	4	10	3
Roughly-squared stone and rubble masonry (with washer)	13	4	10	3	7	2

Note: 1 Calculation of total building height shall be in accordance with those in the note of Table 7.1.2 for this code.

2 Total height for building with fewer transverse walls shall be reduced by 3m and the storey numbers shall be reduced by one storey correspondingly.

11.4.3 The storey height for multi-storey stone masonry building should be less than or equal to 3m.

11.4.4 Seismic transverse wall space of multi-storey stone masonry building shall not be greater than those specified in Table 11.4.4.

Table 11.4.4 Seismic Transverse Wall Space of Multi-storey Stone Masonry Building (m)

Building and roof type	Intensity		
	6	7	8
Cast-in-place and fabricated integral reinforced concrete	10	10	7
Fabricated reinforced concrete	7	7	4

11.4.5 Multi-storey stone masonry building should be adopted with cast-in-place or fabricated integral reinforced concrete building and roof.

11.4.6 Check for strength of stone wall may refer to Section 7.2 of this code and the shear strength shall be determined according to the test data.

11.4.7 Reinforced concrete construction columns shall be installed at the four corners of external wall and staircase in multi-storey stone masonry building and at the joints of internal and external walls.

11.4.8 Horizontal section areas of seismic transverse wall opening shall not be greater than 1/3 of the total section areas.

11.4.9 Ring beams shall be installed for each storey of longitudinal and transverse wall and the section height shall not be less than 120mm; the width should be the same with wall thickness; the longitudinal steel reinforcement shall not be less than 4ø10 and the stirrup space should not be larger than 200mm.

11.4.10 Strip stone masonry without washer shall be adopted at the joints of longitudinal and transverse walls with no constructional column; steel tie bar meshes shall be installed along wall height every other 500mm and each side of each edge shall be extended into the wall for not less than 1m.

11.4.11 Slab-stones shall not be adopted as the bearing components.

11.4.12 Other relevant requirements for details of seismic design shall be in accordance with the relevant requirements of Chapter 7 in this code.

12 Seismically Isolated and Energy-Dissipated Buildings

12.1 General Requirements

12.1.1 This chapter is applicable to the seismic isolation design for building which sets seismically isolated layer to isolate horizontal ground shock as well as the energy-dissipation design for building which sets energy-dissipation components to absorb and consume seismic energy.

Building structure of seismically isolated and energy-dissipated buildings shall be in accordance with those specified in Article 3.8.1 of this code and its seismic protection objectives shall meet those specified in Article 3.8.2 of this code.

Note: 1 Seismic isolation design in this chapter is to install seismically isolated layer (with integral reset function) composed of components, such as rubber, seismic isolation support and damping device at the house foundation, bottom or between substructure and superstructure in order to prolong the natural vibration period of the whole structural system, reduce the horizontal earthquake action input on the superstructure and to reach the expected seismic protection requirements.

2 Energy-dissipation design is to install energy-dissipated devices, through whose relative deformation and relative speed, the additional damping is provided, in the building structure in order to consume the seismic energy input on the structure and reach the expected seismic protection requirements.

12.1.2 When the design scheme of seismic isolation and energy-dissipation designs for building structure is determined, in addition to meet those specified in Article 3.5.1 of this code, comparative analysis with the seismic design plan shall also be carried out.

12.1.3 When the building structure is adopted with seismic isolation design, it shall be in accordance with the following requirements:

1 The height-width ratio of structure should be less than 4 and shall not be greater than the specific requirements for the non-seismic isolation structure in associated standard and specification; the deformation behavior of this structure shall be close to the shear deformation and the maximum height shall meet the requirements for non-seismic isolation structure in this code; when the height-width ratio is greater than 4 or seismic isolation design is adopted for the structures of the relevant requirements for non-seismic isolation structure, special study shall be carried out.

2 Building site should be Class I, II and III and foundations with preferable stability shall be adopted.

3 The gross horizontal force generated by the horizontal load of wind load and other non-earthquake action should be less than or equal to 10% of the total structure gravity force.

4 Necessary vertical bearing capacity, lateral rigidity and damping shall be provided for seismically isolated layer; flexible connection or other effective measures shall be taken for the equipment piping and wiring through the seismically isolated layer in order to adapt to the horizontal drift of rarely earthquake of the seismically isolated layer.

12.1.4 Energy-dissipation design may be applied to the houses with structure of steel, reinforced concrete, steel-concrete.

The structure shall be provided with adequate additional damping by energy dissipating component and design requirements in corresponding chapter of this code shall also be complied with according to structure types.

12.1.5 When seismically isolated and energy-dissipated buildings are designed, the seismic isolation device and the energy dissipating component shall be in accordance with the following requirements:

1 Performance parameters for seismic isolation device and energy dissipating components shall be determined through test.

2 Installation positions for the seismic isolation device and energy dissipating component shall be convenient for check and replacement.

3 Performance requirements for seismic isolation device and energy dissipating component shall be indicated on the design document and testing shall be carried out according to the requirements before installation to make sure that the performance is in accordance with the requirements.

12.1.6 Seismic isolation design and energy dissipation design for building structure, in addition to complying with relevant special standard, shall also be carried out with performance-based design according to the requirements for seismic performance objectives.

12.2 Essentials in Design of Seismically Isolated Buildings

12.2.1 Seismic isolation design shall select proper seismically isolated layer composed of seismic isolation device and wind resistance device according to the anticipated vertical bearing capacity, horizontal seismic absorbing coefficient and drift control requirements.

Check and calculation about the vertical bearing capacity and horizontal drift under rarely earthquake for seismic isolation support shall be carried out.

Horizontal earthquake action of structure above the seismically isolated layer shall be determined according to the horizontal shock absorbing coefficient; its standard Horizontal earthquake action of structure above the seismically isolated layer shall be determined according to the horizontal shock absorbing coefficient; its standard value of vertical earthquake action shall not be less than 20%, 30% and 40% of the representative values of gross gravity loads above the seismically isolated layer respectively for Intensity 8 (0.20*g*), Intensity 8 (0.30*g*) and Intensity 9

12.2.2 Computational analysis on seismically isolated building design shall meet the following requirements:

1 Mass points composed of seismically isolated support and top beam slab shall be added in the calculation diagram of seismically isolation system; structure with deformation behavior of shear pattern may be adopted with shearing model (Figure 12.2.2); when the mass centre of structure above the seismically isolated layer and the seismically isolated layer rigidity center are not overlapped, twisting effect influence shall be counted in. Beam and slab structure on the top of seismically isolated layer shall be calculated and designed as part of its upper structure.

2 Typically, time-history analysis method should be adopted to calculate; response spectrum characteristic and quantity of input seismic waves shall be in accordance with those specified in Article 5.1.2 of this code and the calculated results should adopt envelope values; when it is located within 10km of earthquake fault, near-field influence coefficient shall be considered for input seismic

waves and it should adopt 1.5 within 5km, not less than 1.25 for those beyond 5km.

3 The masonry structure and the structures similar to the basic cycle may be simply calculated according to Appendix L of this code.

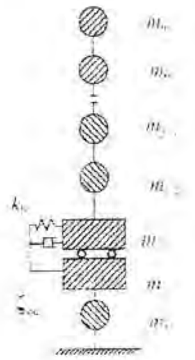


Figure 12.12.2 Calculation Diagram for Seismically Isolated Structure

12.2.3 Rubber seismically isolated support of seismically isolated layer shall be in accordance with the following requirements:

1 Limit horizontal drift of seismically isolated support under pressure stress detailed in Table 12.2.3 shall be greater than the relatively larger value among 0.55 times of its effective diameter and 3 times of the total rubber thickness inside the support.

2 The rigidity and damping characteristic variations of seismically isolated support shall not be greater than ±20% of the initial value after endurance test on corresponding design reference period; the creep shall not be greater than 5% of the total rubber thickness inside support.

3 Vertical pressure stress of rubber seismically isolated support under representative values of gravity load shall not be greater than those specified in Table 12.2.3.

Table 12.2.3 Limit of Pressure Stress for Rubber Seismically Isolated Support

Building category	Category A building	Category B buildings	Category C building
Pressure stress limit (MPa)	10	12	15

Note: 1 Design value of pressure stress shall be calculated according to the combination of permanent load and variable load; thereinto, the floor live load shall be multiplied by the reduction coefficient according to those specified in the current national standard "Load Code for the Design of Building Structures" GB 50009.

2 When the overturning check on structure is carried out, the effect combination of horizontal earthquake actions shall be included; for the structure requiring vertical earthquake action calculation, effect combination of vertical earthquake actions shall also be included.

3 When the second shape coefficient (ratio of effective diameter to total rubber layer thickness) of rubber support is less than 5.0, the limit of pressure stress shall be reduced; when it is less than 5 and greater than or equal to 4, it shall be reduced by 20%; when it is 4, less than 4 or greater than and equal to 3, it shall be reduced by 40%.

4 For the rubber support with external diameter less than 300mm, the pressure stress limit of Category C building shall be 10MPa.

12.2.4 Layout, vertical bearing capacity, lateral rigidity and damping of seismically isolated layer shall be in accordance with the following requirements:

1 Seismically isolated layer shall be installed at the bottom or under the structure; its rubber seismically isolated supports shall be arranged at the position where larger force is exerted and the space should not be oversized; its specification, quantity and distribution shall be determined through calculation according to the requirements of vertical bearing capacity, lateral rigidity and damping. Seismically isolated layer under rarely earthquake shall remain stable and should not have nonrenewable deformation; the tension stress of its rubber support under the action of both horizontal and vertical seismic of rarely earthquake shall not be greater than 1MPa.

2 Horizontal equivalent rigidity and equivalent viscous damping ratio of seismically isolated layer may be calculated according to the following formula:

$$K_h = \Sigma K_j \tag{12.2.4-1}$$

$$\zeta_{ep} = \Sigma K_j \zeta_j / K_h \tag{12.2.4-2}$$

Where ζ_{ep}——Equivalent viscous damping ratio of seismically isolated layer;

K_h——Horizontal equivalent rigidity of seismically isolated layer;

ζ_j——Equivalent viscous damping ratio of seismically isolated support j determined through test; when the damping device is installed, corresponding damping ratio shall be included;

K_j——Horizontal equivalent rigidity of seismically isolated support j (including energy-dissipated devices) determined through test.

3 When design parameters of seismically isolated support are determined through test, pressure stress limits of vertical load detailed in Table 12.2.3 shall be kept; for the calculation of horizontal shock absorbing coefficients, equivalent rigidity for 100% shear deformation and equivalent viscous damping ratio shall be adopted; for rarely earthquake check, equivalent rigidity for 250% shear deformation and equivalent viscous damping ratio should be adopted; when the diameter of seismically isolated support is larger, equivalent rigidity for 100% shear deformation and equivalent viscous damping ratio may be adopted. When time-history analysis is adopted, hysteretic curve acquired through test shall be regarded as reference.

12.2.5 Earthquake action calculation of structure above seismically isolated layer shall meet the following requirements:

1 For multilayer structure, the horizontal earthquake actions may be distributed along height according to the representative values of gravity load.

2 Horizontal seismic influence coefficient calculated by horizontal earthquake action after seismically isolation may be determined according to those specified in Article 5.1.4 and 5.1.5 of this code. Herein, the maximum value of horizontal seismic influence coefficient may be calculated according to following formula:

$$\alpha_{max1} = \beta \alpha_{max} / \psi \tag{12.2.5}$$

Where α_{max1}——Maximum value of horizontal seismic influence coefficient after seismic isolation;

α_{max}——Maximum value of horizontal seismic influence coefficient of non-seismically isolation, which may be adopted according to those specified in Article 5.1.4 of this code.

β——Horizontal seismic absorbing coefficient; for multistoried building, it is the maximum shear force ratio of seismically isolation to non-seismic isolation in each layer acquired through elasticity calculation. For the tall building structure, the maximum over-turning moment ratio of seismically isolation and non-seismic isolation in each layer shall also be calculated then shall be compared with the maximum ratio of shear force between layers and finally the relatively larger value shall be adopted;

[illegible]——Adjustment coefficient; it shall adopt 0.80 for general rubber support; it shall adopt 0.85 for support shearing performance with a deviation being Category S-A; and it shall correspondingly reduce 0.05 where damper is equipped with seismically isolated device.

Note: 1 During elasticity calculation, simplified calculation and response spectrum analysis should be carried out according to the performance parameters where the horizontal shear strain of seismically isolated support is 100%; when time-history analysis method is adopted, it shall be calculated according to the design basic acceleration input of ground motion;

2 The shearing performance deviation of support shall be determined according to the current national standard "Rubber Bearing—Part 3: Elastomeric Seismic-protection Isolators for Buildings" GB 20688.3.

3 The total horizontal earthquake action of structure above seismically isolated layer shall be greater than or equal to that of the non-seismic isolation structure for Intensity 6 precaution and shall be seismically checked; the horizontal earthquake shear force in each storey shall also be in accordance with those specified for the minimum seismic shear force coefficients of precautionary Intensity in this area in Article 5.2.5 of this code.

4 When it is for Intensity 9 and Intensity 8 as well as the horizontal seismic absorbing coefficient is not greater than 0.3, vertical earthquake action of structure above seismically isolated layer shall be calculated. When the standard value of structure vertical earthquake action above seismically isolated layer is calculated, each storey may be regarded as the mass point and the distribution of standard values for vertical earthquake actions along the height shall be calculated according to Formula (5.3.1-2) of this code.

12.2.6 Horizontal shear of seismically isolated support shall be distributed according to the horizontal equivalent rigidity of each seismically isolated support based on the horizontal shear of seismically isolated layer under rare earthquake; when it is calculated according to torsion coupling, torsional rigidity of seismically isolated layer shall be counted in.

Horizontal drift of seismically isolated support corresponding to the horizontal shear of rare earthquake shall be in accordance with the following requirements:

$$u_i \leq [u_i] \qquad (12.2.6\text{-}1)$$

$$u_i = \eta_i u_c \qquad (12.2.6\text{-}2)$$

Where u_i——Horizontal drift of i^{th} seismically-isolated support where taking into account of the torsion under the rare earthquakes;

$[u_i]$——Horizontal drift limit of the i^{th} seismically isolated support; for the rubber seismically isolated support, it shall not exceed the smaller value among 0.55 times of the effective diameter of this support and 3.0 times of the total rubber thickness inside the support;

u_c——Horizontal drift of seismically isolated layer at mass centre part under rare earthquake or where the torsion is not taken into account;

η_i——Torsion influence coefficient of i^{th} seismically-isolated support, the ratio of the calculated drifts of i^{th} support with regard to and without regard to torsion shall be taken into consideration. When the centroid of the structure above the seismically-isolated layer and the rigidity center of the seismically-isolated layer are not eccentric in both main axial directions, the torsion influence coefficient of the side support shall not be less than 1.15.

12.2.7 Seismically isolated measures of seismic insulation structure shall meet the following requirements:

1 The following measure not to hamper the seismically isolated layer from large deformation under rare earthquake shall be taken for the seismic insulation structure:

1) Vertical isolating joint shall be installed at the periphery of superstructure and the joint width should not be less than 1.2 times of the longest horizontal drift of each seismically isolated support under rare earthquake and should not be less than 200mm. The joint width for two adjacent seismic isolation structures shall adopt the sum of the largest horizontal drift values and shall not be less than 400mm.

2) Full-through horizontal isolating joint shall be installed between superstructure and substructure and the joint height may adopt 20mm and shall be filled with flexible materials; when it is difficult to install horizontal isolating joint, reliable horizontal sliding cushion course shall be installed.

3) Porch, stairway, elevator, carriageway and other parts passing the seismically isolated layer shall be protected from possible crashing.

2 The relevant requirements for seismic measure of structure above seismically isolated layer under non-seismic isolation shall not be reduced where the horizontal seismic absorbing coefficient is greater than 0.40 (0.38 when damper is installed); when the horizontal seismic absorbing coefficient is not greater than 0.40 (0.38 when damper is installed), requirements of non-seismically isolated building specified in this code may be properly reduced; however the Intensity reduction shall not exceed Intensity 1 and the details of seismic design relevant to vertical earthquake action resistance shall not be reduced. Here, details of seismic design may be carried out according to Appendix L of this code for masonry structure.

Note: Seismic measures related to vertical earthquake action resistance refer to the requirements of axial compression ratio for wall and column; the relevant requirements of minimum size and ring beam for end wall of external wall.

12.2.8 Connection of seismically isolated layer with superstructure shall meet the following requirements:

1 Beam slab floor shall be installed on the top of seismically isolated layer and shall be in accordance with the following requirements:

1) Cast-in-place concrete beam and slab structure shall be adopted on the relevant positions of seismically isolated support and the cast-in-place plate thickness shall not be less than 160mm;

2) Rigidity and bearing capacity of beam and plate on the top of seismically isolated layer should be greater than those of general floor beam slab.

3) Diecutting and partial bearing of beam and column close to seismically isolated layer shall be calculated; stirrup shall be thickened and steel mesh reinforcement shall be configured as circumstances demand.

2 Connection structure of seismically isolated support and damping device shall be in accordance with the following requirements:

1) Seismically isolated support and damping device shall be installed at the positions convenient for maintenance staff;

2) The connecting piece between seismically isolated support and superstructure or substructure shall be able to transfer the maximum horizontal shear and bending moment of lower support under rare earthquake;

3) Reliable rust prevention measures shall be provided for exposed embedded parts. The anchored reinforcement of embedded parts shall be solidly connected with steel plate; anchor length of anchored reinforcement should be greater than 20 times of the anchored reinforcement diameter and shall not be less than 250mm.

12.2.9 Structure and foundation of structure below seismically isolated layer shall be in accordance with the following requirements:

1 Vertical force, horizontal force and force moment of seismically isolated support bottom of seismically isolated structure under rare earthquake shall be adopted to check.

2 Relevant component parts directly supporting structure above seismically isolated layer among structures (including base plate under basement and seismically isolated tower) below seismically isolated layer shall meet the requirements of built in rigidity ratio and precautionary seismic capacity after seismically isolation and the shear resistant bearing capacity shall be checked according to rare earthquake. Storey drift angle limit structure below seismically isolated layer and above ground under rare earthquake shall be in accordance with those specified in Table 12.2.9.

3 Seismic check and base treatment for seismically isolated building base foundation shall be carried out according to the seismic precautionary Intensity in this area and the anti-liquefaction measures for Category A and B buildings shall be determined by boosting a liquification grade till all the liquefaction settlements are cleared up.

Table 12.2.9 Elasto-plastic Storey Drift Limit of Structure Below Seismically Isolated Layer and Above Ground Under Rare Earthquake Action

Substructure type	$[\theta_p]$
Reinforced concrete frame structure and steel structure	1/100
Reinforced concrete frame-seismic wall	1/200
Reinforced concrete seismic wall	1/250

12.3 Essentials in Design of Energy-dissipated Buildings

12.3.1 When energy dissipation design is carried out, proper energy dissipation components shall be installed according to the anticipated seismic absorbing requirements under frequent earthquakes and the anticipated shift control requirements of structure under rare earthquakes. The energy dissipation components may be composed of energy-dissipated devices, diagonal support, wall, beam and other supporting components. Speed relevant type, drift relevant type or other types may be adopted for energy-dissipated devices.

Note: 1 Speed relevant type energy-dissipated devices refer to the viscous energy-dissipated devices and viscoelastic energy-dissipated devices, etc.;

2 Drift relevant type energy-dissipated devices refer to metal yielding energy-dissipated devices and friction energy-dissipated devices, etc..

12.3.2 Energy dissipation components may be installed respectively along the two principal axis of structure as circumstances demand. The energy dissipation components should be installed at the position of larger deformation and the quantity and distribution shall be determined reasonably through comprehensive analysis and it shall be helpful to boost the energy dissipation capacity for the entire structure and to form uniform and reasonable force system.

12.3.3 Computational analysis on energy dissipation design shall meet the following requirements:

1 When the major structure basically remains in elastic working stage, linear analysis method may be adopted to carry out simplified estimate; and base shear method, mode-decomposition response spectrum method and time-history analysis method shall be respectively adopted according to the deformation behaviors and heights of structure and those specified in Section 5.1 of this code. The seismic influence coefficient of energy dissipated structure may be adopted according to the total damping ratio of energy dissipated structure and those specified in Article 5.1.5 of this code.

Natural vibration period of energy dissipated structure shall be determined according to the total rigidity of energy dissipated structure and the total rigidity shall be the sum of structural rigidity and effective rigidity of energy dissipation component.

The total damping ratio of energy dissipated structure shall be the sum of structural damping ratio and effective damping ratio of structure added by energy dissipation component; the total damping ratio under frequent earthquakes and rare earthquakes shall be respectively calculated.

2 For the conditions of major structure in elasto-plastic stage, static nonlinear analysis method or nonlinear time-history analysis method shall be adopted according to the architectural features of major structure.

During nonlinear analysis, the restoring force model of energy dissipated structure shall include the restoring force model of structure and of energy dissipation component.

3 Elasto-plastic storey drift angle limit of energy dissipated structure shall be in accordance with the anticipated deformation control requirements and should be properly reduced compared with the non-energy dissipated structure.

12.3.4 The effective damping ratio and the effective rigidity of structure added by energy dissipation components may be determined according to the following methods:

1 The effective rigidity of structure added by drift relevant type energy dissipation components and nonlinear speed relevant type energy dissipation components shall be determined by adopting equivalent linearization method.

2 The effective damping ratio of structure added by energy dissipation components may be estimated according to the following Formula:

$$\xi_a = \sum_j W_{cj} / (4\pi W_s) \tag{12.3.4-1}$$

Where ξ_a——Additional effective damping ratio of energy dissipated structure;

W_{cj}——Energy consumed by j^{th} energy-dissipated components recirculating one cycle under drift in the structural expected layer;

W_s——Total strain energy of structure installed with energy dissipation component under anticipated drift.

Note: When the energy dissipation components are uniformly distributed on the structure and where the effective damping ratio added to the structure is less than 20%, the effective damping ratio of structure added by energy dissipation components may also be determined according to the decoupling method by force.

3 When torsion influence is not taken into account, the total strain energy of energy dissipated structure under horizontal earthquake action may be estimated according to the following formula:

$$W_s = (1/2)\sum F_i u_i \tag{12.3.4-2}$$

Where F_i——Standard value of horizontal earthquake action for the i^{th} mass point;

u_i——Drift of i^{th} mass point corresponding to horizontal earthquake action standard value.

4 The energy dissipated by the speed-linear-relevant type energy-dissipated device under horizontal earthquake action may be estimated according to the following formula:

$$W_{cj} = (2\pi^2 / T_1)\ C_j \cos^2\theta_j \Delta u_j^2 \tag{12.3.4-3}$$

Where T_1——The foundational natural vibration period of the energy-dissipated structure;

C_j——The linear damping factor of the j^{th} energy-dissipated device;

θ_j——The included angle between the energy-dissipated direction and the plane for j^{th} energy-dissipated device;

Δu_j——The relative horizontal drift at the two ends of j^{th} energy-dissipated device.

When the damping coefficient and effective rigidity of energy-dissipated device is relevant to the structural vibration cycle, values corresponding to basic natural vibration period of energy dissipated structure may be adopted.

5 The wasted energy for drift relevant type and speed nonlinear relevant type energy-dissipated devices to cycle for one round under horizontal earthquake action may be estimated according to the following formula:

$$W_{cj} = A_j \tag{12.3.4-4}$$

Where A_j——The area of restoring-force characteristics of the j^{th} energy-dissipated device at the relative horizontal drift Δu_j.

The effective rigidity of energy-dissipated devices may be the secant rigidity of restoring-force characteristics of the j^{th} energy-dissipated device at the relative horizontal drift Δu_j.

6 When the effective damping ratio added to the structure by the energy-dissipated components exceeds 25%, it should be counted according to 25%.

12.3.5 Design parameters of energy-dissipated components shall meet the following requirements:

1 For energy-dissipated components consisted of the speed-related energy-dissipated device with supporting components such as the diagonal, wall or beam, the rigidity of the supporting components in the direction of energy-dissipated device may be calculated according to the following formula:

$$K_b \geq (6\pi / T_1) C_D \tag{12.3.5-1}$$

Where K_b——Rigidity of supporting component along energy-dissipated device direction;

C_D——Linear damping factor of the energy-dissipated device;

T_1——Foundational natural vibration period of the energy-dissipated structure

2 Total thickness of viscoelastic materials in viscoelastic energy-dissipated devices shall meet the following formula:

$$t \geq \Delta u / [\gamma] \tag{12.3.5-2}$$

Where t——Total thickness of viscoelastic materials in viscoelastic energy-dissipated devices;

Δu——Maximum possible drift along the direction of energy-dissipated devices;

$[\gamma]$——Maximum allowable shear strain of viscoelastic materials.

3 When energy-dissipated components are composed of drift-related energy-dissipated devices and supporting components such as diagonal support, wall or beam, resilience model parameter of energy-dissipated components should meet the following requirements:

$$\Delta u_{py} / \Delta u_{sy} \leq 2/3 \tag{12.3.5-3}$$

Where Δu_{py}——Horizontal yielding drift or the sliding-start drift of energy dissipation component;

Δu_{sy}——Yielding drift between structural layers of installed energy dissipation component.

4 The limit drift of energy-dissipated device shall not be less than 1.2 times of the maximum drift of energy-dissipated device under rare earthquakes; the limit speed of energy-dissipated device shall not be less than 1.2 times of the maximum speed of energy-dissipated device under earthquake action and the energy-dissipated devices shall meet the bearing capacity requirements at such limit speed.

12.3.6 Performance test on energy-dissipated devices shall meet the following requirements:

1 Sampling inspection shall be carried out for viscous fluid energy-dissipated devices by the third party; the quantity shall be 20% that of the same project, type and specification; however it shall not be less than 2 and the qualification rate for test shall be 100%; and the energy-dissipated devices through testing may be applied to the major structure; as for other type energy-dissipated devices, random inspection quantity shall be 3% of products of the same type and the same specification; when there are few products of the same type and the same specification, 3% of products of the same type and no less than 2 may be inspected, inspection qualification rate is 100%, inspected energy-dissipated devices shall not be used in major structures.

2 For the speed relevant type energy-dissipated device, main design index error and attenuation value of energy-dissipated device shall not be greater than 15% under the design drift design speed amplitude after 30 rounds of cycles in basic frequency of structure; For drift relevant type energy-dissipated devices, main design index error and attenuation value of energy-dissipated devices under design drift amplitude after 30 rounds of cycles shall not be greater than 15% and shall be free from obvious low-cycle fatigue phenomenon.

12.3.7 When energy dissipation design is adopted for structure, the relevant positions of energy dissipation component shall be in accordance with the following requirements:

1 The connection between energy-dissipated device and supporting component shall be in accordance with the structure requirements of relevant component connection in this code and relevant specifications.

2 The connection component between energy-dissipated device and main structure shall be operated within elastic range under the action of the maximum damping force exerted on main structure by energy-dissipated device.

3 When the structural components connected with the energy dissipation components are designed, additional internal force transferred by the energy dissipation components shall be taken into account.

12.3.8 When the seismic performance of energy dissipated structure is obviously boosted, the seismic structure requirements of major structure may be properly reduced. The reduction degree may be determined according to the ratio of the seismic influence coefficient of energy dissipated structure to that of the energy dissipated device-free structure and the maximum reduction degree shall be controlled within 1 Degree.

13 Nonstructural Components

13.1 General Requirements

13.1.1 This chapter is mainly applicable to the connection between nonstructural components and building structure. The nonstructural components include the permanent building nonstructural components and auxiliary mechanical and electrical equipments supported on building structure.

Note: 1 Nonstructural components of building refer to the fixed components and parts (except the supporting skeleton system) in the building, mainly include non bearing wall, components attached to the floor and roof structure, decorating components and parts and large scale storage shelf fixed on the floor, etc..

2 Auxiliary mechanical and electrical equipments of building refer to the accessory machines offering service to the use function of modern architecture, the electrical components, parts and systems, mainly include elevator, illumination and emergency power, communication equipment, pipeline system, space heating and air conditioning system, smoke and fire monitoring and firefighting system as well as common antenna, etc..

13.1.2 Different seismic measures shall be taken for the nonstructural components according to the precautionary category of the building they belong to and the consequence of nonstructural seismic destructiveness as well as the influence scope on the whole building structure in order to reach the corresponding performance-based design objective.

Some methods to realize performance-based seismic design objective for the nonstructural components and auxiliary mechanical and electrical equipments of building may be implemented according to those specified in Section M .2 of Appendix M in this code.

13.1.3 When two nonstructural components with different seismic requirements are connected together, seismic design shall be carried out according to the higher requirements. Herein, when a nonstructural component connection is broken, the nonstructural component with higher requirements connected to it shall not be invalid.

13.2 Essentials in Calculation

13.2.1 When seismic calculation of building structure is carried out, nonstructural component influence shall be taken into account according to the following requirements:

1 When earthquake action is calculated, the gravity of the building component and the building auxiliary mechanical and electrical equipments supported on structural component shall be taken into account.

2 For flexibly connected building components, the rigidity may not be counted in; for the nonstructural components of rigid building embedded into the lateral-force-resisting component plane, its rigidity influence shall be counted in and cycle adjustment and other simplified methods may be adopted; typically, its seismic bearing capacity shall not be counted in; when special structure measures are provided, its seismic bearing capacity may also be counted in according to the relevant requirements.

3 For structural components supporting nonstructural components, earthquake action effect of nonstructural components shall be regarded as additional action and shall be in accordance with the anchoring requirements of connecting pieces.

13.2.2 The earthquake action calculation methods of nonstructural components shall be in accordance with the following requirements:

1 The earthquake force of each component and part shall be imposed on its gravity center and the horizontal seismic force shall be along any horizontal direction.

2 Typically, the earthquake action of nonstructural component generated by its own gravity may be calculated by adopting equivalent lateral force method; for the nonstructural components supported on different stories or at both sides of seismic joint, in addition to the earthquake action generated by its own gravity, the action effect generated by the relative drift between supporting points during seismic shall also be counted in simultaneously.

3 For the auxiliary facility of building (including bracket), where the system natural vibration period is greater than 0.1s and its gravity exceed 1% of the storey gravity where it belong to or the building auxiliary facility gravity exceeds 10% of the gravity of storey it belongs to, seismic design of overall structure model should be carried out or the floor spectrum method specified in Section M.3 in Appendix M of this code may applied to calculate. Herein, the equipment non-flexibly connected with the floor may directly be counted in the analysis on the entire structure accompanied with floor as a mass point to acquire the earthquake action on equipment.

13.2.3 When equivalent lateral force method is adopted, the standard value of horizontal earthquake action should be calculated according to the following formula:

$$F = \gamma\eta\zeta_1\zeta_2\alpha_{\max}G \tag{13.2.3}$$

Where F——Standard value of horizontal earthquake action imposed on the nonstructural component gravity center along the most unfavorable direction;

γ——Function coefficient of nonstructural component, which is determined according to relevant standards or implemented according to those specified in Section M.2 of Appendix M in this code;

η——Category coefficient of nonstructural component, which is determined according to relevant standards or implemented according to those specified in Section M.2 of Appendix M in this code;

ζ_1——State coefficient; it shall adopt 2.0 for prefabricated building component, cantilever component, any equipment with supporting point below mass centre and flexible system and for the other conditions, it may adopt 1.0;

ζ_2——Position coefficient, it should adopt 2.0 for the top of building, 1.0 for the bottom and shall be distributed along height linear; for the structure required to adopt time-history analysis method for supplemental calculation as specified by Chapter 5 of this code, it shall be adjusted according to its calculated results;

$\alpha_{\max}$——Maximum value of seismic influence coefficient; which may be adopted according to the requirements about frequent earthquakes specified in Article 5.1.4 of this code;

G——Gravity of nonstructural component, which shall include the gravity of personnel concerned, medium in vessel and pipeline and objects in cabinet during operation.

13.2.4 The internal force of nonstructural component caused by relative horizontal drift of supporting points may be calculated by multiplying the rigidity of this component along drift direction with the specified relative horizontal drift of supporting points.

The rigidity of nonstructural component along drift direction, rigid connection, hinge connection, elastic connection or slip connection and other simplified mechanical models shall be respectively adopted according to the actual connection state of its end.

The relative horizontal drift of adjacent stories may be adopted according to the limits specified in this code.

13.2.5 The fundamental combination of earthquake action effect (including the effect generated by its self gravity and the effect generated by the relative drift of support) and other load effects shall be calculated according to the relevant requirements of structural components in this code; the combination of earthquake action effect and wind load effect of curtain wall shall be calculated; the action effect generated by vessel temperature, working pressure when the equipment is operating shall also be counted in.

When seismic check of nonstructural component is carried out, the friction force shall not be regarded as the force to resist earthquake action; the seismic adjustment coefficient of bearing capacity may adopt 1.0.

13.3 Essential Measures for Architectural Members

13.3.1 Strengthening measures shall be taken for the positions installing embedded parts and anchoring elements for building nonstructural components, such as connection curtain wall, enclosure wall, partition, parapet wall, rain cover, trademark, billboard, ceiling bracket, large scale storage shelf, etc. in order to bear the earthquake action transmitted to the major structure by nonstructural components of building.

13.3.2 Materials, types and layout of non bearing wall shall be determined through comprehensive analysis according to the Intensity, the height of buildings, the building body type, deformation between structural layers, utilization of self lateral-force-resisting performance for wall and other factors and shall be in accordance with the following requirements:

1 Light wall materials should be adopted for non bearing wall in priority; when the masonry wall is adopted, measures shall be taken to reduce the adverse impact on major structure and tie bar, horizontal tie beam, ring beam, constructional column shall be installed to reliably tie with the major structure.

2 The layout of rigid non bearing wall shall avoid the mutation of rigidity and Intensity distribution for structure; when the enclosure walls are asymmetrically and uniformly laid out, adverse influence of quality and rigidity difference on the seismic resistance of major structure shall be taken into consideration.

3 The major wall structure shall be reliably tied and shall be able to adapt the storey drift of major structure in different directions; deformability to meet the storey dislocation shall be provided for Intensity 8 and 9; when it is connected with cantilever component, it shall also be able to meet the vertical deformation caused by joint rotation.

4 The connecting pieces of external wall board shall be provided with adequate tensibility and proper turning power and should meet the requirements of storey deformation of major structure under precautionary seismic.

5 The masonry parapet wall shall be anchored with major structure at the pedestrian access door and passage; the non-anchored parapet wall height at the non access door should be less than or equal to 0.5m for Intensity 6~8 and shall be anchored where it is for Intensity 9. Adequate width shall be reserved for parapet wall at the seismic joint and the free ends at both sides of joint shall be strengthened.

13.3.3 In the multi-storey masonry structure, the nonstructural components of building such as non bearing wall shall be in accordance with the following requirements:

1 The back built nonbearing partition shall be configured with 2ø6 steel tie bar every other 500mm~600mm along the wall height to tie with bearing wall or column; each edge extended into the human wall shall not be less than 500mm; the top of back built partition with length greater than 5m for Intensity 8 and Intensity 9 shall also be tied with the floor slab or the beam; reinforced concrete construction column should be installed at the limb end of independent wall and at the large door opening edge.

2 The wall shall not be weakened for gas duct, duct and refuse chute; when the wall is weakened, strengthening measures shall be taken for wall; chimney shaft attached to wall without vertical reinforcement and chimney protruding roof should not be adopted.

3 The non anchored reinforced concrete shall not be adopted to prefabricate cantilever eaves.

13.3.4 Masonry filler wall in reinforced concrete structure shall also be in accordance with the following requirements:

1 Plane and vertical layout of filler wall should be uniformly symmetrical and should avoid weak storey or short column.

2 Mortar strength grade of masonry shall not be less than M5; strength grade of solid block should be greater than or equal to MU2.5; strength grade of hollow block should be greater than or equal to MU 3.5 and wall top shall be closely combined with the frame beam.

3 2ø6 tie bar shall be installed on the filler wall every other 500mm~600mm along the total frame column height and the tie bar length extended into the wall should be full through along the full wall length for Intensity 6 and 7 and shall be full through for Intensity 8 and 9.

4 When wall length is greater than 5m, the wall top and the beam should be tied; when wall length is greater than 8m or is 2 times of the storey height, reinforced concrete construction column should be installed; when wall height exceeds 4m, reinforced concrete horizontal tie beam connected with column along the full length should be installed for the half height of wall.

5 The filler wall between staircases and pedestrian passage should also be strengthened with steel mesh mortar layer.

13.3.5 Enclosure wall and partition wall of single-storey factory buildings with R.C. columns shall also be in accordance with the following requirements:

1 The enclosure wall of factory building should be adopted with light wallboard or large wall board of reinforced concrete; the masonry enclosure wall shall be reliably tied with column in externally bonded type; light wall board or large wall board of reinforced concrete shall be adopted

where the external column space is 12m .

2 Rigid enclosure wall should be laid out uniformly and symmetrically along longitudinal direction and should not b externally bonded type on one side and embedment-laying type or open type on the other; and it should not adopt masonry wall on one side and light wall board on the other side.

3 The suspended wall of high-span sealed wall for non-equal-height factory building and at the joint of longitudinal and horizontal factory building should be adopted with light wallboard; when masonry is adopted for Intensity 6 and 7, it shall not be directly built on the low-span roof.

4 Cast-in-place reinforced concrete ring beams shall be installed at the following positions for the masonry enclosure wall:

1) A course shall be installed at the top chord and column top elevation of each trapezoid roof truss; however, when the roof truss end height is not greater than 900mm, they may be installed together;

2) A ring beam shall be additionally installed on the window top about every other 4m according to the principle of "up thick and low thin"; the vertical space of suspended wall and ring beam at the joint of high-low span wall and longitudinal wall span of non equal-height factory building shall not be greater than 3m;

3) Reinforced concrete horizontal beam shall be installed on the gable wall along roof and shall be connected with the ring beam at the top chord elevation part on roof truss end.

5 Ring beam structure shall meet the following requirements:

1) Ring beam should be close; section width of ring beam should be identical with wall thickness and the section height shall not be less than 180mm; the longitudinal steel reinforcement of ring beam, where it is for Intensity 6~8, shall not be less than 4ø12; for Intensity 9, it shall not be less than 4ø14;

2) Longitudinal steel reinforcement of column top ring beam at corner of factory building within range of end bay, where it is for Intensity 6~8, should be greater than or equal to 4ø14; for Intensity 9, should be greater than or equal to 4ø16; stirrup diameter within 1m at both sides of turning should not be less than ø8 and the space should not be larger than 100mm; not less than 3 horizontal diagonal bars with diameter same with that of the longitudinal steel reinforcement shall be additionally installed at the corner of ring beam;

3) Ring beam shall be solidly connected with the column or the roof truss and the horizontal beam of gable wall shall be tied with the roof slab; anchored steel reinforcement of top ring beam connected with column or roof truss should be greater than or equal to 4ø12 and the anchor length should be greater than or equal to 35 times of the reinforcement diameter; the tying of ring beam with roof truss or roof truss at seismic joint should be strengthened.

6 Wall beam should be adopted with cast-in-place; when prefabricated wall beam is adopted, beam bottom shall be solidly tied with brick wall top surface and shall be anchored and tied with column; adjacent wall beams at the corner of factory building shall be reliably connected.

7 Masonry partition should be disconnected or flexibly connected with column; measures to stabilize wall shall be taken and cast-in-place reinforced concrete capping beam shall be installed on the top of partition.

8 For the brick wall foundation, prefabricated foundation beam shall be adopted with cast-in-place joints for Intensity 8 at Class III and IV site; when strip foundation is additionally installed, continuous cast-in-place reinforced concrete ring beam shall be installed at the top elevation of column root and its reinforcement shall not be less than 4ø12.

9 Height of masonry parapet wall should not be larger than 1m and measures to prevent tilting during seismic shall be taken.

13.3.6 Enclosure wall of steel structure factory building shall be in accordance with the following requirements:

1 Enclosure wall of factory building shall adopt light sheets in priority and the pre-cast reinforced concrete wallboard should be flexibly connected with column; when it is for Intensity 9, light sheets should be adopted.

2 The masonry enclosure walls of single-storey factory buildings shall be built close to and tied with columns, and measures shall also be taken to make walls not obstruct longitudinal horizontal drift of factory building colonnade. For Intensity 8 and 9, embedded-laying type shall not be adopted.

13.3.7 Connecting piece of component and floor slab for every kind of ceiling shall be able to bear the dead weights of ceilings, suspended heavy things and relevant electromechanical facilities as well as the additional action of seismic; its anchored bearing capacity shall be larger than that of the connecting piece.

13.3.8 Cantilever rain cover or rain cover supported by Eh column on one end shall reliably connected with the major structure.

13.3.9 The seismic structure of glass curtain wall, prefabricated wall board, cantilever components attached to the building roof and large scale storage shelf shall be in accordance with those specified in relevant special standards.

13.4 Essential Measures for Supports of Mechanical and Electrical Components

13.4.1 For the connecting components and parts of elevator, illumination and emergency power system, smoke and fire monitoring and firefighting system, space heating and air conditioning system, communication system, common antenna attached to the building, their seismic measures shall be determined through comprehensive analysis according to precautionary Intensity, use function of building, house height, structure type, deformation behavior, position where auxiliary facility is located and running requirements.

13.4.2 Seismic precaution requirements may not be taken into account for the following brackets attached to the mechanical and electrical equipments:

1 Equipment with gravity not exceeding 1.8kN.

2 Fuel gas pipeline with internal diameter less than 25mm and electric attached piping with internal diameter less than 60mm.

3 Duct with rectangular section area less than $0.38m^2$ and circular diameter less than 0.70m.

4 Hanger rod suspending pipeline with hanger rod computational length not exceeding 300mm.

13.4.3 Auxiliary mechanical and electrical equipments of building shall not be installed at the position where secondary catastrophes, such as use function failure, may occur; for the equipments with vibration isolation devices, the influence of its intense vibration on the connecting piece shall be paid attention to and the equipment and building structure shall be prevented to occur resonance phenomenon.

Adequate rigidity and strength shall be provided for the bracket of building auxiliary mechanical and electrical equipment; it shall be reliably connected to and anchored with the building structure and shall make the equipment rapidly recover operation after confronting earthquake effects of precautionary Intensity.

13.4.4 Opening installation of pipeline, cable, vent pipe and equipment shall reduce weakening main bearing structure components; reinforcement measures shall be taken on the opening edge.

The connection of pipeline or equipment and building structure shall ensure certain relative dislocation..

13.4.5 Foundation pier or connecting piece of building auxiliary mechanical and electrical equipment shall be able to transfer the earthquake action born by the equipment to the building structure. The position used to fix embedded parts and anchoring elements of building auxiliary mechanical and electrical equipment in the building structure shall be adopted with strengthening measures in order to bear the earthquake action transferred to the major structure by mechanical and electrical equipment.

13.4.6 Elevated water tank in the building shall be reliably connected with the structural components where it is located; and the earthquake action effect caused by the water tank and the water weight it contains on the building structure shall be taken into account.

13.4.7 The auxiliary facilities requiring continuous operation under precautionary seismic should be installed on the position of building structure with less earthquake response; structural components at relevant positions shall be taken with corresponding strengthening measures.

14 Subterranean Buildings

14.1 General Requirements

14.1.1 This chapter is mainly applicable to independent subterranean buildings such as underground garage, underpass, underground substation and underground space complex, excluding subway and urban highway tunnel, etc..

14.1.2 Subterranean buildings should be built on compact, uniform and stable bases. When it is located on the unfavorable sections, such as weak soil, liquefied soil or fault fracture zone, its influence on structural seismic stability shall be analyzed and corresponding measures shall be taken.

14.1.3 Architectural layout of subterranean buildings shall strive to be simple, symmetrical, regular, flat and smooth; form and structure of transverse section should not mutate along vertical direction.

14.1.4 Structural system of subterranean building shall be determined according to the operation requirements, engineering geological conditions of site and construction method and shall be provided with favorable integrity to avoid the lateral rigidity and bearing capacity mutation of lateral-force-resisting structure.

The seismic grade of Category C reinforced concrete infrastructure shall not be less than Grade 4 for Intensity 6 and 7, should be greater than or equal to Grade 3 for Intensity 8 and 9. The seismic grade of Category B reinforced concrete infrastructure should be greater than or equal to Grade 3 for Intensity 6 and 7 and should be greater than or equal to Grade 2 for Intensity 8 and 9 .

14.1.5 For the subterranean buildings located in rocks, reasonable opening structure type shall be selected for the side slope and entrance slope at both sides of access door passage according to topography and geologic conditions and the seismic stability shall be boosted.

14.2 Essentials in Calculation

14.2.1 The following subterranean buildings taking seismic measures according to the requirements in this chapter may not be carried out with earthquake action calculation:

1 Category C subterranean building for Intensity 7, Class I and II site.

2 For Intensity 8 (0.20g) Class I and II site, regularly-shaped medium-small span Category C subterranean buildings with no larger than 2 stories.

14.2.2 The seismic calculation model of subterranean buildings shall be determined according to the actual conditions of structure and shall be in accordance with the following requirements:

1 The actual force conditions of surrounding retaining structure and internal components shall be accurately reflected; the internal structure separated from the surrounding retaining structure may be adopted with the same calculation model with the above-ground buildings.

2 For subterranean buildings with longer longitudinal symmetry axis as well as uniform and regular distribution of surrounding substrates, the structural analysis may select the plane strain analysis model and reaction drift method or accelerated speed method for equivalent horizontal earthquake accelerated speed and equivalent lateral force method to calculate.

3 For subterranean buildings with length-width ratio and height-width ratio all less than 3 and subterranean buildings beyond those specified in Item 2 of this Article, three-dimensional structure analysis calculation model should be adopted and soil layer-structure time-history analysis method to calculate.

14.2.3 Design parameters for seismic calculation of subterranean buildings shall be in accordance with the following requirements:

1 Earthquake action direction shall meet the following requirements:

1) For the infrastructure analyzed according to the plane strain model may only be carried out with transverse horizontal earthquake action calculation;

2) For irregular infrastructure, transverse and longitudinal horizontal earthquake actions should be simultaneously calculated for the structure;

3) As for infrastructure with complex body type, such as underground space complex, the vertical earthquake action shall also be counted in for Intensity 8 and 9

2 The values of earthquake action shall be reduced correspondingly along the underground depth: earthquake action in the rock bed may adopt half of that on the ground, earthquake action at different depth from the ground to the bed rock may be determined according to interpolation method; when the ground surface, soil layer interface and bedrock surface are relatively even, one-dimensional fluctuation method may be adopted to determine; when soil layer interface, bedrock surface or ground surface fluctuates obviously, two-dimensional or three-dimensional finite element method should be adopted to determine.

3 Representative value of structure gravity load shall adopt the sum of the standard value of structure and component deadweight as well as water and earth pressure and the combination value of each variable load.

4 When soil layer-structure time-history analysis method or equivalent horizontal earthquake accelerated speed method is adopted, dynamic characteristic parameters of soil and rock may be determined through test.

14.2.4 Seismic check of subterranean building, in addition to complying with the requirements in Chapter 5 of this code, shall also be in accordance with the following requirements:

1 Seismic check of section bearing capacity and component deformation shall be carried out under frequent earthquake action.

2 For irregular subterranean buildings, underground substation and underground space complex, seismic deformation check under rare earthquake action shall be carried out. The simplified method specified in Section 5.5 of this code may be adopted for calculation. Elasto-plastic storey drift rotation limit $[\theta_p]$ should be taken as 1/250.

3 Subterranean buildings in liquefied base shall check the anti-floating stability during liquification. For the frictional resistance of the liquefied soil layer to the underground continuous wall and uplift pile, its liquefied reduction coefficient should be determined according to the ratio of measured standard penetration blow count to the critical standard penetration blow count.

14.3 Details and Anti-Liquefaction Measures

14.3.1 Seismic structure of reinforced concrete subterranean building shall be in accordance with the following requirements:

1 Cast-in-place structure should be adopted. When partial prefabricated components are required to install, they shall be reliably connected with the surrounding components.

2 The minimum size of underground reinforced concrete frame structure component shall not be less than the requirements for that of the fellow ground structural components.

3 Minimum total reinforcement ratio of longitudinal steel reinforcement for axial column shall increase by 0.2%. The stirrup at the joint of axial column with beam or ceiling, of middle floor slab with base plate shall be thickened and its scope and structure shall be the same with that of the frame structure column on ground.

14.3.2 Ceiling, base plate and floor slab of subterranean building shall be in accordance with the following requirements:

1 Beam and slab structure should be adopted. When slab-column seismic wall structure is adopted, structural concealed beam shall be installed in the sheet strip on column; its structure requirements are identical with the corresponding component of fellow ground structure.

2 At least 50% of the negative moment reinforcement of composite wall, ceiling, base plate and each floor plate for underground diaphragm wall shall be anchored into the underground diaphragm wall and the anchorage length shall be determined through calculation according to the force; inner liner shall be anchored into steel reinforcement with positive moment and shall not be less than the specified anchor length.

3 The opening width for floor slab shall not be larger than 30% of the floor plate width; opening layout should make the structural quality and rigidity uniformly distributed to avoid partial mutation. Edge beam or concealed beam meeting the structure requirements shall be installed around the opening.

14.3.3 When there is liquefied soil layer around the subterranean building and on the base, it shall take the following measures:

1 Measures to eliminate or relieve liquification influence, such as grouting, strengthening and soil replacement, shall be taken for liquefied soil layer.

2 Liquefied floating of underground structure shall be checked, if necessary, corresponding anti-floating measures such as setting of uplift piles and configuration of weights shall be taken.

3 When liquefied soil thin interlayer exists, or underground diaphragm wall enclosure structure whose depth is larger than 20m encounters the liquefaction soil layer during construction, base anti-liquefaction treatment may be omitted, but the influence of soil pressure increase and frictional resistance decrease caused by soil layer liquefaction shall be taken into consideration for the check calculation of its bearing capacity and anti-floating stability.

14.3.4 When the subterranean buildings pass through the ancient stream channel whose bank slope may slide during earthquake or the soft soil zone where obvious uneven settlement may occur, measures such as replacement of soft soil or arrangement of pile foundation shall be taken.

14.3.5 For subterranean buildings located in rocks, the following seismic measures shall be taken:

1 When the gateway passage and the fault fracture zone section without grouting and strengthening treatment adopt composite support structures, the inner lining structure shall adopt reinforced concrete lining and shall not adopt plain concrete lining.

2 When separate lining is adopted, inner linear structure shall be installed with horizontal support at the intersection part with arch wall to prop against enclosing rock.

3 When drilling and blasting method is adopted to construct, it shall be backfilled compactly between the preliminary support and enclosing rock substrate. Grouting shall be implemented to strengthen during backfilling with dry masonry block stones.

Appendix A The Seismic Precautionary Intensity, Design Basic Acceleration of Ground Motion and Design Earthquake Groups of Main Cities in China

This Appendix only specifies seismic precautionary Intensity, design basic acceleration of ground motion and corresponding design earthquake groups which are applied to seismic design of buildings at the center of county-level cities and the higher-level ones in China's seismic precautionary region.

Note: Generally, the "Design Earthquake Group I, Group II and Group III" is referred to as "Group I, Group II and Group III" in this Appendix.

A.0.1 Capital Cities and Municipalities

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group I: Beijing (Dongcheng, Xicheng, Chongwen, Xuanwu, Chaoyang, Fengtai, Shijingshan, Haidian, Fangshan, Tongzhou, Shunyi, Daxing, Pinggu), Yanqing, Tianjin (Hangu) and Ninghe.

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group II: Beijing (Changping, Mentougou, Huairou), Miyun; Tianjin (Heping, Hedong, Hexi, Nankai, Hebei, Hongqiao, Tanggu, Dongli, Xiqing, Jinnan, Beichen, Wuqing, Baochi), Jixian and Jinghai.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Shanghai (Huangpu, Luwan, Xuhui, Changning, Jing'an, Putuo, Zhabei, Hongkou, Yangpu, Minhang, Baoshan, Jiading, Pudong, Songjiang, Qingpu, Nanhui, Fengxian);

Group II: Tianjin (Dagang).

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Shanghai (Jinshan), Chongming; Chongqing (Yuzhong, Dadukou, Jiangbei, Shapingba, Jiulongpo, Nan'an, Beibei, Wansheng, Shuangqiao, Yubei, Ba'nan, Wanzhou, Fuling, Qianjiang, Changshou, Jiangjin, Hechuan, Yongchuan, Nanchuan), Wushan, Fengjie, Yunyang, Zhongxian, Fengdu, Bishan, Tongliang, Dazu, Rongchang, Qijiang, Shizhu and Wuxi *.

Note: The superscript * refers to a city whose center is located on the boundary between this seismic precautionary region and the region with lower seismic precautionary Intensity; the same below.

A.0.2 Hebei Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group I: Tangshan (Lubei, Lunan, Guye, Kaiping, Fengrun, Fengnan), Sanhe, Dachang, Xianghe, Huailai and Zhuolu;

Group II: Langfang (Gubo angyang, Anci).

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group I: Handan (Congtai, Hanshan, Fuxing, Fengfeng Mining Area), Renqiu, Hejian, Dacheng, Luanxian, Yuxian, Cixian, Xuanhua County, Zhangjiakou (Xiahuayuan, Xuanhua District) and Ningjin *;

Group II: Zhuozhou, Gaobeidian, Laishui, Gu'an, Yongqing, Wen'an, Yutian, Qian'an, Lulong, Luannan, Tanghai, Laoting, Yangyuan, Handan County, Daming, Linzhang and Cheng'an.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Zhangjiakou (Qiaoxi, Qiaodong), Wanquan, Huai'an, Anping, Raoyang, Jinzhou, Shenzhou, Xinji, Zhaoxian, Longyao, Renxian, Nanhe, Xinhe, Suning and Baixiang;

Group II: Shijiazhuang (Chang'an, Qiaodong, Qiaoxi, Xinhua, Yuhua, Jingxing Mining Area), Baoding (Xinshi, Beishi, Nanshi),Cangzhou (Yunhe, Xinhua), Xingtai (Qiaodong, Qiaoxi), Hengshui, Bazhou, Xiongxian, Yixian, Cangxian, Zhangbei, Xinglong, Qianxi, Funing, Changli, Qingxian, Xianxian, Guangzong, Pingxiang, Jize, Quzhou, Feixiang, Guantao, Guangping, Gaoyi, Neiqiu, Xingtai County, Wu'an, Shexian, Chicheng, Dingxing, Rongcheng, Xushui, Anxin, Gaoyang, Boye, Lixian, Shenze, Weixian, Gaocheng, Luancheng, Wuqiang, Yizhou, Julu. Shahe. Lincheng, Botou, Yongnian, Chongli and Nangong *;

Group III: Qinhuangdao (Haigang, Beidaihe), Qingyuan, Zunhua, Anguo, Laiyuan, Chengde (Yingshouyingzi *).

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Weichang and Guyuan;

Group II: Zhengding, Shangyi, Wuji, Pingshan, Luquan, Jingxing County, Yuanshi, Nanpi, Wuqiao, Jingxian and Dongguang;

Group III: Chengde (Shuangqiao, Shuangluan), Qinhuangdao(Shanhaiguan), Chengde County, Longhua, Kuancheng, Qinglong, Fuping, Mancheng, Shunping, Tangxian, Wangdu, Quyang, Dingzhou, Xingtang, Zanhuang, Huanghua, Haixing, Mengcun, Yanshan, Fucheng, Gucheng, Qinghe, Xinle, Wuyi, Zaoqiang ,Weixian, Fengning, Luanping, Pingquan, Linxi, Lingshou and Qiu xian.

A.0.3 Shanxi Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group I: Taiyuan (Xinghualing, Xiaodian, Yingze, Jiancaoping, Wanbailin, Jinyuan), Jinzhong, Qingxu, Yangqu, Xinzhou, Dingxiang, Yuanping, Jiexiu, Lingshi, Fenxi, Daixian, Huozhou, Guxian, Hongdong, Linfen, Xiangfen, Fushan and Yongji;

Group II: Qixian, Pingyao and Taigu.

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group I: Datong (Urban Area, Mining Area, Southern Suburban District), Datong County, Huairen, Yingxian, Fanshi, Wutai, Guangling, Lingqiu, Ruicheng and Yicheng;

Group II: Shuozhou (Shuocheng District), Hunyuan, Shanyin, Gujiao, Jiaocheng, Wenshui, Fenyang, Xiaoyi, Quwo, Houma, Xinjiang, Jishan, Jiangxian, Hejin, Wanrong, Wenxi, Linyi, Xiaxian, Yuncheng, Pinglu, Qinquan * and Ningwu *.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Yanggao and Tianzhen;

Group II: Datong (Xinrong), Changzhi (Urban Area, Suburban District), Yangquan (Urban Area, Mining Area, Suburban District), Changzhi County, Zuoyun, Youyu, Shenchi, Shouyang, Xiyang, Anze, Pingding, Heshun, Xiangning, Yuanqu, Licheng, Lucheng and Huguan;

Group III: Pingshun, Yushe, Wuxiang, Loufan, Jiaokou, Xixian, Puxian, Jixian, Jingle, Lingchuan, Yuxian, Qinshui, Qinxian and Shuozhou (Pinglu).

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group III: Pianguan, Hequ, Baode, Xingxian, Linxian, Fangshan , Liulin, Wuzhai, Kelan, Lan xian. Zhongyang, Shilou, Yonghe, Daning, Jincheng, Lvliang, Zuoquan, Xiangyuan, Tunliu, Zhangzi, Gaoping, Yangcheng and Zezhou.

A.0.4 Inner Mongolia Autonomus Region

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.30*g*:

Group I: Tumd Right Banner and Dalad Banner *.

2 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group I: Huhhot (Xincheng, Huimin, Yuquan, Saihan), Baotou (Kundulun, Donghe, Qingshan, Jiuyuan), Wuhai (Haibowan, Hainan, Wuda), Tumd Left Banner, Hanggin Rear banner, Dengkou and Ningcheng;

Group II: Baotou (Shiguai) and Toqtoh *.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group I: Chifeng (Hongshan * and Yuanbaoshan District), Harqin Barnner, Bayannur, Wuyuan, Urad Front Banner and Liangcheng;

Group II: Guyang, Wuchuan and Horinger;

Group III: Alxa Left Banner.

4 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Chifeng (Songshan District), Chahar Right Front Banner, Kailu, Aohan Banner, Zhalantun, Tongliao *;

Group II: Qingshuihe, Ulanqab, Zhuozi, Fengzhen, Urad Rear Banner and Urad Middle Banner;

Group III: Ordos and Jungar Banner.

5 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Manzhouli, New Barag Right Banner, Morin Dawa Banner, Arun Banner, Jalaid Banner, Ongniod Banner, Shangdu, Uxin Banner, Horqin Left Middle Banner, Horqin Left Rear Banne, Naiman Banner, Hure Banner and Sonid Right Banner;

Group II: Xinghe and Chahar Right Rear Banner;

Group III: Darhan Muminggan United Banner, Alxa Right Banner, Otog Banner, Otog Front Banner, Baotou(Baiyun Mining Area), Ejin Horo Banner, Hanggin Banner, Siziwang Banner and Chahar Right Middle Banner.

A.0.5 Liaoning Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group I: Pulandian and Donggang.

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group I: Yingkou (Zhanqian, Xishi, Bayuquan, Laobian), Dandong (Zhenxing, Yuanbao, Zhen'an), Haicheng, Dashiqiao, Wafangdian, Gaizhou and Dalian (Jinzhou).

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Shenyang (Shenhe, Heping, Dadong, Huanggu, Tiexi, Sujiatun, Dongling, Shenbei, Yuhong), Anshan (Tiedong, Tiexi, Lishan, Qianshan), Chaoyang (Shuangta, Longcheng), Liaoyang (Baita, Wenshng, Hongwei, Gongchangling, Taizihe), Fushun (Xinfu, Dongzhou, Wanghua), Tieling (Yinzhou, Qinghe), Panjin (Xinglongtai, Shuangtaizi), Panshan, Chaoyang County, Liaoyang County, Tieling County, Beipiao, Jianping, Kaiyuan, Fushun County *, Dengta, Tai'an, Liaozhong and Dawa;

Group II: Dalian (Xigang, Zhongshan, Shahekou, Ganjingzi, Lvshun), Xiuyan and Lingyuan.

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Benxi (Pingshan, Xihu, Mingshan, Nanfen), Fuxin (Xihe, Haizhou, Xinqiu, Taiping, Qinghemen), Huludao (Longgang, Lianshan), Changtu, Xifeng, Faku, Zhangwu, Diaobingshan, Fuxin County, Kangping, Xinmin, Heishan, Beining, Yixian, Kuandian, Zhuanghe, Changhai and Fushun (Shuncheng);

Group II: Jinzhou (Taihe, Guta, Linghe), Linghai, Fengcheng and Harqin Left Wing;

Group III: Xingcheng, Suizhong, Jianchang and Huludao (Nanpiao).

A.0.6 Jilin Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*: Qian Gorlos and Songyuan.

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Da'an *.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Changchun (Nanguan, Chaoyang, Kuancheng, Erdao, Lvyuan, Shuangyang), Jilin (Chuanying, Longtan, Changyi, Fengman), Baicheng, Qian'an, Shulan, Jiutai and Yongji *.

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Siping (Tiexi, Tiedong), Liaoyuan (Longshan, Xi'an), Zhenlai, Taonan, Yanji, Wangqing, Tumen, Hunchun, Longjing, Helong, Antu, Jiaohe, Huadian, Lishu, Panshi, Dongfeng, Huinan, Meihekou, Dongliao, Yushu, Jingyu, Fusong, Changling, Dehui, Nong'an, Yitong, Gongzhuling, Fuyu and Tongyu *.

Note: All county-level (and above) seismic precautionary cities throughout the Province are divided into Group I of design earthquake groups.

A.0.7 Heilongjiang Province

1 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Suihua, Luobei and Tailai.

2 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Harbin (Songbei, Daoli, Nangang, Daowai, Xiangfang, Pingfang, Hulan, Acheng), Tsitsihar (Jianhua, Longsha, Tiefeng, Ang'angxi, Fularji, Nianzishan,Meilisi), Daqing (Sartu, Longfeng, Ranghulu, Datong, Honggang), Hegang (Xiangyang, Xingshan, Gongnong, Nanshan, Xing'an, Dongshan), Mutankiang (Dong'an, Aimin, Yangming, Xi'an), Jixi (Jiguan, Hengshan, Didao, Lishu, Chengzihe, Mashan), Kiamusze (Qianjin, Xiangyang, Dongfeng, Suburb District), Qitaihe (Taoshan, Xinxing, Qiezihe), Yichun (Yichun District, Wuma, Youhao), Jidong, Wangkui, Muling, Suifenhe, Dongning, Ning'an, Wudalianchi, Jiayin, Tangyuan, Hua'nan, Huachuan, Yilan, Boli, Tonghe, Fangzheng, Mulan, Bayan, Yanshou, Shangzhi, Binxian, Anda, Mingshui, Suiling, Qing'an, Lanxi, Zhaodong, Zhaozhou, Shuangcheng, Wuchang, Nahe, Bei'an, Gannan, Fuyu, Longjiang, Heihe, Zhaoyuan, Qinggang * and Hailin *.

Note: All county-level (and above) seismic precautionary cities throughout the Province are divided into Group I of design earthquake groups.

A.0.8 Jiangsu Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.30*g*:

Group I: Suqian, (Sucheng, Suyu *).

2 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group I: Xinyi, Pizhou and Suining.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group I: Yangzhou (Weiyang, Guangling, Hanjiang), Zhenjiang (Jingkou, Runzhou), Sihong and Jiangdu;

Group II: Donghai, Shuyang and Dafeng.

4 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Nanjing (Xuanwu, Baixia, Qinhuai, Jianye, Gulou, Xiaguan, Pukou, Liuhe, Qixia, Yuhuatai, Jiangning), Changzhou (Xinbei, Zhonglou, Tianning, Qishuyan,Wujin), Taizhou (Hailing, Gaogang), Jiangpu, Dongtai, Hai'an, Jiangyan, Rugao, Yangzhong, Yizheng, Xinghua, Gaoyou, Liuhe, Jurong, Danyang, Jintan, Zhenjiang (Dantu), Liyang, Lishui, Kunshan and Taicang;

Group II: Xuzhou (Longyun, Gonglou, Jiuli, Jiawang, Quanshan), Tongshan, Peixian, Huai'an (Qinghe, Qingpu, Huaiyin), Yancheng (Tinghu, Yandu), Siyang, Xuyi, Sheyang, Ganyu and Rudong;

Group III: Lianyungang (Xinpu, Lianyun, Haizhou) and Guanyun.

5 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Wuxi (Chong'an, Nanchang, Beitang, Binhu, Huishan), Suzhou (Jinchang, Canglang, Pingjiang, Huqiu, Wuzhong, Xiangcheng), Yixing, Changshu, Wujiang, and Taixing and Gaochun;

Group II: Nantong (Chongchuan, Gangzha), Haimen, Qidong, Tongzhou, Zhangjiagang, Jingjiang, Jiangyin, Wuxi (Xishan), Jianhu, Hongze and Fengxian;

Group III: Xiangshui, Binhai, Funing, Baoying, Jinhu, Guannan, Lianshui and Chuzhou.

A.0.9 Zhejiang Province

1 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Daishan, Shengsi, Zhoushan (Dinghai, Putuo) and Ningbo (Beicang, Zhenhai).

2 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Hangzhou (Gongshu, Shangcheng, Xiacheng, Jianggan, Xihu, Binjiang, Yuhang, Xiao shan), Ningbo(Haishu, Jiangdong, Jiangbei, Yinzhou), Huzhou(Wuxing, Nanxun), Jiaxing (Nanhu, Xiuzhou), Wenzhou (Lucheng, Longwan, Ouhai), Shaoxing, Shaoxing County, Changxing, Anji, Lin'an, Fenghua, Xiangshan, Deqing, Jiashan, Pinghu, Haiyan, Tongxiang, Haining, Shangyu, Cixi, Yuyao, Fuyang, Pingyang, Cangnan, Leqing, Yongjia, Taishun, Jingning, Yunhe and Dongtou;

Group II: Qingyuan, Rui'an.

A.0.10 Anhui Province

1 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group I: Wuhe, Sixian.

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Hefei (Shushan, Luyang, Yaohai, Baohe), Bengbu (Bengshan, Longzihu, Yuhui, Huaishan), Fuyang (Yingzhou, Yingdong, Yingquan), Huainan (Tianjia'an, Datong), Zongyang, Huaiyuan, Changfeng, Liu'an (Jin'an, Yu'an), Guzhen, Fengyang, Mingguang, Dingyuan, Feidong, Feixi, Shucheng, Lujiang, Tongcheng, Huoshan, Guoyang, Anqing (Daguan, Yingjiang, Yixiu) and Tongling County *;

Group II: Lingbi.

3 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Tongling (Tongguangshan, Shizishan, Suburb District), Huainan (Xiejiaji, Bagongshan, Panji), Wuhu (Jinghu, Gejiang, Sanjiang, Jiujiang), Ma'anshan (Huashan, Yushan, Jinjiazhuang), Wuhu County, Jieshou, Taihe, Linquan, Funan, Lixin, Fengtai, Shouxian, Yingshang, Huoqiu, Jinzhai, Hanshan, Hexian, Dangtu, Wuwei, Fanchang, Chizhou, Yuexi, Qianshan, Taihu, Huaining, Wangjiang, Dongzhi, Susong, Nanling, Xuancheng, Langxi, Guangde, Jingxian, Qingyang and Shitai;

Group II: Chuzhou (Langya, Nanqiao), Lai'an, Quanjiao, Dangshan, Xiaoxian, Mengcheng, Haozhou, Chaohu, Tianchang;

Group III: Suixi, Huaibei and Suzhou.

A.0.11 Fujian Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group II: Jinmen *.

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group I: Zhangzhou (Xiangcheng, Longwen), Dongshan, Zhaoan and Longhai;

Group II: Xiamen (Siming, Haicang, Huli, Jimei, Tong'an, Xiang'an), Jinjiang, Shishi, Changtai and Zhangpu;

Group III: Quanzhou (Fengze, Licheng, Luojiang and Quangang).

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group II: Fuzhou (Gulou, Taijiang, Cangshan, Jin'an), Hua'an, Nanjing, Pinghe and Yunxiao;

Group III: Putian (Chengxiang, Hanjiang, Licheng, Xiuyu), Changle, Fuqing, Pingtan, Hui'an, Nan'an, An'xi and Fuzhou (Mawei).

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05g:

Group I: Sanming (Meilie, Sanyuan), Pingnan, Xiapu, Fuding, Fu'an, Zherong, Shouning, Zhouning, Songxi, Ningde, Gutian, Luoyuan, Shaxian, Youxi, Minqing, Minhou, Nanping, Datian, Zhangping, Longyan, Taining, Ninghua, Changting, Wuping, Jianning, Jiangle, Mingxi, Qingliu, Liancheng, Shanghang, Yong'an and Jian'ou;

Group II: Zhenghe, Yongding;

Group III: Lianjiang, Yongtai, Dehua, Yongchun, Xianyou and Mazu.

A.0.12 Jiangxi Province

1 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10g:

Xunwu and Huichang.

2 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05g:

Nanchang (East Lake, West Lake, Qingyunpu, Wanli, Qingshanhu), Nanchang County, Jiujiang (Xunyang, Lushan), Jiujiang County, Jinxian, Yugan, Pengze, Hukou, Xingzi, Ruichang, De'an, Duchang, Wuning, Xiushui, Jing'an, Tonggu, Yifeng, Ningdu, Shicheng, Ruijin, Anyuan, Dingnan, Longnan, Quannan and Dayu.

Note: All county-level (and above) seismic precautionary cities throughout the Province are divided into Group I of design earthquake groups.

A.0.13 Shandong Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20g:

Group I: Tancheng, Linmu, Junan, Juxian, Yishui, Anqiu, Yanggu, and Linyi (Hedong).

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15g:

Group I: Linyi (Lanshan, Luozhuang), Qingzhou, Linju, Heze, Dongming, Liaocheng, Shenxian and Juancheng;

Group II: Weifang (Kuiwen, Weicheng, Hanting, Fangzi), Cangshan, Yinan, Changyi, Changle, Zhucheng, Wulian, Changdao, Penglai, Longkou, Zaozhuang (Tai'erzhuang), Zibo (Linzi *) and Shouguang *.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10g:

Group I: Yantai (Laishan, Zhifu, Muping), Weihai, Wendeng, Gaotang, Chiping, Dingtao and Chengwu;

Group II: Yantai (Fushan), Zaozhuang(Xuecheng, Shizhong, Yicheng, Shanting *), Zibo (Zhangdian, Zichuan, Zhoucun), Pingyuan, Dong'e, Pingyin, Liangshan, Yuncheng, Juye, Caoxian, Guangrao, Boxing, Gaoqing, Huantai, Mengyin, Feixian, Weishan, Yucheng, Guanxian, Shanxian *, Xiajin * and Laiwu (Laicheng *, Gangcheng);

Group III: Dongying (Dongying, Hekou), Rizhao (Donggang, Lanshan), Yiyuan, Zhaoyuan, Xintai, Qixia, Laizhou, Pingdu, Gaomi, Kenli, Zibo (Boshan), Binzhou * and Pingyi *.

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Rongcheng;

Group II: Dezhou, Ningyang, Qufu, Zoucheng, Yutai, Rushan and Yanzhou;

Group III: Ji'nan (Shizhong, Lixia, Huaiyin, Tianqiao, Licheng, Changqing), Qingdao (Shinan, Shibei, Sifang, Huangdao, Laoshan, Chengyang, Licang), Tai'an (Taishan, Daiyue), Ji'ning (Shizhong, Rencheng), Leling, Qingyun, Wudi, Yangxin, Ningjin, Zhanhua, Lijin, Wucheng, Huimin, Shanghe, Linyi, Jiyang, Qihe, Zhangqiu, Sihui, Laiyang, Haiyang, Jinxiang, Tengzhou, Laixi, Jimo Jiaonan, Jiaozhou, Dongping, Wenshang, Jiaxiang, Linqing, Feicheng, Lingxian and Zouping.

A.0.14 Henan Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group I: Xinxiang (Weibin, Hongqi, Fengquan, Muye), Xinxiang County, Anyang (Beiguan, Wenfeng, Yindou, Long'an), Anyang County, Qixian, Weihui, Huixian, Yuanyang, Yanjin, Huojia and Fanxian;

Group II: Hebi (Qibin, Shancheng *, Heshan *) and Tangyin.

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group I: Taiqian, Nanle, Shanxian and Wuzhi;

Group II: Zhengzhou (Zhongyuan, Erqi, Guancheng, Jinshui, Huiji), Puyang, Puyang County, Changhuan, Fengqiu, Xiuwu, Neihuang, Xunxian, Huaxian, Qingfeng, Lingbao, Sanmenxia, Jiaozuo (Macun *) and Linzhou *.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Nanyang (Wolong, Wancheng), Xinmi, Changge, Xuchang * and Xuchang County;

Group II: Zhengzhou (Shangjie), Xinzheng, Luoyang (Xigong, Laocheng, Chanhe, Jianxi, Jili, Luolong *), Jiaozuo (Jiefang, Shanyang, Zhongzhan), Kaifeng (Gulou, Longting, Shunhe, Yuwangtai, Jinming), Kaifeng County, Minquan, Lankao, Mengzhou, Mengjin, Gongyi, Yanshi, Qinyang, Bo'ai, Jiyuan, Xingyang, Wenxian, Zhongmou and Qixian *.

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Xinyang (Shihe, Pingqiao), Luohe(Yancheng,Yuanhui,Zhaoling), Pingdingshan (Xinhua, Weidong, Zhanhe, Shilong), Ruyang, Yuzhou, Baofeng, 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, Yexian * and Wuyang *;

Group II: Shangqiu (Liangyuan, Juyang), Yima, Xin'an, Xiangcheng, Jiaxian, Songxian, Yiyang, Yichuan, Dengfeng, Zhecheng, Weishi, Tongxu, Yucheng, Xiayi and Ningling;

Group III: Ruzhou, Suixian, Yongcheng, Lushi, Luoning and Mianchi.

A.0.15 Hubei Province

1 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10g:

Zhuxi, Zhushan and Fangxian.

2 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05g:

Wuhan (Jiang'an, Jianghan, Qiaokou, Hanyang, Wuchang, Qingshan, Hongshan, Dongxihu, Hannan, Caidian, Jiangxia, Huangpi, Xinzhou), Jingzhou (Shashi, Jingzhou), Jingmen (Dongbao, Duodao), Xiangfan (Xiangcheng, Fancheng, Xiangyang), Shiyan (Maojian, Zhangwan), Yichang (Xiling, Wujianggang, Dianjun, Xiaoting, Yiling), Huangshi (Xialu, Huangshigang, Xisaishan, Tieshan), Enshi, Xianning, Macheng, Tuanfeng, Luotian, Yingshan, Huanggang, E'zhou, Xishui, Qichun, Huangmei, Wuxue, Yunxi, Yunxian, Danjiangkou, Gucheng, Laohekou, Yicheng, Nanzhang, Baokang, Shennongjia, Zhongxiang, Shayang, Yuan'an, Xingshan, Badong, Zigui, Dangyang, Jianshi, Lichuan, Gong'an, Xuan'en, Xianfeng, Changyang, Jiayu, Dazhi, Yidu, Zhijiang, Songzi, Jiangling, Shishou, Jianli, Honghu, Xiaogan, Yingcheng, Yunmeng, Tianmen, Xiantao, Hong'an, Anlu, Qianjiang, Tongshan, Chibi, Chongyang, Tongcheng, Wufeng * and Jingshan *.

Note: All county-level (and above) seismic precautionary cities throughout the Province are divided into Group I of design earthquake groups.

A.0.16 Hunan Province

1 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15g:

Changde (Wuling, Dingcheng).

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10g:

Yueyang (Yueyang Tower, Junshan *), Yueyang County, Miluo, Xiangyin, Linli, Lixian, Jinshi, Taoyuan, Anxiang and Hanshou.

3 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05g:

Changsha (Yuelu, Furong, Tianxin, Kaifu, Yuhua), Changsha County, Yueyang (Yunxi), Yiyang (Heshan, Ziyang), Zhangjiajie (Yongding, Wulingyuan), Chenzhou (Beihu, Suxian), Shaoyang (Daxiang, Shuangqing, Beita), Shaoyang County, Luxi, Ruanling, Loudi, Yizhang, Zixing, Pingjiang, Ningxiang, Xinhua, Lengshuijiang, Lianyuan, Shuangfeng, Xinshao, Shaodong, Longhui, Shimen, Cili, Huarong, Nanxian, Linxiang, Ruanjiang, Taojiang, Wangcheng, Xupu, Huitong, Jingzhou, Shaoshan, Jianghua, Ningyuan, Daoxian, Linwu, Xiangxiang *, Anhua *, Zhongfang * and Hongjiang *.

Note: All county-level (and above) seismic precautionary cities throughout the Province are divided into Group I of design earthquake groups.

A.0.17 Guangdong Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Shantou (Jinping, Haojiang, Longhu, Chenghai), Chaoan, Nan'ao, Xuwen and Chaozhou *.

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Jieyang, Jiedong, Shantou (Chaoyang, Chaonan) and Raoping.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Guangzhou (Yuexiu, Liwan, Haizhu, Tianhe, Baiyun, Huangpu, Fanyu, Nansha, Luogang), Shenzhen (Futian, Luohu, Nanshan, Bao'an, Yantian), Zhanjiang (Chikan, Xiashan, Potou, Mazhang), Shanwei, Haifeng, Puning, Huilai, Yangjiang, Yangdong, Yangxi, Maoming (Maonan, Maogang), Huazhou, Lianjiang, Suixi, Wuchuan, Fengshun, Zhongshan, Zhuhai (Xiangzhou, Doumen, Jinwan) Dianbai, Leizhou, Foshan (Shunde, Nanhai, Chancheng *), Jiangmen (Pengjiang, Jianghai, Xinhui *) and Lufeng *.

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Shaoguan (Zhenjiang, Wujiang, Qujiang), Zhaoqing (Duanzhou, Dinghu), Guangzhou (Huadu) , Shenzhen (Yougang), Heyuan, Jiexi, Dongyuan, Meizhou, Dongguan, Qingyuan, Qingxin, Nanxiong, Renhua, Shixing, Ruyuan, Yingde, Fogang, Longmen, Longchuan, Pingyuan, Conghua, Meixian, Xingning, Wuhua, Zijin, Luhe, Zengcheng, Boluo, Huizhou (Huicheng,Huiyang), Huidong, Sihui, Yunfu, Yun'an, Gaoyao, Foshan, (Sanshui, Gaoming), Heshan, Fengkai, Yunan, Luoding, Xinyi, Xinxing, Kaiping, Enping, Taishan, Yangchun, Gaozhou, Wengyuan, Lianping, Heping, Jiaoling, Dapu and Xinfeng *.

Note: All county-level (and above) seismic precautionary cities throughout the Province are divided into Group I of design earthquake groups, except that Dabu belongs to Group II.

A.0.18 Guangxi Zhuang Autonomous Region

1 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Lingshan and Tiandong.

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Yulin, Xingye, Hengxian, Beiliu, Baise, Tianyang, Pingguo, Longan, Pubei, Bobai and Leye *.

3 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Nanning (Qingxiu, Xingning, Jiangnan, Xixiangtang, Liangqing, Yongning), Guilin (Xiangshan, Diecai, Xiufeng, Qixing, Yanshan), Liuzhou (Liubei, Chengzhong, Yufeng, Liunan), Wuzhou (Changzhou, Wanxiu, Dieshan), Qinzhou (Qinnan, Qinbei), Guigang (Gangbei, Gangnan), Fangchenggang (Gangkou, Fangcheng), Beihai(Haicheng, Yinhai), Xing'an, Lingchuan, Lingui, Yongfu, Luzhai, Tian'e, Donglan, Bama, Du'an, Dahua, Mashan, Rong'an, Xiangzhou, Wuxuan, Guiping, Pingnan, Shanglin, Binyang, Wuming, Daxin, Fusui, 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: All county-level (and above) seismic precautionary cities throughout the Autonomous Region are divided into Group I of design earthquake groups.

A.0.19 Hainan Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.30*g*:

Haikou (Longhua, Xiuying, Qiongshan, Meilan).

2 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Wenchang and Ding'an.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Chengmai.

4 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Lingao, Qionghai, Danzhou and Tunchang.

5 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Sanya, Wanning, Changjiang, Baisha, Baoting, Lingshui, Dongfang, Ledong, Wuzhishan, and Qiongzhong.

Note: All county-level (and above) seismic precautionary cities throughout the Province are divided into Group I of design earthquake groups, except that Tunchang and Qiongzhong belong to Group II.

A.0.20 Sichuan Province

1 The seismic precautionary Intensity is greater than or equal to Intensity 9 while the design basic acceleration of ground motion is greater than or equal to 0.40*g*:

Group II: Kangding and Xichang.

2 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0. 30*g*:

Group II: Mianning *.

3 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group I: Maoxian, Wenchuan and Baoxing;

Group II: Songpan, Pingwu, Beichuan (Before Wenchuan Earthquake), Dujiangyan, Daofu, Luding, Ganzi, Luhuo, Xide, Puge, Ningnan, Litang;

Group III: Jiuzhaigou, Shimian and Dechang.

4 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group II: atang, Dege, Mabian, Leibo, Tianquan, Lushan, Danba, Anxian, Qingchuan, Jiangyou, Mianzhu, Shifang, Pengzhou, Lixian and Jiange *;

Group III: Yingjing, Hanyuan, Zhaojue, Butuo, Ganluo, Yuexi, Yajiang, Jiulong, Muli, Yanyuan, Huidong and Xinlong.

5 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Zigong (Ziliujing, Da'an, Gongjing, Yantan);

Group II: Mianyang (Fucheng, Youxian), Guangyuan (Lizhou, Yuanba, Chaotian), Leshan (Shizhong, Shawan), Yibin, Yibin County, E'bian, Muchuan, Pingshan, Derong, Ya'an, Zhongjiang, Deyang, Luojiang, Emeishan Moutain and Maerkang;

Group III: Chengdu (Qingyang, Jinjiang, Jinniu, Wuhou, Chenghua, Longzequan, Qingbaijiang, Xindu, Wenjiang), Panzhihua (East District, West District and Renhe), Ruoergai, Seda, Rangtang, Shiqu, Baiyu, Yanbian, Miyi, Xiangcheng, Daocheng, Shuangliu, Leshan (Jinkouhe, Wutongqiao), Mingshan, Meigu, Jinyang, Xiaojin, Huili, Heishui, Jinchuan, Hongya, Jiajiang, Qionglai, Pujiang, Pengshan, Danling, Meishan, Qingshen, Pixian, Dayi, Chongzhou, Xinjin, Jintang and Guanghan.

6 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Luzhou (Jiangyang, Naxi, Longmatan), Neijiang (Shizhong, Dongxing), Xuanhan, Dazhou, Daxian, Dazhu, Linshui, Quxian, Guang'an, Huaying, Longchang, Fushun, Nanxi, Xingwen, Xuyong, Gulin, Zizhong, Tongjiang, Wanyuan, Bazhong, Langzhong, Yilong, Xichong, Nanbu, Shehong, Daying, Lezhi and Ziyang;

Group II: Nanjiang, Cangxi, Wangcang, Yanting, Santai, Janyang, Luxian, Jiang'an, Changning, Gaoxian, Gongxian, Renshou and Weiyuan;

Group III: Qianwei, Rongxian, Zitong, Junlian, Jingyan, A'ba and Hongyuan.

A.0.21 Guizhou Province

1 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Wangmo;

Group III: Weining.

2 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Guiyang (Wudang *, Baiyun *, Xiaohe, Nanming, Yunyan, Huaxi), Kaili, Bijie, Anshun, Duyun, Huangping, Fuquan, Guiding, Majiang, Qingzhen, Longli, Pingba, Nayong, Zhijin, Puding, Liuzhi, Zhenning, Huishui, Changshun, Guanling, Ziyun, Luodian, Xingren, Zhenfeng, An'long, Jinsha, Yinjiang, Chishui, Xishui and Sinan *;

Group II: Liupanshui, Shuicheng and Ceheng;

Group III: Hezhang, Pu'an, Qinglong, Xingyi and Panxian.

A.0.22 Yunnan Province

1 The seismic precautionary Intensity is greater than or equal to Intensity 9 while the design basic acceleration of ground motion is greater than or equal to 0.40*g*:

Group II: Xundian and Kunming (Dongchuan);

Group III: Lancang.

2 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.30*g*:

Group II: Jianchuan, Songming, Yiliang, Lijiang, Yulong, Heqing, Yongsheng, Luxi, Longling, Shiping and Jianshui;

Group III: Gengma, Shuangjiang, Cangyuan, Menghai, Ximeng and Menglian.

3 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group II: Shilin, Yuxi, Dali, Qiaojia, Jiangchuan, Huaning, E'shan, Tonghai, Eryuan, Binchuan, Midu, Xiangyun, Huize and Nanjian;

Group III: Kunming (Panlong, Wuhua, Guandu, Xishan), Pu'er (Fomer Simao City), Baoshan, Malong, Chenggong, Chengjiang, Jinning, Yimen, Yangbi, Weishan, Yunxian, Tengchong, Shidian, Ruili, Lianghe, Anning, Jinghong, Yongde, Zhenkang, Lincang, Fengqing * and Longchuan *.

4 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group II: Shangri-la, Lushui, Daguan, Yongshan and Xinping *;

Group III: Qujing, Mile, Luliang, Fumin, Luquan, Wuding, Lanping, Yunlong, Jinggu, Ning'er (former Pu'er), Zhanyi, Gejiu, Honghe, Yuanjiang, Lufeng, Shuangbai, Kaiyuan, Yingjiang, Yongping, Changning, Ninglang, Nanhua, Chuxiong, Mengla, Huaping and Jingdong *.

5 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group II: Yanjing, Suijiang, Deqin, Gongshan and Shuifu;

Group III: Zhaotong, Yiliang, Ludian, Fugong, Yongren, Dayao, Yuanmou, Yao'an, Mouding, Mojiang, Lvchun, Zhenyuan, Jiangcheng, Jinping, Fuyuan, Shizong, Luxi, Mengzi, Yuanyang, Weixi and Xuanwei.

6 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Weixin, Zhenxiong, Funing, Xichou, Malipo and Maguan;

Group II: Guangnan;

Group III: Qiubei, Yanshan, Pingbian, Hekou, Wenshan and Luoping.

A.0.23 Tibet Autonomous Region

1 The seismic precautionary Intensity is greater than or equal to Intensity 9 while the design basic acceleration of ground motion is greater than or equal to 0.40*g*:

Group III: Dangxiong and Motuo.

2 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.30*g*:

Group II: Shenzha;

Group III: Milin and Bomi.

3 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group II: Pulan, Nielamu and Saga;

Group III: Lhasa, Duilongdeqing, Nimu, Renbu, Nima, Luolong, Longzi, Cuona, Qusong, Naqu, Linzhi (Bayi Town) and Linzhou.

4 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group II: Zhada, Jilong, Lazi, Xietongmen, Yadong, Luozha and Angren;

Group III: Ritu, Jiangzi, Kangma, Bailang, Zhanang, Cuomei, Sangri, Jiazha, Bianba, Basu, Dingqing, Leiwuqi, Naidong, Qiongjie, Gongga, Langxian, Dazi, Nanmulin, Bange, Langkazi, Mozhugongka, Qushui, An'duo, Nierong, Rikaze * and Ge'er *.

5 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Gaize:

Group II: Cuoqin, Zhongba, Dingjie and Mangkang;

Group III: Changdu, Dingri, Sajia, Gangba, Baqing, Gongbujiangda, Suoxian, Biru, Jiali, Chaya, Zuogong, Chayu, Jiangda and Gongjue.

6 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group II: Geji.

A.0.24 Shaanxi Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group I: Xi'an (Weiyang, Lianhu, Xincheng, Beilin, Baqiao, Yanta, Yanliang*, Lintong), Weinan, Huaxian, Huayin, Tongguan and Dali;

Group III: Long xian.

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group I: Xianyang (Qindu, Weicheng), Xi'an (Chang'an), Gaoling, Xingping, Zhouzhi, Huhu and Lantian;

Group II: Baoji (Jintai, Weibin, Chencang), Xianyang (Yangling Special Region), Qianyang, Qishan, Fengxiang, Fufeng, Wugong, Meixian, Sanyuan, Fuping, Chengcheng, Pucheng, Jingyang, Liquan, Hancheng, Heyang and Lueyang;

Group III: Fengxian.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Ankang and Pingli;

Group II: Luonan, Qian County, Mian County, Ningqiang, Nanzheng and Hanzhong;

Group III: Baishui, Chunhua, Linyou, Yongshou, Shangluo (Shangzhou), Taibai, Liuba, Tongchuan (Yaozhou, Wangyi, Yintai *) and Zhashui *.

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Yan'an, Qingjian, Shenmu, Jiaxian, Mizhi, Suide, Ansai, Yanchuan, Yanchang, Zhidan, Ganquan, Shangnan, Ziyang, Zhenba, Zichang * and Zizhou *;

Group II: Wuqi, Fuxian, Xunyang, Baihe, Langao and Zhenping;

Group III: Dingbian, Fugu,Wubao, Luochuan, Huangling, Xunyi, Yangxian, Xixiang, Shiquan, Hanyin, Ningshan Chenggu ,Yichuan, Huanglong, Yijun, Changwu, Binxian, Foping, Zhen'an, Danfeng and Shanyang.

A.0.25 Gansu Province

1 The seismic precautionary Intensity is greater than or equal to Intensity 9 while the design basic acceleration of ground motion is greater than or equal to 0.40*g*:

Group II: Gulang.

2 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0. 30*g*:

Group II: Tianshui (Qinzhou, Maiji), Lixian and Xihe;

Group III: Baiyin (Pingchuan District).

3 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20g:

Group II: Dangchang, Subei, Longnan, Chengxian, Huixian, Kangxian and Wenxian;

Group III: Lanzhou (Chengguan, Qilihe, Xigu, Anning), Wuwei, Yongdeng, Tianzhu, Jingtai, Jingyuan, Longxi, Wushan, Qin'an, Qingshui, Gangu, Zhangxian, Huining, Jingning, Zhuanglang, Zhangjiachuan, Tongwei, Huating, Liangdang and Zhouqu.

4 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15g:

Group II: Kangle, Jiayuguan, Yumen, Jiuquan, Gaotai, Linze and Sunan;

Group III: Baiyin (Baiyin District), Lanzhou(Honggu District), Yongjing, Minxian, Dongxiang, Hezheng, Guanghe, Lintan, Zhuoni, Diebu, Linzhao, Weiyuan, Gaolan, Chongxin, Yuzhong, Dingxi, Jinchang, Akesai, Minle, Yongchang and Pingliang.

5 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10g:

Group II: Zhangye, Hezuo, Maqu and Jinta;

Group III: Dunhuang, Guazhou, Shandan, Linxia, Linxia County, Xiahe, Luqu, Jingchuan, Lingtai, Minqin, Zhenyuan, Huanxian and Jishishan.

6 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05g:

Group III: Huachi, Zhengning, Qingyang, Heshui, Ningxian and Xifeng.

A.0.26 Qinghai Province

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20g:

Group II: Maqin;

Group III: Maduo and Dari.

2 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15g:

Group II: Qilian;

Group III: Gande, Menyuan, Zhiduo and Yushu.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10g:

Group II: Wulan, Chengduo, Zaduo and Nangqian;

Group III: Xining (Middle, East, West, North), Tongren, Gonghe, Delingha, Haiyan, Huangyuan, Huangzhong, Ping'an, Minhe, Hualong, Guide, Jianzha, Xunhua, Ge'ermu, Guinan, Tongde, Henan, Qumacai, Jiuzhi, Banma, Tianjun, Gangcha, Datong, Huzhu, Ledu, Dulan and Xinghai.

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05g:

Group III: Zeku.

A.0.27 Ningxia Hui Autonomous Region

1 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.30g:

Group II: Haiyuan.

2 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20g:

Group I: Shizuishan (Dawukou, Huinong) and Pingluo;

Group II: Yinchuan (Xingqing, Jinfeng, Xixia), Wuzhong, Helan, Yongning, Qingtongxia, Jingyuan, Lingwu and Guyuan;

Group III: Xiji, Zhongning, Zhongwei, Tongxin and Longde.

3 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15g:

Group III: Pengyang.

4 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05g:

Group III: Yanchi.

A.0.28 Xinjiang Uygur Autonomous Region

1 The seismic precautionary Intensity is greater than or equal to Intensity 9 while the design basic acceleration of ground motion is greater than or equal to 0.40g:

Group III: Wuqia and Tashikuergan.

2 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.30g:

Group III: Atushi, Kashi and Shufu.

3 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20g:

Group I: Balikun;

Group II: Urumqi (Tianshan, Shayibake, Xinshi, Shuimogou, Toutunhe, Midong), Urumqi County, Wensu, Akesu, Keping, Zhaosu, Tekesi, Kuche, Qinghe, Fuyun and Wushi *;

Group III: Nileke, Xinyuan, Gongliu, Jinghe, Wusu, Kuitun, Shawan, Manasi, Shihezi, Kelamayi (Dushanzi), Shule, Jiashi, Aketao and Yingjisha.

4 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15g:

Group I: Mulei *;

Group II: Kuerle, Xinhe, Luntai, Hejing, Yanqi, Bohu, Bachu, Baicheng, Changji and Fukang *;

Group III: Yining, Yining County, Huocheng, Hutubi, Chabuchaer and Yuepuhu.

5 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Shanshan;

Group II: Urumqi (Dawanching), Turfan, Hetian, Hetian County, Jimusaer, Luopu, Qitai, Yiwu, Tuokexun, Heshuo, Weili, Moyu, Zeller and Hami *;

Group III: Wujiaqu, Kelamayi (Kelamayi District), Bole, Wenquan, Aheqi, Awati, Shaya, Tumushuke, Shache, Zepu, Zecheng, Maigaiti and Pishan.

6 The seismic precautionary Intensity is Intensity 6 while the design basic acceleration of ground motion is 0.05*g*:

Group I: Emin and Hebukesaier;

Group II: Yutian, Habahe, Tacheng, Fuhai and Kelamayi (Maerhe);

Group III: Aletai, Tuoli, Minfeng, Ruoqiang, Buerjin, Jimunai, Yumin, Kelamayi (Baijiantan), Qiemo and Alaer.

A.0.29 Hongkong and Macao Special Administrative Region and Taiwan Province

1 The seismic precautionary Intensity is greater than or equal to Intensity 9 while the design basic acceleration of ground motion is greater than or equal to 0.40*g*:

Group II: Taizhong;

Group III: MiaoLi, Yunlin, Jiayi and Hualian.

2 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.30*g*:

Group II: Tainan;

Group III: Taibei, Taoyuan, Jilong, Yilan, Taidong and Pingdong.

3 The seismic precautionary Intensity is Intensity 8 while the design basic acceleration of ground motion is 0.20*g*:

Group III: Gaoxiong and Penghu.

4 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.15*g*:

Group I: Hongkong.

5 The seismic precautionary Intensity is Intensity 7 while the design basic acceleration of ground motion is 0.10*g*:

Group I: Macao.

Appendix B Requirements for Seismic Design of High Strength Concrete Structures

B.0.1 The concrete strength grade for high strength concrete structures shall meet the requirements of Article 3.9.3 in this code; its seismic design shall not only meet the requirements for seismic design of ordinary concrete structures, but also meet the requirements of this Appendix.

B.0.2 Among the section shear force design limits of structural components, the item with the design value (f_c) of concrete axes compression strength shall be multiplied by the influence coefficient (β_c) of concrete strength. It shall take 1.0 and 0.8 where the concrete strength grade is C50 and C80 respectively; it shall take the interpolated value where the concrete strength grade is between C50 and C80.

During calculation on the compressive area height and check for bearing capacity of structural components, the item with the design value (f_c) of concrete axes compression strength in the Formula shall also be multiplied by the corresponding influence coefficient of concrete strength in accordance with the relevant requirements of the national standard "Code for Design of Concrete Structures" GB 50010.

B.0.3 The details of seismic design for high strength concrete frames shall meet the following requirements:

1 The reinforcement ratio of longitudinal tension reinforcement at beam end should be less than or equal to 3% (Grade HRB335 steel reinforcement) and 2.6% (Grade HRB400 steel reinforcement). The minimum diameter of stirrup in stirrup densified area at beam end shall be 2mm greater than that of ordinary concrete beam stirrup.

2 The axial compression ratio limit of column should be adopted according to the following requirements: The axial compression ratio limit of concrete column less than or equal to C60 may be the same as that of ordinary concrete column; the axial compression ratio limit of concrete column between C65~C70 should be 0.05 less than that of ordinary concrete column; the axial compression ratio limit of concrete column between C75~C80 should be 0.1 less than that of ordinary concrete column.

3 When the concrete strength grade is greater than C60, the minimum total reinforcement ratio of column longitudinal steel reinforcement shall be 0.1% greater than that of ordinary concrete column.

4 The minimum stirrup characteristic value at the densified area of column should be determined according to the following requirements: When the concrete strength grade is higher than C60, it should adopt compound stirrup, compound spiral stirrup or continuous compound rectangular spiral stirrup as stirrup.

1) When the axial compression ratio is less than or equal to 0.6, the minimum stirrup characteristic value at the densified area of column should be 0.02 greater than that of ordinary concrete column;

2) When the axial compression ratio is greater than to 0.6, the minimum stirrup characteristic value at the densified area of column should be 0.03 greater than that of ordinary concrete column.

B.0.4 When the concrete strength grade of seismic wall is higher than C60, it shall make special research and take strengthening measures.

Appendix C Requirements for Seismic Design of Prestressed Concrete Structures

C.0.1 This Appendix is applicable to the seismic design of pre-tensioning and post-tensioning bonded prestressed concrete structures under Intensity 6, 7 and 8; under Intensity 9, special research shall be carried out.

As for the seismic design of unbounded prestressed concrete structures, measures shall be taken to prevent the relaxation of effective pre-applied force outside plastic hinge zone of structural components under rarely earthquake action; and it shall also meet the special requirements.

C.0.2 As for the prestressed concrete structures in seismic design, measures shall be taken to ensure that they have good deformation and seismic energy dissipation ability so as to reach the basic requirements of ductile structures; it shall avoid the shear failure of components ahead of bending failure, the joint is damaged before connected components and that the anchoring of prestressed tendon is damaged before components.

C.0.3 During seismic design, the transfer beam of prestressed frames, gantry and transfer storey should be fabricated with bonded prestressed tendon. The tension members of bearing structures and the frames with seismic Grade 1 shall not be fabricated with unbonded prestressed tendon.

C.0.4 During seismic design, the seismic grade of prestressed concrete structures and the adjustment of corresponding seismic combination internal force shall meet the requirements for armored concrete structures in Chapter 6 of this code.

C.0.5 The concrete strength grade of prestressed concrete structures, the frames and the transfer components at transfer storey should be greater than or equal to C40. The prestressed concrete members of other anti-lateral force shall be greater than or equal to C30.

C.0.6 The seismic calculation of prestressed concrete structures shall not only meet the requirements of Chapter 5 in this code, but also meet the following requirements:

1 The damping ratio of prestressed concrete structures may be 0.03; and it may adopt the equivalent damping ratio which is converted from the proportion of total deformation energy of armored concrete structures and prestressed concrete structures in the whole structures.

2 During component section seismic check of prestressed concrete structures, prestressed action effect item shall be added in the fundamental combination of earthquake action effect in Article 5.4.1 of this code; generally, it subitem coefficient shall be 1.0. When the prestressed action effect is unfavorable to the bearing capacity of components, it shall be 1.2.

3 When the prestressed tendon crosses the joint core zone of frames, the section seismic check of joint core zone shall give consideration to the influence by the effective check width of effective pre-applied force and prestressed tunnel-weakening core zone.

C.0.7 The seismic construction of prestressed concrete structures shall not only meet the following requirements, but also meet the requirements of armored concrete structures in Chapter 6 of this code.

1 The prestressed concrete members of anti-lateral force shall be reinforced with mixed prestressed tendon and non-prestressed tendon; their proportion shall be controlled based on seismic grade according to the relevant requirements; the prestressed strength ratio should be less than or equal to 0.75.

2 As for the maximum reinforcement ratio of longitudinal tension steel reinforcement at prestressed concrete frame beam end as well as the non-prestressed steel reinforcement ratio of bottom surface and top surface, it shall meet the requirements of armored concrete frame beam after corresponding conversion of prestressed strength ratio.

3 Asymmetric reinforcement may be used for prestressed concrete frame column. The calculation of its axial compression ratio shall give consideration to the design value of axial pressure formed by total effective pre-applied force of prestressed tendon; and it shall meet the requirements of corresponding frame column in armored concrete structures. The stirrup should be densified throughout the total height.

4 In the slab-column aseismic wall structures, the section area of prestressed steel reinforcement shall give consideration to the slab-bottom-crossed continuous steel reinforcement within the range of column section.

C.0.8 The anchorage device of post-tensioning prestressed tendon should not be set up at the core zone of beam-column joint; the anchorage performance of prestressed tendon and anchorage assembly parts shall meet special requirements.

Appendix D Section Seismic Check for the Beam-column Joint Core Zone of Frames

D.1 Beam-column Joint of Ordinary Frames

D.1.1 The design value for assembly shear force at beam-column joint core zone of Grade 1, 2 and 3 frames shall be determined according to the following formula:

$$V_j = \frac{\eta_{jb}\sum M_b}{h_{b0} - \alpha'_s}\left(1 - \frac{h_{b0}-\alpha'_s}{H_c - h_b}\right) \tag{D.1.1-1}$$

The Grade 1 frame structures and Grade 1 frames of Intensity 9 may not be determined according to the above formula, but it shall meet the following formula:

$$V_j = \frac{1.15\sum M_{bua}}{h_{b0} - \alpha'_s}\left(1 - \frac{h_{b0}-\alpha'_s}{H_c - h_b}\right) \tag{D.1.1-2}$$

Where V_j——Design value of assembly shear force at beam-column joint core zone;

h_{b0}——Effective height of beam section, which may take the average value where the beam section heights on both sides of the joint are unequal;

α'_s——Distance between the force concurrence point of beam compressive reinforcement and the compressive edge;

H_c——Column calculated height, which may take the distance between inflection points on upper and lower column of joints;

h_b——Beam section height, which may take the average value where the beam section height on both sides of the joint are unequal;

η_{jb}——Strong joint coefficient; as for frame structures, taking 1.5 for Grade 1, 1.35 for Grade 2 and 1.2 for Grade 3; as for frames in other structures, taking 1.35 for Grade 1, 1.2 for Grade 2 and 1.1 for Grade 3;

ΣM_b——Sum of assembly bending moment design values at counterclockwise or clockwise direction of joint right-left beam end; the bending moment with smaller absolute value shall take zero where the joint right-left beam end of Grade 1 frames refers to hogging moment).

ΣM_{bua}——Sum of bending moment values corresponding to the reinforced normal section seismic bend bearing capacity at counterclockwise or clockwise direction of joint right-left beam end; it may be determined according to the reinforced area of steel reinforcement (counting compressed tendon) and the standard value of material strength.

D.1.2 The section effective check width at core zone shall be determined according to the following requirements:

1 As for the section effective check width at core zone, it may adopt the section width of lateral column where the beam section width at the check direction is greater than or equal to 1/2 section width of lateral column; it may adopt the smaller value of the following two values where the beam section width at the checki direction is less than 1/2 section width of lateral column.

$$b_j=b_b+0.5h_c \tag{D.1.2-1}$$

$$b_j=b_c \tag{D.1.2-2}$$

Where b_j——Section effective check width at joint core zone;

b_b——Beam section width;

h_c——Column section height at the check direction;

b_c——Column section width at the check direction.

2 When the center lines of beam and column misalign and the eccentricity is less than or equal to 1/4 of the column width, the section effective check width at core zone the smaller value of the above value and the value calculated from the following formula.

$$b_j=0.5(b_b+b_c)+0.25h_c-e \tag{D.1.2-3}$$

Where e——Eccentricity between center lines of beam and column.

D.1.3 Design value of assembly shear force at joint core zone shall meet the following requirements:

$$V_j \le \frac{1}{\gamma_{RE}}\left(0.30\eta_j f_c b_j h_j\right) \tag{D.1.3}$$

Where η_j——Confined influence factor of orthogonal beam.It may take 1.5 where the floor slab is cast-in-place, the center lines of beam and column superposes, the beam section width of four sides is greater than or equal to 1/2 sectional width of lateral column and the orthogonal beam height is greater than or equal to 3/4 of frame beam height; it should adopt 1.25 for Grade 1 of Intensity 9 and 1.0 for other conditions;

h_j——Section height at joint core zone, which may take column section height at the check direction;

γ_{RE}——Seismic adjustment coefficient of bearing capacity, which may take 0.85.

D.1.4 The section seismic shear bearing capacity at joint core zone shall be checked according to the following formula:

$$V_j \le \frac{1}{\gamma_{RE}}\left(0.1\eta_j f_t b_j h_j + 0.05\eta_j N\frac{b_j}{b_c} + f_{yv}A_{svj}\frac{h_{b0}-\alpha_s'}{s}\right) \tag{D.1.4-1}$$

Grade 1 of Intensity 9

$$V_j \le \frac{1}{\gamma_{RE}}\left(0.9\eta_j f_t b_j h_j + f_{yv}A_{svj}\frac{h_{b0}-\alpha_s'}{s}\right) \tag{D.1.4-2}$$

Where N——Smaller value of upper-column assembly axial pressure corresponding to design value of assembly shear force; it shall be less than or equal to 50% of product between the column section area and the design value of concrete axes compression strength; when the N refers to tension, taking N= 0;

f_{yv}——Design value of stirrup tensile strength;

f_t——Design value of concrete axes tensile strength;

A_{svj}——Gross section area of stirrup at the same section check direction within the range of effective check width at core zone;

s——Space between stirrup.

D.2 Beam-column Joint of Flat-beam Frames

D.2.1 When the beam width of flat-beam frames is greater than the column width, the beam-column joint shall meet the requirements of this segment.

D.2.2 As for the beam-column joint core zone of flat-beam frames, the shear bearing capacity shall be checked within and beyond the range of column width according to the section area ratio of beam longitudinal reinforcement within and beyond the column width range.

D.2.3 The check method at core zone shall not only meet the requirements of beam-column joint for ordinary frames, but also meet the following requirements:

1 When the shear force limit at core zone is checked according to Formula (D.1.3) of this code, the effective width of core zone may take the average value for the sum of beam width and column width;

2 The confined influence factor with beams on four sides may take 1.5 where checking the shear bearing capacity of core zone within the column width range; it should take 1.0 where checking the shear bearing capacity of core zone beyond the column width range;

3 When checking the shear bearing capacity of core zone, the value of axial force may be the same as that of ordinary beam-column joint at the core zone within the column width range; the favourable effect of axial force on shear bearing capacity may not be considered at the core zone beyond column width;

4 The upper-beam steel reinforcement inside anchored column should be greater than 60% of its gross section area.

D.3 Beam-column Joint of Column Frames

D.3.1 When the center lines of beam and column superpose, the design value of assembly shear force at beam-column joint core zone of column frames shall meet the following requirements:

$$V_j \leq \frac{1}{\gamma_{RE}}\left(0.30\eta_j f_c A_j\right) \tag{D.3.1}$$

Where η_j——Confined influence factor of orthogonal beam shall be determined according to Article D.1.3 of this code; the width of the column section shall be determined according to the column diameter;

A_j——Effective section area at joint core zone. When the beam width (b_b) is greater than or equal to half of column diameter (D), taking $A_j = 0.8D^2$; when the beam width (b_b) is less than half of column diameter (D) and is greater than or equal to $0.4D$, taking $A_j = 0.8D\,(b_b + D/2)$.

D.3.2 When the center lines of beam and column superpose, the section seismic shear bearing capacity at beam-column joint core zone of column frames shall be checked according to the following requirements:

$$V_j \le \frac{1}{\gamma_{RE}}\left(1.5\eta_j f_t A_j + 0.05\eta_j \frac{N}{D^2} A_j + 1.57 f_{yv} A_{sh} \frac{h_{b0} - \alpha'_s}{s} + f_{yv} A_{svj} \frac{h_{b0} - \alpha'_s}{s}\right) \quad \text{(D.3.2-1)}$$

Grade 1 under Intensity 9

$$V_j \le \frac{1}{\gamma_{RE}}\left(1.2\eta_j f_t A_j + 1.57 f_{yv} A_{sh} \frac{h_{b0} - \alpha'_s}{s} + f_{yv} A_{hvj} \frac{h_{b0} - \alpha'_s}{s}\right) \quad \text{(D.3.2-2)}$$

Where A_{sh}——Section area of single round stirrup;

A_{svj}——Gross section area of tie bar and non-round stirrup at the same section check direction;

D——Column section diameter;

N——Design value of axial force; it may be taken according to the requirements of ordinary beam-column joint.

Appendix E　Requirements for Seismic Design of the Transfer Storey Structures

E.1　Design Requirements for Frame-supported Floor Slab of Rectangular Plane Seismic Wall Structure

E.1.1　Cast-in-place floor slab shall be used for frame-supported storey; its thickness should be greater than or equal to 180mm and its concrete strength grade should be greater than or equal to C30. It shall adopt double-layer two-way reinforcement and the reinforcement ratio for each storey at each direction shall be greater than or equal to 0.25%.

E.1.2　The design value for shear force of frame-supported floor slab in partial frame-supported seismic wall structure shall meet the following requirements:

$$V_{\mathrm{f}} \leq \frac{1}{\gamma_{\mathrm{RE}}}(0.1 f_{\mathrm{c}} b_{\mathrm{f}} t_{\mathrm{f}}) \tag{E.1.2}$$

Where　V_{f}——Design value for assembly shear force of frame-supported floor slab, which is calculated according to rigid floor slab where the shear force is transmitted from non-ground seismic wall to ground seismic wall. Under Intensity 8, it shall be multiplied by enhancement coefficient 2; under Intensity 7, it shall be multiplied by enhancement coefficient 1.5; when checking ground seismic wall, this enhancement coefficient shall be given no consideration;

b_{f}, t_{f}——Width and thickness of frame-supported floor slab respectively;

γ_{RE}——Seismic adjustment coefficient of bearing capacity; it may take 0.85.

E.1.3　As for the conjoining section between frame-supported floor slab and ground seismic wall of partial frame-supported seismic wall structure, its shear bearing capacity shall be checked according to the following formula:

$$V_{\mathrm{f}} \leq \frac{1}{\gamma_{\mathrm{RE}}}(f_{\mathrm{y}} A_{\mathrm{s}}) \tag{E.1.3}$$

Where　A_{s}——Section area of all steel reinforcement in frame-supported storey floor (including beam and plate) crossing ground seismic wall.

E.1.4　The edge of frame-supported floor slab and the surrounding of large openings shall be set up with boundary beam, whose width should be greater than or equal to 2 times of plate thickness; the reinforcement ratio of longitudinal steel reinforcement shall be greater than or equal to 1%. The joints of steel reinforcement should be connected mechanically or welded; the steel reinforcement of floor slab shall be anchored inside the boundary beam.

E.1.5　As for long or irregular architectural plane and frame-supported storey with great phase difference between internal force of seismic walls, simplified method may be adopted to check the flexual and shear bearing capacity inside floor level if necessary.

E.2 Requirements for Seismic Design of Tube-structure Transfer Storey

E.2.1 The structural quality center at upper and lower transfer storey should superpose approximately (excluding podiums); the lateral rigidity ratio between upper and lower transfer storey should be less than or equal to 2.

E.2.2 The vertical anti-lateral force component (wall, column) at upper transfer storey should directly fall on the main structure of transfer storey.

E.2.3 The transfer-storey structures of thick plates should not be used for tall building at and above Intensity 7.

E.2.4 The floor of transfer storey shall be free from large opening; the inside of plane should be approximate to rigidity.

E.2.5 The floor of transfer storey shall be reliably connected with drum and seismic wall; the seismic check and construction of transfer-storey floor slab should meet the relevant requirements of frame-supported floor slab in Article E.1 of this Appendix.

E.2.6 Vertical earthquake action shall be considered for transfer-storey structure under Intensity 8.

E.2.7 No transfer-storey structure shall be adopted in case of Intensity 9.

Appendix F Requirements for Seismic Design of Small Armored Concrete Hollow Block Buildings with Seismic Wall

F.1 General Requirements

F.1.1 The maximum height of small armored concrete hollow block buildings with seismic wall applicable to this Appendix shall be in accordance with those specified in Table F.1.1-1; the ratio between the total height and total width of the buildings should be less than or equal to the requirements of Table F.1.1-2.

Table F.1.1-1 Applicable Maximum Height of Small Armored Concrete Hollow Block Buildings with Seismic Wall (m)

Minimum wall thickness	Intensity 6	Intensity 7		Intensity 8		Intensity 9
(mm)	0.05g	0.10g	0.15g	0.20g	0.30g	0.40g
190	60	55	45	40	30	24

Note: 1 When the height of buildings exceeds the height specified in the Table, it shall carry otu special research and demonstration and take effective strengthening measures;

2 When the building area of rooms with the bay at certain storey (stories) greater than 6.0m accounts for over 40% of the building area at corresponding storey, the data in the Table shall reduce 6m correspondingly.

3 The height of buildings refers to the height between outdoor ground and the top of main roof slab (excluding partial roof projection).

Table F.1.1-2 Maximum Height-width Ratio of Small Armored Concrete Hollow Block Buildings with Seismic Wall

Intensity	Intensity 6	Intensity 7	Intensity 8	Intensity 9
Maximum height-width ratio	4.5	4.0	3.0	2.0

Note: When the horizontal and vertical layout of buildings is irregular, the maximum height-width ratio shall be reduced properly.

F.1.2 Different seismic grades shall be applied to small armored concrete hollow block buildings with seismic wall according to the seismic precautionary category and Intensity as well as building height; and it shall meet the requirements of corresponding calculation and construction measures. The seismic grade of Category C buildings should be determined according to those specified in Table F.1.2.

Table F.1.2 Seismic Grade of Small Armored Concrete Hollow Block Buildings with Seismic Wall

Intensity	Intensity		Intensity 7		Intensity 8		Intensity 9
Height (m)	≤24	>24	≤24	>24	≤24	>24	≤24
Seismic grade	Grade 4	Grade 3	Grade 3	Grade 2	Grade 2	Grade 1	Grade 1

Note: The seismic grade may be determined according to building irregularity as well as site and base conditions when it is approximate or equal to height boundary.

F.1.3 The small armored concrete hollow block buildings with seismic wall shall avoid applying irregualar building structure scheme specified in Section 3.4 of this code and shall meet the following requirements:

1 The planeform should be simple, regular and moderate in unevenness; the vertical layout should be regular, even and free of excessive outward projection and inward contraction.

2 The vertical and horizontal seismic wall should be connected and lined up; the length of each independent wall independent wall segment should be less than or equal to 8m and should be greater than or equal to 5 times of the wall thickness; the ratio between total height and length of the wall segment should be greater than or equal to 2; the door opening should be aligned from up to down in a row.

3 When cast-in-place armored concrete buildings are adopted, the maximum space between seismic transverse walls shall be in accordance with those specified in Table F.1.3.

Table F.1.3 The Maximum Space between Small Armored Concrete Hollow-block Seismic Transverse Walls

Intensity	Intensity 6	Intensity 7	Intensity 8	Intensity 9
Maximum space (m)	15	15	11	7

4 When the buildings need be set up with seismic joint, the minimum width of joint shall meet the following requirements:

When the height of buildings is less than or equal to 24m, it may take 100mm; when the height of buildings is greater than 24m, it shall increase by 6m, 5m, 4m and 3m for Intensity 6, Intensity 7, Intensity 8 and Intensity 9 correspondingly; and it should be widened for 20mm.

F.1.4 The storey height of small armored concrete hollow block buildings with seismic wall shall meet the following requirements:

1 The storey height of bottom reinforced part should be less than or equal to 3.2m for Grade I and II and shall be less than or equal to 3.9m for Grade III and IV.

2 The storey height of other positions shall be less than or equal to 3.9m for Grade I and II and shall be less than or equal to 4.8m for Grade III and IV.

Note: The bottom reinforced part refers to the height range greater than or equal to 1/6 of building height and greater than or equal to the second storey at bottom; it shall take the first storey when the total height of building is less than 21m.

F.1.5 The short wall of small armored concrete hollow block buildings with seismic wall shall meet the following requirements:

1 It shall not adopt reinforcement small-block seismic wall structure with entire short wall; and it shall form seismic wall structure when the short seismic wall and ordinary seismic wall resist horizontal earthquake action together. Short wall should not be adopted under Intensity 9.

2 Under the action of specified horizontal force, the bottom seismic overturning moment of ordinary seismic wall shall be greater than or equal to 50% of the total overturning moment on structures; the ratio between section area of short seismic wall and gross section area of seismic wall at two principal axis directions should be less than or equal to 20%.

3 The short wall should be set up with wing wall; no buildings and roof beams shall be arranged outside I-shaped short wall plane to intersect with it unilaterally.

4 The seismic grade of short wall shall be used by increasing one grade according to the requirements of Table F.1.2; for Grade I, the reinforcement shall be improved according to the requirements of Intensity 9.

Note: The short seismic wall refers to seismic wall with a ratio of 5~8 between height and width of wall column section; the ordinary seismic wall refers to seismic wall with the height-width ratio of wall column section greater than 8. The long and short property of "L-shaped", "T-shaped" and "+-shaped" multiple walls shall be determined by the longer one.

F.2 Essentials in Calculation

F.2.1 During seismic calculation of small armored concrete hollow block buildings with seismic wall, it shall adjust the earthquake action effect according to the requirements of this section. Under Intensity 6, it may not carry out section seismic check, but details of seismic design shall be taken according to the relevant requirements of this Appendix. The small armored concrete hollow block buildings with seismic wall shall be carried out with seismic check for deformation under frequent earthquake action. The maximum drift angle between elastic layers inside the storey should be less than or equal to 1/1200 for ground floor and should be less than or equal to 1/800 for other storey.

F.2.2 As for the bearing capacity calculation of small armored concrete block seismic wall, the design value for assembly shear force of bottom reinforced section shall be adjusted according to the following requirements:

$$V=\eta_{vw}V_w \tag{F.2.2}$$

Where V——Design value for assembly shear force of bottom reinforced section on seismic wall;

V_w——Calculated value for assembly shear force of bottom reinforced section on seismic wall;

η_{vw}——Enhancement coefficient of shear force; it shall take 1.6 for Grade 1, 1.4 for Grade 2, 1.2 for Grade 3 and 1.0 for Grade 4.

F.2.3 The design value for assembly shear force of small armored concrete hollow-block seismic wall section shall meet the following requirements: The shear span ratio is greater than 2

$$V\leq\frac{1}{\gamma_{RE}}(0.2f_g bh) \tag{F.2.3-1}$$

The shear span ratio is less than or equal to 2

$$V\leq\frac{1}{\gamma_{RE}}(0.15f_g bh) \tag{F.2.3-2}$$

Where f_g——Design value for small-block masonry compressive strength of pouring holes;

b——Section width of seismic wall;

h——Section height of seismic wall;

γ_{RE}——Seismic adjustment coefficient of bearing capacity, taking 0.85.

Note: The shear span ratio shall be calculated according to Formula (6.2.9-3) of this code.

F.2.4 The shear bearing capacity of small armored concrete hollow-block seismic wall section under eccentric compression shall be checked according to the following formula:

$$V \leq \frac{1}{\gamma_{RE}}\left[\frac{1}{\lambda-0.5}\left(0.48 f_{gv} b h_0 + 0.1 N\right) + 0.72 f_{yh} \frac{A_{sh}}{s} h_0\right] \quad (F.2.4\text{-}1)$$

$$0.5 V \leq \frac{1}{\gamma_{RE}}\left(0.72 f_{yh} \frac{A_{sh}}{s} h_0\right) \quad (F.2.4\text{-}2)$$

Where N——Axial pressure design value of seismic wall assembly; when $N>0.2f_g bh$, taking $N=0.2f_g bh$;

λ——Shear span ratio at calculated section, taking $\lambda=M/Vh_0$; when it is less than 1.5, taking 1.5; when it is greater than 2.2, taking 2.2;

f_{gv}——Design value for small-block masonry shear strength of grouting holes; $f_{gv}= 0.2 f_g^{0.55}$;

A_{sh}——Section area of horizontal steel reinforcement on the same section;

s——Space between horizontally-distributed tendon;

f_{yh}——Design value for tensile strength of horizontally-distributed tendon ;

h_0——Effective height of seismic wall section.

F.2.5 There shall be no small eccentric tension on the wall column of small armored concrete hollow-block seismic wall under frequent earthquake action assembly. The diagonal section shear bearing capacity of small armored concrete hollow-block seismic wall under large eccentric tension shall be calculated according to the following formula:

$$V \leq \frac{1}{\gamma_{RE}}\left[\frac{1}{\lambda-0.5}\left(0.48 f_{gv} b h_0 - 0.17 N\right) + 0.72 f_{yh} \frac{A_{sh}}{s} h_0\right] \quad (F.2.5\text{-}1)$$

$$0.5 V \leq \frac{1}{\gamma_{RE}}\left(0.72 f_{yh} \frac{A_{sh}}{s} h_0\right) \quad (F.2.5\text{-}2)$$

When $0.48 f_{gv} b h_0 - 0.17 N \leq 0$, taking $0.48 f_{gv} b h_0 - 0.17 N = 0$

Where N——Axial tension design value of seismic wall assembly.

F.2.6 Armored concrete connection beam should be used for connection beam with the span-height ratio of small armored concrete hollow-block seismic wall greater than 2.5; the design value of section assembly shear force and the shear bearing capacity of diagonal section shall meet the relevant requirements of connection beam in the current national standard "Code for Design of Concrete Structures" GB 50010.

F.2.7 When small armored concrete hollow-block masonry connection beam is used for seismic wall, it shall meet the following requirements:

1 The section of connection beam shall meet the requirements of the following formula:

$$V \leq \frac{1}{\gamma_{RE}}(0.15 f_g b h_0) \quad (F.2.7\text{-}1)$$

2 The diagonal section shear bearing capacity of connection beam shall be calculated according to the following formula:

$$V \leq \frac{1}{\gamma_{RE}}\left(0.56 f_{gv} b h_0 + 0.7 f_{yv} \frac{A_{sv}}{s} h_0\right) \quad (F.2.7\text{-}2)$$

Where A_{sv}——Gross section area of each limb stirrup inside the same section;

f_{yv}——Design value for tensile strength stirrup.

F.3 Details of Seismic Design

F.3.1 The small armored concrete hollow block buildings with seismic wall shall apply hole-grouting concrete with large slump, excellent flowability and workability and good combination with masonry block. The strength grade of hole-grouting concrete shall be greater than or equal to Cb20.

F.3.2 The seismic wall of small armored concrete hollow block buildings with seismic wall shall be fully filled with hole-grouting concrete.

F.3.3 The transversely and vertically distributed steel reinforcement of small armored concrete hollow-block seismic wall shall be in accordance with those specified in Table F.3.3-1 and F.3.3-2. The transversely distributed steel reinforcement should be arranged in double row; the space between tie bars among double-row distributed steel reinforcement shall be less than or equal to 400mm; the diameter shall be greater than or equal to 6mm. The vertically distributed steel reinforcement should be arranged in single row; the diameter shall be less than or equal to 25mm.

Table F.3.3-1 Detailing Requirements for Transversely Distributed Steel Reinforcement in Small Armored Concrete Hollow-block Seismic Wall

Seismic grade	Minimum reinforcement ratio (%)		Maximum space (mm)	Minimum diameter (mm)
	General position	Strengthening position		
Grade 1	0.15	0.15	400	ø8
Grade 2	0.13	0.13	600	ø8
Grade 3	0.11	0.13	600	ø8
Grade 4	0.10	0.10	600	ø6

Note: Under Intensity 9, the reinforcement ratio shall be greater than or equal to 0.2%; the maximum space shall be less than or equal to 400mm at the top storey or the bottom strengthening position.

Table F.3.3-2 Detailing Requirements for Vertically Distributed Steel Reinforcement in Small Armored Concrete Hollow-block Seismic Wall

Seismic grade	Minimum reinforcement ratio (%)		Maximum space (mm)	Minimum diameter (mm)
	General position	Strengthening position		
Grade 1	0.15	0.15	400	ø12
Grade 2	0.13	0.13	600	ø12
Grade 3	0.11	0.13	600	ø12
Grade 4	0.10	0.10	600	ø12

Note: Under Intensity 9, the reinforcement ratio shall be greater than or equal to 0.2%; the maximum space shall be properly reduced at the top storey or the bottom strengthening position.

F.3.4 The axial compression ratio of small armored concrete hollow-block seismic wall under the action of gravity-load representative value shall meet the following requirements:

1 As for the bottom strengthening position of common wall, it should be less than or equal to 0.4, 0.5 and 0.6 for Grade 1 (Intensity 9), Grade 1 (Intensity 8) and Grade 2 and 3 respectively; it should be less than or equal to 0.6 for general position.

2 As for the overall height of short wall, it should be less than or equal to 0.50 and 0.60 for Grade 1 and Grade 2 and 3 respectively; as for I-shaped short wall without any flange, the axial compression ratio limit shall be reduced for 0.1 correspondingly.

3 If the wall column section in all directions refers to independent small wall column with $3b<h<5b$, it should be less than or equal to 0.4 and 0.5 for Grade 1 and Grade 2 and 3 respectively; the axial compression ratio limit of independent small wall column without any flange shall be reduced for 0.1 correspondingly.

F.3.5 The wall column end of small armored concrete hollow-block seismic wall shall be set up with boundary components; when the axial compression ratio of bottom strengthening position is greater than 0.2 and 0.3 for Grade 1 and Grade 2 respectively, ig shall set up constrained boundary components. The reinforcement scope of structural boundary components: 3-hole reinforcement at the ends of flange-free wall; 3-hole reinforcement at L-shaped corner joint; 4-hole reinforcement at T-shaped corner joint. horizontal stirrup shall be set up within the range of boundary components; the minimum reinforcement shall be in accordance with those specified in Table F.3.5. As for the range of constrained boundary components, one hole shall be added along the forced direction than structural boundary components. The horizontal stirrup shall be strengthened correspondingly; or it may also be strengthened with concrete frame column.

Table F.3.5 Reinforcement Requirements for Boundary Components of Seismic Wall

Seismic grade	Minimum reinforcement quantity of vertical steel reinforcement in each hole		Minimum diameter of horizontal stirrup	Horizontal stirrup
	Bottom strengthening position	General position		Maximum space
Grade 1	1ø20	1ø18	ø8	200mm
Grade 2	1ø18	1ø16	ø6	200mm
Grade 3	1ø16	1ø14	ø6	200mm
Grade 4	1ø14	1ø12	ø6	200mm

Note: 1 The horizontal stirrup of boundary components should adopt the form of overlapping spot-welding meshes;

2 Under Grade 1, 2 and 3, the stirrup of boundary components shall apply hot rolled reinforcement greater than or equal to Grade HRB335;

3 When the axial compression ratio under Grade 2 is greater than 0.3, the minimum diameter of horizontal stirrup for bottom strengthening position shall be greater than or equal to 8mm.

F.3.6 The overlapping length of vertically and transversely distributed steel reinforcement in small armored concrete hollow-block seismic wall shall be greater than or equal to 48 times of reinforcement diameter; the anchorage length shall be greater than or equal to 42 times of reinforcement diameter.

F.3.7 The transversely distributed steel reinforcement in small armored concrete hollow-block seismic wall shall be arranged continuously along the wall; the anchorage on both ends shall meet the following requirements:

1 As for Grade 1 and 2 seismic wall, the transversely distributed steel reinforcement may bend for 180 degree into a hook along vertical main reinforcement; the length of straight reach at hook end should be greater than or equal to 12 times of reinforcement diameter. The transversely distributed steel reinforcement shall not be less than also bend into the end hole-grouting concrete; the anchorage length shall be greater than or equal to 30 times of steel reinforcement and shall be greater than or equal to 250mm.

2 As for Grade 3 and 4 seismic wall, the transversely distributed steel reinforcement may bend into end hole-grouting concrete; the anchorage length shall be greater than or equal to 25 times of steel reinforcement diameter and shall be greater than or equal to 200mm.

F.3.8 In small armored concrete hollow-block seismic wall, masonry connection beam may be adopted for those with span-height ratio less than 2.5; its construction shall meet the following requirements:

1 As for the upper and lower longitudinal steel reinforcement on connection beam, the anchored length into the wall shall be greater than or equal to 1.15 and 1.05 times of anchored length for Grade 1 and 2 as well as Grade 3 and 4 respectively; and it shall be greater than or equal to the anchored length for Grade 4. All of them shall be greater than or equal to 600mm.

2 The stirrup of connection beam shall be set up along the full length of beam; the diameter of stirrup shall be 10mm and 8mm for Grade 1 and Grade 2, 3 and 4 respectively. The space between stirrup shall be less than or equal to 75mm, 100mm and 120mm for Grade 1, Grade 2 and Grade 3 respectively.

3 Structural stirrup with a space of less than or equal to 200mm shall be set up for top connection beam within the range of longitudinal steel reinforcement anchored into the wall; its diameter shall be the same as the diameter of stirrup in connection beam.

4 Within the range between 200mm from beam top surface and 200mm from beam bottom surface, waist reinforcement shall be added and its space shall be less than or equal to 200mm. The quantity of waist reinforcement at each storey shall be greater than or equal to 2ø12 and 2ø10 for Grade 1 and Grade 2~4 respectively. The length of waist reinforcement anchored into the wall shall be greater than or equal to 30 times of steel reinforcement diameter and shall be greater than or equal to 300mm.

5 No holes shall be cut on connection beam; when it need cut holes, the following requirements shall be met:

1) Steel sleeve with the external diameter less than or equal to 200mm shall be embedded at 1/3 of mid-span beam height;

2) The upper and lower effective height of opening shall be greater than or equal to 1/3 of beam height and shall be greater than or equal to 200mm.

3) The opening shall be erected with strengthening steel reinforcement; the shear bearing capacity shall be checked for section weakened by the opening.

F.3.9 The ring-beam construction of small armored concrete hollow-block seismic wall shall meet the following requirements:

1 The wall shall be set up with cast-in-place armored concrete ring-beam at the foundation and the elevation of each storey; the width of ring-beam shall be same as the wall thickness; its section height should be greater than or equal to 200mm.

2 The ring-beam concrete compressive strength shall be greater than or equal to the corresponding hole-grouting small-block masonry strength and shall be greater than or equal to C20.

3 The diameter of ring-beam longitudinal steel reinforcement shall be greater than or equal to that of transversely distributed steel reinforcement in the wall and shall be greater than or equal to 4ø12. The ring-beam longitudinal reinforcement in the foundation shall be greater than or equal to 4ø12; the diameters of ring-beam and foundation ring-beam stirrup shall be greater than or equal to 8mm and the space between them shall be less than or equal to 200mm. When the ring-beam height is greater than 300mm, waist reinforcement shall be set up along the direction of ring-beam section height and the space between them shall be less than or equal to 200mm; the diameter shall be greater than or equal to 10mm.

4 When the ring-beam bottom is embedded into the small block hole, the depth should be greater than or equal to 30mm; the top of ring-beam shall be in rough surface.

F.3.10 Cast-in-place armored concrete slab shall be used for the building, roof and tall building of small armored concrete hollow block buildings with seismic wall or shall be adopted under Intensity 9. Cast-in-place armored concrete slab should be adopted for multi-storey building; when the seismic grade is Grade 4, it may also adopt assembled monolithic armored concrete floor.

Appendix G Requirements for Seismic Design of Buildings with Steel Support-Concrete Frame Structures and Steel Frame-Armored Concrete Core Tube Structures

G.1 Steel Support-Armored Concrete Frames

G.1.1 When the seismic precautionary Intensity is between Intensity 6~8 and the height of buildings exceeds the maximum applicable height of armored concrete frame structures in Article 6.1.1 of this code, it may use lateral resistant system composed of steel support-concrete frame structures.

When the seismic design is carried out according to the requirements of this section, the applicable maximum height should be less than or equal to the average value for the maximum applicable height of armored concrete frame structures and frame-wall structures in Article 6.1.1 of this code. The buildings exceeding the maximum applicable height shall be carried out with special research and demonstration and effective strengthening measures shall also be taken.

G.1.2 Different seismic grade shall be selected for buildings with steel support-concrete frame structures according to different precautionary category, Intensity and building height; and it shall meet the requirements of corresponding calculation and construction measures. The steel support frame part of Category C building shall be higher than the requirements of frame structure in Article 8.1.3 and Article 6.1.2 of this code by one seismic grade; the armored concrete frame part shall still be determined according to the frame structures in Article 6.1.2 of this code.

G.1.3 The layout of steel support-concrete frame structures shall meet the following requirements:

1 The steel support frames shall be set up simultaneously in the two principal axis direction of structures.

2 The steel support should be laid continuously at up and down; when continuous layout can't be carried out due to the influence of building scheme, it should continue layout at adjoining span.

3 The steel support should adopt cross support, or it may also adopt zig-zag support or V-shaped support; when single support is used, the diagonal member on both directions shall be basically symmetrical.

4 The planar layout of steel support shall avoid producing torsional effect. As for buildings without large opening between steel supports, the length-width ratio of roof should meet the requirements of space between seismic walls in Article 6.1.6 of this code the staircase should be set up with steel support.

5 As for steel support frames at the ground floor, the seismic overturning moment distributed according to rigidity shall be greater than 50% of the total seismic overturning moment.

G.1.4 The seismic calculation of steel support-concrete frame structures shall also meet the following requirements:

1 The damping ratio of structures shall be less than or equal to 0.045; or it may use the equivalent damping ratio which is converted from the proportion of concrete frame part and steel support part in the total deformation energy of the structures.

2 When diagonal member of steel support frames may be calculated according to the hinge bar at ends. When the axes of diagonal member deviate from the concrete column axes by 1/4 of the column width, additional bending moment shall be considered.

3 The earthquake action undertaken by concrete frames shall be calculated according to the two models of frame structure and supporting frame structure and the larger value should be taken.

4 The storey drift limit of steel support-concrete frame should be interpolated according to frames and frame-wall structures.

G.1.5 The connecting construction between steel support and concrete column shall meet the relevant requirements of the support and column connection for single-storey armored concrete concrete column factory buildings in Section 9.1 of this code. The connecting construction between steel support and concrete beam shall meet the requirement that the connection doesn't come before support failure.

G.1.6 The steel support in steel support-concrete frame structures shall be designed according to the requirements of Chapter 8 in this code as well as the current national standard "Code for Design of Steel Structures" GB 50017; the armored concrete frames shall be designed according to the requirements of Chapter 6 in this code.

G.2 Steel frame-Armored Concrete Core Tube Structures

G.2.1 When the seismic precautionary Intensity is between Intensity 6~8 and the height of buildings exceeds the maximum applicable height of concrete frame-core tube structure in Article 6.1.1 of this code, it may adopt lateral resistant system composed of Steel-frame concrete core tube structures.

When the seismic design is carried out according to the requirements of this section, the applicable maximum height should be less than or equal to the average value for the maximum applicable height of reinforced-concrete frame core tube structures in Article 6.1.1 of this code and the maximum applicable height of steel frame-center support structure in Article 8.1.1 of this code. The buildings exceeding the maximum applicable height shall be carried out with special research and demonstration and effective strengthening measures shall also be taken.

G.2.2 Different seismic grade shall be selected for buildings Steel-frame concrete core tube structures according to different precautionary category, Intensity and buiding height; and it shall meet the requirements of corresponding calculation and construction measures. The steel frame part of Category C building shall still be determined according to Article 8.1.3 of this code. The concrete part shall increase by one seismic grade (more than one grade under Intensity 8) than the requirements of Article 6.1.2 in this code.

G.2.3 The structural layout of buildings with Steel frame-armored concrete core tube structure shall also meet the following requirements:

1 Rigid connection shall be used for the beam and column connection of external steel frame for steel frame-core tube structure; and it should adopt steel floor beam. Tge rigid connection position of concrete wall and steel beam should be set up with construction profile steel for connection.

2 The maximum storey seismic shear force of steel frame which is distributed according to rigidity calculation should be greater than or equal to 10% of the total structural seismic shear force. When it is less than 10%, the earthquake action undertaken by the core tube wall shall be increased properly; when the seismic grade of wall construction is increased by one grade, it shall be improved under Grade 1.

3 The floor of steel frame-core tube structure shall have good rigidity and ensure integrity under rare earthquake action. The floor shall adopt assembly floor with profiled steel sheet or cast-in-place armored concrete floor slab; and measures shall be taken to strengthen the connection between floor and steel beam. When there is large opening on floor or the floor has transfer storey, cast-in-place solid floor shall be used for strengthening.

4 When the profile steel concrete column is adopted for the lower part of steel frame column, the frame column joint between different materials shall be set up with transition layer to avoid mutation of rigidity and bearing capacity. After the steel column in transition layer is counted into envelope concrete, the section rigidity may take the average value for the two section rigidity of profile steel concrete column at the lower part of transition layer and the steel column at the upper part of transition layer.

G.2.4 The seismic calculation of Steel frame-armored concrete core tube structure shall also meet the following requirements:

1 The damping ratio of structures shall be less than or equal to 0.045; or it may use the equivalent damping ratio which is converted from the proportion of armored concrete tube part and steel frame part in the total deformation energy of the structures.

2 As for any storey with steel frame except console strengthened storey and adjacent storey, the seismic shear force distributed according to calculation shall be multiplied by enhancement coefficient so as to take the smaller value of greater than or equal to 20% of the total seismic shear force at structure bottom and 1.5 times of maximum storey seismic shear force for frames; and it shall be greater than or equal to 15% of the seismic shear force at structure bottom. Corresponding adjustment shall be made for all calculated values of shear force, bending moment and axial force on the storey frame component caused by earthquake action.

3 The structural calculation should give consideration to the influence of different axial deformation of steel frame column and armored concrete wall.

4 The limit of structural storey drift may refer to the limit of armored concrete structure.

G.2.5 The steel structure and concrete structure in buildings with steel frame-armored concrete core tube structure shall also be designed according to the requirements of Chapter 6 and Chapter 8 of this code as well as the current national standard "Code for Design of Steel Structures" GB 50017.

Appendix H Requirements for Seismic Design of Multi-storey Factory Buildings

H.1 Factory Buildings with Armored Concrete Frame-bent Structures

H.1.1 This section is applicable to the seismic design for factory buildings of lateral frame-bent structures connected with armored concrete frames and bent frames and factory buildings of vertical frame-bent structures with armored concrete frames at bottom and bent frames at top. When there is no requirement in this section, it shall comply with the relevant requirements of Chapter 6 and Section 9.1 in this code.

H.1.2 Different seismic grades shall be adopted for frames factory buildings with frame-bent structure according to different Intensity, structure type and height; and it shall meet the requirements of corresponding calculation and construction measures.

Where no storage bin is set up, the seismic grade may be determined according to Chapter 6 of this code; when storage bin is set up, the seismic grade of lateral bent frames may be selected according to the requirements of current national standard "Design Code for Anti-seismic of Special Structures" GB 50191. The seismic grade of vertical frame-bent structure shall be determined according to the height boundary (reducing 4m) of frames in Chapter 6 of this code.

Note: The frames are set up with storage bin, but the span-height ratio of vertical wall is greater than 2.5 and the seismic grade is still determined according to frames without storage bin.

H.1.3 The structural layout of factory buildings shall meet the following requirements:

1 The factory buildings shall have rectangular plane as well as simple and symmetrical facade.

2 Within the plane of structural units, the frames, column support and other anti-lateral force components shall be arranged symmetrically and uniformly to avoid mutation on lateral rigidity and bearing capacity of anti-lateral force structure.

3 The equipment with large mass should not be arranged in edge storey of structural units and should be set up at a position close to the rigidity center. When it is inevitable, the equipment platform shall be separated from major structure; or it shall be laid at low position where the technical requirements are met.

H.1.4 The structural layout of factory buildings with vertical frame-bent structure shall meet the following requirements:

1 The roof should adopt non-purlin roof. When other roof is adopted, the connection between roof support and component to ensure the roof has adequate horizontal rigidity.

2 The longitudinal end shall be set up with roof truss, roof beam or frame structure to bear load instead of gable wall; the transverse wall and bent frame shall not be used together to bear load inside the bent frame span.

3 The bent frame span at the top storey shall meet the following requirements:

1) The gravity center of bent frame should be approximate to or overlap with the rigidity center of substructure. The multi-span bent frame should have equal height and length;

2) The floor shall be cast-in-place; the top bent built-in storey shall avoid large opening; the thickness of floor slab should be greater than or equal to 150mm;

3) The bent frame column shall extend to the bottom vertically and continuously.

4) When the longitudinal column support is set up for bent frame at top storey, the floor shall not be equipped with staircase or punch. The central line of column support diagonal member shall meet the central line of beam column at the joint on one point.

H.1.5 The earthquake action calculation of factory buildings with vertical bent frame shall meet the following requirements:

1 The calculation of earthquake action should adopt space-filling model; the mass point should be set up at the intersection point of beam column axes, at the bracket, column cap and column variable cross section or at the concentrated load of column.

2 When determining the representative value of gravity load, the variable loads shall be based on industrial characteristic and take the coefficient of corresponding combination value for floor live load. The coefficient of load combination value for bin storage may take 0.9.

3 When there is storage bin and equipment with high support gravity center at the storey, the support member and connection shall count in the additional bending moment of storage bin and equipment under horizontal earthquake action. The horizontal earthquake action may be calculated according to following formula:

$$F_{s}=\alpha_{max}(1.0+H_{x}/H_{n})G_{eq} \tag{H.1.5}$$

Where F_s——Standard value for horizontal earthquake action at gravity center of equipment or hopper ;

α_{max}——Maximum value of horizontal seismic influence coefficient;

G_{eq}——Representative value for gravity load of equipment or hopper;

H_x——Distance between gravity center of equipment or hopper and outdoor flooring;

H_n——Height of factory buildings.

H.1.6 The adjustment and seismic check of earthquake action effect for factory buildings with vertical bent frame structure shall meet the following requirements:

1 After the frame column of vertical wall for Grade 1~4 support storage bin is adjusted according to Article 6.2.2, 6.2.3 and 6.2.5 of this code, the design values of assembly bending moment and shear force shall be multiplied by enhancement coefficient; the enhancement coefficient shall be greater than or equal to 1.1.

2 At the top-storey frame joint where the vertical bent frame structure is connected with bent frame column, the design value for assembly bending moment at column end shall be adjusted according to Article 6.2.2 of this code. As for the bending moment at the beam end and column end of other top-storey frame joint, the design value may not be adjusted.

3 When the bent frame at top storey is set up with longitudinal column support, the earthquake-induced additional axial force of lower frame column for bent frame column connected with column support and of Grade 1 and 2 frame column shall be multiplied by adjustment coefficient 1.5 and 1.2 respectively; when calculating the axial compression ratio, the additional axial force may not be multiplied by adjustment coefficient.

4 The seismic check of factory buildings with frame-bent structure shall meet the following requirements:

1) Under Intensity 8 with Class III and IV site as well as under Intensity 9, the bent frame column of frame-bent structure and the single column outstretching frame-span rooftop and supporting bent-frame-span roof shall be carried out with elastic-plastic deformation check; the limit of elastic-plastic drift angle may take 1/ 30.

2) When the beam section height difference on both sides of Grade 1 and 2 frame beam column joint is greater than 25% of large beam section height or greater than 500mm, the seismic shear bearing capacity of joint lower column shall be checked according to the following formula:

$$\frac{\eta_{jb} M_{b1}}{h_{01} - a_s'} - V_{col} \leq V_{RE} \tag{H.1.6-1}$$

Under Intensity 9 and Grade 1, it may not meet the above formula, but shall comply with:

$$\frac{1.15 M_{b1ua}}{h_{01} - a_s'} - V_{col} \leq V_{RE} \tag{H.1.6-2}$$

Where η_{jb}——Enhancement coefficient of joint shear force, taking 1.35 and 1.2 for Grade 1 and Grade 2 respectively;

M_{b1}——Design value for assembly bending moment of higher beam end and beam bottom;

M_{b1ua}——Bending moment value corresponding to the seismic bend bearing capacity of actually-allocated beam bottom normal section by higher beam end; it shall be determined according to the standard value for actually-allocated steel reinforcement area (including compression steel reinforcement) and material strength;

h_{01}——Effective height of higher beam section;

a'_s——Distance between force concurrence point of compression steel reinforcement and compressive edge under tensioning of higher beam end and beam end;

V_{col}——Design value for calculated shear force at joint lower column;

V_{RE}——Design value for seismic shear bearing capacity at joint lower column.

H.1.7 The basic details of seismic design for factory buildings with vertical bent frame structure shall meet the following requirements:

1 The axial compression ratio of frame column for support storage bin should be less than or equal to the specified value (reducing 0.05) of frame structure in Table 6.3.6 of this code.

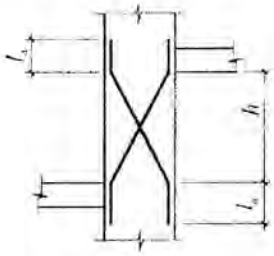
2 The minimum total reinforcement ratio of longitudinal steel reinforcement in frame column for support storage bin shall be greater than or equal to the requirements of opposite angle column in Table 6.3.7 of this code.

3 When longitudinal column support is set up for top-storey bent frame of vertical bent frame structure, the lower frame column of bent frame column connected with column support, the reinforcement ratio of longitudinal steel reinforcement as well as the configuration of stirrup shall meet the requirements of framed column in Article 6.3.7 of this code. The densified area of stirrup shall take the overall height of column.

4 When the shear span ratio of frame column is less than or equal to 1.5, it shall meet the following requirements:

1) The stirrup shall be configured by improving seismic grade by one grade. Under Grade 1 seismic grade, the requirements for stirrup shall be enhanced properly;

2) The frame column shall be set up with two diagonal bar on each direction (Figure H.1.7). The diameter of diagonal bar shall be greater than or equal to 20mm and 18mm for Grade 1 and 2 frames as well as 16mm for Grade 3 and 4 frames; the anchorage length of diagonal bar shall be greater than or equal to 40 times of diagonal bar diameter.



h—Net height of short column ; l_a—Anchorage length of diagonal bar

Figure H.1.7

5 When bracket is set up inside the trestle type column, the stirrup of frame column shall be densified within 500mm away from bracket and the up-down; when the ratio between the net height of bracket up-down column and the height of column section is greater than 4, the column stirrup shall be densified through the overall height.

H.1.8 The layout of lateral frame-bent structure, the adjustment and seismic check of earthquake action effect as well as the layout of non-purlin roof and purlin roof support shall meet the relevant requirements of current national standard "Design Code for Anti-seismic of Special Structures" GB 50191.

H.2 Factory Buildings with Multi-storey Steel Structures

H.2.1 This section is applicable to multi-storey factory buildings with steel structural frames, supporting frames, bent frames and other structural system. When there is no requirement in this section, the multi-storey part may comply with the relevant requirements of Chapter 8 in this code; the height boundary of seismic grade shall be reduced for 10m than the requirements in Section 8.1 of this code. The single-storey part may comply with the requirements of Section 9.2 of this code.

H.2.2 The layout of factory buildings with multi-storey steel structural shall not only meet the relevant requirements of Chapter 8 in this code, but also meet the following requirements:

1 When there are complex planeform, large height difference among different structures or obvious difference between storey loads, it shall set up seismic joint or take other measures. When seismic joint is set up, the joint width shall be greater than or equal to 1.5 times of corresponding concrete structure buildings.

2 The heavy equipment shall be placed at low position.

3 When the equipment weight is directly supported by the foundation and the equipment need cross the storey vertically, the stories of factory buildings shall be separated from the equipments. The joint width between equipment and storey shall be greater than or equal to width of seismic joint.

4 The equipment on the storey shall not cross the layout of seismic joint. When the conveyer, pipeline and other strip equipment must cross the layout of seismic joint, the equipment shall have the capacity to adapt to the structural deformation under earthquake or measures shall be taken to prevent rupture.

5 The operating platform structure inside the factory buildings and the frame structure of factory buildings should adopt the layout preventing earthquake joint disconnection. When connected with the factory building structure into an integral, storey elevation the elevation of platform structure should be consistent with the corresponding storey elevation of factory building frames.

H.2.3 The support layout of factory buildings with multi-storey steel structure shall meet the following requirements:

1 The column support should be placed among columns with large load and it shall be thoroughly connected in the same column. When they must stagger with each other under limited conditions, it shall be laid continuously between columns; and it should properly increase adjacent storey or overlapping storey of roof horizontal support or column support to ensure the horizontal earthquake action undertaken by the support is transmitted to the foundation reliably.

2 The column-removed structure shall be increased with horizontal support for adjacent storey and roof; and vertical support shall be set up for adjacent columns.

3 When there is great difference between lateral rigidity of roof truss frames and the column support is laid irregularly, the floor with steel planking shall be set up with horizontal support.

4 The longitudinal rigidity of colonnades should be equal or close.

H.2.4 The floor of factory buildings should adopt cast-in-place concrete composite floor slab, assembled monolithic floor or steel planking. It shall meet the following requirements:

1 The concrete floor shall be reliably connected with steel beam.

2 When holes are cut on floor slab, reliable measures shall be taken to ensure the floor slab to transmit earthquake action.

H.2.5 The frame-bent structure shall be set up with complete roof support and it shall meet the following requirements:

1 As for the connection support between roof cross beam of bent frames and multistorey frames, its elevation should be consistent with the corresponding storey elevation of multistorey frames; and roof longitudinal horizontal support shall be set up along the full length of single-storey and multi-storey connection colonnade.

2 The high span and low span should be based on relative independent closed support system composed of individual elevation.

H.2.6 The earthquake action calculation of factory buildings with multi-storey steel structure shall meet the following requirements:

1 Typically, it should adopt space-filling model analysis. When the structural layout is regular and the mass distribution is uniform, it may carry out check along the transverse and longitudinal directions of structures. When the surface of cast-in-place armored concrete floor slab has small openings and the shear connection pieces and the steel beam are connected into an integral, it may be regarded as rigid floor.

2 Under frequent earthquakes, the structural damping ratio may take 0.03~0.04; under rarely earthquake, the damping ratio may take 0.05.

3 When determining the representative value of gravity load, the variable loads shall be based on the industrial characteristics while the coefficient of corresponding combination value shall be adopted for floor repair load, finished product or material stacking floor load as well as the equipment, hopper and materials inside the pipes.

4 The components and connection of directly supporting equipment and hopper shall be counted into the earthquake action caused by equipment and so on. The horizontal earthquake action produced by general equipment on support components and their connection may be calculated according to the requirements of Article H.1.5 in this Appendix. The bending moment and torsion moment caused by horizontal earthquake action on the support components shall be calculated based on the distance between equipment gravity center and support component centroid.

H.2.7 The seismic capacity check for components and joints of factory buildings with multi-storey steel structure shall meet the following requirements:

1 When the full plastic bearing capacity of right-left beam ends and up-down column ends of joints is checked according to Formula (8.2.5) in this code, the strong column coefficient of frame column shall take 1.25, 1.20 and 1.10 for Grade 1 and earthquake action, Grade 2 and 1.5 times of earthquake action as well as Grade 3 and 2 times of earthquake action respectively.

2 The following cases may not meet the requirements of Formula (8.2.5) in this code:

1) The column cap of single-storey frames or top multistorey frames;

2) As for the frame column not complying with Formula (8.2.5) of this code, the sum of shear bearing capacity along the check direction is less than 20% of the storey frame shear bearing capacity; as for the frame column of each colonnade in the storey not complying with Formula (8.2.5) of this code, the sum of shear bearing capacity is less than 33% of the sum for entire frame column shear bearing capacity in the colonnade.

3 The ratio between design internal force of column support member bar and the design value of its bearing capacity should be less than or equal to 0.8; when the column support undertakes at least 70% storey shear force, it should be less than or equal to 0.65.

H.2.8 The basic details of seismic design for factory buildings with multi-storey steel structure shall meet the following requirements:

1 The slenderness ratio of frame column should be less than or equal to 150; when the axial compression ratio is greater than 0.2, it should be less than or equal to 125 $(1-0.8N/Af)\sqrt{235/f_y}$.

2 The width-to-thickness ratio of plate pieces for frame column and beam in factory buildings shall meet the following requirements:

1) The single-storey part and multi-storey part with total height less than or equal to 40m, it may comply with the requirements of Section 9.2 of this code.

2) When the total height of multi-storey part is greater than 40m, it may comply with the requirements of Section 8.3 of this code.

3 The maximum stress area of frame beam and column shall not change its flange section suddenly; the upper and lower flange shall be set up with lateral support. The distance between this supporting point and adjacent supporting point shall meet the relevant requirements of plastic design in "Code for Design of Steel Structures" GB 50017.

4 The column support components should meet the following requirements:

1) The column support of multistorey frames should make X shape with frame crossgirder or other anti-seismic form; its slenderness ratio should be less than or equal to 150;

2) The width-to-thickness ratio of plate pieces for support bar shall meet the requirements of Section 9.2 of this code.

5 When frame beam adopts high-strength-bolt frictional splicing, its position should keep clear of the maximum stress area (the larger value of 1/10 clear beam span and 1.5 times of beam height). In case of beam flange splicing, the high strength bolts parallel to the internal force direction should be greater than or equal to 3 rows. The section modulus of splice plate shall be greater than 1.1 times of sectional modulus under splicing.

6 The column base of factory buildings shall be able to ensure transmitting the bearing capacity of column; it should adopt embedded type, plug-in type or encased column base and shall comply with the requirements of Section 9.2 of this code.

Appendix J Earthquake Action Effect Adjustment for Transverse Plane Bent Frame of Single-storey Factory Buildings

J.1 Adjustment on Basic Natural Vibration Period

J.1.1 When the transverse earthquake action of factory buildings is calculated according to plane bent frame, the basic natural vibration period of bent frame shall give consideration to the connection solidification of longitudinal wall as well as roof truss and column. It may be adjusted according to the following requirements:

1 As for the bent frame composed of armored concrete roof truss or steel roof truss and steel armored concrete column, it shall take 80% of the period calculated value where there is longitudinal wall and 90% of the period calculated value where there is no longitudinal wall;

2 As for the bent frame composed of armored concrete roof truss or steel roof truss and brick column, it shall take 90% of the period calculated value;

3 As for the bent frame composed of wood roof truss, wood-and-steel composite truss or light steel roof truss and brick column, it shall take the period calculated value.

J.2 Adjustment Coefficient for Seismic Shear Force and Bending Moment of Bent Frame Column

J.2.1 As for single-storey steel reinforcement concrete column factory buildings with armored concrete roof, the basic natural vibration period shall be determined according to Article J.1.1 of this code and the seismic shear force and bending moment of bent frame column shall be calculated according to plane bent frame. When they meet the following requirements, it may consider the space operation and torsion influence and they may be adjusted according to the requirements of Article J.2.3 in this code:

1 Intensity 7 and Intensity 8;

2 The ratio between unit roof length and total span of factory buildings is less than 8 or the total span of factory buildings is greater than 12m;

3 The thickness of gable wall is greater than or equal to 240mm; the horizontal section area of punch is less than or equal to 50% of the total area and it is well connected with the roof;

4 The height of column cap is less than or equal to 15m.

Note: 1 The roof length refers to the space between gable walls. When there is only gable wall on one end, it shall take the distance between bent frame under consideration and the gable wall;

2 As for factory buildings with unequal height and large high-low span, the total span may not include low span

J.2.2 As for single-storey brick column factory buildings with armored concrete roof, the basic natural vibration period shall be determined according to Article J.1.1 of this code and the seismic shear force and bending moment of bent frame column shall be calculated according to plane bent frame. When they meet the following requirements, it may consider the space operation and torsion influence and they may be adjusted according to the requirements of Article J.2.3 in this code:

1 Intensity 7 and Intensity 8;

2 With bearing gable wall on both ends;

3 The thickness of gable wall or bearing (seismic) transverse wall is greater than or equal to 240mm; the horizontal section area of punch is less than or equal to 50% of the total area and it is well connected with the roof;

4 The length of gable wall or bearing (seismic) transverse wall should be greater than or equal to its height;

5 The ratio between unit roof length and total span is less than 8 or the total span of factory buildings is greater than 12m.

Note: The roof length refers to the space between gable wall and gable wall or bearing (seismic) transverse wall.

J.2.3 The shear force and bending moment of bent frame column shall be multiplied by corresponding adjustment coefficients respectively. The value of steel armored concrete column except column at high-low span joint may be selected according to those specified in Table J.2.3-1 while the value of brick column with gable wall on both ends may be selected according to those specified in Table J.2.3-2.

Table J.2.3-1 Effect Adjustment Coefficient of Steel Armored Concrete Column (Except Column at High-low Span Joint) Considering Space Operation and Torsion Influence

Roof	Gable wall		Roof length (m)											
			≤30	36	42	48	54	60	66	72	78	84	90	96
Armored concrete roof without purlin	Gable wall on both ends	Factory building with equal height	—	—	0.75	0.75	0.75	0.80	0.80	0.80	0.85	0.85	0.85	0.90
		Factory building with unequal height	—	—	0.85	0.85	0.85	0.90	0.90	0.90	0.95	0.95	0.95	1.00
	Gable wall on one end		1.05	1.15	1.20	1.25	1.30	1.30	1.30	1.30	1.35	1.35	1.35	1.35
Armored concrete roof with purlin	Gable wall on both ends	Factory building with equal height	—	—	0.80	0.85	0.90	0.95	0.95	1.00	1.00	1.05	1.05	1.10
		Factory building with unequal height	—	—	0.85	0.90	0.95	1.00	1.00	1.05	1.05	1.10	1.10	1.15
	Gable wall on one end		1.00	1.05	1.10	1.10	1.15	1.15	1.15	1.20	1.20	1.20	1.25	1.25

Table J.2.3-2 The Effect Adjustment Coefficient of Brick Column Considering Space Action

Roof type	Space between gable wall or bearing (seismic) transverse wall (m)										
	≤12	18	24	30	36	42	48	54	60	66	72
Armored concrete roof with purlin	0.60	0.65	0.70	0.75	0.80	0.85	0.85	0.90	0.95	0.95	1.00
Armored concrete roof with purlin or wood roof with densified roof slab	0.65	0.70	0.75	0.80	0.90	0.95	0.95	1.00	1.05	1..05	1.10

J.2.4 As for all section above support low-span roof bracket of steel armored concrete column at high-low span joint, the seismic shear force and bending moment calculated according to base shear method shall be multiplied by enhancement coefficient; their values may be determined according to the following formula:

$$\eta=\zeta\left(1+1.7\frac{n_{\mathrm{h}}}{n_0}\cdot\frac{G_{\mathrm{EL}}}{G_{\mathrm{Eh}}}\right) \tag{J.2.4}$$

Where η——Enhancement coefficients of seismic shear force and bending moment;

ζ——Space operation influence coefficient at low-span joint of factory buildings with unequal height; it may be selected according to those specified in Table J.2.4;

n_{h}——Quantity of high spans;

n_0——Quantity of calculated spans. It shall take the total span quantity where there is low span only on one side; it shall take the sum of total span quantity and high span quantity where there is low span on both sides;

G_{EL}——Representative value of total gravity load at low-span roof elevation on one side of joint;

G_{Eh}——Representative value of total gravity load at high-span column cap elevation.

Table J.2.4 Space Operation Influence Coefficient of Armored Concrete Upper Column at High-low span Joint

Roof	Gable wall	Roof length (m)										
		≤36	42	48	54	60	66	72	78	84	90	96
Armored concrete roof without purlin	Gable wall on both ends	—	0.70	0.76	0.82	0.88	0.94	1.00	1.06	1.06	1.06	1.06
	Gable wall on one end	1.25										
Armored concrete roof with purlin	Gable wall on both ends	—	0.90	1.00	1.05	1.10	1.10	1.15	1.15	1.15	1.20	1.20
	Gable wall on one end	1.05										

J.2.5 As for upper column section at crane girder top elevation of single-storey factory building with steel armored concrete column, the seismic shear force and bending moment induced by crane bridge shall be multiplied by enhancement coefficient. When they are calculated according to base shear method and other simplified calculation method, their values may be selected according to those specified in Table J.2.5.

Table J.2.5 Enhancement Coefficient of Seismic Shear Force and Bending Moment Induced by Bridge Framework

Roof type	Gable wall	Side column	High-low span column	Other central column
Armored concrete roof without purlin	Gable wall on both ends	2.0	2.5	3.0
	Gable wall on one end	1.5	2.0	2.5
Armored concrete roof with purlin	Gable wall on both ends	1.5	2.0	2.5
	Gable wall on one end	1.5	2.0	2.0

Appendix K Seismic Check for Single-storey Factory Buildings in Longitudinal Direction

K.1 Modifying Stiffness Method for Longitudinal Seismic Calculation of Single-storey Factory Buildings with Armored Concrete Columns

K.1.1 Calculation on basic natural vibration period

When the longitudinal earthquake action of single-span or contour multi-span factory building with armored concrete columns is calculated according to this Appendix, the elevation of column cap is less than or equal to 15m and the average span is less than or equal to 30m, the longitudinal fundamental period may be determined according to the following formula:

1 The factory buildings with brick enclosure wall may be calculated according to following formula:

$$T_1 = 0.23 + 0.00025\psi_1 l\sqrt{H^3} \tag{K.1.1-1}$$

Where ψ_1——Coefficient of roof type; it may take 1.0 for large-scale roof slab armored concrete roof truss and 0.85 for steel roof truss;

l——Span of factory buildings (m). The multi-span factory buildings may take the average value of all spans;

H——Height between foundation top surface and column cap (m).

2 The factory buildings with flexible connection between open, semi-open or wallboard and pillar may be calculated according to Formula (K.1.1-1) and multiplied by the following influence coefficient of enclosure wall:

$$\psi_2 = 2.6 - 0.002l\sqrt{H^3} \tag{K.1.1-2}$$

Where ψ_2——Influence coefficient of enclosure wall. It shall take 1.0 where it is less than 1.0.

K.1.2 Calculation on colonnade earthquake action.

1 As for factory buildings with contour multi-span armored concrete roof, the standard value for earthquake action at column cap elevation of longitudinal colonnade may be determined according to the following formula:

$$F_i = \alpha_1 G_{eq}\frac{K_{ai}}{\Sigma K_{ai}} \tag{K.1.2-2}$$

$$K_{ai} = \psi_3\psi_4 K_i \tag{K.1.2-2}$$

Where F_i——Standard value for longitudinal earthquake action at column cap elevation of i colonnade;

α_1——Horizontal seismic influence coefficient correspond to the longitudinal basic natural vibration period of factory buildings; it shall be determined according to Article 5.1.5 of this code;

G_{eq}——Representative value for total equivalent gravity load of unit colonnade in factory buildings; it shall include the representative value of roof gravity load determined according to Article 5.1.3 of this code, 70% of the longitudinal wall deadweight; 50% deadweight of transverse wall and gable wall as well as converted column deadweight (10% of column deadweight where there is crane and 50% of column deadweight where there is no crane);

K_i——Total lateral rigidity of column cap in i colonnade. It shall include the sum for lateral rigidity of pillar inside i colonnade and upper-lower column support and the reduced lateral rigidity of longitudinal wall. The reduction coefficient of closely-laid brick enclosure wall may take 0.2~0.6 according to the side drift value of colonnade;

K_{ai}——Adjustment lateral rigidity of column cap in i colonnade;

ψ_3——Enclosure wall influence coefficient of colonnade lateral rigidity. I t may be selected according to those specified in Table K.1.2-1. As for four-span or five-span factory buildings with longitudinal brick enclosure wall, the third colonnade from side column may be determined according to 1.15 times of corresponding value in the Table;

ψ_4——Column support influence coefficient of colonnade lateral rigidity. As for brick enclosure wall in longitudinal direction, it may take 1.0 for side colonnade; the central colonnade may be selected according to those specified in Table K.1.2-2.

Table K.1.2-1 The Influence Coefficient of Enclosure Wall

Category and Intensity of enclosure wall		Colonnade and roof type				
		Side colonnade	Central colonnade			
			Roof without purlin		Roof with purlin	
240 brick wall	370 brick wall		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
Without wall, asbestos tile or clevis		0.90	1.1	1.1	1.2	1.2

Table K.1.2-2 The Column Support Influence Coefficient of Central Colonnade for Longitudinal Brick Enclosure Wall

Quantity of columns with lower-column support in factory building units	Slenderness ratio of lower-column support diagonal member for central colonnade					Central colonnade without support
	≤40	41~80	81~120	121~150	>150	
One column	0.9	0.95	1.0	1.1	1.25	1.4
Two columns	—	—	0.9	0.95	1.0	

2 As for factory buildings with contour multi-span armored concrete roof, the standard value

for earthquake action at top crane girder elevation of colonnades may be determined according to the following formula:

$$F_{ci} = \alpha_1 G_{ci} \frac{H_{ci}}{H_i} \tag{K.1.2-3}$$

Where F_{ci}——Standard value for longitudinal earthquake action at top crane girder elevation of i colonnade;

G_{ci}——Representative value for equivalent gravity load at top crane girder elevation of i colonnade. It shall include the representative value for gravity load of crane girder and suspender determined according to Article 5.1.3 of this code and 40% of pillar deadweight;

H_{ci}——Height of top crane girder in i colonnade;

H_i——Height of column cap in i colonnade.

K.2 Earthquake Action Effect and Check of Column Support for Single-storey Factory Buildings with Armored Concrete Columns

K.2.1 As for the column support with slenderness ratio of diagonal member less than or equal to 200, the horizontal drift under unit lateral force action may be determined according to the following formula:

$$u = \sum \frac{1}{1+\varphi_i} u_{ti} \tag{K.2.1}$$

Where u——Drift of unit lateral force action point;

φ_i——Axial compression stability coefficient ofinter-joint I inter-joint diagonal member; it shall be determined according to the current national standard "Code for Design of Steel Structures" GB 50017;

u_{ti}——Relative drift only considering tie bar load for i inter-jointinter-joint under unit lateral force action.

K.2.2 As for diagonal member section with slenderness ratio less than or equal to 200 may only be checked according to the tensile; considering the unloading influence of compression bar, the tension may be determined according to the following formula:

$$N_t = \frac{l_i}{(1+\psi_c \varphi_i)s_c} V_{bi} \tag{K.2.2}$$

Where N_t——Design value of axial tension for checking tensile of i inter-joint support diagonal member;

l_i——Full length of i inter-joint diagonal member;

ψ_c——Unloading coefficient of compression bar; it may take 0.7, 0.6 and 0.5 while the slenderness ratio of compression bar is 60, 100 and 200;

V_{bi}——Design value of seismic shear force undertaken by i inter-joint support;

s_c——Clear distance between support columns.

K.2.3 The longitudinal colonnade of non-bonding wall building, the upper column support and the lower column support in the same row should be designed under equal strength.

K.3 Section Seismic Check for Joint Embedded Parts at Column Support End of Single-storey Factory Buildings with Armored Concrete Columns

K.3.1 When anchor bar is adopted for anchoring embedded parts at column support and column connecting joint, its section seismic bearing capacity should be checked according to the following formula:

$$N \leq \frac{0.8 f_y A_s}{\gamma_{RE}\left(\frac{\cos\theta}{0.8\zeta_m \psi} + \frac{\sin\theta}{\zeta_r \zeta_v}\right)} \tag{K.3.1-1}$$

$$\psi = \frac{1}{1+\frac{0.6e_0}{\zeta_r s}} \tag{K.3.1-2}$$

$$\zeta_m = 0.6 + 0.25t/d \tag{K.3.1-3}$$

$$\zeta_v = (4-0.08d)\sqrt{f_c/f_y} \tag{K.3.1-4}$$

Where A_s——Gross section area of anchor bar;

γ_{RE}——Seismic adjustment coefficient of bearing capacity; it may take 1.0;

N——Diagonal tension of embedded plate; it may adopt 1.05 times of support diagonal member axial force calculated based on yield point strength of total cross section;

e_0——Eccentricity between diagonal tension and anchor-bar resultant action line; it shall be less than 20% (mm) of the distance between anchor bars at outer row;

θ——Included angle between diagonal tension and its horizontal projection;

ψ——Influence coefficient of eccentricity;

s——Distance (mm) between anchor bars at outer row;

ζ_m——Influence coefficient of embedded plate flexural deformation;

t——Thickness (mm) of embedded plate;

d——Diameter (mm) of anchor bar;

ζ_r——Influence coefficient of anchor-bar row quantity in the check direction; it may take

1.0, 0.9 and 0.85 for the second, the third and the fourth row;

ζ_v——Shear influence coefficient of anchor bar; it shall take 0.7 where it is greater than 0.7.

K.3.2 When angle steel is adopted for anchoring embedded parts at column support and column connecting joint, its section seismic bearing capacity should be checked according to the following formula:

$$N \leq \frac{0.7}{\gamma_{re}\left(\frac{\cos\theta}{N_{u0}}+\frac{\sin\theta}{V_{u0}}\right)} \tag{K.3.2-1}$$

$$V_{uo} = 3n\zeta_r\sqrt{W_{min}bf_af_c} \tag{K.3.2-2}$$

$$N_{uo} = 0.8nf_aA_s \tag{K.3.2-3}$$

Where n——Quantity of angle steel;

b——Width of angle steel limb;

W_{min}——Minimum section modulus of angle steel perpendicular to direction of shear force;

A_s——Section area of foot angle steel;

f_a——Design value for tensile strength of angle steel.

K.4 Modifying Stiffness Method for Longitudinal Seismic Calculation of Single-storey Factory Buildings with Brick Columns

K.4.1 This section is applicable to the longitudinal seismic check on contour multi-span single-storey factory buildings with brick columns where there is armored concrete roof with or without purlin.

K.4.2 The longitudinal basic natural vibration period of single-storey factory buildings with brick columns may be calculated according to following formula:

$$T_1 = 2\psi_T\sqrt{\frac{\Sigma G_s}{\Sigma K_s}} \tag{K.4.2}$$

Where ψ_T——Correction coefficient of period; it may be selected according to those specified in Table K.4.2;

G_s——Concentrated gravity load of s colonnade; it includes the gravity load of gable wall and roof with half span on both sides of colonnade as well as the wall and column gravity load at column cap or wall crown which is converted according to the kinetic energy equivalence principle;

K_s——Lateral rigidity of s colonnade.

Table K.4.2 The Correction Coefficient of Longitudinal Basic Natural Vibration Period for Factory Buildings

Roof type	Armored concrete roof without purlin		Armored concrete roof with purlin	
	Side span without skylight	Side span with skylight	Side span without skylight	Side span with skylight
Correction coefficient of period	1.3	1.35	1.4	1.45

K.4.3 The standard value for total horizontal earthquake action on single-storey factory buildings with brick columns in longitudinal direction may be calculated according to following formula:

$$F_{EK} = \alpha_1 \Sigma G_s \tag{K.4.3}$$

Where α_1——Seismic influence coefficient of longitudinal basic natural vibration period T_1 corresponding to single-storey factory buildings with brick columns;

G_s——Representative value for gravity load at wall crown, which is converted from s colonnade according to the principle of colonnade bottom shear force equivalence.

K.4.4 The horizontal earthquake action on the upper end of s upper end in the longitudinal direction of factory buildings may be calculated according to following formula:

$$F_s = \frac{\psi_s k_s}{\Sigma \psi_s K_s} F_{Ek} \tag{K.4.4}$$

Where ψ_s——Adjustment coefficient of colonnade rigidity reflecting the influence of roof horizontal deformation; it shall be determined based on roof type and colonnade longitudinal wall layout according to those specified in Table K.4.4.

Table K.4.4 The Adjustment Coefficient of Colonnade Rigidity

Longitudinal wall layout		Roof type			
		Armored concrete roof without purlin		Armored concrete roof with purlin	
		Side colonnade	Central colonnade	Side colonnade	Central colonnade
Brick-column open shed		0.95	1.1	0.9	1.6
Colonnade with wall-column brick wall		0.95	1.1	0.9	1.2
Side colonnade with wall-column brick wall	Central colonnade longitudinal wall with at least 4 bays	0.7	1.4	0.75	1.5
	Central colonnade longitudinal wall with less than 4 bays	0.6	1.8	0.65	1.9

Appendix L Simplified Calculation of Seismic Isolation Design and Seismic Isolation Measures for Masonry Structures

L.1 Simplified Calculation of Seismic Isolation Design

L.1.1 When seismic isolation design is adopted for multi-storey masonry structures and structures with equivalent period of masonry structures, the total horizontal earthquake action of superstructure may be subject to according to Formula (5.2.1-1) of this code, but it shall meet the following requirements:

1 The horizontal seismic absorbing coefficient should be determined based on the fundamental period of whole system after seismic isolation according to the following formula:

$$\beta = 1.2\eta_2(T_{gm}/T_1)^{\gamma} \tag{L.1.1-1}$$

Where β——Horizontal seismic absorbing coefficient;

η_2——Damping adjustment coefficient of Seismic influence coefficient; it shall be determined based on the equivalent damping of seismic isolation storey according to Article 5.1.5 of this code;

γ——Damped exponential of seismic influence coefficient at curvilinear descending section; it shall be determined based on the equivalent damping of seismic isolation storey according to Article 5.1.5 of this code;

T_{gm}——Characteristic period of masonry structures for adopting seismic isolation scheme; it shall be determined based on the local design earthquake groups according to Article 5.1.4 of this code; but it shall take 0.4s where it is less than 0.4s;

T_1——System fundamental period after seismic isolation; it shall be less than or equal to 2.0s and the larger value for 5 times of characteristic period.

2 The horizontal seismic absorbing coefficient of structures with equivalent period as masonry structures should be determined based on the fundamental period of whole system for post-seismic isolation according to the following formula:

$$\beta = 1.2\eta_2(T_g/T_1)^{\gamma}(T_0T_g)^{0.9} \tag{L.1.1-2}$$

Where T_0——Computation period of non-seismic isolation structures; it shall take the value of characteristic period where it is less than the characteristic period;

T_1——System fundamental period after seismic isolation; it shall be less than 5 times of characteristic period;

T_g——Characteristic period; the rest symbols are as above.

3 As for masonry structures and structures with equivalent fundamental period, the system fundamental period after seismic isolation may be calculated according to following formula:

$$T_1 = 2\pi\sqrt{G / K_h g} \tag{L.1.1-3}$$

Where T_1——Fundamental period of seismic isolation system;

G——Representative value for gravity load of structures above seismic isolation storey;

K_h——Horizontal equivalent rigidity of seismic isolation storey; it may be calculated according to the requirements of Article 12.2.4 in this code;

g——Gravity acceleration.

L.1.2 As for masonry structures and structures with equivalent fundamental period, the horizontal shear force of seismic isolation storey under rarely earthquake action may be calculated according to following formula:

$$V_c = \lambda_s \alpha_1(\zeta_{eq})G \tag{L.1.2}$$

Where V_c——Horizontal shear force of seismic isolation storey under rarely earthquake action.

L.1.3 As for masonry structures and structures with equivalent fundamental period, the horizontal drift of seismic isolation storey with mass center under rarely earthquake action may be calculated according to following formula:

$$u_e = \lambda_s \alpha_1(\zeta_{eq})G / K_h \tag{L.1.3}$$

Where λ_s——Near field coefficient. It shall take 1.5 within 5km away from seismogenic fault; it shall take at least 1.25 between (5~10) km;

α_1 (ζ_{eq})——Seismic influence coefficient under rarely earthquake action; it may be calculated based on seismic isolation storey parameters according to the requirements of Article 5.1.5 in this code;

K_h——Horizontal equivalent rigidity of seismic isolation storey under rarely earthquake action; it may be calculated according to the requirements of Article 12.2.4 in this code.

L.1.4 When the plane layout of seismic isolation support is rectangular or close to rectangular and the superstructure mass center and seismic isolation storey rigidity center misalign, the torsion influence coefficient of seismic isolation support may be determined according to the following methods:

1 When only the torsion under unidirectional earthquake action is considered (Figure L.1.4), the torsion influence coefficient may be estimated according to the following formula:

$$\eta = 1 + 12es_i / (a^2 + b^2) \tag{L.1.4-1}$$

Where e——Eccentricity of the superstructure mass center and seismic isolation storey rigidity center in the direction perpendicular to the earthquake action;

s_i——Distance between i seismic isolation support and seismic isolation storey rigidity center in the earthquake action direction;

a, b——Length of two sides on seismic isolation storey plane.

The torsion influence coefficient of opposite side support should be greater than or equal to 1.15; when effective torsion resistant measures are taken for seismic isolation storey and superstructure or the torsion period is less than 70% of the translation period, the torsion influence coefficient may take 1.15.

2 While considering the torsion of bi-directional earthquake action, the torsion influence coefficient may still be calculated according to Formula (L.1.4-1); but the eccentricity value (e) shall be replaced with the larger value calculated from the following formula:

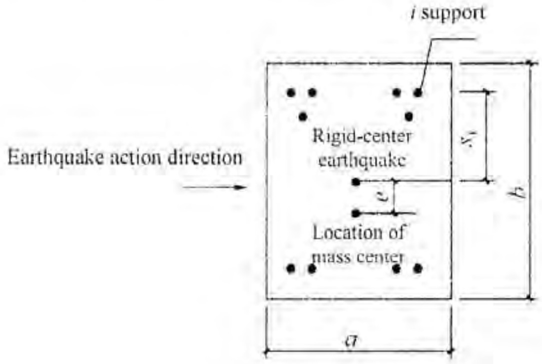


Figure L.1.4 Sketch Map of Torsion Calculation

$$e=\sqrt{e_x^2+(0.85e_y)^2} \tag{L.1.4-2}$$

$$e=\sqrt{e_y^2+(0.85e_x)^2} \tag{L.1.4-3}$$

Where e_x——Eccentricity under earthquake action in the y direction;

e_y——Eccentricity under earthquake action in the x direction.

The torsion influence coefficient of opposite side support should be greater than or equal to 1.2.

L.1.5 When seismic check under vertical earthquake action is carried out for masonry structures according to the requirements of Article 12.2.5 in this code, the normal stress influence coefficient of masonry seismic shear strength should be determined according to the mean compression stress after deducting the vertical earthquake action effect.

L.1.6 The longitudinal and transverse beam of top seismic isolation storey in masonry structures may be calculated according to single-span simple-supported beam or multi-span continuous beam which bears uniformly distributed load. The uniformly distributed load may be determined according to the requirements of armored concrete bressummer for bottom frame brick buildings in Article 7.2.5 of this code. When the sagging moment calculated from continuous beam is less than 0.8 times mid-span bending moment of single-span simple-supported beam, it shall be carried out with reinforcement according to 0.8 times mid-span bending moment of single-span simple-supported beam.

L.2 Seismic Isolation Measures for Masonry Structures

L.2.1 When the horizontal seismic absorbing coefficient is less than or equal to 0.40 (0.38 while setting up damper), the storey number, total height and height-width ratio limit of masonry structures and houses in Category C buildings may be adopted according to the relevant requirements of Section 7.1 in this code by reducing one Intensity.

L.2.2 The construction of seismic isolation storey for masonry structures shall meet the following requirements:

1 When the seismic isolation storey of multi-storey masonry building is located in top basement, the seismic isolation support should not be directly placed on the masonry wall and it shall check the partial pressure-bearing of masonry.

2 The top longitudinal and transverse beam construction of seismic isolation storey shall meet the requirements of armored concrete bressummer for bottom frame brick buildings in Article 7.5.8 of this code.

L.2.3 The details of seismic design for upper masonry structures in Category C buildings after seismic isolation shall meet the following requirements:

1 As for the minimum distance between end of load-bearing outer wall and the hole side of door and window section as well as the reinforcement construction of ring-beam, they shall meet the relevant requirements of Section 7.1, 7.3 and 7.4 in this code.

2 When the horizontal seismic absorbing coefficient of armored concrete construction column for multi-storey brick buildings is greater than 0.40 (0.38 while setting up damper), it shall meet the requirements of Table 7.3.1 in this code; when the horizontal seismic absorbing coefficient under Intensity (7~9) is less than or equal to 0.40 (0.38 while setting up damper), it shall be in accordance with those specified in Table L.2.3-1.

Table L.2.3-1 The Requirements for Constructional Column Layout in Brick Buildings After Seismic Isolation

<table>
<tr><th colspan="3">Stroey number of buildings</th><th colspan="2" rowspan="2">Position</th></tr>
<tr><th>Intensity 7</th><th>Intensity 8</th><th>Intensity 9</th></tr>
<tr><td>III, IV</td><td>II, III</td><td></td><td rowspan="4">Four corners of buildings and elevator rooms; wall corresponding to upper and lower ends of stairway inclined segment; four corners of external wall and corresponding outer corners; joints between transverse wall at split-storey position and outside longitudinal wall; both sides of large openings; joints of internal and external wall for large rooms</td><td>Every other 12m or joint between unit transverse wall and external wall</td></tr>
<tr><td>V</td><td>IV</td><td>II</td><td>Every other 12m or joint between unit transverse wall and external wall</td></tr>
<tr><td>VI</td><td>V</td><td>III, IV</td><td>Joint between transverse wall(axes) of partition room and external walls; joint between gable wall and inner longitudinal wall; joint between outside longitudinal wall and internal wall (axes) at fourth storey under Intensity 9</td></tr>
<tr><td>VII</td><td>VI, VII</td><td>V</td><td>Joints between internal wall (axes) and external wall; partial small piers of internal wall; joints between inner longitudinal wall and transverse wall (axes)</td></tr>
</table>

3 When the horizontal seismic absorbing coefficient of core column for small concrete block buildings is greater than 0.40 (0.38 while setting up damper), it shall meet the requirements of Table 7.4.1 in this code; when the horizontal seismic absorbing coefficient under Intensity (7~9) is less than or equal to 0.40 (0.38 while setting up damper), it shall be in accordance with those specified in Table L.2.3-2.

4 When the horizontal seismic absorbing coefficient of superstructure for other details of seismic design is greater than 0.40 (0.38 while setting up damper), it shall meet the requirements of Chapter 7 in this code; when the horizontal seismic absorbing coefficient under Intensity (7~9) is less than or equal to 0.40 (0.38 while setting up damper), it shall be in accordance with the requirements of Chapter 7 in this code by reducing one Intensity.

Table L.2.3-2 The Requirements for Constructional Column Layout in Small Concrete Block Buildings After Seismic Isolation

<table>
<tr><th colspan="3">Stroey number of buildings</th><th rowspan="2">Position</th><th rowspan="2">Quantity</th></tr>
<tr><th>Intensity 7</th><th>Intensity 8</th><th>Intensity 9</th></tr>
<tr><td>III, IV</td><td>II, III</td><td></td><td>Outer corners of external walls, four corners of staircase; wall corresponding to upper and lower ends of stairway inclined segment; joints of internal and external wall for large rooms; every other 12m or joints between unit transverse wall and external wall</td><td rowspan="2">Corners of external wall; joint of external wall inside fully-grouted 3 holes; fully-grouted 4 holes</td></tr>
<tr><td>V</td><td>IV</td><td>II</td><td>Outer corners of external walls: four corners of staircase; wall corresponding walls at up and down ends at inclined ladders of staircases; joints between internal and external walls of large rooms; joints between gable wall and inner longitudinal wall; joints between transverse wall (axes) of three bays and outside longitudinal wall</td></tr>
<tr><td>VI</td><td>V</td><td>III</td><td>Outer corners of external walls; four corners of staircase; wall corresponding walls at up and down ends at inclined ladders of staircases; joints between internal and external walls of large rooms; joints between transverse wall (axes) of partition room and outside longitudinal wall; joints between gable wall and inner longitudinal wall; joint between outside longitudinal wall and transverse wall (axes) under Intensity 8 and 9; both sides of large opening</td><td>Outer corners of external wall; joint of external wall inside fully-grouted 5 holes; fully-grouted 1 hole on both sides of fully-grouted 5 holes</td></tr>
<tr><td>VII</td><td>VI</td><td>IV</td><td>Outer corners of external walls: four corners of staircase; wall corresponding walls at up and down ends at inclined ladders of staircases; joints between internal and external walls (axes) and external wall; joints between inner longitudinal wall and transverse wall (axes); both sides of hole at the proportion of 1:3</td><td>Outer corners of external walls; joint of external wall in fully-grouted 7 holes; joint of internal wall in fully-grouted 4 holes; full-grouted 1 hole on both sides of fully-grouted 4~5 holes</td></tr>
</table>

Appendix M Reference Methods for Achieving Performance-Based Seismic Design

M.1 Methods for Performance-Based Seismic Design of Structural Components

M.1.1 The structural components select seismic bearing capacity, deformability and construction seismic grade according to the following requirements, which will helps realize the requirements of seismic performance. The same or different seismic performance requirements may be selected for components at different positions of the whole structure, vertical components as well as horizontal components:

1 When priority is given to the improvement of seismic safety, the reference index for bearing capacity of structural components corresponding to different performance requirements may be selected according to those specified in Table M.1.1-1.

Table M.1.1-1 Reference Index Diagram of Bearing Capacity for Structural Components to Meet the Seismic Performance Requirements

Performance requirement	Frequent earthquakes	Precautionary earthquake	Rare earthquake
Performance 1	In good condition; according to the routine design	In good condition; the bearing capacity is rechecked according to the design value of earthquake effect which is adjusted by seismic grade	Basically in good condition; the bearing capacity is rechecked according to the design value of earthquake effect which is not adjusted by seismic grade
Performance 2	In good condition; according to the routine design	Basically in good condition; the bearing capacity is rechecked according to the design value of earthquake effect which is adjusted by seismic grade	Slight~moderate damage; the bearing capacity is rechecked according to the limit value
Performance 3	In good condition; according to the routine design	Slight damage; the bearing capacity is rechecked according to the standard value	Moderate damage, where the bearing capacity reaches the limit, it can keep stable and the reduction is less than 5%
Performance 4	In good condition; according to the routine design	Low~medium level damage; the bearing capacity is rechecked according to the limit value	No severe damage, where the bearing capacity reaches the limit it can basically keep stable and the reduction is less than 10%

2 When the service performance need be determined according to earthquake residual deformation, not only the structural components diagram meet the performance requirements of improving seismic safety, but also the reference index for storey drift with different performance requirements may be selected according to those specified in Table M.1.1-2.

3 The seismic grade for detailed construction of structural components corresponding different performance requirements may be selected according to those specified in Table M.1.1-3. Different components at the same position of the structure may be divided into vertical components and horizontal components; and the corresponding seismic construction grade may be selected according to their lowest performance requirement.

Table M.1.1-2 Reference Index Diagram of Storey Drift for Structural Components to Meet the Seismic Performance Requirements

Performance requirement	Frequent earthquakes	Precautionary earthquake	Rare earthquake
Performance 1	In good condition; the deformation is far less than the limit of elastic drift	In good condition; the deformation is less than the limit of elastic drift	Basically in good condition, the deformation is slightly greater than the limit of elastic drift
Performance 2	In good condition; the deformation is far less than the limit of elastic drift	Basically in good condition, the deformation is slightly greater than the limit of elastic drift	Slight plastic deformation; the deformation is less than 2 times of the elastic drift limit
Performance 3	In good condition; the deformation is obviously less than the limit of elastic drift	slight damage: the deformation is less than 2 times of the elastic drift limit	Obvious plastic deformation; the deformation is about 4 times about of the elastic drift limit
Performance 4	In good condition; the deformation is less than the limit of elastic drift	Slight~moderate damage; the deformation is less than 3 times of the elastic drift limit	No severe damage, the deformation is less than or equal to 0.9 times of the plastic deformation limit

Note: The deformation calculation under precautionary Intensity and rare earthquake shall give consideration to gravity second-order effect and it may deduct the whole flexural deformation.

Table M.1.1-3 Construction Seismic Grade Diagram of Structural Components Corresponding to Different Performance Requirements

Performance requirement	Construction seismic grade
Performance 1	Basic seismic construction. It may be adopted by lowering two intensities according to the relevant requirements of routine design; but it shall be greater than or equal to Intensity 6 and there shall be no brittle failure
Performance 2	Low-ductility construction. It may be adopted by lowering one Intensity according to the relevant requirements of routine design; when the bearing capacity of components is two intensities higher than frequent earthquake, it may be adopted by reducing two intensities; all of them shall be greater than or equal to Intensity 6 and there shall be no brittle failure
Performance 3	Medium-ductility construction. When the bearing capacity of component is one Intensity higher than frequent earthquake, it may be adopted by lowering one Intensity according to the relevant requirements of routine design and it shall be greater than or equal to Intensity 6; otherwise, it shall be still adopted according to the relevant requirements of routine design
Performance 4	High-ductility construction. It shall be still adopted according to the relevant requirements of routine design

M.1.2 When the bearing capacity of structural components is rechecked according to different requirements, the calculation and adjustment of earthquake internal force, the combination of earthquake action effect as well as the value and checking method of material strength shall meet the following requirements:

1 The bearing capacity of structural components under precautionary Intensity includes the compression, tension, shear and flextual bearing capacity of concrete members concrete components as well as the tension, compression, flextual and stable bearing capacity of steel components. When they are rechecked according to the design value for earthquake effect adjustment, it shall adopt the fundamental combination of earthquake action effect which corresponds to seismic grade and counts no wind load effect; and they shall be checked according to the following formula:

$$\gamma_G S_{GE}+\gamma_E S_{EK}(I_2,\lambda,\zeta)\leq R/\gamma_{RE} \tag{M.1.2-1}$$

Where I_2——Precautionary ground motion; the seismic isolation structure includes horizontal seismic absorbing influence;

λ——Earthquake effect adjustment coefficient considering seismic grade according to non-seismic performance design;

ζ——Additional damping influence of rigidity reduction or energy dissipation and seismic absorbing structure considering the entry of partial secondary components into plasticity.

Other symbols are the same as those for non-seismic performance design.

2 When the bearing capacity of structural components is rechecked according to the design value considering no earthquake action effect adjustment, it shall adopt the fundamental combination considering no wind load; and it shall be checked according to the following formula:

$$\gamma_G S_{GE}+\gamma_E S_{EK}(I,\zeta)\leq R/\gamma_{RE} \tag{M.1.2-2}$$

Where I——Ground motion or rare ground motion under precautionary Intensity; the seismic isolation structure includes horizontal seismic absorbing influence;

ζ——Additional damping influence of rigidity reduction or energy dissipation and seismic absorbing structure considering the entry of partial secondary components into plasticity.

3 When the bearing capacity of structural components is rechecked according to the standard value, it shall adopt the standard combination of earthquake action effect counting no wind load effect; and it shall be checked according to the following formula:

$$S_{GE}+S_{EK}(I,\zeta)\leq R_k \tag{M.1.2-3}$$

Where I——Precautionary ground motion or rare ground motion; the seismic isolation structure includes horizontal seismic absorbing influence;

ζ——Additional damping influence of rigidity reduction or energy dissipation and shock absorbing structure considering the entry of partial secondary components into plasticity;

R_k——Bearing capacity calculated according to the standard value of material strength.

4 When structural components is rechecked according to the ultimate bearing capacity, it shall adopt the standard combination of earthquake action effect counting no wind load effect; and it shall be checked according to the following formula:

$$S_{GE}+S_{EK}(I,\zeta)< R_u \tag{M.1.2-4}$$

Where I——Precautionary ground motion or rare ground motion; the seismic isolation structure includes horizontal seismic absorbing influence;

ζ——Additional damping influence of rigidity reduction or energy dissipation and shock absorbing structure considering the entry of partial secondary components into plasticity;

R_u——Bearing capacity calculated according to the minimum limit of material strength; the steel strength may take the minimum limit value, the reinforcement strength may take 1.25 times of the yield strength and the concrete strength may take 0.88 times of the cube strength.

M.1.3 The storey elastoplastic deformation of vertical structural components under precautionary earthquake and rare earthquake action shall be rechecked according to different control objectives. The calculation of earthquake storey shear force, the adjustment of earthquake action effect as well as the calculation and checking method of component storey drift shall meet the following requirements:

1 Different methods shall be adopted to adjust the earthquake storey shear force and earthquake action effect according to the elastoplastic degree of different positions in the whole structure. When the components are in cracking stage on the whole or just enter the yield stage, it may take equivalent rigidity and equivalent damping and make estimation according to equivalent linear method; when the bearing capacity of components are between yield stage and ultimate stage on the whole, it make estimation according to statical or dynamic elastoplastic analysis method; when the bearing capacity of components is in descending stage, it shall make estimation according to the dynamic elastoplastic analysis method of counting the parameters at descending branch.

2 Under precautionary earthquake, long-term rigidity should be adopted for the initial rigidity of concrete components.

3 The storey elastoplastic deformation of components shall be calculated according to its actual bearing capacity and shall count in gravity second-order effect according to the requirements of this code; the deformation under wind load and gravity action will take part in no earthquake combination.

4 The storey elastoplastic deformation of components may be checked according to the following formula:

$$\Delta u_p(I,\zeta,\xi_y,G_E)<[\Delta u] \tag{M.1.3}$$

Where $\Delta u_p(\cdots)$——Elastoplastic storey drift angle of vertical components counting gravity second-order effect and damping influence under precautionary earthquake or rare earthquake, which depends on its actual bearing capacity; the influence of whole rotation may be deducted for structures with height-width ratio greater than 3;

$[\Delta u]$——Elastoplastic drift angle limit; it shall be determined according to the performance control objective. As for vertical components at the position with maximum deformation in the whole structure, the slight damage may take half of moderate damage; the moderate damage may take the average value of those specified in Table 5.5.1 and Table 5.5.5 of this code; no severe damage may be controlled at less than 0.9 times of those specified in Table 5.5.5 of this code.

M.2 Methods for Performance-Based Seismic Design of Building Component Support and Building Accessory Equipment Support

M.2.1 When the performance design of non-structural building components and auxiliary electrom chanical devices is carried out according to the special requirements of use function, the performance requirement under the influence of earthquake at precautionary Intensity may be selected according to those specified in Table M.2.1.

Table M.2.1 Reference Performance Level of Building Components and Auxiliary Electromechanical Devices

Performance level	Function description	Deformation index
Performance 1	The appearance may be damaged, but it will not affect application and fire prevention; the safety glass cracks; the application and emergency system may run as usual	The connected structural components may endure 1.4 times of design deflection for building components and equipment support
Performance 2	It may basically runs normally or recover quickly; the fire resistance period reduces by 1/4; the toughened glass crushes; the application system operates after overhaul; the emergency system may run as usual	The connected structural components may endure 1.0 times of design deflection for building components and equipment support
Performance 3	The fire resistance period reduce obviously, the glass drops off and the exit is obstructed by fragment; the application system is apparently damaged and can recover function only after repairing; the emergency system is damaged but can still run basically	The connected structural components can only endure 0.6 times of design deflection for building components and equipment support

M.2.2 When performance-based seismic design is carried out for building enclosure wall, auxiliary components and fixed cabinets, the category coefficient and function coefficient of components under earthquake action may be determined by reference to Table M.2.2.

Table M.2.2 Category Coefficient and Function Coefficient of Non-structural Components in Buildings

Name of components and parts	Category coefficient of components	Function coefficient	
		Category B	Category C
Non-bearing outer wall:			
Enclosure wall	0.9	1.4	1.0
Glass curtain wall	0.9	1.4	1.4
Connection:			
Wall connection pieces	1.0	1.4	1.0
Veneer connection pieces	1.0	1.0	0.6
Fireproof ceiling connection pieces	0.9	1.0	1.0
Non-fireproof ceiling connection pieces	0.6	1.0	0.6
Auxiliary components:			
Marks or billboard	1.2	1.0	1.0
Cabinet support higher than 2.4m:			
File cabinet on (in) storage rack (cabinet)	0.6	1.0	0.6
Antiquity cabinet	1.0	1.4	1.0

M.2.3 When performance-based seismic design is carried out for building auxiliary equipment support and connection pieces, the category coefficient and function coefficient of components under earthquake action may may determined by reference to Table M.2.3.

M.3 Floor Spectra Method for Seismic Calculation of Building Components and Building Auxiliary Equipments

M.3.1 The floor spectra of nonstructural components shall reflect the self dynamic characteristics of specific structures supporting nonstructural components, the storey position of nonstructural components as well as the amplification action of structural and nonstructural damping characteristic on the ground seismic motion where the structures are located.

Table M.2.3 Category Coefficient and Function Coefficient of Auxiliary Equipment Components in Buildings

System of components and parts	Category coefficient of components	Function coefficient	
		Category B	Category C
Master control system, generator and refrigeration compressor for emergency power supply	1.0	1.4	1.4
Supporting structure, guide track, bracket and compartment guide components for elevator	1.0	1.0	1.0
Suspended or pendular light fittings	0.9	1.0	0.6
Other light fittings	0.6	1.0	0.6
Cabinet-type equipment support	0.6	1.0	0.6
Support for water tank cooling tower	1.2	1.0	1.0
Support for boiler and pressure vessel	1.0	1.0	1.0
Common antenna support	1.2	1.0	1.0

When calculating the floor spectra, single mass point model may be used for nonstructural components under general situation while multiple supporting point system should be used for nonstructural components with relative drift between supports.

M.3.2 When floor response spectrum method is adopted, the standard value for horizontal earthquake action of nonstructural components may be calculated according to the following formula:

$$F = \gamma \eta \beta_s G \tag{M.3.2}$$

Where β_s——Floor response spectrum value pf nonstructural components; it depends on the cycle ratio, mass ratio and damping between precautionary Intensity, site conditions, nonstructural components and structural systems as well as the supporting position, quantity and connection property of nonstructural components in structures;

γ——Function coefficient of nonstructural components; it depends on the precautionary category and operation requirement of buildings. Generally, it is divided into three levels: 1.4, 1.0 and 0.6;

η——Category coefficient of nonstructural components; it depends on the material performance of components and other factors. Generally, its value ranges between 0.6~1.2.

Explanation of Wording in This Code

1 Words used for different degrees of strictness are explained as follows in order to mark the difference in executing the requirements in this code:

1) Words denoting a very strict or mandatory requirement:

"Must" is used for affirmation; "must not" for negation.

2) Words denoting a strict requirement under normal conditions:

"Shall" is used for affirmation; "shall not" for negation.

3) Words denoting a permission of a slight choice or an indication of the most suitable choice when conditions permit:

"Should" is used for affirmation; "should not" for negation.

4) "May" is used to express the option available, sometimes with the conditional permission.

2 "Shall be in accordance with……" or "shall execute based upon……" is used in this code to indicate that it is necessary to comply with the requirements stipulated in other relative standards and codes.

List of Quoted Standards

1 "Code for Design of Building Foundation" GB 50007

2 "Load Code for the Design of Building Structures" GB 50009

3 "Code for Design of Concrete Structures" GB 50010

4 "Code for Design of Steel Structures" GB 50017

5 "Design Code for Anti-seismic of Special Structures" GB 50191

6 "Code for Acceptance of Constructional Quality of Concrete Structures" GB 50204

7 "Standard for Classification of Seismic Protection of Building Constructions" GB 50223

8 "Technical Code for Building Slope Engineering" GB 50330

9 "Rubber Bearing—Part 3: Elastomeric Seismic-Protection Isolators for Buildings" GB 20688.3

10 "Steel Plate with Through-thickness Characteristics" GB/T 5313