CODE 2/1997

REPUBLIC OF IRAQ

IRAQI SEISMIC CODE REQUIREMENTS FOR BUILDINGS

BUILDING RESEARCH CENTRE

General Commission for Industrial Research and Development
Ministry of Industry and Minerals

APROVED BY CENTRAL ORGANIZATION FOR STANDARDIZATION AND QUALITY CONTROL

IRAQI SEISMIC CODE REQUIREMENTS FOR BUILDINGS
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CHAPTER 1 - SCOPE AND PURPOSE

1.1- This code provides design and construction requirements for an earthquake resistant buildings, towers, chimneys and similar structures.

The method specified do not cover nuclear power plants, large dams, and similar installations which require special site and structural investigations.

1.2- The purpose of an earthquake resistant design is :
- to prevent loss of life and human injury.
- to ensure continuity of vital services.
- to minimize damage to property.

CHAPTER 2 - PRINCIPLES OF EARTHQUAKE - RESISTANT DESIGN

2.1- Basic Concept

The basic concept of the requirements provided in this code is that complete protection against total damage is not economically feasible for all types of buildings and structures. This concept is fulfilled by the following criteria:

a) The structure should withstand, without any structural and non-structural damage, the effects of slight seismic motion.

b) The structure should withstand, with limited non-structural damage and limited non-linear behaviour of structural members, the effects of moderate seismic motion (design earthquake).

c) The structure should not collapse under sever or maximum expected earthquake.

2.2- Structural Layout

For better earthquake resistance, it is necessary that buildings and structures have simple forms, in both plan and elevation, and of structural elements which resist horizontal seismic actions be arranged in such a way that torsional effects are minimized. Non symmetrical distribution of volumes, masses and stiffnesses in buildings should be avoided in order to control torsional effects.

Generally, the design and construction of buildings and structures with irregular or complicated layout shall be avoided due to the potential occurrence of critical additional stresses in the regions of discontinuities. When these requirements cannot be met, the structure shall be separated by seismic joints, each part having an adequate shape and a proper distribution of volumes, masses and rigidities. Otherwise proper considerations of irregularities should be taken, such as by performing appropriate dynamic analysis.

2.3- Structural System

The structural system should be clearly defined so that rational analysis can be applied. In computing earthquake response of a building, the influence of not only structural systems, but also non-structural elements (infill walls, partitions, windows, etc.) should be considered as well.

2.4- Ductility

The structural system and its structural elements should have adequate strength and ductility for the applied seismic actions. Structural elements which have sufficient ductility are capable of absorbing energy. Special attention should be given to the brittleness of structural elements such as shear failure, joint fracture, buckling, bond failure and anchorage failure.

2.5- Deformations

The deformations of the structure under seismic actions should be limited. Generally, there are two kinds of deformations to be controlled: the inter-storey drift (relative lateral displacement within a storey) and the absolute lateral displacement relative to the base.

2.6- Site Selection

The construction sites should be properly selected in accordance with the microzonation criterion. When available sites with active faults, sloping soil profiles, undesirable settlement properties and possible liquefaction, etc. should be carefully evaluated.

2.7- Seismic Joints

Seismic joints should be provided to separate various parts of buildings and structures, in particular with different dynamic characteristics, in order to allow them to vibrate independently. Seismic joints are provided for buildings with irregular plans and for buildings of non-uniform heights. The width of the joints is determined in such a way that during the earthquake the parts of the building separated by the joints do not affect each other by collision. For rigid buildings with height up to 15m, the minimum width of the seismic joints is 25mm in seismic zones I and II and 40mm in seismic zone III. For buildings and structures over 15m in height, and for flexible structures, the joint's width is determined by the following formula:

$$d \geq (\delta_1 + \delta_2 + 15) \quad \text{.................................} \quad (2\text{-}1)$$

But not less than :

25mm for seismic zones I, II
and 40mm for seismic zone III

Where:

d - width of joint (mm).

δ_1 , δ_2 - total lateral displacements of the two parts of the building under the seismic action (see clause 4.8). For buildings of Class I, they should be determined by dynamic response analysis.

2.8- Floor Structures

Floor structures should be designed in such a way to behave as rigid horizontal diaphragms monolithically joined in a structural system, which should transmit lateral effects to the vertical structural system. For structures not meeting the above requirement, they shall be treated as deformable structural elements in the analysis.

CHAPTER 3 - EVALUATION OF SEISMIC ACTIONS

3.1- General

3.1.1- The seismic analysis of structures shall take the dynamic properties of the structure into consideration either by dynamic analysis or by equivalent static analysis. A dynamic analysis is highly recommended for specific structures such as slender high-rise buildings and structures with irregularities of geometry or mass distribution or rigidity distribution.

Ordinary structures may be designed by the equivalent static method using conventional linear elastic analysis. Appropriate post-elastic performance shall be provided by adequate choice of structural system and ductile detailing. Non-linear methods of analysis should be employed to verify the sequence of inelastic behavior and the formation of collapse mechanism.

Note: If it is essential that services, e.g. mechanical and electrical equipment and pipings, retain their functions during and after a severe earthquake, the design of these services should preferably be done using dynamic analysis procedures based on the earthquake response of the structure which supports them.

3.1.2- Seismic design forces shall be applied at points where masses are assumed to be concentrated.

The actual mass distribution may be substituted by a distribution which simplifies the analysis without affecting appreciably the final results (mass concentration at floor levels in multistorey buildings; mass concentrations at an adequate number of equidistant levels in tall constructions like chimneys, towers, etc.).

For structural design, the directions of seismic actions in horizontal plane should be taken at least in two orthogonal directions.

Cantilevers and structures in which vertical seismic effects are significant, should be analyzed in the vertical direction taking into account these effects.

3.1.3- The masses used for analysis have to correspond to the dead and probable live loads.

For different Classes of buildings as defined in 3.2.2, the probable live load shall be taken as 50% for structures of Class II and 25% for structures of Class III and Class IV, of floor live loads determined by the existing regulations.

For structures with significant live load such as (Warehouses, Silos, Libraries, Storage rooms and similar structures), the seismic design forces should be determined for the most unfavorable combination of maximum, and / or minimum actual loading.

Live loads of cranes should not be considered for determination of seismic design forces.

Total weight of permanent equipments should be included. Snow loads may be considered in the calculations at 50% of its normal value.

Wind load should not be considered in combination with seismic actions.

3..2- Evaluation of Seismic Design Forces for Equivalent Static Analysis

The total horizontal seismic design force acting on buildings and structures shall be determined according to the following formula, but shall not be less than (0.02W).

$$V = Z.I.S.K.W \quad \cdots\cdots\cdots\cdots \quad (3\text{-}1)$$

Where:

- V - Total unfactored horizontal seismic design force.
- Z - Seismic hazard zoning coefficient (clause 3.2.1).
- I - Importance factor related to the use of structure (clause 3.2.2).
- S - Dynamic coefficient related to soil category (clause 3.2.3).
- K - Structural system coefficient, specified for various types of structures (clause 3.2.4).
- W - Total weight of the structure including permanent and probable live load (clause 3.1.3 and clause 3.2.5).

3..2.1- Seismic Hazard and Zoning Coefficient - Z

The evaluation of seismic hazard in different seismic areas for the design of buildings and structures shall be performed according to the seismic zoning map of Iraq (Appendix A).

The value of coefficient Z is as follows:

Table 3.1 - Zoning Coefficient Z

Zone	Z
I	0.05
II	0.07
III	0.09

- The buildings and structures located in zone 0 need not be designed to satisfy the requirements of this code, except for buildings and structures of Class I. For the design of buildings and structures of Class I, (clause 3.2.2), it is necessary to determine the seismicity of the site by detailed investigations to estimate the design and the maximum expected earthquakes on the basis of the regional and local seismic hazard investigations.

3..2.2- Importance Factor - I

Depending on how possible a damage may affect public safety, and according to the importance of buildings and structures, different requirements shall be imposed for safety against earthquake. For this reason, buildings and structures are classified as follows:

Class I :

This Class includes all those structures which are of special importance to the public, and which must, consequently, not only withstand an earthquake but remain operational after its occurrence. The following types of structures form part of this Class:

- Structures containing toxic or flammable materials and similar installations and large dams, which require additional safety precautions.

- Hospitals and other medical buildings having surgery and emergency treatment facilities.

- Installations dealing with the consequences of disasters, e.g. fire brigades and other vital civil defence centers.

- Buildings and structures related to stand by power generating equipments for essential facilities.

- Structures for communications and tele-communications and other facilities required for emergency response.

- Tanks or other structures containing, housing or supporting, water or other fire-suppression materials or equipments required for the protection of hazardous facilities.

Class II :

This Class includes buildings and structures of high importance to community, for which high level of reliability is required. The following structures form part of this Class:

- Water supply installations, water reservoirs and silos.

- Oil and gas installations, chemical plants, refineries, and other lifeline systems.

- Structures and installations related to power generating stations.

Class III :

This Class includes buildings and structures in which relatively large number of people are likely to congregate, and which are likely to be endangered to a great degree in the event of earthquake. The following buildings form part of this Class:

- High-rise buildings over 6 stories.

- Public buildings of high occupancy rate (greater than 300 persons), like mosques, sport buildings, cinema-halls, theatres, schools, hospitals and health facilities, industrial buildings, museums, libraries, and similar cultural buildings.

Class IV :

This Class includes buildings and structures in which large congregations of people are not anticipated. The structures listed below form part of this Class:

- Residential buildings, restaurants, warehouses, public buildings, industrial buildings and all structures having occupancies or functions not classified in Classes I, II and III .

The value of the importance factor (I) for buildings and structures of the described Classes is according to Table 3.2 .

Table 3.2 - Importance Factor - I

Classes of Buildings - Structures	Importance Factor - I
Class I	1.50
Class II	1.25
Class III	1.00
Class IV	0.75

3.2.3- Dynamic Coefficient - S

The dynamic coefficient (S) shall be determined according to diagrams shown in Fig.1 depending on the type of soil profile as specified in Table 3.3 .

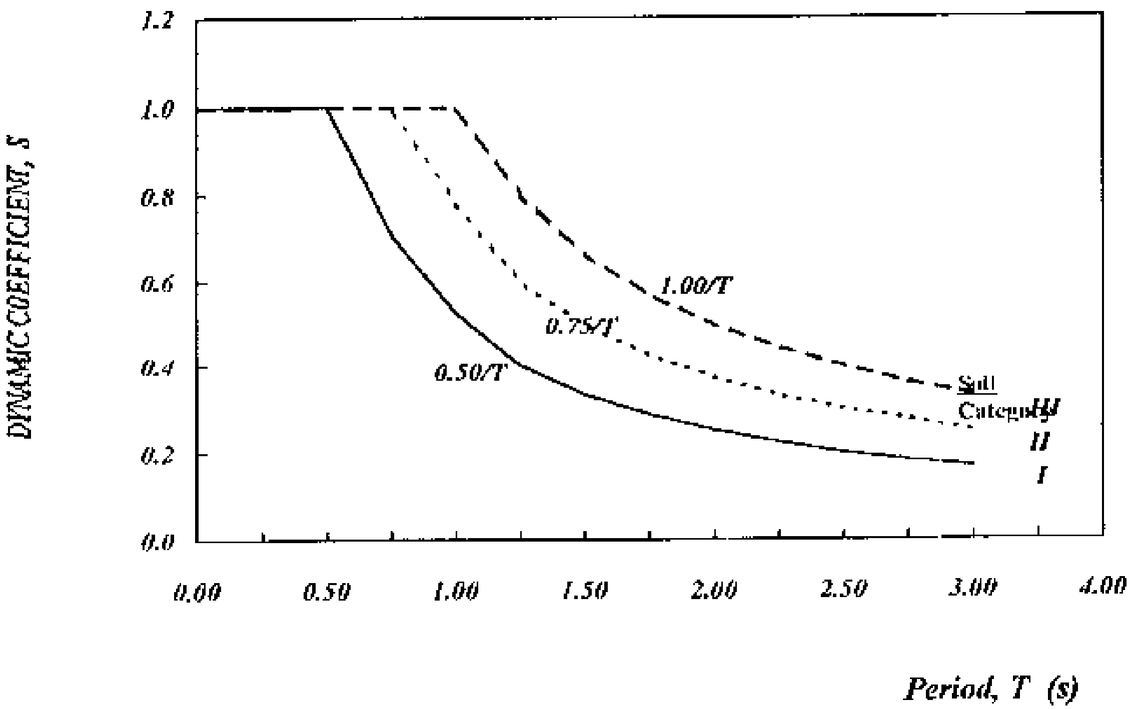

Fig.1 Site-Dependent Normalized Response Spectra

The fundamental period of vibration (T) should be determined using the methods of structural dynamics. In the absence of such calculations the following empirical formula may be used:

$$T = \frac{0.09\ H}{\sqrt{D}} \quad \cdots\cdots\cdots\cdots \quad (3\text{-}2)$$

For buildings in which lateral force resisting system consists of moment resisting space frames capable of resisting 100% of the applied lateral forces, (T) should be determined by the formula:

$$T = 0.10\ N \quad \cdots\cdots\cdots\cdots \quad (3\text{-}3)$$

Where:

T - Fundamental period of vibration of the structure in the direction under consideration (seconds).
H - Height of building from ground level (m).
D - The dimension of building in direction parallel to the applied forces (m).
N - Total number of stories.

The influence of local soil conditions should be taken into account when determining seismic effects on buildings and structures by means of dynamic response spectrum coefficient depending on the category of ground upon which the building is to be constructed. The category of soil should be determined according to the classification given in Table 3.3 on the basis of the results of geotechnical investigations of the construction site, of engineering, geological, and hydrogeological data, and of geophysical and other investigations of the soil profiles.

3.2.4- Structural system coefficient - K

Structural system coefficient (K), takes into account the ductility of the structure, the capacity of stress redistribution, the damping characteristics, and the supplementary strength capacity due to the effects that have not been considered in the design. The structural system coefficient (K) depending on the type of the structure should be determined from Table 3.4 .

Table 3.3 - Categories of Soil Profiles

Category of Soil	Characteristics of Soil Profile	Predominant Period of Soil Profile - T_s (Sec.)
I	Rock or rock like grounds (crystalline,shell-like and carbonate rocks; limestone, marl stone, well-cemented conglomerate, and similar rock - like material), very dense and hard soil deposits characterized by shear wave velocities $V_s \geq 500$ m/s of thickness less than 60m consisting of stable layers of gravel, sand or stiff clay underlayed by firm and stable geological formation.	$T_s < 0.50$ sec.
II	Dense to medium dense soil deposits of thickness not more than 60m, as well as very dense and hard deposits of thickness over 60m consisting of stable layers of gravel, sand and stiff to medium stiff clays overlaying firm geological formation.	0.50 sec. $\leq T_s \leq 0.75$ sec.
III	Deposits of low density and soft soil deposits of thickness greater than 10m consisting of loose gravels, saturated loose to medium dense sands, silty sands (from soft - plastic to flow plastic), plastic clays, organic soft soils, hydraulic - fill, and other soft and loose manually back filled soils with or without sandy or other cohesionless materials.	$T_s \geq 0.75$ sec.

Table 3.4 - Structural System Coefficient - K

Type No.	Type of the Structure	K
1	Buildings with moment resisting space frames with high ductility designed elements.	0.85
2	a) Buildings with ductile moment resisting space frames. b) Buildings with dual system consisting of ductile frames and reinforced concrete structural walls in both directions.	1.00
3	a) Buildings with reinforced concrete structural walls in both directions. b) Buildings with braced space frames.	1.30
4	a) Masonry buildings strengthened with vertical reinforced concrete columns and horizontal belts. b) Buildings with reinforced masonry bearing walls. c) Slim structures with small damping such as chimneys, water towers, etc.	1.60
5	a Buildings with flexible (soft story) or with an abrupt change in their structural rigidity. b) Unreinforced masonry buildings with plane concrete walls.	2.00

3.2.5- **Total weight of buildings and structures - (W)** shall be considered as the weight on the top of the foundation including probable live load (clause 3.1.3). For structures with rigid basement storey, the weight (W) shall be considered above that storey.

3.2.6- Influence of soil and foundation conditions

a) Construction of buildings and structures in soils susceptible to dynamic instability (such as, loose fine sands, soft silt and other soils liable to liquefaction, land and rock slide areas, faulting sites, and sites of expected excessive settlement) should be avoided. However if such soil conditions are unavoidable, the design and construction of buildings and structures should be based on detailed field and laboratory dynamic investigations of the foundation materials.

b) For generally unfavorable soil conditions sufficiently rigid foundations should be provided taking into consideration the effects of non-linear deformations of soil below the entire foundation area.

c) Attention should be paid to the need for controlling the deformation of the foundations and their influence upon the entire structural system of buildings and structures.

d) Foundations should be designed so that during seismic actions excessive differential settlements will be avoided.

e) The subgrade below the entire area of the building should preferably be of the same type of soil. Wherever this is not possible, suitably located joints should be provided.

f) For each structural unit the foundations should be at the same level.

g) Isolated footings shall be connected by tie beams in both orthogonal directions. For strip foundations the tie beam shall be provided in the perpendicular direction.

h) In the case of pile foundations, individual racking piles used as rigid horizontal bearing may produce unfavorable static system. For this reason it is advisable to use vertical piles only.

3.3- Distribution of Seismic Forces

The total horizontal seismic design force V should be distributed over the height of the building in accordance with the following formula:

$$V_i = V \cdot \frac{W_i \; H_i}{\sum_{j=1}^{n} W_j \; H_j} \qquad \text{(3-4)}$$

Where:

V_i	- Horizontal seismic design force in i-th level.
W_j, W_i	- Weight of i-th and j-th floor.
H_i, H_j	- Height of i-th and j-th floor from the top of the foundation.
n	- Total number of levels.

For buildings and structures with more than five levels, 0.15V shall be considered to be concentrated at the top level while the remaining 0.85V shall be distributed in accordance with the above formula.

3.3.1- When dynamic response analysis is required, clause 3.3 is not mandatory.

The distribution of seismic design forces in structures which have highly irregular shapes, i.e., large difference in lateral resistance or in stiffness between adjacent stories, or other unusual structural features, should be determined by methods of dynamic analysis.

3.3.2- For buildings and structures, where vertical seismic force effect can be critical (namely cantilevers, prestressed members, or horizontal members with clear spans greater than 20m), separate control to the vertical seismic influence shall be performed considering the relevant vertical seismic design force R determined by the formula:

$$R = 0.7\ Z\ I\ S\ K\ W_p \quad \cdots\cdots\cdots \quad (3\text{-}5)$$

Where:

R	- Total vertical seismic design force.
Z	- Seismic hazard zoning coefficient (clause 3.2.1).
I	- Importance factor (clause 3.2.2).
S	- Dynamic coefficient related to soil category (clause 3.2.3).
K	- Structural system coefficient (clause 3.2.4).
W_p	- Weight of parts under consideration (clause 3.1.3 and clause 3.2.5).

This force (R) shall be considered in addition to all other relevant loads except wind.

3.3.3- The seismic design forces in any horizontal direction shall be distributed to the various elements of the lateral force resisting system proportionally to their stiffness, considering the rigidity of the horizontal bracing system or diaphragm.

3.3.4- Horizontal Torsional Moments

Due to an eccentricity between the centre of mass and the centre of rigidity it is necessary to take into account the effect of torsional moment at floor levels of the structure in each direction. The torsional moment (T_i) is calculated for each floor of the structure by the formula:

$$T_i = V_i \ (e_i \ \mp \ e) \cdots\cdots\cdots\cdots\cdots\cdots \quad (3\text{-}6)$$

Where:

- T_i - Torsional moment at the i-th level.
- V_i - Value of the horizontal transverse seismic shear force along each considered direction separately for the i-th level.
- e_i - Distance between the rigidity centre and the mass centre at the i-th level.
- e - Accidental eccentricity (the effect of nonsynchronous seismic movement along the building) at the i-th level.
  The accidental eccentricity (e) is taken as:
  e – 0.05D - for the usual type of buildings.
  e = 0.07D - for building with an irregular distribution of structural elements.
- D - Dimension of the building perpendicular to the considered direction at the i-th level.

The structure should be designed in such a way that e_i [0.15D. In the case when this condition cannot be satisfied seismic joints should be provided. For buildings with more than 7 levels, or buildings with irregular rigidity where $e_i > 0.15D$, torsional effects should be taken into account through a three dimensional analysis.

3.3.5- Elements of structures, nonstructural components and their anchorage shall be designed to resist seismic design force given by the following formula:

$$R_e = Z \quad K_e \quad W_e \cdots\cdots\cdots\cdots\cdots\cdots \quad (3\text{-}7)$$

Where:

- R_e - Seismic design force of elements.
- Z - Seismic hazard zoning coefficient, (clause 3.2.1).
- K_e - Coefficient related to the type of elements (Table 3.5).
- W_e - Weight of element for which the seismic force is calculated.

Table 3.5 - Seismic Coefficient Related to the Elements of Structures and Non-Structural Components (K_e)

Elements of Structures and Non-Structural Components	Direction of Force	Value of K_e
Exterior and interior non-bearing walls, partitions and masonry fences.	Normal to Flat Surface	2.5
Cantilevers and cantilever parapet walls	Normal to Flat Surface	7.5
Exterior and interior ornamentations and appendages	Any Direction	10.0
When connected to or a part of a building: towers, tanks, storage racks, chimneys, smoke stacks and penthouses.	Any Direction	2.5
When connected to or a part of a building: rigid and rigidly mounted equipment and machinery not required for continued operation of essential occupancies.	Any Horizontal Direction	2.5
When resting on the ground: tank plus effective mass of it's contents.	Any Direction	2.5

3.4- Method of Dynamic Time History Analysis

3.4.1- The dynamic analysis of buildings and structures should be performed for determination of the elastic and post-elastic dynamic response of the structure to the representative earthquake ground motions at the site. The stress and deformation conditions of buildings and structures shall be determined for the criteria of design and maximum expected earthquake. The acceptable level of damages to the structural and non-structural elements for maximum expected earthquakes should be considered.

The seismic analysis by the dynamic time history analysis method is obligatory for all buildings and structures of Class 1 .

3.4.2- Earthquake ground motions for dynamic analysis of buildings and structures should be based on the geological, seismic and seismotectonic regional investigations as well on dynamic investigations of foundation material properties associated with the specific site. Ground motion time histories developed for the specific site shall be representative of actual earthquake motions.

The parameters of ground motion time histories should be determined considering the return period of earthquake occurrence at the site, life-time and usage of buildings and structures and the acceptable level of seismic risk.

The parameters of ground motion time histories should be determined for the criteria of design and the maximum expected earthquake.

The dynamic analysis requires several earthquake time histories, to insure adequate coverage of the problem. In the absence of actual strong motion earthquake records, artificial earthquake ground motions shall be developed on the basis of probabilistic methods and shall be used as an alternative.

The time history analysis shall be applied to both elastic and inelastic mathematical model of the structural system.

3.4.3- When setting up a mathematical model representing the dynamic properties of the real structure, reference should be made to examples of realistic models with which the validity of the dynamic analysis has been demonstrated. Consideration should be given to:

a) Coupling effects of the structure with its foundation and supporting ground.

b) Damping in fundamental and higher modes of vibration; For design purposes, the damping ratio for the fundamental mode of regular structures is often taken as 0.05. Structures that have few sources for frictional energy dissipation, such as bare welded steel structures, may posses lower values of damping.

c) Restoring force-distortion relationships of the structural elements in the elastic and inelastic range.

d) Effects of non-structural elements on the rigidity of the structure.

e) Torsional effects of earthquake response.

For buildings and structures of Class I, where verification of stability is performed by dynamic time history analysis, it is obligatory for the development of mathematical models of structures to use dead plus probable live load (clauses 3.1.3 and 3.2.5) without load factors.

3.4.4- The total horizontal seismic force V obtained by this analysis should not be smaller than 75% of the design force obtained by the method of equivalent static analysis (clause 3.2). The total horizontal seismic force should not be taken smaller than 0.02W.

CHAPTER 4 - VERIFICATION OF STRUCTURAL BEHAVIOUR, STRESS CONDITIONS AND DEFORMATIONS

The members of buildings and structures should be designed taking into consideration the following criteria:

4.1- Proportions of the sections for reinforced concrete elements and elements of steel structures shall preferably be determined on the basis of the limit state principles.

4.2- The design and the control of the buildings and structures and structural elements should be provided using the design methods required by the acceptable design code of practice.

4.3- The verification of the deformations at limit state is especially necessary for flexible structures (for example frame structures of multi-storey buildings where large deformations would involve some excessive damage to the infill walls and to the other non-structural elements, as well as buildings where large horizontal displacements would cause P-Δ effects, and an increase of unfavourable effect such as oscillations of water tanks).

4.4- If calculation is made by the elastic design theory, the allowable stresses can be increased by 33%.

4.5- The allowable stresses in the soil, for the most unfavourable combination of seismic and other effects, should be determined in a way that the factor of safety against shear failure in soil is not less than (1.5). For structures constructed in a soil profile of category III, the factor of safety shall not be less than (1.8) .

4.6- In the analysis of the structure and structural elements designed by the limit state theory, the following load factors should be used:

- For reinforced and prestressed concrete	$1.1D + 1.3L + 1.4E$
but not less than	$1.30\,(D + L_R + E)$
- For steel structures	$1.15(D + L_R + E)$
- For bearing masonry structures	$1.50\,(D + L_R + E)$
- When live load provide a relieving effect	$0.9D + 1.4E$

Where:

D - is the dead load.
L - is the live load.
L_R - is the probable live load.
E - is the earthquake load.

4.7- Relative Floor Displacement:

a) The maximum relative floor displacement for the seismic design force of the structure should not be larger than

$$h_i/200.$$

Where:

h_i - Height of the i-th floor.

For buildings and structures with light-weight non-brittle partitions or without partitions (open frame buildings like shopping centres, garages, etc.), the maximum relative floor displacement for the design seismic force shall not be larger than $h_i/150$. For other types of buildings and structures, the relative floor displacement may be limited according to the necessities, depending on the safety and serviceability of building and life safety of occupants.

b) If for the design of the structure a dynamic response analysis is performed for the purpose of determining the behaviour of the structural elements in the post-elastic range, the maximum relative floor displacement for the design seismic action (Design Earthquake) shall not be larger than $h_i/150$.

c) In calculating the relative storey displacements, infill walls in framed structures of Classes III & IV should not be taken into account.

For the determination of maximum relative floor displacement, using the method of equivalent static analysis, the total design lateral seismic force V shall be increased by the following coefficient:

- For building of Class III	2.5
- For building of Class IV	2.0

4.7- The maximum horizontal deflection of buildings and structures when determined for seismic design force shall not be larger than:

$$\frac{H}{600}$$

Where:

H - Is the height of the structure above ground.

For industrial and other similar buildings, the maximum horizontal deflection of the structure may be larger than H/600 if the stability of the building and the structure is analytically and/or experimentally confirmed.

CHAPTER 5 - CONSTRUCTION OF EARTHQUAKE RESISTANT STRUCTURES

5.1- Reinforced Concrete Structures

5.1.1- General, Ductility and Strength Requirements

The post-elastic deformation capacity of reinforced concrete structural elements in practice is measured by the ductility factor, defined as the ratio between the ultimate deformation and the onset of yield. Based on this definition, the ductility factor of structural elements and the whole structure can be evaluated.

The procedure for evaluation of ductility and ductility factor is generally difficult and complicated. It involves two main problems:

I - The estimation of the seismic loading effects by site and seismicity investigation, and

II - The determination of the mathematical model of structure for linear and/or non-linear dynamic response analysis.

According to this code the ductility requirements are satisfied by design and detailing requirements for structural elements and structures. (Additional dynamic analysis is required for structures of Class I). These conditions and requirements are generally referred to as:

a) Limitation on the use of non-ductile reinforcing steel bars for elements where ductility capacity is required, especially for structural elements subjected directly to the seismic actions.

b) For elements subjected to bending or to bending and compression with large eccentricity, the adoption of appropriate reinforcement percentages and position of reinforcement that will ensure a ductile deformation of elements up to the ultimate limit state.

c) For elements subjected to compression loads with small eccentricity, to compensate for low ductility it is necessary to implement appropriate design conditions for the concrete in the cross section and percentage of longitudinal and transverse reinforcement.

d) Elements subjected to eccentric compression have to be provided with additional requirements for sufficient ductility capacity to avoid any type of local failure due to shear. All critical members including joints, must be checked for a shear force corresponding to the development of the ultimate moments of the sections where hinges are expected.

e) Plastic hinges development during sever earthquakes are acceptable only in elements with high ductility. Plastic hinges should be aimed to form in beams rather than in columns.

f Anchorage and splices: Reinforcement at critical sections should be detailed to avoid bond failure.

Steel reinforcing bars with F_y equal to 250, 340, 410 MPa are accepted for ductile structural elements (columns, structural walls, beams of moment resisting frames, etc.) Welded wire fabric can be used only in horizontal diaphragms and partially in vertical structural walls and shear walls. Generally these types of reinforcement are not used as structural reinforcement for seismic loads.

5.1.2- General Classification

According to the basic structural system buildings and structures are classified as follows:

a) **Frame system:**
A system in which both vertical and lateral loads are resisted by space frames.

b) **Wall system:**
A system in which both vertical and lateral loads are resisted by vertical structural walls either single or coupled.

c) **Dual system:**
A system in which vertical loads are mainly carried by space frame. Resistance to lateral forces is provided partly by the frame system and partly by structural walls.

5.1.3- Frame Systems

Frame structures are designed as structural systems in both directions of the building itself. As a rule, the stiffness of the beams should be smaller than the stiffness of the columns, in order to ensure the occurrence of non-linear deformations (plastic hinges) at the ends of the beams.

Frame systems are designed in a way that the structural elements are able to dissipate the seismic energy by bending and the occurrence of non-linear deformations at the ends of the beams. The non-linear deformations at the columns should be avoided.

The joints are designed so that they remain in the linear range even after the occurrence of non-linear deformations in the elements they join.

a) **Columns:** The design of columns which are subjected totally or partially to seismic influence should be based on the following requirement:

$$A_c \geq \frac{P}{\Phi_c \; f_{cu}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (5\text{-}1)$$

Where:

A_c - is the gross area of the section of the column (mm^2).
P - is the total axial force in column due to factored gravity loads (D+L) (Newtons).
F_{cu} - is the characteristic compressive strength of concrete in (N/mm^2).
Φ_c - is the reduction coefficient given in Table 5.1.

■ Percentages of total longitudinal reinforcement should fall between the minimum and maximum limits given in Table 5.2.

Table 5.1 - Reduction Coefficient (Φ_c)

Type of column	Values of Φ_c	
	Zone I, II	Zone III
Interior	0.28	0.25
Perimeter	0.25	0.22
Corner	0.22	0.18

Table 5.2 - Percentage of Minimum and Maximum Longitudinal Steel Reinforcement

Column Type	Minimum longitudinal steel reinforcements (%)			Maximum longitudinal steel reinforcement (%)		
	f_y (MPa)			f_y (MPa)		
	250	340	410	250	340	410
Interior	0.8	0.7	0.6	6	5	4
Perimeter	0.9	0.8	0.7	6	5	4
Corner	1.0	0.9	0.8	6	5	4

In case when the section of the column is defined by architectural reasons, so that reinforcement is not determined from design considerations, the minimum percentage of reinforcement related to the gross concrete section will be considered as 0.5% for all types of steel.

It is preferable that the diameter of the longitudinal bars may not exceed 32mm and the distance between centers of bars should not exceed 250mm.

- Splices: Lap splices, in general, should be away from the potential hinge regions and be within areas of small tensile stresses.

When there are several bars in a column which are not welded, half of these reinforcements should be extend to cover two floors. This means, that 50% of the reinforcement are lapped at each floor.

For structures constructed in Zone III, the lapping of reinforcement with diameter larger than 28mm, should be made by welding.

The design shear force in a column should be estimated by ultimate capacity analysis with ultimate bending moment at both ends of the column according to the following formula:

$$1.5\ Vic \leq \frac{M_{u_1} + M_{u_2}}{h_c} \leq 3\ Vic \quad \cdots\cdots\cdots\cdots \quad (5\text{-}2)$$

Where:

M_{u1}, M_{u2} - are the ultimate moment capacities considered positive at upper and lower ends of the column under axial load condition ($D+L_R$).

h_c - is the clear height of the column.

Vic - is the contribution of seismic shear force of the column in the i-th floor.

Short columns with h_c/b | 2 should be avoided.

Where:

b - is the dimension of column cross-section in the considered direction.

- The following condition should be satisfied for the cross-section of reinforced concrete columns:

$$A_c \geq \frac{M_{u_1} + M_{u_2}}{0.14 h_c \sqrt{f_{cu}}} \qquad (5\text{-}3)$$

All units are in (N, m).

<u>Note:</u> The shear control for columns given above is obligatory for buildings of Class I, II, III and IV. In seismic zones II and III, in addition to the requirement of Iraqi Code 1/1987 for reinforced concrete.

- The minimum transverse reinforcement in each direction of the section of the column should not be less than 0.20% . The transverse reinforcement is calculated by the following formula:

$$\rho_t = \frac{A_t}{s\ b} 100\% \qquad (5\text{-}4)$$

Where:

ρ_t - is the percentage of transverse reinforcement.

A_t - is the total area of ties intersected by a vertical plane parallel to the side of the column (b).

S_b - is the distance between ties.

In zones II and III within a minimum length of 500mm from the joints, the distance between the ties should not exceed 150mm and the percentage of transverse reinforcement of the section of column should not be less than 0.25%. This transverse reinforcement in the columns should be continued through the joints. Closing of the ties is made by overlapping extended to the whole length of the shorter side.

b) **Beams:**

- The moment resisting frame systems designed in seismic zone III should have beams with a depth limited by the following conditions:

  Width of beam ⟨ 0.4 depth of beam
  Width of beam ⟨ 0.5 width of column

- The minimum percentage of compression reinforcement (ρ') placed at the support should be:

  0.30 ρ in seismic zone I and II
  0.40 ρ in seismic zone III.

By more favourable value of the percentages of tensile reinforcement (ρ) and of compression reinforcement (ρ'), the ductility of the beam is increased.

The percentage of total longitudinal reinforcement ($\rho+\rho'$) should not exceed:

4.5% for steel : $f_y = 250$ MPa
4.0% for steel : $f_y = 340$ MPa
3.5% for steel : $f_y = 410$ MPa

For buildings designed in seismic zone III, the maximum shear force in beams should be estimated by the ultimate bending moment at each end of the beam according to the following formula:

$$V_{max} = \frac{M_{UL} + M_{UR}}{L_b} + V_g \quad \text{.......................} \quad (5\text{-}5)$$

Also, the following conditions should be satisfied for the cross section of reinforced concrete beams:

$$b\,d \geq \frac{V_{max}}{0.14\sqrt{f_{cu}}} \quad \text{.......................} \quad (5\text{-}6)$$

Where:

M_{UL},M_{UR} - are the absolute values of the ultimate moment capacity at the ends of the beam.

L_b - is the clear span of beam.

V_g - is the shear force from ($D+L_R$).

b,d - are the dimensions of the active beam cross-section.

f_{cu} - is the characteristic compressive strength of concrete.

The units in the above formula are in Newtons and millimeters.

- The spacing of ties in the beams near the joints, for a distance equal to double the height of the beam should not exceed 150mm, and the area of stirrups should be at least 0.2%. In Zone III, anchorage of the ties should be made by overlapping extended to the whole length of the shorter side.

c) **Joints:**

- The joint's core should be designed in a way that it can transmit the ultimate limit state forces that can occur in the connected elements (beams and columns) without damage.
- When the width of column is larger than the width of connected beams, all column reinforcement located outside the core of joint is required to interact with the beam. In this case it is preferable to use additional longitudinal reinforcement in the column through the joint.

d) **Infill Walls:**

The infill walls of the frame systems should be made as light as possible. If by structural measures and calculations, it is proved that it is necessary to have the infill walls be anchored to the basic system (by special connectors or joints, etc.), the anchoring of the infill walls should not increase the rigidity of the basic structural system.

If the structural system is flexible, i.e., it can undergo relative deformations at the floors larger than that given in clause 4.7 under seismic effect, the stability and damage level of infill walls should be controlled by using experimental data. The stability of the infill walls should also be checked for the direction orthogonal to the wall, according to clause 3.3.5 of this code.

5.1.4- Wall Systems

a) General Design Considerations:

- Wall systems are systems which have reinforced concrete walls as the main structural system in both directions.
- All walls, on which lateral earthquake load is applied should be designed in such a way that they can dissipate seismic energy by flexural yielding.
- Appropriate design procedures should be used to ensure that the ultimate shear strength of walls should be in excess of the maximum shear force when flexural strength capacity is reached.
- When two or more structural walls are inter-connected in the same plane by substantially ductile beams, part of the seismic energy to be dissipated should be assigned to the coupling system. Capacity design procedures (non-linear) should be used to ensure that the energy dissipation in the coupling system can be maintained at its flexural strength capacity. Structural walls, coupled shear walls and diaphragms should be considered as integral units.
- The area of the transverse section of the walls, for each orthogonal direction should not be smaller than 1.2% of the gross floor area of the building.
- The design of walls which are subjected totally or partially to seismic influence should be based on the following requirement:

$$A_w \geq \frac{P}{\Phi_w f_{cu}} \qquad (5\text{-}7)$$

Where:

A_w - is the gross area of the horizontal section of the wall (mm^2).
f_{cu} - is the characteristic compressive cube strength of concrete (N/mm^2).
Φ_w - is a coefficient given in Table 5-3.
P - is the total axial force in wall due to gravity loads (D+L) in (Newton).

Table 5.3 - Values of Φ_w for Different Zones of Seismic Activity

Coefficient	Zone I and II	Zone III
Φ_w	0.18	0.15

- The ratio of the total height to the length of each structural wall should not be smaller than 2.0 .

- Openings in walls not regularly arranged to form coupled walls should preferably be avoided, unless their influence on the behaviour of the wall under seismic action is either insignificant or accounted for by rational analysis.

- The thickness of the bearing walls should not be less than 150mm.

- Bearing walls must be well anchored to floors, roofs, columns, pilasters, buttresses and intersecting walls.

- In case of Class 1 buildings, for which the structural analysis is made by dynamic procedure in accordance with this code, the ultimate shear forces in the plastic zones should be entirely resisted by reinforcement.

- The structural wall system must ensure the global stability of the structure to overturning.

b) **Vertical Reinforcement:**

- The ratio of vertical reinforcement in any part of the section should not be less than 0.25% or $0.8/f_y$ and not greater than 3.5% or $16/f_y$ of the cross sectional area of that part of the wall. If the walls are reinforced with less than the above minimum steel they should be designed in accordance with the clause 3.2.4, Table 3.4, (Type No.5).

- The ratio of the reinforcement at each end of the wall should not be smaller than 0.15% of the total horizontal section of the wall to be placed within a distance of 1/10 of the wall length at each end. The minimum percentage of steel for the middle part is 0.15%. The middle part may be reinforced with welded wire mesh.

- The splicing of the vertical reinforcement in the middle part of the wall section may be made by overlapping. Reinforcement at ends are spliced by welding or the reinforcement is extended over two floors which means splicing of 50% of the reinforcement by overlapping at each level.

c) **Horizontal and other Reinforcements:**

- The horizontal reinforcement of walls is determined by calculations, so that the total calculated seismic shear force for the considered level should be resisted by the horizontal reinforcement. The horizontal reinforcement ratio in any part of the section should not be less than 0.25%.

- Transverse reinforcement (ties) may be used to confine concrete in regions where large inelastic compressive strains can occur; to satisfy the intended sectional ductility; and to restrain the vertical bars from buckling.

- Where diagonal reinforcement is used in coupling beams, they should be enclosed by rectangular ties in each direction.

5.1.5- Dual Systems

In this system the vertical loads are mainly carried by frames and the resistance to lateral forces is provided partly by the frame system and partly by the structural walls.

- The distribution of the seismic forces is performed according to the deformation characteristics of each element of the basic structural system.

- The frames should be designed to take at least 25% of the total seismic force. The structural walls are designed for the value of the shear forces obtained by analysis according to the requirements of this code.

5.2- Prestressed Concrete Structures

5.2.1- Under the expression prestressed concrete structure in this code it means a concrete structure in which the seismic effects and the seismic energy dissipation is taken by prestressed elements. If the structural elements, in addition to the prestressing steel reinforcement, also contains longitudinal ordinary steel reinforcement of at least 0.45%, such a structure is considered as a reinforced concrete structure.

- The stability of the system and the elements of the structure should be proved by analytical and/or experimental procedure.

- The elements of prestressed concrete structures are designed to dissipate the seismic energy by bending and by the occurrence of non-linear deformations.

- The prestressed concrete elements in addition to steel tendons for prestressing, should contain at least 0.20% ordinary steel reinforcement to provide for seismic energy dissipation.

- At the critical sections, where non-linear deformations are expected, closely spaced transverse reinforcement should be provided to resist the total ultimate shear force. which corresponds to the ultimate moment in the section increased by 10%.

- The ratio of the total height to the length of each structural wall should not be smaller than 2.0 .

- Openings in walls not regularly arranged to form coupled walls should preferably be avoided, unless their influence on the behaviour of the wall under seismic action is either insignificant or accounted for by rational analysis.

- The thickness of the bearing walls should not be less than 150mm.

- Bearing walls must be well anchored to floors, roofs, columns, pilasters, buttresses and intersecting walls.

- In case of Class I buildings, for which the structural analysis is made by dynamic procedure in accordance with this code, the ultimate shear forces in the plastic zones should be entirely resisted by reinforcement.

- The structural wall system must ensure the global stability of the structure to overturning.

b) Vertical Reinforcement:

- The ratio of vertical reinforcement in any part of the section should not be less than 0.25% or $0.8/f_y$ and not greater than 3.5% or $16/f_y$ of the cross sectional area of that part of the wall. If the walls are reinforced with less than the above minimum steel they should be designed in accordance with the clause 3.2.4, Table 3.4, (Type No.5).

- The ratio of the reinforcement at each end of the wall should not be smaller than 0.15% of the total horizontal section of the wall to be placed within a distance of 1/10 of the wall length at each end. The minimum percentage of steel for the middle part is 0.15%. The middle part may be reinforced with welded wire mesh.

- The splicing of the vertical reinforcement in the middle part of the wall section may be made by overlapping. Reinforcement at ends are spliced by welding or the reinforcement is extended over two floors which means splicing of 50% of the reinforcement by overlapping at each level.

c) Horizontal and other Reinforcements:

- The horizontal reinforcement of walls is determined by calculations, so that the total calculated seismic shear force for the considered level should be resisted by the horizontal reinforcement. The horizontal reinforcement ratio in any part of the section should not be less than 0.25%.

- Transverse reinforcement (ties) may be used to confine concrete in regions where large inelastic compressive strains can occur; to satisfy the intended sectional ductility; and to restrain the vertical bars from buckling.

- Where diagonal reinforcement is used in coupling beams, they should be enclosed by rectangular ties in each direction.

5.1.5- Dual Systems

In this system the vertical loads are mainly carried by frames and the resistance to lateral forces is provided partly by the frame system and partly by the structural walls.

- The distribution of the seismic forces is performed according to the deformation characteristics of each element of the basic structural system.

- The frames should be designed to take at least 25% of the total seismic force. The structural walls are designed for the value of the shear forces obtained by analysis according to the requirements of this code.

5.2- Prestressed Concrete Structures

5.2.1- Under the expression prestressed concrete structure in this code it means a concrete structure in which the seismic effects and the seismic energy dissipation is taken by prestressed elements. If the structural elements, in addition to the prestressing steel reinforcement, also contains longitudinal ordinary steel reinforcement of at least 0.45%, such a structure is considered as a reinforced concrete structure.

- The stability of the system and the elements of the structure should be proved by analytical and/or experimental procedure.

- The elements of prestressed concrete structures are designed to dissipate the seismic energy by bending and by the occurrence of non-linear deformations.

- The prestressed concrete elements in addition to steel tendons for prestressing, should contain at least 0.20% ordinary steel reinforcement to provide for seismic energy dissipation.

- At the critical sections, where non-linear deformations are expected, closely spaced transverse reinforcement should be provided to resist the total ultimate shear force. which corresponds to the ultimate moment in the section increased by 10%.

- The anchoring of prestressing reinforcement should be outside the expected plastic hinge zones.

- The structural deformations should be restricted depending on the function of the building and on the effect of the deformation upon the structural elements of the building.

5.2.2- **Joints of the elements are designed so that:**

a) The ultimate strength capacity of the joint should be at least equal to the ultimate strength of the elements joining into it.

b) Joints should be ductile, assuring their deformability;

c) Joints should be reinforced with adequate shear reinforcement which should completely resist the ultimate shear force.

5.3- Steel Structures

5.3.1- Steel structures should be designed so that the structural elements are able to dissipate the seismic energy by bending and by the occurrence of non-linear deformations. In case of frame systems, non linear deformations are allowed at the beam ends or at the diagonal bracings.

5.3.2- Local buckling should not be allowed in zones of plastic hinges. Furthermore, proportioning of the joints should be made so that they provide for the transmission of ultimate bending moments and the corresponding shear forces from one element to another, without occurrence of non-linear deformations in the joint's zone. In other words, the joints should be proportioned to work always in the elastic range.

5.3.3- In addition to the above, steel structures must conform to the following requirements.

a) For one story industrial buildings, the transfer of the forces from the roof level is recommended to be through structural walls or by bracing systems with an adequate rigidity to ensure the limitation of the deformations within the roof plane. For structures with more than one storey, vertical structural walls or bracings are also recommended.

b) Structural solution has to be provided after careful consideration of the deformation compatibility of various structural and non-structural elements.

c) For the support of the principal structural elements slides and rollers should be avoided. In case when such supports are used, measures to limit the lateral or vertical displacements should be considered.

5.4- Prefabricated Structures

5.4.1- The stability of the structural system and the system of joints of prefabricated reinforced concrete, prestressed and other prefabricated structures, should be proved by experimental and/or analytical study.

5.4.2- The structural system, as well as the system of joints shall be as simple as possible. The system of joints between the elements should ensure the overall integrity of the structure.

5.4.3- The reinforcement that receives the tension stresses should be extended so that the yield stress in the reinforcement can be developed by anchorage bond.

5.4.4- The structural floor should be designed as rigid diaphragms in their own planes.

5.4.5- The horizontal joints which join the floor elements, as well as the vertical bearing elements should be constructed to provide monolithic state of the joints and stability to the structural system in general.

5.5- Masonry Structures

The basic system of masonry structures are the bearing walls in both directions of the building, connected by sufficiently rigid floor system. The term masonry structures in this code includes the following:

a) Simple masonry structures.

b) Masonry structures with vertical reinforced concrete elements.

c) Composite masonry and concrete with or without reinforcement.

d) Reinforced masonry structures with reinforcement in the joints.

5.5.1- Simple masonry structures; are walls of clay bricks, blocks, or other material elements connected with mortar with strength of at least 2.5 N/mm^2. It is preferable to use cement-lime mortar in the construction.

5.5.2- Masonry structures with vertical reinforced concrete elements; are walls constructed with masonry units with vertical reinforced concrete elements cast in place after the construction of masonry.

5.5.3- Composite masonry and concrete with or without reinforcement; are walls constructed with one or two sides of masonry units with a concrete cast insitu with minimum thickness of 80mm. If reinforcement is used for these walls the amount of vertical and horizontal reinforcement is to be estimated by analysis. The minimum amount of reinforcement is 0.1% of the total thickness of wall including masonry part.

5.5.4- Reinforced masonry structures with reinforcement in the joints; are masonry walls constructed with mortar of strength of at least 5.0 N/mm^2. with steel reinforcement in horizontal and/or vertical directions. The reinforcement should be made of steel bars placed at equal distances of not more than 500mm.

5.5.5- The walls which provide rigidity are distributed as uniform as possible in both directions of the building considering the following as applicable.

a) The minimum wall thickness is 200mm.

b) The floor slab shall be rigid. If prefabricated elements are used, a topping cast-in-situ slab with minimum thickness of 40mm reinforced with steel mesh has to be used.

c) For structural floors with insufficient rigidity the height of the building above ground shall be limited to two levels. These floors can be considered as rigid floors if cast-in-situ concrete of minimum 40mm reinforced with steel mesh on top is used.

d) The maximum distance between walls which provide rigidity in each direction in the building should be 10m.

e) The width of portions of masonry walls between openings when floors are not sufficiently rigid should be at least 1/3 of smaller opening dimension.

f) Application of combined system, i.e., the lower part of the building to be reinforced concrete skeleton, and the upper part of bearing walls; is not allowed.

g) In Zone III, free-standing walls and parapets above the floor structure, higher than 800mm, have to be tied together (possibly with reinforced concrete ties).

5.5.6- Checking of the resistance of masonry structures is to be made according to the method of allowable stresses or the limit state method. In case of buildings with ratio of height to width over 2.0, the walls should also be checked for bending and shear, an increase of allowable stresses by 50% is permitted.

a) If checking of the resistance is made by the method of allowable stresses, the principal tensile stresses in different elements (walls) shall be controlled. The factor of safety should not be less than 1.5.

The principal tensile stresses in different elements (walls) shall be obtained from the following expression:

$$\sigma_m = \sqrt{\frac{\sigma_o^2}{4} + (1.5\,\tau_o)^2} - \frac{\sigma_o}{2} \leq \sigma_{ma} \quad \cdots\cdots\cdots \quad (5\text{-}8)$$

Where:

σ_{ma} - is the allowable principal tensile stress.

τ_o - is the average shear stress in the wall element due to seismic effect.

σ_o - is the average stress in the wall element due to vertical loads.

b) If checking of the resistance is made by the limit state method, the resistance of the structure should not be less than the factored total horizontal seismic force according to clause 3.2 . The load factors are to be in accordance with clause 4.6 .

The resistance of each individual wall element can be calculated according to the following expression:

$$\tau_o = \frac{\sigma_{mr}}{1.5}\sqrt{1 + \frac{\sigma_o}{\sigma_{mr}}} \quad \cdots\cdots\cdots \quad (5\text{-}9)$$

Where:

σ_{mr} - is the principal tensile stress at failure of walls constructed of different material.

5.5.7- Number of Storeys

a) The maximum number of storeys including ground floor for different systems of masonry structures is given in Table 5.4 .

- **Table 5.4 - Maximum allowed number of storeys for masonry structures.**

Type of Masonry Structure	Seismic Zone	
	I , II	III
a) Simple (clause 5.5.1).	3	2
b) With vertical reinforced concrete elements (clause 5.5.2).	4	3
c) Composite (clause 5.5.3)		
- Non reinforced	4	3
- Reinforced	No Limit *	No Limit *
d) Reinforced masonry (clause5.5.4).	5	4

* According to design calculations

b) The number of storeys in Table 5.4 may be increased if proved by analytical study with sufficient experimental data.

c) In case masonry buildings are not analyzed for seismic effects, but otherwise conform to the requirements of this code, the allowable number of storeys independently from the structural system, is limited to:

3 storeys in seismic Zone I and II.
4 storeys in seismic Zone III.

5.6- Foundations

5.6.1- General

The design and construction of foundations, foundation components, and the connection of the superstructure elements thereto, shall conform to the requirements of this section and other applicable requirements in Seismic Zone I, II and III.

5.6.2- Soil Capacities

The capacity of the foundation soil in bearing or the capacity of the soil interface between pile, pier or caisson and the soil shall be sufficient to support the structure with all prescribed loads, other than earthquake forces, taking due account of the settlement that the structure is capable of withstanding. For the load combination including earthquake, the soil capacity must be sufficient to resist loads at acceptable strains considering both the short time of loading and the dynamic properties of the soil. Allowable soil stress may be increased by more than 33 percent if substantiated by geotechnical data. For piles, this refers to pile capacity as determined by pile-soil friction or bearing.

5.6.3- Superstructure-to-Foundation Connection

The connection of superstructure elements to the foundation shall be adequate to transmit to the foundation the forces for which the elements are designed.

5.6.4- Foundation-Soil Interface

for regular buildings, the force at top of building (0.15V) as per section 3.3 may be omitted when determining the overturning moment to be resisted at the foundation-soil interface.

5.6.5- Special Requirements for Piles and Caissons

a) Piles and caissons shall be designed for flexure whenever the top of such members will be displaced by earthquake motions. The criteria and detailing requirements of column design by this code shall apply for a length of such members equal to 120 percent of the flexural length.

b) Footing Interconnection

1) Pile caps shall be completely interconnected by structural members (tie beams) or approved equivalent means.

2) All strut members shall be capable of resisting in tension or compression a force not less than 10 percent of the larger footing or column load unless it can be demonstrated that equivalent restraint can be provided by other approved means.

Appendix A

LIST OF REPORTS OF INVESTIGATIONS FOR ELABORATION OF PRELIMINARY SEISMIC DESIGN CODE OF IRAQ

1. Volume I, Report IZIIS 88-84. SIESMIC HAZARD EVALUATION AND SEISMIC ZONING MAPS OF IRAQ.
2. Volume II, Report IZIIS 88-85. COMPILATION AND ANALYSIS OF MICROTREMORS RECORDED IN BAGHDAD.
3. Volume III, Report IZIIS 88-86. GEOTECHNICAL MODELLING OF SELECTED FREE FIELD AND BUILDING SITES IN BAGHDAD, BASRAH AND MOSUL, TRANSFER FUNCTION ANALYSIS AND DETERMINATION OF REPRESENTATIVE SOIL PROFILES FOR DYNAMIC SITE RESPONSE ANALYSIS.
4. Volume IV, Report IZIIS 88-87. DYNAMIC PROPERTIES OF THE REPRESENTATIVE SOIL DEPOSITS IN BAGHDAD.
5. Volume V, Report IZIIS 88-88. DYNAMIC RESPONSE ANALYSIS OF REPRESENTATIVE SOIL PROFILES IN BAGHDAD, BASRAH AND MOSUL.
6. Volume VI, Report IZIIS 88-89. PRELIMINARY SEISMIC MICROZONING OF BAGHDAD METROPOLITAN AREA AND EARTHQUAKE DESIGN SPECTRA.
7. Volume VII, Report IZIIS 88-90. DYNAMIC PROPERTIES FROM FULL-SCALE AMBIENT VIBRATION TESTS OF THE REPRESENTATIVE BUILDINGS IN BAGHDAD.
8. Volume VIII, Report IZIIS 88-91. VERIFICATION CRITERIA BASED ON SEISMIC STABILITY ANALYSIS OF BIOLOGICAL RESEARCH CENTRE.
9. Volume IX, Report IZIIS 88-92. VERIFICATION CRITERIA BASED ON SEISMIC STABILITY ANALYSIS OF APARTMENT BUILDING NO. 15 IN COMPLEX NO. 10 (8 STORIES).
10. Volume X, Report IZIIS 88-93. VERIFICATION CRITERIA BASED ON SEISMIC STABILITY ANALYSIS OF APARTMENT BUILDING IN HAIFA COMPLEX NO, 8 (16 STORIES).
11. Volume XI, Report IZIIS 88-94. VERIFICATION CRITERIA BASED ON SEISMIC STABILITY ANALYSIS OF COMMERCIAL BUILDING IN JAMUHURIA STREET.
12. Volume XII, Report IZIIS 88-95. VERIFICATION CRITERIA BASED ON SEISMIC STABILITY ANALYSIS OF SCHOOL BUILDING IN EL CAMALIA.
13. Volume XIII, Report IZIIS 88-96. EVALUATION OF SOIL STRUCTURE INTERACTION EFFECTS OF SELECTED BUILDINGS AND DYNAMIC PROPERTIES OF TYPICAL ELEVATED WATER TANK.
14. Volume XIV, Report IZIIS 88-97. VULNERABILITY ASSESSEMENT AND EVALUATION OF ACCEPTABLE SEISMIC RISK.

15. Volume XV, Report IZIIS 88-98. PRELIMINARY DRAFT OF IRAQI SEISMIC DESIGN CODE (ISDC).
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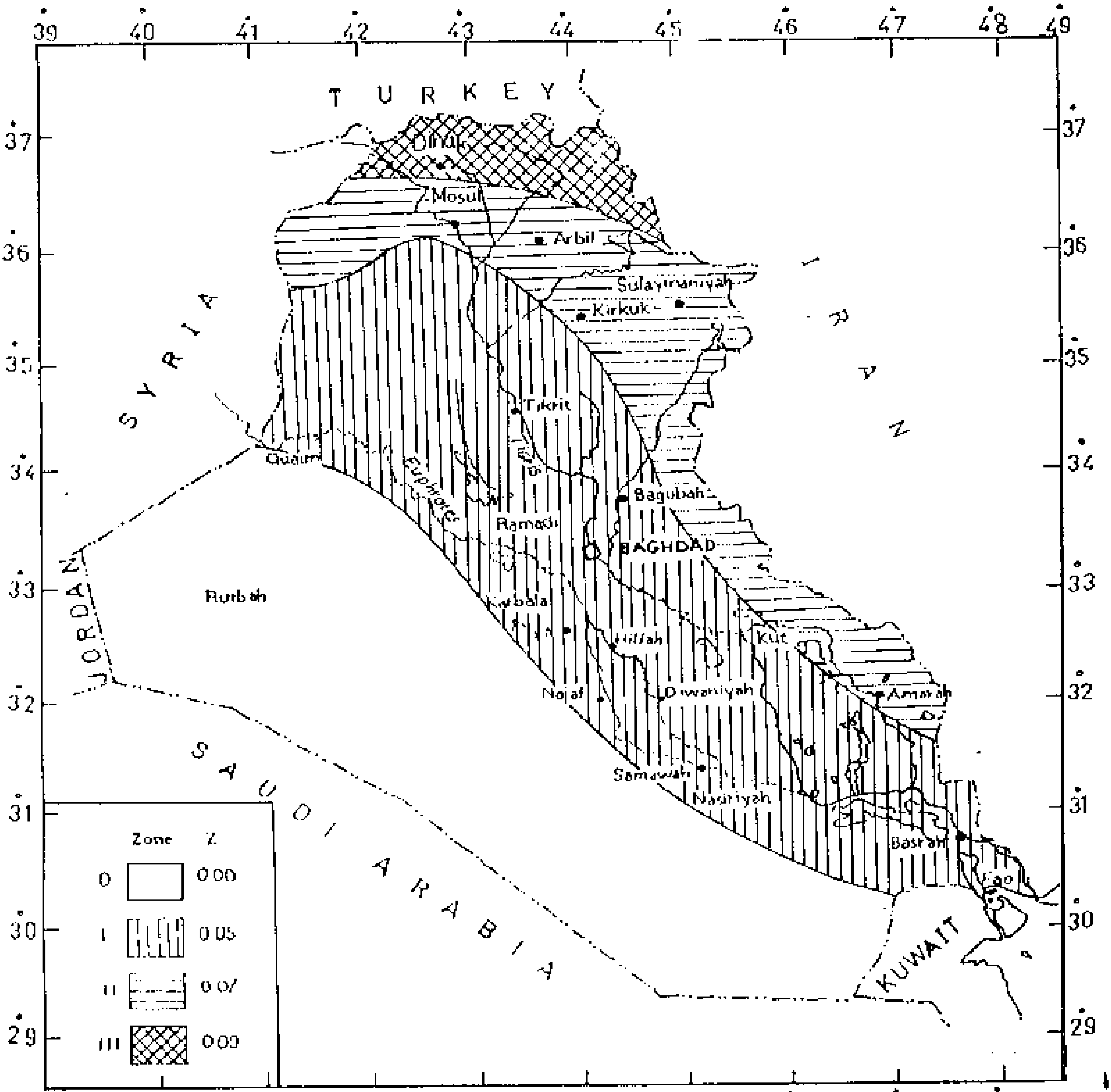
22. RECOMMENDED LATERAL FORCE REQUIREMENTS AND TENTATIVE COMMENTARY, By Seismology Committee of Structural Engineers Association of California - 1988.

APPENDEX B



SEISMIC ZONING MAP OF IRAQ

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