1. General

1.0.1 This code is prepared for the purpose of carrying out the policy of giving priority to the prevention of earthquake disasters so that when buildings are made earthquake-resistant, damage to buildings, loss of life and economic losses will be minimized.

When buildings designed based on the code are subjected to the influence of frequently occurred earthquakes with an intensity of less than the fortification intensity of the region, the buildings will not be, or will be only slightly damaged and will continue to be servicable without repair; when they are subjected to the influence of earthquakes equal to the fortification intensity of the region, they may be damaged but will still be servicable after ordinary repair or without repair; when they are subjected to the influence of expected rare earthquakes with an intensity higher than the fortification intensity of the region, they will not collapse nor suffer damage that would endanger human lives.

1.0.2 The code is applied to seismic design of ordinary buildings (including chimneys, water towers, etc.) in regions of fortification intensity of Ⅵ to Ⅸ.

The design of buildings in regions of fortification intensity X and/or with specific professional requirements shall follow related special provisions.

1.0.3 Fortification intensity shall be determined by documents (maps) approved and issued by the authorized central government agency. Normally, the basic intensity may be used; for cities where a disaster prevention plan has been drawn up, the approved seismic fortification zoning may be used (by using the fortification intensity or the design ground motion parameters).

1.0.4 Buildings shall be classified into the following four types based on their importance.

Type A——Buildings with special requirements, such as those that would result in serious consequences if they are damaged by an earthquake. They must be approved by competent State authorities.

Type B——Buildings of life-line systems in the main designated cities of the nation.

Type C——Buildings not included in types A, B or D.

Type D——Buildings of less importance, where earthquake damage will not likely cause death or injury to people and/or considerable economic losses.

1.0.5 Seismic design of all types of buildings shall conform to the following requirements.

1. Seismic actions of Type A buildings shall be calculated by ground motion parameters obtained from specific study; seismic actions of buildings of other types shall be determined from the fortification intensity of the region. However, buildings in regions of fortification intensity Ⅵ need not undergo any calculation of seismic actions except where there are specific provisions in the code.

2. Specific seismic measures shall be taken for Type A buildings; seismic measures for Type B buildings, except as specified in the code, shall be those taken for the region, the fortification intensity of which is one grade higher than that of the local region. However, when the fortification intensity is Ⅸ, seismic measures taken may be appropriately more stringent than those for fortification intensity Ⅸ; seismic measures for Type C buildings shall be taken based on the fortification intensity of the region; for Type D buildings, based on the fortification intensity one grade lower than that of the region; when the fortification intensity is Ⅵ, no reduction of intensity is necessary.

Note: The term " fortification intensity" in the code is usually shortened as " intensity". For example, fortification intensity of Ⅵ, Ⅶ, Ⅷ, or Ⅸ is shortened as intensity Ⅵ, Ⅶ, Ⅷ or Ⅸ respectively.

1.0.6 The code was revised in accordance with provisions of the national standard: "Uniform Standard for the Design of Building Structures, GBJ 68-84". Notations, units and glossary of terms used in the code are in accordance with the national standard: "Notations, Units and Glossary of Terms Commonly Used in the Design of Building Structures, GBJ 83-85".

1.0.7 Seismic designs based on this code should be coordinated also with requirements specified in other current design and geotechnical codes concerned.

2. Basic requirements of seismic design

2.1 Seismic effect, site and subsoil

2.1.1 If seimic effects upon the region of given fortification intensity, where buildings are located, are caused by an earthquake either in the region or in a region of intensity one grade higher than the fortification intensity of the region, the buildings shall be designed in accordance with provisions of the code for design near-earthquake ; if seismic effects upon the region are caused by an earthquake in a region of intensity two grades or more higher than the fortification intensity of the region, the buildings shall be designed by provisions of the code for design far-earthquake .

Note: In the code, the term "design near-earthquake" is generally simplified as "near-earthquake"; "design far-earthquake" as "far-earthquake".

2.1.2 When selecting a construction site, a comprehensive evaluation should be made based on the requirements of the project, seismicity of the region, and geotechnical and geological data of the site. The location favorable to earthquake resistance should be selected, while unfavorable locations should be avoided. If unfavorable locations are unavoidable, appropriate seismic measures shall be taken. Type A, B, and C buildings shall not be built in hazardous regions.

2.1.3 When the construction site is a site of Type I, seismic constructional measures shall be taken as those stipulated for an intensity one grade lower than the given intensity, except Type D buildings, while the earthquake actions shall still be calculated based on the given intensity. However, when the given intensity equals Ⅵ, stringency of constructional measures shall not be reduced.

2.1.4 Design of subsoil and foundation should be in accordance with the following requirements:

1. The same structural unit should not be placed on subsoil with entirely different characteristics.

2. The same structural unit should not be placed partly on natural subsoil and partly on pile foundation.

3. For subsoil with layers consisted of soft clay, potentially liquefied soil, recently back-filled soil, or with extremely non-uniform distribution, measures should be taken to strengthen the integrality and rigidity of the foundation.

2.2 Configuration of plan and elevation

2.2.1 Configuration of plan and elevation of buildings should be regular and symmetric; distribution of mass and change in stiffness should be even; and the floors of the building should not be staggered.

2.2.2 Seismic joints of buildings may be installed based on the actual need of the structure. When no seismic joints are installed in a building with irregular configuration, the computing model selected shall agree with its actual conditions and a more precise seismic analysis shall be performed; local stresses and deformation concentrations, as well as torsional effects shall all be estimated, vulnerable parts shall be identified, and appropriate measures shall be taken to enhance its seismic resistance. If seismic joints are installed, the structure shall be split into regular structural units. Seismic joints shall have enough clearance in accordance with the intensity, site category, and type of building. The superstructures on the two sides of the joint shall be separated completely.

Expansion joints and settlement joints shall comply with the requirements of seismic joints.

2.3 Seismic structural system

2.3.1 The seismic structural system of a building shall be determined through comprehensive analysis of the technical and economic conditions based on the following factors: importance of the building, fortification intensity, building height, site, subsoil, foundation, material used, and construction technology.

2.3.2 Seismic structural system shall comply with the following requirements:

1. It shall have a clear computing model and reasonable path for seismic action transfer;

2. It should have several lines of defense against earthquakes. It should avoid loss of either earthquate resistance capacity or gravity load capacity of the whole system due to damage to part of the structure or members;

3. It shall possess the necessary strength, adequate deformability, and better energy dissipation ability;

4. It should possess a rational distribution of stiffness and strength, avoid weakening of some parts of the structure due to local weakening or abrupt changes; avoid appearance of extremely large concentration of stress and plastic deformation; when weak parts do appear, measures should be taken to enhance their earthquake resistantee capacity.

2.3.3 Earthquake structural elements shall comply with the following requirements:

1. For unit masonry, in order to improve deformability, reinforced concrete ring beams, construction columns, and core columns shall be installed as specified, or reinforced masonry or composite masonry columns shall be used.

2. For unit concrete structural members, dimensions of members shall be selected rationally, longitudinal reinforcement and hoops shall be installed so that shearing failure does not occur before flexual failure; crushing of concrete does not occur before the yielding of steel bars, and anchorage and cohesion failure of steel bar does not occur before member failure.

3. For steel members, dimensions of members shall be rationally selected to avoid buckling.

2.3.4 The connections of seismic structural members shall comply with the following requirements:

1. The load bearing capacity of member joints shall not be less than that of the member;

2. The anchoring load bearing capacity of pre-embedded members shall not be less than that of connecting members;

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

2.3.5 The earthquake resistant bracing system shall guarantee the stability of the structure during an earthquake.

2.4 Non-structural members

2.4.1 Secondary structural members shall be reliably connected or anchored to the main structure so that human injury or damage to important equipment due to their collapse can be avoided.

2.4.2 Due consideration should be given to the favorable or unfavorable effects of curtain walls and partition walls on the earthquake resistance of the structure; irrational layout of walls that would cause damage to the main structure shall be avoided.

2.4.3 Veneers shall be firmly adhered to the main structure; falling of suspended ceilings shall be avoided to prevent human injury; suspended or adhered heavy decorative ornaments shall also be avoided; when unavoidable, safety measures shall be taken.

2.5 Materials and construction

2.5.1 Specific material and construction requirements for earthquake resistant structures shall be clearly stated in the design documents.

2.5.2 Except as stipulated in different chapters of the code, the property indices of structural materials shall possess the minimum requirements as shown below:

1. The strength grade of clay bricks shall not be less than MU 7.5; the strength grade of mortar for brick masonry should not be less than M 2.5; the strength grade of mortar for a brick chimney should not be less than M 5;

2. The strength grade of concrete blocks should not be less than MU 10 for medium-size and MU 5, for small-size blocks; the strength grade of mortar for concrete blocks should not be less than M 5;

3. The strength grade of concrete for beams, columns and joints of a frame with Seismic Category 1 should not be less than C 30; for constructional columns, core columns, ring beams, and spread foundations should not be less than C 15; for other members shall not be less than C 20;

4. The strength grade of steel reinforcements: longitudinal bars should be Grade Ⅱ or Ⅲ deformed bars; hoops should be Grade Ⅰ or Ⅱ bars; the bars for constructional columns and core columns may be Grade Ⅰ or Ⅱ bars.

2.5.3 In the construction of structures, main bearing steel reinforcements should not be replaced by those with strength grade higher than that used in the original design. When replacement is necessary, the conversion shall be made based on the actual yielding strength of the reinforcement.

2.5.4 For construction of constructional columns, core columns and frames with infilled walls in a brick masonry building with a framed first story, the brick walls shall be laid before the concrete column is cast.

2.5.5 The bricks at the intersection of longitudinal and transverse walls in a brick masonry building shall be laid with racking, otherwise tying measures shall be adapted.

2.5.6 The inspection and acceptance of materials and construction quality shall be in accordance with the requirements specified in the relevant current national standards.

3. Site, subsoil and foundation

3.1 Site

3.1.1 In the selection of a construction site, the ground sections, including those favorable, unfavorable, and hazardous to earthquake resistance, shall be identified by Table 3.1.1.

Table 3.1.1 Identification of Ground Sections

Ground section category	Geological, topographical and geomorphical description
Favorable to earthquake resistance	Stiff soil or dense and homogeneous medium-stiff soil in a wide open area.
Unfavorable to earth-quake resistance	Soft soil; liquefiable soil; stripe-shaped protruding ridge; high isolated hill; non-rocky steep slope; river banks and edges of slopes; soil strata having obviously heterogeneous distribution in plane for cause of formation, lithology, and state (such as abandoned and filled river beds, fracture zone of fault, and hidden swamp, creek, gully and pit, as well as subsoil with partial excavation and filling.)
Hazardous to earthquake resistance	Places where landslide, avalanche, subsidence, formation of cracks and mudrock flow are liable to occur during an earthquake; location of causative fault on which ground dislocation may occur.

3.1.2 The site soil should be classified according to the shear-wave velocity of the soil layer as shown in Table 3.1.2.

Table 3.1.2 Classification of Site Soil

Type of site soil	Shear-wave velocity of soil layer (m/s)
Stiff soil	$v_s > 500$
Medium-stiff site soil	$500 \geqslant v_{sm} > 250$
Medium-soft site soil	$250 \geqslant v_{sm} > 140$
Soft site soil	$v_{sm} \leqslant 140$

Note: v_s is the shear-wave velocity of soil layer; b_{sm} is the weighted mean shear-wave velocity of soil layers, according to the thickness of all soil layers within 15 m under the ground level but not greater than the thickness of the overlaying layer.

3.1.3 If no measured data of shear-wave velocity for Type C and Type D buildings is available, the soil may be classified according to Table 3.1.3 and the type of site soil may be determined based on the following principles: In case of single-layer soil, the type of soil is the same as the type of site soil; in case of multi-layer soil, the type of site soil may be evaluated comprehensively based on the types and thicknesses of the soil layers within 15 m below ground level, but not greater than the thickness of the overlaying layer at the site.

Table 3.1.3 Classification of Soil

Type of soil	Geotechnical description
Stiff soil	Stable rock, dense gravel
Medium-stiff soil	Medium dense or slightly dense gravel; dense or medium-dense gravel, coarse or medium sand; cohesive soil and silt with $f_k > 200$.
Medium-soft soil	Slightly dense gravel, coarse or medium sand; fine and silty sand other than that which is loose; cohesive soil and silt with $f_k \leqslant 200$; fillland with $f_k \geqslant 130$.
Soft soil	Muck and mucky soil; loose sand; new alluvial sediment of cohesive soil and silt; fill land with $f_k < 130$.

Note: f_k is the nominal value of static bearing capacity of subsoil in *kPa*.

3.1.4 The overlaying thickness of the site is the distance from the ground level to a soil layer where the shear-wave velocity is more than 500 m/s or to the top of the stiff soil.

3.1.5 Construction sites shall be classified into four categories according to type of site soil and the overlaying thickness at the site, and should also comply with Table 3.1.5. If sufficient basis is available, the site category may be modified accordingly.

3.1.6 The geological exploration of the site shall be carried out not only based on the provisions of relevant national standards, but also on the classification of the ground sections-whether favorable, unfavorable, or hazardous to buildings, providing the site category of the building and geotechnical stability evaluation for earthquake resistance (whether landslide or avalanche would occur) according to the actual condition. For buildings where a supplementary time-history analysis is required, the relevant dynamic parameters and the overlaying thickness of the site shall also be provided as required by design.

Table 3.1.5 Construction Site Categories

Type of site soil	Thickness of overlaying layer at site, d_{ov}(m)				
	0	$0<d_{ov}\leqslant 3$	$3<d_{ov}\leqslant 9$	$9<d_{ov}\leqslant 80$	$d_{ov}>80$
Stiff site soil	I				
Medium-stiff site soil		I		II	
Medium-soft site soil		I	II		III
Soft site soil		I	II	III	IV

3.2 Seismic checking for subsoil and foundation

3.2.1 For the following types of buildings, the bearing capacity of natural subsoil and foundation need not be checked for earthquake resistance:

1. Unit masonry buildings, multi-story brick buildings with inner frames or framed first story, and water towers;

2. Ordinary single-story factory buildings, single-story spacious buildings, multi-story framed civil buildings, and multi-story framed buildings with eqivalent foundation loading to that of multi-story framed civil buildings, provided that no weak cohesive soil layers lie within the range of the main bearing layer of the subsoil;

3. Chimneys not exceeding 100 meters in height when the intensity is VII or VIII;

4. Structures, of which seismic checking specified in this code need not be conducted for their superstructure .

Note: Weak cohesive soil layer refers to soil layer with nominal static bearing capacity less than 80, 100 and 120 kPa when the intensity is VII, VIII, and IX respectively.

3.2.2 In the checking of foundation on natural subsoil for earthquake resistance, the seismic bearing capacity of subsoil shall be calculated by the following equation:

$$f_{sE}=\zeta_s f_s \tag{3.2.2}$$

where f_{sE}—adjusted design value of seismic bearing capacity of subsoil;

ζ_s—adjusting coefficient for seismic bearing capacity of subsoil, and shall be taken from Table 3.2.2 ;

f_s—design value of static bearing capacity of subsoil and shall be taken as the value as given in the current national standard: "Design Code for Subsoil and Foundation for Buildings".

Table 3.2.2 **Adjusting Coefficient for Seismic Bearing Capacity of Subsoil**

Geotechnical description	ζ_s
Rock; dense gravel; dense gravel, dense coarse or medium sand; cohesive soil or silt with $f_k \geqslant 300$	1.5
Medium dense or slightly dense gravel; medium or slightly dense gravel; coarse or medium sand; dense and medium dense fine or silty sand; cohesive soil or silt with $150 \leqslant f_k < 300$	1.3
Slightly dense fine sand and silty sand; cohesive soil or silt with $100 \leqslant f_k < 150$; newly alluvial sediment deposit of cohesive soil and silt.	1.1
Silt, silty soil, loose sand , and fill	1.0

3.2.3 In the checking of vertical bearing capacity of natural subsoil under seismic action, the mean pressure on the base of the foundation and the maximum pressure at the edge of the foundation shall comply with the following equations; also, the area of zero stress between the foundation base and the subsoil shall not be more than 25% of the total area of the foundation base; the zero-stress area for chimney foundations should comply with the requirements specified in the current national standard: "Design Code for Chimneys".

$$p \leqslant f_{sE} \qquad (3.2.3-1)$$

$$p_{max} \leqslant 1.2 f_{sE} \qquad (3.2.3-2)$$

where p—mean design value of pressure of combined seismic action on the foundation base;

p_{max}—maximum design value of pressure of combined seismic action at the edge of foundation base.

3.2.4 For pile foundation with low pile caps and mainly supporting vertical load, the seismic bearing capacity of the pile foundation need not be checked for the following buildings types if there is no liquefaction potential soil layer, no silt or silty soil surrounding the pilecap, or no fill with nominal value of static bearing capacity not greater than 100 kPa:

1. Buildings as specified in Clauses 1, 3, 4 of Article 3.2.1 .

2. When the intensity is Ⅶ or Ⅷ, ordinary single-story factory buildings, single-story spacious buildings, multi-story framed civil buildings, and multi-story

factory buildings with foundation load equivalent to that of multi-storey framed civil buildings.

3.3 Seismic measures for subsoil

3.3.1 Under ordinary circumstances, discrimination of the potential of liquefaction of the saturated soil and adoption of measures to prevent liquefaction need not be considered when intensity is Ⅵ, but for Type B buildings that are sensitive to the settlement caused by liquefaction, measures for intensity Ⅶ may be used. When the intensity is Ⅶ to Ⅸ, for Type B buildings, discrimination of the potential of liquefaction and adoption of relevant measures may be considered using those stipulated for the original intensity.

3.3.2 If one of the following conditions is satisfied, saturated sand or saturated silt may be preliminarily discriminated as non-liquefiable soil, or effects of liquefaction need not be considered:

1. If the geological period of the soil is pleistocene of the Quatenary period (Q_3) or earlier, the soil may be considered non-liquefiable.

2. If the clay particle content (particle diameter less than 0.005 mm) of silt is not less than 10%, 13%, and 16%, when the intensity is Ⅶ, Ⅷ, and Ⅸ respectively, the soil may be considered non-liquefiable.

Note: Clay particle content is determined by use of sodium hexametaphosphate as the dispersant. When using other methods, appropriate conversions should be used based on related provisions.

3. For buildings resting on natural subsoil, effects of liquefaction need not be considered when the thickness of the non-liquefiable overlaying layer and the depth of underground water level comply with one of the following conditions:

$$d_u > d_0 + d_b - 2 \quad (3.3.2-1)$$

$$d_w > d_0 + d_b - 3 \quad (3.3.2-2)$$

$$d_u + d_w > 1.5d_0 + 2d_b - 4.5 \quad (3.3.2-3)$$

where d_w—depth of underground water level (m) for which the mean annual highest level during the service of the building should be used or the annual highest level in recent years may also be used.

d_u—thickness of the non-liquefiable overlaying layer (m); when computing, silt and silty soil layer should be deduced.

d_b—buried depth of foundation (m); if it is not greater than 2 m, then 2 m shall be used.

d_0—characteristic depth of liquefaction-potential soil (m), values in Table

3.3.2 may be used.

Table 3.3.2 Characteristic Depth of Liquefaction-Potential Soil(m)

Type of saturated soil	Intensity		
	Ⅶ	Ⅷ	Ⅸ
Silt	6	7	8
Sand	7	8	9

3.3.3 If it is considered in the preliminary discrinination that further discrimination of the potential of liquefaction is necessary, the standard penetration tests shall be performed. For the liquefaction-potential soil situated within a depth of 15m below the ground level, the following equations shall be satisfied. Other discriminating methods, if already proved successful, may also be used.

$$N_{63.5} < N_{cr} \tag{3.3.3-1}$$

$$N_{cr} = N_0[0.9 + 0.1(d_s - d_w)]\sqrt{\frac{3}{\rho_c}} \tag{3.3.3-2}$$

where $N_{63.5}$—measured value of standard penetration resistance in number of blow counts for saturated soil (correction for the length of rod not yet considered);

N_{cr}—critical value of standard penetration resistance in number of blow counts for liquefaction discrimination;

N_0—reference value of standard penetration resistance in number of blow counts for liquefaction discrimination, it shall be taken fromTable 3.3.3;

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

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

Table 3.3.3 Reference Value of Standard Penetration Resistance in Number of Blow Counts.

Near- or far-earthquake	Intensity		
	Ⅶ	Ⅷ	Ⅸ
Near-earthquake	6	10	16
Far-earthquake	8	12	—

3.3.4 For the subsoil with liquefaction-potential soil layers, the depth and thickness of each layer shall be investigated and the liquefaction index shall be calculated using the following formula:

$$I_{lE} = \sum_{i=1}^{n} \left(1 - \frac{N_i}{N_{cri}}\right) d_i w_i \qquad (3.3.4)$$

where I_{lE}—liquefaction index.

n—total number of standard penetration test points in each bore within the depth of 15m under the ground surface.

N_i , N_{cri}—measured value and critical value of standard penetration resistance, in number of blow counts, at the ith point respectively; when the measured value is greater than the critical value, the latter should be used.

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

w_i—weighted function value of the ith soil layer (in m^{-1}), considering the effect of the layer depth of the unit soil layer thickness; when the depth of the midpoint of the layer is less than 5 m, a value of 10 shall be used; when it equals 15 m, a value of zero shall be used; when it is between 5 m and 15 m, the value shall be taken by linear interpolation.

3.3.5 For the subsoil with liquefaction-potential soil layers, its category of liquefaction shall be classified according to the liquefaction index as shown in Table 3.3.5.

3.3.6 Anti-liquefaction measures for subsoil shall be determined comprehensively considering the importance of the building, category of liquefaction of the subsoil, and other actual conditions. For the flat and uniform liquefaction-potential soil layer, measures shown in Table 3.3.6 may be selected. Untreated liquefaction-potential soil layer shall not be used as bearing layer of natural subsoil except for Type D buildings.

Table 3.3.5 Category of Liquefaction

Liquefaction Index	$0 < I_{le} \leqslant 5$	$5 < I_{le} \leqslant 15$	$I_{le} > 15$
Category of liquifaction	light	moderate	serious

Table 3.3.6 Anti-Liquefaction Measures

Building type	Category of liquefaction of subsoil		
	Light	Moderate	Serious
B	Layers, liable to settle during liquefaction, to be partially eliminated, or foundation and super-structure on the layers to be treated.	Layers, liable to settle during liquefaction, to be completely eliminated, or partially to be eliminated together with foundation and superstructure to be treated.	Lsyers, liable to settle during liquifaction, to be completely eliminated.
C	Foundation and super-structure to be treated, or measures may not be taken.	Foundation and super-structure to be treated, or other more strict measures to be taken.	Layers, liabe to settle during liquefaction, to be completely eliminated or partially to be eliminated together with foundation and superstructure to be treated.
D	Measures may not be taken	Measures may not be taken	Foundation and super-structure to be treated, or other low-cost measures to be taken.

3.3.7 Measures taken to eliminate all layers liable to settle during liquefaction shall comply with the following requirements:

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

2. When deep foundations are used, the depth of the foundation base embeded in the stable soil layer below the liquefaction depth shall not be less than 500 mm;

3. When a compaction method is used for strengthening (e.g. vibrating impact, vibrating compaction, sand pile compaction, and strong ramming), compaction shall be carried out down to the lower margin of liquefaction-potential depth, and the measured value of the standard penetration resistance of the compacted soil layer in

number of blow counts shall be greater than the corresponding critical value.

4. Excavate out the entire liquifaction-potential soil layers.

3.3.8 Measures taken to eliminate part of layers liable to settle during liquefaction should comply with the following requirements:

1. Excavation shall be carried out to a depth so that the liquefaction index of the subsoil shall be not greater than 4. For single foundations and strip footings, the depth of excavation shall not be less than 5m also under the base of the footing or not less than the width of the footing, whichever is greater;

2. In the range of the depth of excavation, the liquefaction-potential soil layers shall either be excavated or strengthened by compaction, so that the measured value of the standard penetration resistance of the treated soil layer, in number of blow counts, shall be greater than the corresponding critical value.

3.3.9 The following measures can be taken for treatment of the foundation and the superstructure to reduce the effect of liquefaction based on comprehensive considerations:

1. Select the appropriate buried depth of foundation.

2. Regulate the foundation base area to reduce the eccentricity of the foundation;.

3. Upgrade the integrality and rigidity of the foundation. For example, the use of caisson or raft footing, the use of footing in the shape of a cross, adding foundation ring beams or connecting beams;

4. Decrease the load; upgrade the integrality, uniformity, and symmetry of the superstructure; install rational settlement joints, and avoid the use of a structural configuration that is vulnerable to unequal settlement.

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

3.3.10 If, in the range of the main bearing layer of subsoil, there is a layer of soft cohesive soil, measures such as adoption of pile foundation, strenghtening of subsoil, or those specified in Article 3.3.9 shall be taken based on comprehensive consideration in the light of specific conditions.

3.3.11 If abandoned and filled rivers beds, hidden ditches, partially excavated or partially filled slopes, partial or non-uniform liquefaction-potential soil layers, or other soil layers that possess serious non-uniform distribution in plane of formation, lithology, and state that are obviously different from each other can not be avoided for a site, the geological, geomorphical and topographical features of layers shall be thoroughly investigated and appropriate measures shall be taken in the light of specific conditions.

3.3.12 If sloping bank of a river where sliding and ground-cracking may be induced during an earthquake or the boundary section of abandoned and filled river beds cannot be avoided for a site, relevant stabilization measures for the subsoil shall be taken.

4. Seismic action and seismic checking for structures

4.1 General

4.1.1 Seismic action on all types of building structure shall be considered according to the following principles:

1. Under ordinary circumstances, horizontal seismic actions may be considered and checked separately along the two main axial directions of the building structure; the horizontal seismic action shall be resisted totally by the corresponding lateral force-resisting elements;

2. For structures with braced lateral force-resisting elements, the horizontal seismic action in the direction of each lateral force-resisting element should be considered separately;

3. For structures with obvious asymmetric and non-uniform mass and stiffness distribution, the torsion effects caused by horizontal seismic action shall be considered;

4. For large-span structures, long-cantilevered structures, chimneys, and similar tall structures in regions of intensity Ⅷ or Ⅸ, and for high-rise buildings in regions of intensity Ⅸ, vertical seismic action shall be considered;

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

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

2. For building structures other than those as stated in the above clause, the response spectrum method for modal analysis should be used; for free-standing chimneys not higher than 100m , an approximate method in Chapter 11 of this code may be used;

3. For buildings with extremely irregular configuration, buildings of type A, and high rise buildings within the height range given in Table 4.1.2, a time-history analysis procedure should be used as an additional safeguard.

The number of acceleration records or synthesized acceleration curves for time-history analysis should be selected according to intensity, near- or far-earthquake, and site category. However, the base shear obtained from the time-history analysis shall be not less than 80% of that calculated by the method specified in clause 1 or 2 of this article.

Table 4.1.2 Building Height Range for Using Time-History Analysis Procedure

Intensity Ⅶ; Intensity Ⅷ at sites of category Ⅰ, Ⅱ	>80m
Intensity Ⅷ at sites of category Ⅲ, Ⅳ; and intensity Ⅸ	>60m

4.1.3 In the computation of seismic action, the representative value of gravity load of the building shall be taken as the sum of standard values of the weight of the structure and components plus the combination values of variable loads on the structure. Combination coefficients for different variable loads shall be taken from Table 4.1.3.

Table 4.1.3 Combination Coefficients

Types of variable load	Combination coefficient
Snow load	0. 5
Accumulated dust load on the roof	0. 5
Live load on the roof	no need to consider
Live load on the floor, based on actual conditions	1. 0
Live load on the floor, considered as equivalent uniform load: 1. Library stack room, file storage room 2. Other civil buildings	 0. 8 0. 5
Suspended load of cranes 1. Cranes with rigid hook 2. Cranes with flexible hook	 0. 3 no need to consider

Note: When the suspended load of cranes with rigid hook is comparatively large, the combination coefficient may be determined in the light of specific conditions.

4.1.4 Seismic effect coefficient of a building structure shall be determined from Figure 4.1.4, based on near- or far- earthquake, site category, and natural period of structure. Its lower limit shall be not less than 20% of the maximum value. In checking of cross sections for earthquake resistance, the maximum value of horizontal seismic effect coefficient shall be taken from Table 4.1.4-1.

Table 4.1.4 − 1 Maximum Value of Horizontal Seismic effect Coefficient for Seismic Checking Cross of Sections

Intensity	Ⅵ	Ⅶ	Ⅷ	Ⅸ
α_{max}	0.04	0.08	0.16	0.32

Table 4.1.4 − 2 Characteristic Period Value(s)

Near- or far-earthquake	Site category			
	Ⅰ	Ⅱ	Ⅲ	Ⅳ
Near-earthquake	0.20	0.30	0.40	0.65
Far-earthquake	0.25	0.40	0.55	0.85

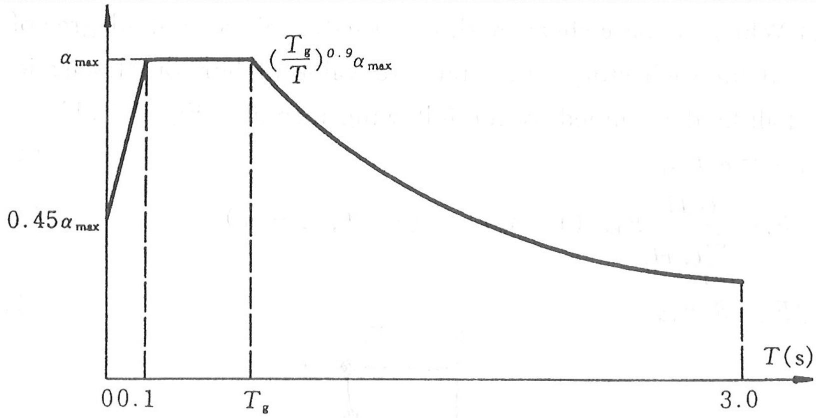

Fig.4.1.4 Seismic Effect Coefficient Curve

α—seismic effect coefficient

α_{max}—maximum value of seimic effect coefficient

T—natural vibration period of structure

T_g—characteristic period taken from Table 4.1.4-2 , based on the site category and near- or far-earthquake.

4. 1. 5 Natural vibration period of a structure may be determined by theoretical computation or by empirical formula. When it is determined by theoretical computation, calculation model and elastic stiffness of the structure corresponding to those used in the seismic checking shall be used, and the evaluated result shall be reduced appropriately considering the effect of non-structural members; when determined by empirical formula, it shall comply with corresponding practical conditions.

4.1.6 Checking of structures for earthquake resistance shall comply with the following requirements:

1. Seismic checking of cross section may not be carried out for: (1) buildings in a region of intensity Ⅵ (except comparatively high buildings and tall structures built on a site of category Ⅳ), and (2) structures that need not be checked according to related chapters of this code. However, the relevant requirements for seismic measures shall be satisfied;

2. For structures other than those as stated in clauses 1 and 3 of this article, seismic checking of cross section shall be conducted according to section 4.4.

3. For structures meeting the requirements of section 4.5, not only the seismic checking in cross section shall be conducted by the provisions of section 4.4, but also the corresponding checking of deformation should be carried out.

4.2 Calculation of horizontal seismic action

4.2.1 When the base shear method is used, only a single-degree of freedom may be considered for each story; the standard value of horizontal seismic action of the structure shall be determined by the following formulas (Fig. 4.2.1):

$$F_{Ek} = \alpha_1 G_{eq} \quad (4.2.1-1)$$

$$F_i = \frac{G_i H_i}{\sum_{j=1}^{n} G_j H_j} F_{Ek} (1 - \delta_n) \quad (i = 1, 2 \cdots n) \quad (4.2.1-2)$$

$$\Delta F_n = \delta_n F_{Ek} \quad (4.2.1-3)$$

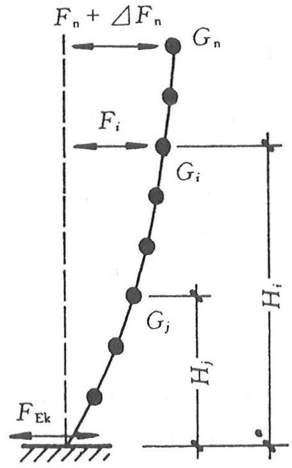

Fig. 4.2.1 Sketch for Computation of the Horizontal Seismic Action

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

α_1—horizontal seismic effect coefficient corresponding to the fundamental period of the structure, which shall be determined using Article 4.1.4. For multi-story masonry buildings and multi-story brick buildings with framed first story or inner frames, the maximum value of horizontal seismic effect coefficient may be taken;

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

F_i—standard value of horizontal seismic action applied on mass.

G_i, G_j—representative values of gravity load concentrated at the masses of i and j respectively, which shall be determined by Article 4.1.3.

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

δ_n—additional seismic action coefficient at the top of the building; for multi-story reinforced concrete buildings, it may be taken using Table 4.2.1 ;for multi-story brick buildings with inner frames, a value of 0.2 may be used;no need to consider for other buildings.

$\triangle F_n$—additional horizontal seismic action applied at top of the building.

Table 4.2.1 Additional Seismic Action Coefficient at Top of the Building

T_g(s)	$T_1>1.4\,T_g$	$T_1\leqslant1.4T_g$
≤0.25	$0.08T_1+0.07$	
0.3~0.4	$0.08T_1+0.01$	No need to consider
≥0.55	$0.08T_1-0.02$	

Note: T_1—foundamental period of the structure

4.2.2 When the response spectrum method is used for modal analysis, if the torsional effect of a structure is not considered, the seismic action and the action effect may be calculated in accordance to the following requirements:

1. The standard value of horizontal seismic action on mass i of the structure, corresponding to jth mode, shall be determined by the following formulas:

$$F_{ji}=\alpha_j\gamma_jX_{ji}G_i \quad (i=1,\ 2\cdots n;\ j=1,\ 2,\ \cdots m) \qquad (4.2.2-1)$$

$$\gamma_j=\sum_{i=1}^{n}X_{ji}G_i/\sum_{i=1}^{n}X_{ji}^2G_i \qquad (4.2.2-2)$$

where F_{ji}—standard value of horizontal seismic action of mass i corresponding to *mode j*;

α_j—seismic effect coefficient corresponding to the natural period of jth mode of the structure, it shall be determined by article 4.1.4;

X_{ji}—relative horizontal displacement of mass i corresponding to jth mode

γ_j—mode participation factor of jth mode.

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

$$S=\sqrt{\sum S_j^2} \tag{4.2.2-3}$$

where S—horizontal seismic action effect

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

4.2.3 When the base shear method is used, the effect of seismic action of penthouse, parapet, and chimney on the roof should be multiplied by an amplification factor of 3.0; the increase part of the effect should be assigned to the roof, not to the lower part of the structure; when modal analysis method is used, the projecting part may be considered as a mass; the amplification factor of seismic action effect of the projecting sky-light frame of a single-story factory building shall comply with relevant requirements of Chapter 8 of this code.

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

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

2. For buildings with flexible diaphragms, such as wood floors and roofs, the distribution may be carried out in proportion to the representative values of gravity load acting on the floor area, braced by the lateral force-resisting members;

3. For buildings with ordinary prefabricated reinforced concrete floors and roofs, the average value of the results obtained from the above-mentioned two methods of distribution may be used;

4. When the effect of spatial action, deformation of diaphragms, elasto-plastic deformation of the wall, and torsional effect are considered, the results of distribution may be appropriately regulated in accordance with relevant requirements

specified in this code.

4.2.5 When considering the torsional effect, the building may be modeled with three degrees of freedom, including two orthogonal horizontal displacements, and one angular rotation for each floor level of the building. The seismic action and action effect may be calculated by the modal analysis method shown below. The simplified method for the determination of the seismic action effect may be used, provided reliable basis is available.

1. Standard value of horizontal seismic action applied to the ith floor, corresponding to the jth mode of natural vibration of the structure shall be determined by the following equations:

$$\left.\begin{aligned} F_{xji} &= \alpha_j \gamma_{tj} X_{ji} G_i \\ F_{yji} &= \alpha_j \gamma_{tj} Y_{ji} G_i \\ F_{tji} &= \alpha_j \gamma_{tj} r_i^2 \varphi_{ji} G_i \end{aligned}\right\} \quad (i = 1, 2 \cdots n; j = 1, 2 \cdots m) \qquad (4.2.5-1)$$

where F_{xji}, F_{yji}, F_{tji}—standard values of horizontal seismic action applied to the i th floor, corresponding to the j th mode in the directions of x, y and angular rotation respectively;

X_{ji}, Y_{ji},—relative horizontal displacements of the center of mass of the i th floor, corresponding to the j th mode in the directions of x and y respectively;

φ_{ji}—relative angular rotation of the i th floor corresponding to the jth mode;

r_i—radius of gyration of mass for the i th floor, which is the square root (positive value) of the quotient obtained by dividing the rotational moment of inertia of the mass in the i th floor by the mass of the floor;

γ_{tj}—mode participation factor of the j th mode, considering torsion effect, which may be determined by the following formulas:

When only the seismic action in the x direction is considered:

$$\gamma_{tj} = \sum_{i=1}^{n} X_{ji} G_i / \sum_{i=1}^{n} (X_{ji}^2 + Y_{ji}^2 + \varphi_j^2 \gamma_i^2) G_i \qquad (4.2.5-2)$$

When only the seismic action in the y direction is considered:

$$\gamma_{tj} = \sum_{i=1}^{n} Y_{ji} G_i / \sum_{i=1}^{n} (X_{ji}^2 + Y_{ji}^2 + \varphi_j^2 \gamma_i^2) G_i \qquad (4.2.5-3)$$

2. The effect of seismic action considering torsion effect shall be determined by the following equations:

$$S = \sqrt{\sum_{j=1}^{m} \sum_{k=1}^{m} \rho_{jk} S_j S_k} \tag{4.2.5-4}$$

$$\rho_{jk} = \frac{0.02(1+\lambda_T)\lambda_T^{1.5}}{(1-\lambda_T^2)^2 + 0.01(1+\lambda_T)^2\lambda_T} \tag{4.2.5-5}$$

where S_j, S_k—action effects caused by the seismic action of the j th and k th modes respectively (the first 9 to 15 modes taken may be sufficient);

ρ_{jk}—coupling coefficient of the j th and the k th modes;

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

4.2.6 In the seismic computation of a structure, in general, the interaction of subsoil and structure may be ignored. However, for reinforced concrete high-rise buildings with caisson or a relatively rigid raft foundation on sites of categories Ⅲ or Ⅳ, if the subsoil-structure interaction is considered, the horizontal seismic action may be reduced by 10% - 20%, depending on the type of structure and the category of the site, based on the assumption of a rigid subsoil condition. The story-drifts of the building may be determined on the basis of the reduced floor shear force.

4.3 Calculation of vertical seismic action

4.3.1 For chimneys and similar tall structures as well as high-rise buildings, the standard value of vertical seismic action shall be determined by the following equations (Fig. 4.3.1); the effects of vertical seismic action at the floor level may be distributed in proportion of the representative value of gravity loads acting on the members.

$$F_{Evk} = \alpha_{v,max} G_{eq} \tag{4.3.1-1}$$

$$F_{vi} = \frac{G_i H_i}{\sum G_j H_j} F_{Evk} \tag{4.3.1-2}$$

where F_{Evk}—standard value of the total vertical seismic actions applied to the structure;

F_{vi}—standard value of vertical seismic action at the level of mass i;

$\alpha_{v,max}$—maximum value of vertical seismic effect coefficient, which may be taken as 65% of the maximum value of the horizontal seismic effect coefficient;

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

4.3.2 For a flat network roof structure and for trusses with a span greater than 24m, the standard value of vertical seismic action may be taken as the product of the representative value of the gravity load and the coefficient of vertical seismic action. Values for the coefficient of vertical seismic action may be obtained using Table 4.3.2 .

Table 4.3.2 Coefficients of Vertical Seismic Action

Type of roof structure	Intensity	Site category		
		I	II	III, IV
Flat network structure and steel truss	VIII	no need to consider	0.08	0.10
	IX	0.15	0.15	0.20
Reinforced concrete roof truss	VIII	0.10	0.13	0.13
	IX	0.20	0.25	0.25

4.3.3 For long-cantilver and other large-span structures in regions of intensity VIII and IX, the standard value of the vertical seismic actions may be taken as 10% and 20% of the representative value of gravity load of the structure or the member respectively.

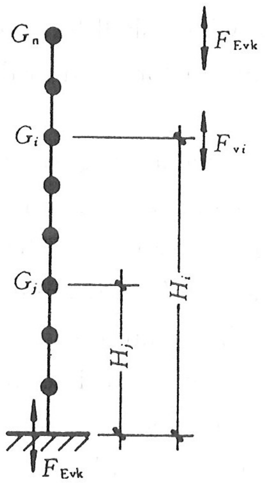

Fig.4.3.1 Schematic Diagram for the Computation of Vertical Seismic Action

4.4 Seismic checking for cross section

4.4.1 Fundamental combination of effects of seismic action and other loads on structural members shall be calculated by the following equation:

$$S = \gamma_G C_G G_E + \gamma_{Eh} C_{Eh} E_{hk} + \gamma_{Ev} C_{Ev} E_{vk} + \psi_w \gamma_w C_w w_k \quad (4.4.1)$$

Table 4.4.1 Partial Coefficient of Seismic Actions

Seismic actions	γ_{Eh}	γ_{Ev}
Considering horizontal seismic action only	1.3	need not consider
Considering vertical seismic action only	need not consider	1.3
Considering both horizontal and vertical seismic actions	1.3	0.5

where S—design value of combination of inner forcesin a structural member, including design value of combination of bending moment, axial force and shear force;

γ_G—partial coefficient of gravity load, a value of 1.2 shall be used in ordinary conditions; when the effect of gravity load is favorable to the loading capacity of the member, a value of 1.0 may be used;

γ_{Eh}, γ_{Ev}—partial coefficients for horizontal and vertical seismic action respectively, which shall be taken using Table 4.4.1;

γ_w—partial coefficient for wind load, for which a value of 1.4 shall be used;

G_E—representative value of gravity load, which shall be taken from Article 4.1.3, but when a crane is available , the standard value of the hanging gravity load of the crane shall be included;

E_{hk}—standard value of horizontal seismic action, which shall be obtained by the requirements of section 4.2 ;

E_{vk}—standard value of vertical seismic action, which shall be abtained by the requirements of section 4.3 ;

w_k—standard value of wind load;

ψ_w—coefficient for combination value of wind load, which need not be considered for ordinary structures; for chimneys, water towers with considerable height and high-rise buildings, 0. 2 may be used;

C_G, C_{Eh}, C_{Ev}, C_w—action effect coefficients for gravity load, horizontal seismic action, vertical seismic action, and wind load respectively; in ordinary circumstances, they shall be determined by the national standard, "Uniform standard for the Design of Building Structures, GBJ 68-84". When the requirements are available in various chapters of this code, they shall be multiplied by corresponding amplification factors and adjustment factors.

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

Table 4.4.2 Adjusting Coefficient for Load-bearing Capacity (γ_{RE})

Material	Type of Structural Member	Stress type	γ_{RE}
Steel	Column	Eccentric compression	0.7
	Column braces in factory buildings with steel structural systems		0.8
	Column braces in factory buildings with reinforced concrete structural systems		0.9
	Welded-joint of members		1.0
Masonry	Seismic shear walls with constructional columns or core columns at both ends	Shear	0.9
	Other seismic shear walls		1.0
Reinforced concrete	Beam	Bending	0.75
	Columns with axial compression ratio<0.15	Eccentric compression	0.75
	Columns with axial compression ratio ≥0.15	Eccentric compression	0.80
	Seismic shear wall	Eccentric compression	0.85
	All types of members	Shear, eccentric tension	0.85

4.4.2 In the checking of cross-section of structural members for seismic resistance, the following expression shall be used for design:

$$S \leq R/\gamma_{RE} \tag{4.4.2}$$

where γ_{RE}—seismic adjusting coefficient for load-bearing capacity of the structural member, which shall be obtained by using Table 4.4.2 except that requirements are available in other chapters of this code;

R—design value of load-bearing capacity of the structural member, which shall be calculated by the pertinent code provisions.

4.4.3 When vertical seismic action is only considered, the seismic adjusting coefficient may be taken as 1.0 for all structural members.

4.5 Seismic checking for deformation

4.5.1 For frames (including frames with infilled walls) and frame-shear wall structures (including frames with soft stories) the seismic deformation of a structure should be checked for the action of earthquakes frequently ocurred with intensity lower than the fortification intensity of the region. The inter-story elastic displacement shall comply with the following requirement:

$$\Delta u_e \leqslant [\theta_e] H \qquad (4.5.1)$$

where Δu_e—inter-story elastic displacement caused by the standard value of the frequently occured earthquake action. When calculating, the maximum value of horizontal seismic effect coefficient shall be taken from Table 4.1.4 - 1; all partial coefficients of various actions shall be taken as 1.0, and elastic stiffness may be used for reinforced concrete members;

$[\theta_e]$ —limit value of inter-story elastic displacement rotation which may be taken in accordance with Table 4.5.1;

H—height of story

Table 4.5.1 Limit Value of Inter-Story Elastic Displacement Rotation

Type of structure	Conditions	$[\theta_e]$
Frame	Considering anti-lateral action of filled brick walls	1/550
	Others	1/450
Frame-shear wall	Public buildings with stringent requirements for ornamental decorations	1/800
	Others	1/650

4.5.2 The following structures should be checked for seismic deformation at their weak stories (or locations) under the action of an expected seldomly occurred earthquake with an intensity higher than the fortification intensity of the region:

1. Transversal bents of single-story factory buildings with tall reinforced concrete columns and large spans situated in regions of intensity Ⅷ for site category of Ⅲ and Ⅳ, or of intensity of Ⅸ for all sites;

2. Framed structures and brick wall buildings with a frame in the first story, situated in regions of intensity Ⅶ to Ⅸ, when the yield strength coefficient of the story is less than 0.5.

3. Reinforced concrete structures of Type A building.

Note: Yield strength coefficient of a storey is the ratio between the story shear bearing capacity, calculated by standard value of the actual strength of reinforcement and other material, and the story elastic seismic shear force. For bent column, it refers to the ratio between the bending bearing capacity of the normal cross section, calculated by the standard value of the actual cross section area of reinforcement and the strength of the material as well as the axial force, and the elastic seismic moment.

4.5.3 Elasto-plastic deformation of a weak story (or location) of a structure under the action of a seldomly occurred earthquake may be calculated by the following methods:

1. For framed structures and those with filled walls that do not exceed 12 stories and with no abrupt change of story stiffness and single-story factory buildings with reinforced concrete column, the simplified method in Article 4.5.5 may be used.

2. For buildings with more than 12 stories and Class A buildings the time-history analysis method may be used.

4.5.4 In the computation of the standard value of seldomly occurred earthquake action, the horizontal seismic effect coefficient shall be taken from Figure 4.1.4, with maximum values taken from Table 4.5.4:

Table 4.5.4 Maximum Value of Horizontal Seismic Effect Coefficient under the Action of Seldomly Occured Earthquake

Intensity	Ⅶ	Ⅷ	Ⅸ
α_{max}	0.50	0.90	1.40

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

1. The weak story (or location of a structure) may be identified as follows:

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

the weak story.

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

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

2. The inter-story elasto-plastic displacement may be calculated by the following equations:

$$\Delta u_p = \eta_p \Delta u_e \quad (4.5.5-1)$$

or

$$\Delta \mu_p = \mu \Delta u_y = \frac{\eta_p}{\xi_y} \Delta u_y \quad (4.5.5-2)$$

Table 4. 5. 5 Amplification Factor for Elasto-Plastic Displacement

Type of structure	Total number of stories or locations	ξ_y			
		0.5	0.4	0.3	0.2
Multi-story structure with uniform elevation	2-4	1.30	1.40	1.60	2.10
	5-7	1.50	1.65	1.80	2.40
	8-12	1.80	2.00	2.20	2.80
Single-story factory building	Upper part of column	1.30	1.60	2.00	2.60

where Δu_p—inter-story elasto-plastic displacement;

Δu_y—inter-story yield displacement;

μ—story ductility factor;

Δu_e—inter-story displacement induced by the action of seldomly occurred earthquake, calculated by elastic analysis;

η_p—amplification factor for elasto-plastic displacement; when the yield strength coefficient of the weak story(location)is not less than 80% of the average value of coefficients of the neighboring stories(location), η_p may be taken from Table 4.5.5, but when the yield strength coefficient is not more than 50% of the above-mentioned average value, the corresponding values in the Table should be multiplied by 1.5. In other conditions, the factor may be determined by interpolation;

ξ_y—story yield strength coefficient.

4. 5. 6 The inter-story elasto-plastic displacement in the weak stories (locations) of a structure shall comply with the following requirement:

$$\Delta u_p \leqslant [\theta_p] H \qquad (4.5.6)$$

where H—height of the weak story (location) or the height of the upper column in single-story factory building;

$[\theta_p]$ —limit value of inter-story elasto-plastic displacement rotation, which can be taken from Table 4. 5. 6; for frame structures, when axial compression ratio is less than 0. 4, values in Table 4. 5. 6 can be increased by 10%; when the upper limit values of the steel ratio of stirrups in Table 6.3.10 in this code are used for the entire height of the columns, the values in Table 4.5.6 can be increased by 20%, but the total increase should not be greater than 25%.

Table 4.5.6 Inter-Story Elasto-Plastic Displacement Rotation

Type of structure	$[\theta_p]$
Bent with reinforced concrete columns in single-story strucure	1/30
Frame structure and frame structure with filled walls	1/50
Frame in the first story of brick building	1/70

Editorial Note The remainder of the current code has been organized as follows.